City of Riverside Public Works Department

# UPDATE OF THE INTEGRATED MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

## VOL. 4 Wastewater Treatment System


Chapter 1: Existing Facilities
Chapter 2: Summary of Planning Studies
Chapter 3: Process Design and Reliability Criteria
Chapter 4: Preliminary Treatment
Chapter 5: Primary Treatment
Chapter 6: Secondary Treatment
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Chapter 8: Advanced Water Treatment
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# Abbreviations

| | |
|---|---|
| αF | alpha factor |
| µg/L | micrograms per liter |
| A/A Trunk Sewer | Acorn/Arlanza Trunk Sewer |
| AACE | Advancement of Cost Engineering |
| AADF | annual average daily flow |
| AAF | average annual flow |
| AB | Assembly Bill |
| ABS | Acrylonitrile-Butadiene-Styrene |
| ACI | American Concrete Institute |
| ACP | asbestos cement pipe |
| ACS | American Community Survey |
| ACT treatment train | Activated treatment train |
| ADC | alternative daily cover |
| ADEQ | Arizona Department of Environmental Quality |
| ADF | average daily flow |
| ADWF | average dry weather flow |
| AFY | acre-feet per year |
| APAD | acid-phase anaerobic digestion |
| AQMD | Air Quality Management District |
| AQMP | Air Quality Management Plan |
| ARB | California Air Resources Board |
| ARVs | air release valves |
| ASCE | American Society of Civil Engineers |
| AWT | Advanced Water Treatment |
| BACT | best available control technology |
| BCM | Best Available Control Measures for Fugitive Dust Sources |
| BFP | belt filter press |
| BMP | best management practices |
| BNR | Biological nutrient removal |
| BOD | biochemical oxygen demand |
| $BOD_5$ | 5-day biochemical oxygen demand |
| BPTC | Best Practicable Treatment or Control |
| Btu/lb | British thermal unit per pound |
| BWF | base wastewater flow |
| C | Celsius |
| $CaCO_3$ | calcium carbonate |
| Carollo | Carollo Engineers, Inc. |

| | |
|---|---|
| CASA | California Association of Sanitation Agencies |
| CCB | chlorine contact basin |
| CCF | hundred cubic feet |
| CCI | Construction Cost Index |
| CCTV | Closed Circuit Television |
| CDFW | California Division of Fish and Wildlife |
| CDM | Camp Dresser & McKee, Inc. |
| CECs | Contaminants of Emerging Concern |
| CEQA | California Environmental Quality Act |
| CESA | California Endangered Species Act |
| cfd | cubic feet per day |
| cfm | cubic feet per minute |
| cfs | cubic feet per second |
| CIP | Capital Improvement Program |
| City | City of Riverside |
| CIWQS | California's Integrated Water Quality System |
| CMB | Combustion Sources |
| CMMS | Computerized Maintenance Management System |
| CNG | compressed natural gas |
| $CO_2$ | carbon dioxide |
| COD | chemical oxygen demand |
| COS | cost-of-service |
| cP | centipoise |
| CSCI | California Stream Condition Index |
| CSD | Community Services Districts |
| CTS | Coatings and Solvents |
| cu ft | cubic feet |
| CWA | Clean Water Act |
| CWC | California Water Code |
| CWEA | California Water Environment Association |
| DAF | dissolved air flotation |
| DAFT | dissolved air flotation thickeners |
| days/week | days per week |
| DCR | demand-capacity ratio |
| DDW | California Division of Drinking Water |
| DG | digester gas |
| DIR | Department of Industrial Relations |
| DMR | discharge monitoring report |
| DU | dwelling unit |

| | |
|---|---|
| DWF | dry weather flow |
| EBRT | empty bed residence time |
| EC | Emerging Constituents |
| EDR | electro-dialysis reversal |
| EDU | equivalent dwelling units |
| EGM | Emission Growth Management |
| EIR | Environmental Impact Report |
| EnerTech | EnerTech Environmental California, LLC |
| ENR | Engineering News Record |
| EPA | Environmental Protection Agency |
| EQ | equalization |
| ESA | Endangered Species Act |
| F | Fahrenheit |
| FDA | Food & Drug Administration |
| Flo-Dar | Marsh-McBirney Flo-Dar™ |
| FLX | Compliance Flexibility Program |
| FOG | fats, oils, and grease |
| fps | feet per second |
| ft | feet |
| FTE | Full Time Employee |
| FUG | Fugitive Emissions |
| FY | fiscal years |
| g | grams |
| gal | gallons |
| GBT | gravity belt thickeners |
| GHG | Greenhouse Gas |
| GIS | Geographic Information System |
| GL | General Ledger |
| gpcd | gallons per capita day |
| gpd | gallons per day |
| gpd/ac | gallons per day per acre |
| gpm | gallons per minute |
| gpm/sq ft | gallons per minute per square feet |
| GWI | groundwater infiltration |
| $H_2S$ | hydrogen sulfide |
| HGL | hydraulic grade line |
| HID | High-Intensity Discharge |
| hp | horsepower |
| hr | hour |

| | |
|---|---|
| hrs/day | hours per day |
| $HS^-$ | hydrogen sulfide ion |
| HVAC | heating, ventilation, and air conditioning |
| I/I | Infiltration/inflow |
| IEBL | Inland Empire Brine Line |
| IEUA | Inland Empire Utilities Agency |
| in. | inch |
| iPACS | Internet-based POTW Administration and Compliance System |
| IRWD | Irvine Ranch Water District |
| IT | Information Technology |
| IWWMP | Integrated Wastewater Master Plan |
| klb/d | thousand pounds per day |
| klbN/d | thousand pounds of nitrogen per day |
| KPI | Key Performance Indicator |
| kWh | kilowatt hour |
| lbs | pounds |
| lbs/cfd | pounds per cubic feet per day |
| $lbs/ft^3$ | pounds per cubic feet |
| LF | linear feet |
| LIMS | Laboratory Information Management System |
| LM | Longitudinal Motion |
| LOTO | Lock Out / Tag Out |
| LRO | legally responsible official |
| Master Plan | Integrated Master Plan for the Wastewater Collection and Treatment Facilities |
| MBR | Membrane Bioreactor |
| MBR treatment train | Membrane Bioreactor treatment train |
| MCC | motor control center |
| MCS | Multiple Component Sources |
| MDD | maximum day demand |
| MDL | Method Detection Limits |
| MF | microfiltration |
| MFR | multi-family residential |
| MG | million gallons |
| mg-min/L | milligrams per minute per liter |
| mg/L | milligrams per liter |
| mgd | million gallons per day |
| mgN/L | milligrams of nitrogen per liter |
| min | minute |

| | |
|---|---|
| min/hr | minutes per hour |
| ML&C | mortar lined and coated |
| MLSS | mixed liquor suspended solids |
| mm | millimeter |
| MMBtu | million British thermal units |
| MMBtu/hr | million British thermal units per hour |
| MMRP | Measurement, Monitoring and Reporting Procedures |
| MOB | Mobile Source Programs |
| MP | Master Plan |
| MPN | most probable number |
| MRP | Monitoring and Reporting Program |
| msl | mean sea level |
| mV | millivolt |
| N/L | nitrogen per liter |
| $N_2O$ | nitrous oxide |
| NACWA | National Association of Clean Water Agencies |
| $NaHSO_3$ | sodium bisulfite |
| NaOCl | sodium hypochlorite |
| NASSCO | National Association of Sewer Service Companies |
| NEC | National Electric Code |
| NELAC | National Environmental Laboratory Accreditation Conference |
| NELAP | National Environmental Laboratory Accreditation Program |
| NFPA | National Fire Protection Association |
| NGO | Non-Governmental Organizations |
| $NH_3$-N | ammonia nitrogen |
| NOAA | National Oceanic and Atmospheric Association |
| NOI | notice of intent |
| $NO_X$ | Nitrogen oxides |
| NPDES | National Pollutant Discharge Elimination System |
| NTU | nephelometric turbidity unit |
| O&M | Operations and Maintenance |
| OCSD | Orange County Sanitation District |
| OERP | Overflow Emergency Response Plan |
| OES | Office of Emergency Services |
| OJT | On-the-Job Training |
| ORP | Oxidation-Reduction Potential |
| P/L | phosphorus per liter |
| PACP | Pipeline Assessment Certification Program |
| PAYGO | Pay-As-You-Go |

| | |
|---|---|
| PDR | Preliminary Design Report |
| PEIR | Programmatic Environmental Impact Report |
| PFRP | Process to Further Reduce Pathogens |
| PLC | programmable logic controller |
| POTW | Publicly Operated Treatment Work |
| ppbv | parts per billion by volume |
| ppcd | pounds per capita per day |
| ppd | pounds per day |
| ppd/cu ft | pounds per day per cubic feet |
| ppd/sq ft | pounds per day per square feet |
| pph | pounds per hour |
| ppm | parts per million |
| psf | pounds per square foot |
| psi | pounds per square inch |
| PTZ | Pan-Tilt-Zoom |
| PVC | Polyvinyl Chloride |
| PWS | potable water salinity |
| PWWF | peak wet weather flow |
| QICS | Qualitative Intelligence and Communication System |
| R&R | rehabilitation and repair |
| RAS | return activated sludge |
| RCNLD | Replacement Cost New Less Depreciation |
| RCP | reinforced concrete pipe |
| RDII | Rain Derived Infiltration and Inflow |
| RDT | rotary drum thickeners |
| RECLAIM | Regional Clean Air Incentives Market |
| Regional Board | California Regional Water Quality Control Board |
| RG | rain gauge |
| RNG | renewable natural gas |
| RO | reverse osmosis |
| RPU | Riverside Public Utilities |
| RST | rotary screw thickeners |
| RTP | Regional Transportation Plan |
| RWQCB | Regional Water Quality Control Board |
| RWQCP | Regional Water Quality Control Plant |
| $S_2^-$ | sulfide ion |
| SARDA | Santa Ana River Dischargers Association |
| SART | Santa Ana River Trail |
| SB | Senate Bill |

| | |
|---|---|
| sBOD | Soluble biochemical oxygen demand |
| SBT | sludge blending tank |
| SCADA | supervisory control and data acquisition |
| SCAG | Southern California Association of Governments |
| SCAP | Site Cleanup Subaccount Program |
| SCAQMD | South Coast Air Quality Management District |
| scfm | standard cubic feet per minute |
| sCOD | soluble chemical oxygen demand |
| SECAP | System Evaluation and Capacity Assurance Plan |
| SFR | single-family residential |
| SFY | square feet per year |
| SIU | Significant Industrial Users |
| SLCP | Short Lived Climate Pollutant |
| SLR | solids loading rate |
| SOC | Strengths, Opportunities, and Concerns |
| SOP | Standard Operating Procedures |
| SOR | surface overflow rate |
| South Star | South Star Engineering & Consulting, Inc. |
| $SO_X$ | Sulphur oxides |
| sq mi | square mile |
| SQR | Structural Quick Rating |
| SRF | State Revolving Fund |
| SRT | solids retention time |
| SS | stainless steel |
| SSC | Sustainable Communities Strategy |
| SSMP | Sewer System Management Plan |
| SSO | Sanitary Sewer Overflows |
| State Water Board | California State Water Resources Board |
| SWMM | Storm Water Management Model |
| SWRCB | State Water Resources Control Board |
| TDS | total dissolved solids |
| TIN | total inorganic nitrogen |
| TKN | total Kjeldahl nitrogen |
| TL | |
| TM | Technical Memorandum |
| TMDL | total maximum daily load |
| TN | total nitrogen |
| TOC | total organic carbon |
| TP | total phosphorus |

| | |
|---|---|
| TS | total solids |
| TSS | total suspended solids |
| TST | Test for Significant Toxicity |
| URS | URS Corporations |
| USACE | United States Army Corps of Engineers |
| USBR | U.S. Department of the Interior Bureau of Reclamation |
| USFWS | United States Fish and Wildlife Service |
| USGS | United States Geological Survey |
| UV/AOP | Ultraviolet/Advanced Oxidation Process |
| V&A | V&A Consulting Engineers |
| VCP | Vitrified Clay Pipe |
| VFA | volatile fatty acids |
| VFD | variable frequency drive |
| VSR | volatile solids reduction |
| VSS | volatile suspended solids |
| WaPUG | Wastewater Planning Users Group |
| WAS | waste activated sludge |
| WDR | Waste Discharge Requirements |
| WLAM | Waste Load Allocation Model |
| WMWD | Western Municipal Water District |
| WQMP | Water Quality Management Plan |
| WRCRWA | Western Riverside County Regional Wastewater Authority |
| wt | wet ton |
| WTPD | wet tons per day |
| WWTP | wastewater treatment plant |
| ZLD | Zero Liquid Discharge |

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 1

## Wastewater Treatment System

## Existing Facilities

Engineers...Working Wonders With Water®

City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 1: EXISTING FACILITIES

FINAL | June 2019

City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 1: EXISTING FACILITIES

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:08:11-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 1

# EXISTING FACILITIES

## 1.1 Purpose

The purpose of this Chapter is to summarize and describe the existing facilities of the liquid treatment portion of the City RWQCP. This Chapter presents design criteria of unit processes, but does not include an evaluation of the design and reliability criteria of the unit processes, which will be described in Volume 4, Chapter 3, Process Design and Reliability Criteria. Furthermore, this Chapter is limited only to the liquid treatment side and does not include solids handling facilities, which are described in Volume 5, Chapter 1, Existing Facilities.

## 1.2 Background

The RWQCP began serving the City in 1947. In 1978, the RWQCP transitioned into a regional facility by expanding its service district to include the Rubidoux, Jurupa, and Edgemont Community Service Districts and later, in 2015, Highgrove. As part of the 1978 expansion, primary, secondary, and tertiary treatment capacity was constructed to accommodate the increased plant flows. Subsequent projects have added capacity and upgraded existing primary, secondary, tertiary, solids handling, and storage facilities to increase the treatment capacity and meet water quality standards and goals at the RWQCP.

The RWQCP is near the end of an expansion/rehabilitation project that will increase the capacity from 40 to 46 mgd ADWF. This project will be completed by the end of the summer of 2017.

## 1.3 Existing Supporting and Treatment Facilities Background

Figure 1.1 shows the layout of the existing facilities. The RWQCP consists of approximately 121 AC of land, including an additional 25 AC on the east end acquired in 1990. The main entrance to the plant is on Acorn Street.

The major crossroads near the RWQCP are Van Buren Road to the west, Jurupa Avenue to the south, and Payton Road to the east. There is not much of a buffer zone around the plant except along the Santa Ana River on the north side. There are businesses to the immediate east and south sides of the plant.

Figure 1.2 shows a flow schematic for the plant based on current operation. The RWQCP incorporates two separate treatment trains (the ACT treatment train (Plant 2) and the MBR treatment train (Plant 1)). Sewage comes into the RWQCP at the headworks. From the headworks, flow is split to the two trains for independent treatment to an equivalent tertiary level.

Flow to the ACT treatment train (Plant 2) goes through four circular primary clarifiers, the diurnal flow equalization basin (through a scalping process), six rectangular aeration basins, and four circular secondary clarifiers. Secondary effluent is stored in two equalization basins prior to tertiary treatment. Flow from the EQ basins is either sent to Tertiary Treatment Train Filters 1 through 10 followed by CCB 1 and CCB 3, or to Filters 11 through 16 followed by CCB 3 directly. From CCB 3, combined ACT treatment train (Plant 2) flow is diverted to the Final Effluent Metering Flow Control Outflow structure.

For the MBR treatment train (Plant 1), flow from the headworks is sent to the primary influent splitter box and split between four circular primary clarifiers. At the MBR primary effluent diversion box, flow can either be scalped to the diurnal flow equalization basin, or sent to the MBR treatment train (Plant 1) fine screens. From the fine screens, flow is routed through the five rectangular aeration basins followed by eight submerged membrane filters. The filtrate from the MBR treatment train (Plant 1) is pumped directly to CCB 2. From CCB 2, it is pumped for use as recycled water or it blends with ACT treatment train (Plant 2) effluent at the Final Effluent Metering Flow Control Outflow structure.

At the Final Effluent Metering Flow Control Outflow structure, sodium bisulfite is added as discussed in Section 1.7 below followed by discharge to the Santa Ana River.

The following Sections describe the major treatment processes in further detail.

## 1.4 Preliminary Treatment

### 1.4.1 Influent Sewers

The RWQCP receives influent from five lines: the Arlanza trunk, the Riverside/Hillside trunk, the Acorn trunk, and the Jurupa and Rubidoux force mains. The Acorn and Arlanza trunk lines enter the plant separately, the merge at the Acorn/Arlanza Junction Vault upstream of the Headworks facility. Flow travels from the junction vault in a common pipeline to the Headworks facility. The Acorn/Arlanza combined flow is measured upstream of the Headworks facility within the common pipeline. The remaining three truck sewers are all metered separately, and all flow is sampled for $BOD_5$, suspended solids, $NH_3$-N, and other parameters. The City and each of the CSDs are responsible for the O&M of their own collection facilities. In 2014, URS completed a PDR for the City making recommendations for improvements to the influent metering structures. The improvements have not been made. Table 1.1 describes the meter types, trunk sizes, and ADWF as of the year 2016.

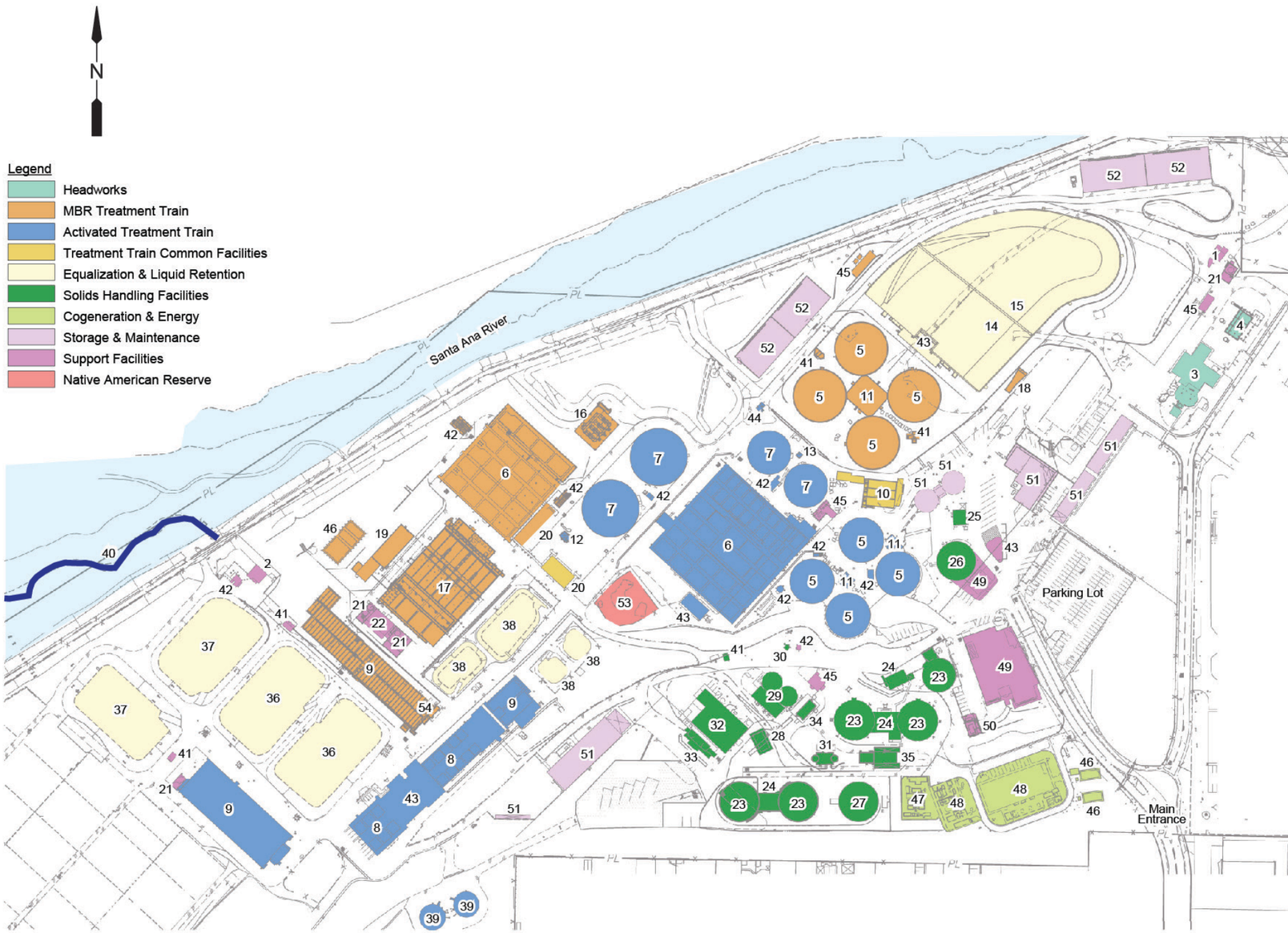
Table 1.1 Influent Sewers

| Influent Line | Current Meter Type | Trunk Size (in) | 2016 ADWF Flows (mgd) |
|---|---|---|---|
| Arlanza Acorn | Combined with Acorn Trunk; Flo-Dar Meter | 51<br>36 | 8.39 |
| Santa Ana (Riverside/Hillside) | Combined with Hillside Trunk; Flo-Dar Meter | 45<br>24 | 13.62 |
| Jurupa | Flo-Dar Meter | 18 | 3.41 |
| Rubidoux | MagMaster | 18 | 1.96 |

Notes:
(1) Source: 2014 PDR for the Influent Flow Metering Project by URS Corporation.

### 1.4.2 Headworks Facilities

Headworks facilities include screening and grit removal facilities for the protection of downstream equipment and processes. Both screenings and grit are washed, dewatered, and sent to a sanitary landfill. The current headworks facility was constructed in 1999. This facility combines the flow from the incoming sewers. The combined flow is passed through four parallel screens and two vortex grit basins. Once the water has been screened and degritted, it is divided between the ACT treatment train (Plant 2) and MBR treatment train (Plant 1) plants using the ACT treatment train (Plant 2) control valves. Table 1.2 describes the headworks treatment processes based on a design ADWF of 50 mgd and a peaking factor of 2.0.




| KEY | |
|---|---|
| 1. | Influent Flow Monitoring |
| 2. | Final Effluent Metering Flow Control Outflow |
| 3. | Headworks Facility |
| 4. | Headworks Odor Control Facility |
| 5. | Primary Clarifier |
| 6. | Aeration Basin |
| 7. | Secondary Clarifier |
| 8. | Tertiary Filter |
| 9. | Chlorine Contact Basin |
| 10. | Primary Clarifiers Odor Control Facility |
| 11. | Primary Sludge and Scum Pump Station |
| 12. | RAS/WAS Pump Station |
| 13. | RAS Pump Station |
| 14. | Diurnal Flow Equalization Basin |
| 15. | Wet Weather Equalization Basin |
| 16. | MBR Fine Screen Facility |
| 17. | MBR Filtration Units |
| 18. | MBR Train Influent Flow Meter |
| 19. | MBR Air Scour/Blower Building |
| 20. | Blower Building |
| 21. | Chemical Storage and Feed Area |
| 22. | Dechlorination Building |
| 23. | Digester |
| 24. | Digester Control Building |
| 25. | Digested Sludge Pump Station |
| 26. | Digested Sludge Storage Tank |
| 27. | Digester Gas Storage Facility |
| 28. | Digester Flares |
| 29. | Dissolved Air Flotation Thickeners |
| 30. | Solids Metering Facility |
| 31. | Bio-Solids Blending Facility |
| 32. | Bio-Solids Dewatering Facility |
| 33. | Bio-Solids Silos/Loading Facility |
| 34. | TWAS Disintegration Facility |
| 35. | FOG Receiving Station |
| 36. | Secondary Effluent Equalization Pond |
| 37. | Stormwater Retention Pond |
| 38. | Waste Backwash Lagoon |
| 39. | Backwash Storage Tank |
| 40. | RWQCP Plant Effluent Outfall |
| 41. | Flow Monitoring or Flow Control Structure |
| 42. | Splitter Box or Diversion Structure |
| 43. | Pump Station/Switchgear Room |
| 44. | Sampling Station |
| 45. | Electrical or MCC Structure |
| 46. | Power Generation or Standby Generators |
| 47. | Fuel Cell |
| 48. | Cogeneration |
| 49. | Administration Building |
| 50. | Cooling Tower |
| 51. | Maintenance, Storage, or Misc. |
| 52. | Material Storage Bed |
| 53. | Native American Rock Reserve Area |
| 54. | Recycled Water Pump Station |

Figure 1.1 2017 Existing Facilities Layout

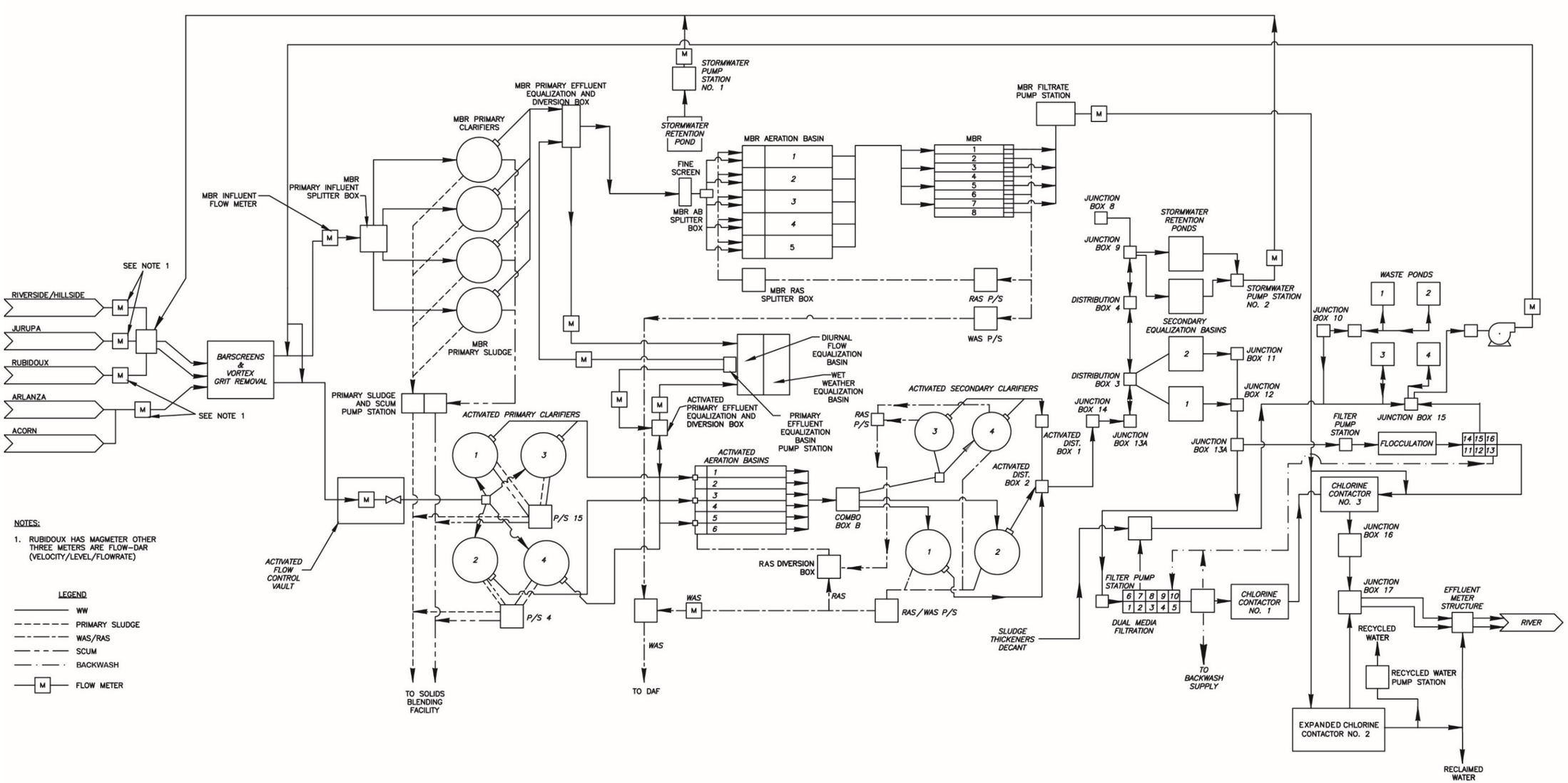



Figure 1.2 Process Flow Schematic

Table 1.2 Headworks Facilities

| Description | Value |
|---|---|
| **Influent Screenings Generation, cu. yds./day[(1)]** | |
| Average (wet screenings) | 15 |
| Average (washed/compacted) | 7.5 |
| Maximum (wet screenings) | 22.5 |
| Maximum (washed/compacted) | 11 |
| **Grit Production, cu. yds./day [(1)]** | |
| Average | 4 |
| Maximum | 8 |
| **Influent Bar Screens** | |
| Number | 4 duty |
| Width, ft. | 3.5 |
| Clear Openings Between Bars, in | 0.5 |
| Type | Climber |
| **Wet Screenings Conveyor** | |
| Number | 2 duty |
| Type | Shaftless Screw |
| **Screenings Washing Compactor Unit** | |
| Number | 2 duty |
| **Dry Screenings Conveyor[(2)]** | |
| Number | 2 duty |
| Type | Shaftless Screw |
| **Vortex Grit Removal Units** | |
| Number | 2 duty |
| Diameter, ft. | 20 |
| Capacity, each, mgd | 50[(3)] |
| Grit Removal Efficiency, percent | |
| >50 Mesh | 95 |
| >100 Mesh | 65 |
| **Grit Pumps** | |
| Number per tank | 1 |
| Capacity, gpm | 250 |
| Head, ft. | 45 |
| Type | Centrifugal Recessed Impeller |
| **Grit Classifiers (Teacup)** | |
| Number | 2 duty |
| Type | Hydraulic Vortex |
| Diameter, in | 42 |
| Capacity, gpm/each | 250 |
| **Grit Dewatering Equipment (Snail)** | |
| Number | 2 duty |
| Belt Width, in | 18 |

Notes:
(1) Based on a design ADF of 50 mgd and a peaking factor of 2.0.
(2) Compacted screenings and washed grit are conveyed together to the roll-away basin.
(3) Based on manufacturer's rated capacity.
(4) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.

In 2000, an in-ground, organic-media biofilter with an air header distribution system was built to treat foul air extracted from the headworks building. However, as part of the MBR treatment train (Plant 1) expansion in 2015/2016, that facility was completely demolished including existing fans, and a new headworks biofilter was built. The design criteria for this new headworks biofilter is shown in Table 1.3. The new biofilter is sized to handle 12 air changes per hour in the headworks building.

Table 1.3 Headworks Odor Control Facility Design Criteria

| Description | Value |
|---|---|
| Influent $H_2S$, PPM | 40 Peak; 20 Average |
| Airflow, scfm | 13,800 |
| EBRT, sec | 30 |
| Bed Depth, ft. | 6 |
| Area Required, sq. ft. | 1,150 |
| Discharge Odor Level, D/T | 400 |
| Fans | |
| Number | 1 Duty + 1 Standby |
| Capacity, scfm | 14,000 |
| Discharge Pressure, inches Water Column | 15 |
| Horsepower | 50 |

Notes:
(1) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.

## 1.5 Diurnal Flow Equalization and Wet Weather Equalization Basin

In order to optimize the ACT treatment train (Plant 2) and MBR treatment train (Plant 1) capacity, a diurnal flow equalization structure with a secondary compartment intended for wet weather equalization was constructed as part of the MBR treatment train (Plant 1) expansion. The diurnal flow equalization structure is intended to receive primary effluent flow from either or both the ACT treatment train (Plant 2) and MBR treatment train (Plant 1). The design intent of the diurnal flow equalization basin is to normalize flow through the two trains by scalping excess flows independently from each train by use of flowmeters and control valves. During the day, primary effluent is stored in the diurnal flow equalization basin, then pumped back into the treatment trains at night during lower flows. This process stabilizes flow to the downstream processes and optimizes capacity.

Furthermore, the basin was designed to allow the operational flexibility to overflow primary effluent into the secondary compartment. The diurnal flow equalization basin pump station can pump primary effluent stored in the equalization basin to either of the two train diversion

structures for further treatment in either the ACT treatment train (Plant 2) or MBR treatment train (Plant 1). Table 1.4 describes the equalization basins and the pump station.

Table 1.4 Diurnal Flow Equalization and Wet Weather Equalization Basins

| Description | Value |
|---|---|
| **MBR Treatment Train (Plant 1) Excess Flowmeter to Diurnal Equalization Basin** | |
| Type | Parshall flume |
| Size, in | 30 |
| Minimum flow, mgd | 0.52 |
| Maximum flow, mgd | 26.9 |
| **ACT Treatment Train (Plant 2) Excess Flowmeter to Diurnal Equalization Basin** | |
| Type | Parshall flume |
| Size, in | 18 |
| Minimum flow, mgd | 0.33 |
| Maximum flow, mgd | 15.8 |
| **Diurnal Flow Equalization Basin** | |
| Invert Elevation, ft. | 677.9 Low, 679.0 High |
| Top of berm elevation, ft. | 702 |
| Water elevation, ft. | 694 |
| Overflow elevation to wet weather equalization basin, ft. | 693.5 |
| Freeboard, ft. | 8 |
| Approximate Volume, MG | 5 |
| Depth from top water surface, ft. | 16 |
| Side Slope (2 walls), H:V(1) | 2:1 |
| **Wet Weather Flow Equalization Basin** | |
| Invert Elevation, ft. | 681 |
| Top of berm elevation, ft. | 702 |
| Water elevation, ft. | 694 |
| Overflow elevation from Diurnal Equalization basin, ft. | 693.5 |
| Freeboard, ft. | 8 |
| Approximate Volume, MG | 5 |
| Depth from top water surface, ft. | 12 |
| Side Slope (3 walls), H:V(1) | 2:1 |
| **Return Flow to Secondary Process** | |
| Time to empty Equalization basin, hrs | 6 |
| Pump Type | Submersible (wet well) |
| Number | 5 duty + 1 swing standby |
| Flow, each, gpm | 6,400 |
| Total Dynamic Head, ft. | 36 |
| Horsepower | 75 |
| Drive | Variable Speed |

Notes
(1) Remaining walls are vertical surfaces.
(2) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.

## 1.6 MBR Treatment Train (Plant 1)

### 1.6.1 Primary Clarifiers

From the headworks effluent splitter box, flow to the MBR treatment train (Plant 1) is routed to the MBR treatment train (Plant 1) influent flowmeter vault and MBR treatment train (Plant 1)

primary influent splitter box before being distributed to the four new MBR treatment train (Plant 1) primary clarifiers. Depending upon flow rate, flow from the primary clarifiers enters a diversion structure that allows flow to be diverted to the fine screens and/or the diurnal flow equalization basin. Sludge and scum removed from the MBR treatment train (Plant 1) primary clarifiers is pumped from the primary sludge and scum pump station to the solids blending facility. Table 1.5 describes the MBR treatment train (Plant 1) primary processes.

Table 1.5 MBR Treatment Train (Plant 1) Primary Clarifiers

| Description | Value |
|---|---|
| **Primary Clarifiers** | |
| Number | 4 |
| Diameter, ft. | 120 |
| Volume, total MG | 4.1 |
| Surface Area, total sq. ft. | 145,240 |
| Overflow Rate, gal/day/ sq. ft. | 707 |
| Detention Time, hrs | 3 |
| Side Water Depth, ft. | 18 |
| **Sludge Pumps** | |
| Number | 4 duty + 2 standby |
| Type | Progressive Cavity |
| Capacity, gpm/each | 365 |
| Total Dynamic Head, psi | 158 Max |
| Horsepower | 75 |
| Drive | Variable Speed |
| **Sludge In-Line Grinders** | |
| Number | 2 duty + 2 standby |
| Size, in | 12 |
| Max Flow, gpm | 1,200 |
| Pressure drop, psi | 1.59 |
| Horsepower | 5 |
| Drive | Constant Speed |
| **Scum Pumps** | |
| Number | 2 duty + 2 standby |
| Design Flow, gpm | 165-230 |
| Total Dynamic Head, ft. | 83-74 |
| Horsepower | 15 |
| Drive | Constant Speed |
| **Scum In-Line Grinder** | |
| Number | 1 duty + 0 standby |
| Size, in | 12 |
| Max Flow, gpm | 1,200 |
| Pressure Drop, PSI | 1.59 |
| Horsepower | 5 |
| Drive | Constant Speed |

Notes:
(1) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.

As part of MBR treatment train (Plant 1) expansion, a new odor control system was built to treat foul air from new and existing primary clarifiers. Design criteria for this new primary clarifier biofilter are shown in Table 1.6.

Table 1.6 Primary Clarifier Odor Control Facility Design Criteria

| Description | Value |
|---|---|
| Influent $H_2S$, ppm | 40 Peak; 20 Average |
| Airflow, scfm | 44,000 |
| EBRT, sec | 30 |
| Bed Depth, ft. | 6 |
| Area Required, sq. ft. | 3,667 |
| Discharge Odor Level, D/T | 400 |
| Fans | |
| Number | 1 Duty + 1 Standby |
| Capacity, scfm | 44,000 |
| Discharge Pressure, inches Water Column | 18 |
| Horsepower | 175 |

Notes:
(1) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.

### 1.6.2 MBR Process

The MBR process utilizes suspended growth biological treatment in an activated sludge process followed by membrane filtration to achieve solids-liquid separation and a tertiary effluent quality product. The MBR is similar to conventional treatment processes (like the activated sludge treatment) except that membranes are used in place of secondary clarifiers and tertiary filters. Because this process incorporates a physical membrane barrier, it produces a very consistent and low turbidity effluent regardless of quality changes in the feed water, as long as membrane integrity is maintained.

#### 1.6.2.1 Fine Screens

Abrasive solids can wear through the membrane filters and cause failures while hair and other stringy materials wrap around the membrane fibers and cause clumping of the mixed liquor. Fine screens are installed at the head of the MBR treatment train (Plant 1) to protect downstream equipment. Table 1.7 describes the fine screen process units.

Table 1.7 MBR Treatment Train (Plant 1) Fine Screens

| Description | Value |
|---|---|
| Fine Screens | |
| Number | 3 duty + 1 standby |
| Type | In-channel Rotary Drum Screen |
| Channel Width | 8 ft, 6 in |
| Screen Perforation, in (mm) | 0.08(2) |
| Nominal screening basket diameter, in | 102 |
| Nominal screenings auger diameter, in | 20 |
| Average flow through each screen, mgd | 10.7 |
| Max flow through each screen, mgd | 16 |
| Horsepower | 5 |
| Washwater flowrate, gpm | 65 |
| Minimum washwater supply pressure, psi | 60 |

Notes:
(1) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.

#### 1.6.2.2 Aeration Basins

Part of the MBR treatment train (Plant 1) process includes aeration basins similar to conventional activated sludge basins but with a higher MLSS concentration in which suspended growth biological treatment occurs. For this reason, the aeration basins previously part of Plant 1 were rehabilitated and converted to MBR treatment train (Plant 1) aeration basins and a fifth aeration basin was added. Table 1.8 describes some of the aeration basin processes in more detail.

Table 1.8 MBR Treatment Train (Plant 1) Aeration Basin

| Description | Value |
|---|---|
| **Aeration Basins** | |
| Number | 5 |
| Volume in Service, MG | 5.04 |
| Anoxic Faction, percent | 25 |
| Diffused Aeration | |
| Air flow, scfm | 5,000 |
| Diffusers | |
| Type | Fine bubble |
| Number | 395 |
| **Blowers in Building**(1) | |
| Number | 2 duty + 1 common standby |
| Types | Single-Stage Centrifugal, Vertical Split |
| Flow, scfm each | 9,000 |
| Discharge Pressure, psig | 8.8 |
| Inlet Temp, degrees F | 110 |
| **Blowers in Building 2** | |
| Number | 1 duty |
| Types | Turblex Single Stage |
| Flow, scfm each | 9,000 (minimum) |
| Discharge Pressure, psig | 23 (minimum) |
| Inlet Temp, degrees F | 110 |

Notes:
(1) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.

#### 1.6.2.3 Submerged Membrane Filtration

Once out of the MBR treatment train (Plant 1) aeration basins, flow is routed to the MBR units (reconfigured Plant 1 rectangular secondary clarifiers). Each unit has a second stage anoxic zone, membrane basins (second stage oxic zones), and deoxygenation zones. Both the anoxic zones and deoxygenation zones use submersible propeller mixers.

Table 1.9 provides details regarding the submerged membrane filtration units.

Table 1.9 MBR Treatment Train (Plant 1) Submerged Membrane Filtration

| Description | Value |
|---|---|
| **Reconfigured Final Clarifier/MBR Treatment Train (Plant 1) Filtration Units** | |
| Second Stage Anoxic Zones | |
| Number | 8 |
| Volume, each, cu. ft. | 15,460 |
| Membrane Basins | |
| Number | 8 |
| Volume, each, cu. ft. | 18,644 |
| Deoxygenation Zones | |
| Number | 4 |
| Volume, each, cu. ft. | 18,813 |
| Deoxygenation Mixers | 2 each zone - submersible propeller |
| Anoxic Mixers | 2 each zone - submersible propeller |
| **Membrane System** | |
| Basins | 8 |
| Cassettes per basin, installed/spare | 14/6 |
| Surface area per basin, sq. ft. | 248,625 |
| Surface area, total, sq. ft. | 1,989,000 |
| Membrane Flux | |
| Average Day Flow, all basins online | 13.1 |
| Average Day Flow, 7 online/1 standby | 14.9 |
| Max Month Flow, all basins online | 15.4 |
| Max Month Flow, 7 online/1 standby | 17.6 |
| Max Day Flow, all basins online | 19.6 |
| Max Day Flow, 7 online/1 standby | 22.4 |
| **Filtrate Pumps** | |
| Type | End suction centrifugal |
| Capacity, each, gpm | 4,167 |
| Motor Horsepower | 75 |
| **Backwash Pumps** | |
| Type | Horizontal Centrifugal Split Case |
| Number | 2+1 standby |
| Capacity, each, gpm | 2,600 |
| Motor Horsepower | 30 |
| **Air Compressors** | |
| Type | Rotary Screw |
| Number | 1+1 standby |
| Capacity, each, scfm | 100 |

Table 1.9 MBR Treatment Train (Plant 1) Submerged Membrane Filtration (continued)

| Description | Value |
|---|---|
| **Membrane Scour Aeration Blowers** | |
| Number | 5+1 standby |
| Type | High speed turbo blower |
| Maximum air delivery, scfm | 36,190 |
| Rated capacity, each, scfm | 8,200 |
| Rated discharge pressure, psi | 5.25 |
| Motor horsepower | 300 |
| **RAS Pumps** | |
| Type | Vertical axial flow |
| Number | 2+1 standby |
| Capacity, gpm each | 18,000 |
| Motor horsepower | 75 |
| **WAS Pumps** | |
| Type | Non-clog centrifugal |
| Number | 1+2 standby |
| Capacity, gpm each | 440 |
| Motor horsepower | 10 |

Notes:
(1) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.

### 1.6.3 Disinfection

The existing facilities prior to the MBR treatment train (Plant 1) expansion project had CCB 2 out of service with all plant flows treated at CCBs 1 and 3. During MBR treatment train (Plant 1) expansion, CCB 2 was expanded by 9,500 sq. ft. to serve the MBR treatment train (Plant 1) creating a dedicated Title 22 reuse facility. In 2017, the City was required to meet a CT value of 900 mg-min/L under an interim directive by regulatory agencies, although Title 22 disinfection requires a 90-minute model contact time at peak flow and a 450 mg-min/L CT value (product of contact time and residual chlorine concentration). In November 2018, Carollo worked with the City to prepare and submit an updated Title 22 report in order to modify the City's disinfection requirements. The results of that effort are discussed in Volume 4, Chapter 9, Disinfection, of this update to the Master Plan. Chemical storage and feed systems were relocated during MBR treatment train (Plant 1) expansion and is discussed in Section 1.7. Table 1.10 describes the MBR (Plant 1) disinfection system in 2017. Volume 4, Chapter 9 provides detailed updates to this Section as it pertains to both the MBR treatment train (Plant 1) and ACT treatment train (Plant 2).

Table 1.10 MBR Treatment Train (Plant 1) Disinfection

| Description | Value |
|---|---|
| **CCB 2** | |
| Total Volume, gals | 3,240,000 |
| Length to Width Ratio, L:W | Varies |
| Surface area, sq. ft. | 169,014 |

Table 1.10 MBR Treatment Train (Plant 1) Disinfection (continued)

| Description | Value |
|---|---|
| Depth, ft. | 19.17 |
| Contact time at peak flow, min | 90 |
| CT Value, mg-min/L | 450 |

Notes:
(1) CT value revised based on conversations with the City staff regarding current regulatory requirements.
(2) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.

## 1.7 ACT Treatment Train (Plant 2)

### 1.7.1 Primary Clarifiers

The purpose of the primary clarifiers is to remove settleable solids from the wastewater. Primary clarifiers typically remove about 70 percent of the incoming TSS and about half of the BOD. The primary effluent from the ACT treatment train (Plant 2) primaries flows by gravity to either the diurnal flow equalization basin by scalping or to the ACT treatment train (Plant 2) aeration basins. The diurnal flow equalization basin is described in further detail in Section 1.5. Equalized flow is returned and split between the ACT treatment train (Plant 2) aeration basins. The ACT treatment train (Plant 2) primary clarifier solids are thickened in the primaries and pumped directly to the anaerobic digesters. Table 1.11 describes the ACT treatment train (Plant 2) primary processes.

Table 1.11 Activated Primary Clarifiers

| Description | Value |
|---|---|
| **Primary Clarifiers** | |
| Number | 4 |
| Diameter, ft. | 95 |
| Overflow Rate, gal/day/sq. ft. | 705 |
| Detention Time, hrs | 2.54 |
| Side Water Depth, ft. | 9 |
| **Sludge Pumps** | |
| Number | 6 |
| Type | Progressive Cavity |
| Size, gpm/each | 100 |

Notes:
(1) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.
(2) Source: 2014 Capital Improvement Program and Rate Development Study.

### 1.7.2 Aeration Basins

Aeration basins provide biological treatment. In these units, the wastewater is actively mixed with a large concentration of microorganisms and oxygen (from air) that break down the soluble and insoluble organic matter and convert it to $CO_2$ and more biomass. In addition, $NH_3$-N is converted to nitrogen gas in a two-step process. The ACT treatment train (Plant 2) aeration basins include an anoxic zone at the front of the basin. The anoxic zones in these basins occupy 25 percent of the total volume of the aeration basins. The rest of the aeration basin compartments are aerobic. Two motor driven, single-stage centrifugal, vertical split type blowers are used to supply variable volumes of air to the aeration basins. The ACT treatment train (Plant 2) and MBR treatment train

(Plant 1) share a common standby blower. Table 1.12 describes some of the aeration basin processes in more detail.

Table 1.12 Activated Aeration Basin

| Description | Value |
|---|---|
| **Aeration Basins** | |
| Number | 6 |
| Volume in Service, MG | 7.85 |
| Anoxic Faction, percent | 25 |
| Diffused Aeration | |
| Air flow, scfm | 5,000 |
| Diffusers | |
| Type | Fine Bubble |
| Number, each | 694 |
| **Blowers** | |
| Number | 2 duty + 1 common standby |
| Types | Single-Stage Centrifugal, Vertical Split |
| Flow, scfm each | 12,500 |
| Discharge Pressure, psig | 9.2 |
| Inlet Temp, degrees F | 110 |

Notes:
(1) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.
(2) Source: 2014 Capital Improvement Program and Rate Development Study.

### 1.7.3 Secondary Clarifiers

Secondary clarifiers settle out the MLSS from the aeration basins. The majority of the solids removed from the wastewater stream, referred to as RAS, are returned to the aeration basins to maintain the MLSS concentration in the aeration basin. The remaining solids, known as WAS, are thickened in the DAFT before being sent to anaerobic digestion. Table 1.13 describes the secondary clarifiers.

Table 1.13 Activated Secondary Clarifiers

| Description | Value |
|---|---|
| **Units 1 and 2** | |
| Diameter, ft. | 130 |
| Side Water Depth, ft. | 14 |
| Overflow Rate, gal/day/sq. ft. | 494 |
| Detention Time, hrs | 5.0 |
| **Units 3 and 4** | |
| Diameter, ft. | 100 |
| Side Water Depth, ft. | 10.3 |
| Overflow Rate, gal/day/sq. ft. | 494 |
| Detention Time, hrs | 3.7 |
| Surface Area, total sq. ft. | 42,250 |

Table 1.13 Activated Secondary Clarifiers (continued)

| Description | Value |
|---|---|
| **RAS Pumping** | |
| Number | 4 duty + 2 standby |
| Capacity, gpm each | 3 at 3,000; 3 at 5,700 |
| Type | Vertical Centrifugal |
| **WAS Pumping**(1) | |
| Number | 1 duty + 1 standby |
| Capacity, gpm each | 2 at 480 |
| Type | Vertical Centrifugal |

Notes:
(1) There are also two WAS pumps rated at 315 gpm that are no longer in service.
(2) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.
(3) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.
(4) Source: 2014 Capital Improvement Program and Rate Development Study.

### 1.7.4 Secondary Flow Equalization and Storm Water Retention

Secondary flow equalization precedes the final filtration and disinfection steps. The secondary equalization basins dampen daily variations in flow and provide the filters with a stable influent flow. The flow equalization basins are filled by gravity from the secondary clarifiers. As part of the MBR treatment train (Plant 1) plant expansion, two of the flow equalization basins were converted to storm water retention ponds due to the fact that upstream primary effluent equalization was provided. During high flow storm events, secondary effluent can be diverted to the storm water retention ponds and later be pumped back to the headworks for treatment. Table 1.14 describes these ponds.

Table 1.14 Activated Secondary Flow Equalization and Stormwater Retention Ponds

| Description | Value |
|---|---|
| **Secondary Effluent Flow Equalization Ponds** | |
| Number | 2 |
| Volume, each, MG | 1.5 |
| **Stormwater Retention Ponds** | |
| Number | 2 |
| Volume, each, MG | 1.5 |

Notes:
(1) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.
(2) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.
(3) Source: 2014 Capital Improvement Program and Rate Development Study.

### 1.7.5 Tertiary Treatment

Filtration removes most of the suspended solids present in the effluent from the secondary clarifiers. This reduces the chlorine demand of the water, improves the disinfection process, and results in production of a Title 22 tertiary effluent. The current installation includes 16 dual media filters operating in two separate trains. Both trains consist of a 24-in layer of anthracite above a 15-in layer of silica sand. Filters 1 through 10 feed directly to CCB 1 which in turn feeds to CCB 3. Prior to the MBR treatment train (Plant 1) expansion, CCB 1 could feed to either CCB 2 or CCB 3. Typical operation in 2017 had CCB 1 feeding into CCB 3 to achieve the necessary Title 22 contact

time. Filters 11 through 16 are preceded by a chemical flocculation step through the injection of Alumer™ that aggregates very small particles so that they can be efficiently removed in the filters. Currently, the flocculation basins are out of service. However, Alumer™ is dosed continuously to meet Title 22 standards. Filters 11 through 16 discharge to CCB 3 directly. Table 1.15 describes these tertiary filters.

Table 1.15 Activated Tertiary Treatment

| Description | Value |
|---|---|
| **Filter Influent Pumps** | |
| Filters 1 - 10 | |
| Number | 2 duty + 1 standby |
| Capacity, each gpm | 13,050 |
| Horsepower | 250 |
| Filters 11 - 16 | |
| Number | 2 duty + 1 standby |
| Capacity, each gpm | 8,000 |
| Horsepower | 125 |
| **Tertiary Filtration** | |
| Number of Filters | 16 |
| Anthracite Depth, in | 24 |
| Silica Sand Depth, in | 15 |
| Filters 1 through 10 | |
| Surface Area, each, sq. ft. | 552 |
| Average Loading Rate, gpm/ sq. ft. | 3.33 |
| Maximum Loading Rate, gpm/ sq. ft. | 5.00 |
| Capacity, gpm/filter (Average/Max.) | 1,838/2,760 |
| Filters 11 through 16 | |
| Surface Area, each, sq. ft. | 650 |
| Average Loading Rate, gpm/ sq. ft. | 3.33 |
| Maximum Loading Rate, gpm/ sq. ft. | 5.00 |
| Capacity, gpm/filter (Average/Max.) | 2,165/3,250 |
| Flocculation Basins[1] | |
| Number | 10 |
| Number of Stages, each | 2 |
| Volume per basin, gal | 13,740 |
| **Main Backwash Pumps (Filters 1-10/Filters 11-16)** | |
| Number | 2 duty + 1 standby |
| Capacity, gpm | 14,100 |
| Horsepower | 150 |
| Max. filter backwash rate, ft. rise/min | 3.41/3.48 |
| Max. filter backwash rate, gpm/sq. ft. (unit) | 25.5/26.0 |
| Max. filter backwash rate, gpm/filter | 14,082/16,900 |
| Capacity, gpm | 1,650 |

Table 1.15 Activated Tertiary Treatment (continued)

| Description | Value |
|---|---|
| **Subsurface Wash Pumps (Filters 1-10/Filters 11-16)** | |
| Number | 1 duty + 1 standby |
| Horsepower | 150 |
| Max. filter wash rate, ft. rise/min | 0.4/0.3 |
| Max. filter wash rate, gpm/sq. ft. (unit) | 3.0/2.54 |
| Max. filter wash rate, gpm/filter | 1,650 |
| **Waste Backwash Pumps** | |
| Number | 1 duty + 1 standby |
| Capacity, gpm | 5,220 |
| Horsepower | 100 |
| **Backwash Storage Tanks** | |
| Number | 2 |
| Volume, each, MG | 0.66 |
| Average Alumer Dosage, mg/L | 0.2 |

Notes:
(1) Currently, the flocculation basins are not fully utilized. Instead Alumer™ is continuously dosed to meet Title 22 requirements.
(2) There are also two WAS pumps rated at 315 gpm that are no longer in service.
(3) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.
(4) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.
(5) Source: 2014 Capital Improvement Program and Rate Development Study.
(6) Source: 1981 Record Drawings for the Tertiary Filtration Plant (T-13) Project.
(7) Source: 1990 Tertiary Expansion Project (T-26).

### 1.7.6 Disinfection

Disinfection of the tertiary effluent is accomplished by adding sodium hypochlorite and providing adequate contact time to meet discharge permit requirements. Dechlorination, removal of chlorine residual through the addition of sodium bisulfite is required by the permit to protect aquatic life after discharge to the Santa Ana River. This is done at the end of the CCBs before the water is discharged.

Prior to the Title 22 report, flow for the ACT treatment train (Plant 2) side flowed from the tertiary filters into CCB 1 then into CCB 3 for disinfection treatment. From CCB 3, ACT treatment train (Plant 2) effluent was blended with MBR treatment train (Plant 1) effluent by way of Junction Box 16 and 17. However, with the completion of the Title 22 report, significant changes were made to the disinfection processes at RWQCP. These changes are discussed in detail in Volume 4, Chapter 9 of this update to the Master Plan.

Table 1.16 describes the ACT treatment train (Plant 2) disinfection system as it existed in 2017. Similar to MBR disinfection, the City was required to meet a CT value of 900 mg-min/L under an interim directive by regulatory agencies.

Table 1.16 ACT Treatment Train (Plant 2) Disinfection

| Description | Value |
|---|---|
| **CCBs - ACT Treatment Train (Plant 2) Side** | |
| Number of Units | 2 |
| CCB 1 Volume, gals | 448,320 |
| CCB 1 Length to Width Ratio, L:W | 18.5:1 |
| CCB 1 Surface Area, sq. ft. | 3,534 |
| CCB 1 Depth, ft. | 11 |
| CCB 3 Volume, gals | 2,900,000 |
| CCB 3 Length to Width Ratio, L:W | 48:1 |
| CCB 3 Surface Area, sq. ft. | 20,475 |
| CCB 3 Depth, ft. | 18 |
| Total contact time at peak flow, min | 120 |
| CT Value, mg/L-min(1) | 900 |

Notes:
(1) CT value revised based on conversations with the City of Riverside staff regarding current regulatory requirements.
(2) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.
(3) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.
(4) Source: 2014 Capital Improvement Program and Rate Development Study.

## 1.8 Disinfection Chemical Storage and Feed Facility

As part of the MBR treatment train (Plant 1) expansion, the chemical storage and feed systems were relocated and enhanced. The gravity feed sodium hypochlorite system was replaced with a pump system. The new system is near the abandoned dechlorination building between CCB 2 and the MBR treatment train (Plant 1) filter units. Chemical metering pumps were located within the dechlorination building. In addition to the disinfection requirements for the ACT treatment train (Plant 2) and MBR treatment train (Plant 1), sodium hypochlorite is necessary for the clean-in-place procedure for the submerged membrane filters in the MBR treatment train (Plant 1).

Similarly, the two existing sodium bisulfite storage tanks were relocated to north of the abandoned sulfur dioxide storage building with pumps located in the sulfur dioxide storage room. Additionally, the injection point for sodium bisulfite was located at the effluent of CCB 3. With the addition of the MBR treatment train (Plant 1), sodium bisulfite injection has been relocated to Junction Box 17 just upstream of the Final Effluent Metering Flow Control Outflow. Furthermore, a chlorine residual sampling location was added to the Effluent Meter Structure to provide input for controlling sodium bisulfite dosage. Table 1.17 describes the chemical storage and feed facility details.

Table 1.17 Disinfection Chemical Storage and Feed Facilities

| Description | Value |
|---|---|
| **Sodium Hypochlorite** | |
| Storage Tank | |
| Number | 3 (2 Existing, 1 New) |
| Delivery | 12.5 Percent, Liquid |
| Storage, Days | 29 |
| Capacity, gal | 20,000 |
| Material | FRP |
| Pumps | |
| Type | Diaphragm |
| Number for CCB 2 – MBR treatment train (Plant 1) | 1 duty + 1 standby |
| Number for CCB 1 – ACT treatment train (Plant 2) | 1 duty + 1 shared standby with CCB 3 |
| Number for CCB 3 – ACT treatment train (Plant 2) | 1 duty + 1 shared standby with CCB 1 |
| MBR treatment train (Plant 1) Clean-in-Place Pump | |
| Type | Peristaltic |
| Number | 1 duty + 1 standby |
| Capacity, gph | 240-300 |
| MBR treatment train (Plant 1) Clean-in-Place Pump | |
| Type | Centrifugal |
| Number | 1 duty + 1 shelf spare |
| Capacity, gpm | 50 |
| Containment | |
| Capacity, gals | 24,300 |
| Pipe | |
| Material | CPVC, double containment |
| Diameter, In | Various |
| **Sodium Bisulfite (Dechlorination)** | |
| Storage Tank | |
| Number | 2 |
| Delivery | 38 percent Liquid |
| Storage, days | 33 |
| Capacity, gal | 12,000 |
| Material | FRP |
| Pumps | |
| Number | 3 duty + 1 standby |
| Capacity, gph | 10-160 |
| Containment | Double Wall Tanks |
| Pipe | |
| Material | CPVC, double containment |
| Diameter, in | Various |

Notes:
(1) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.
(2) Source: 2012 Conformed Drawings for the Riverside RWQCP Phase I Plant Expansion.
(3) Source: 2014 Capital Improvement Program and Rate Development Study.

## 1.9 Recycled Water Pump Station

The recycled water pump station is located in the south-east corner of CCB 2. At present, three 2,400 gpm pumps are installed in a 2 duty + 1 standby configuration. This pump station is intended to provide Title 22 disinfected tertiary recycled water both on-site throughout the RWQCP and off-site to various customers. As the market for recycled water continues to expand, the pump station is designed with pump pads available for total capacity of up to 15,500 gpm (assuming the same size pumps are installed). Table 1.18 describes the recycled water pump station facility details.

Table 1.18 Recycled Water Pump Station Facilities

| Description | Value |
|---|---|
| **Installed** | |
| Type | Vertical Turbine |
| Number | 3 (2 duty + 1 standby) |
| Size, each gpm | 2,400 |
| Horsepower | 250 |
| **Future Installations** | |
| Available Equipment Pads | 7 |

Notes:
(1) Source: 2008 Bid Set Drawings and Technical Specifications for Riverside Quality Control Plant Recycled Water Pump Station at CCB 2.

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 2

## Wastewater Treatment System

## Summary of Planning Studies

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Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 2: SUMMARY OF PLANNING STUDIES

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Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 2: SUMMARY OF PLANNING STUDIES

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Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:11:21-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
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Chapter 2

# SUMMARY OF PLANNING STUDIES

## 2.1 Purpose

The purpose of this chapter is to summarize the past planning studies that have been done for the liquid treatment facilities since the 2008 Master Plan was completed for the City's RWQCP.

## 2.2 Planning Studies Reviewed

The three planning studies that were reviewed for this chapter are:

1. 2014 CIP and Rate Development Study (2014 Update).
2. 2015 Salinity Study.
3. 2014 Influent Metering Flow Project.
4. 2017 Biosolids Handing Assessment Study.

Brief summaries of each of these studies are described in the following sections. Summaries of the major findings and recommendations for each study are also included in this chapter. For completeness, a brief description of the MBR Plant Expansion is also included in this chapter. Furthermore, a Title 22 Engineering Report will be completed for the revised CCB 2. However, no additional information is available regarding this study at the time of this update to the Master Plan.

## 2.3 2014 Capital Improvement Program and Rate Development Study

The 2014 Update served as an interim update to the 2008 Master Plan and specifically focused on updates to the CIP and rate structures.

### 2.3.1 Purpose

The purpose of the 2014 update was to provide a revised CIP for the collection system and RWQCP improvements and to develop a financial plan and rate structure to carry the City through the 2018/19 fiscal year. This study reviewed future conditions, collection system improvements, RWQCP improvements, the CIP and implementation schedule, and the financial plan and rate structure. All of these topics culminated in a recommendations section discussed below.

### 2.3.2 Summary of Findings

1. The 2014 updated flow projections were based on a value of 77 gpcd, which was reflective of both water conservation and economic conditions at the time. CSD's flow projections were based on the 2008 IWWMP projected rates of increase applied to the 2012/13 fiscal year measured flows. Furthermore, the CSD flows to the RWQCP were capped at the contractual capacities listed below:
   a. Edgemont CSD - 0.89 mgd.
   b. Jurupa CSD - 4.0 mgd until 2030 when it increases to 5.0 mgd.
   c. Rubidoux CSD - 3.055 mgd.
   d. Highgrove Community - 4.4 mgd.

2. Concentrations of wastewater constituents that impact treatment capacity and performance increased 50 percent between 1997 and 2014.
3. At the time of the 2014 Update, the Phase 1 Plant Expansion was just getting underway and thus all work associated with that expansion was excluded from the 2014 CIP Update. Other onsite facilities that are independent of the MBR Plant Expansion that were identified for improvements or rehabilitation include:
   a. Additional sludge dewatering equipment replacement.
   b. Upgrade the cogeneration system to meet changing regulations.
   c. Rehabilitate tertiary filter piping and systems, and determine long-term rehabilitation needs for that process.
   d. Begin the Phase 2 Expansion to address the predicted load limitations resulting from increasing influent loading concentrations.
   e. Rehabilitate the Plant 2 Activated Sludge process.
   f. Replace influent metering systems.
   g. Replace two major electrical switchgear installations.
   h. Improve site security with added lighting and perimeter fencing.
   i. Improve flood protection to comply with regulatory requirements.
4. In the initial 5 years of the 2014 update, $122 million of specific and necessary capital projects were identified.
5. A capital funding strategy was developed to provide funds sufficient to implement the 2014 CIP Update, which projected an escalated $222 million in future treatment and collections system improvements over the 5-year study period. Additionally, cash funded capital expenditures of $17.8 (escalated) million were projected in the O&M budget.
6. The primary source of funding for the CIP was intended to be bond proceeds with a small amount of funding from and cash on hand.
7. In total, the 2014 CIP Update included $795 million (2014 dollars) in treatment plant and collections systems projects between FY 2014/15 and 2035/2036. Additional bond issuances are required to fund CIPs beyond the 2018/19 fiscal years.
8. The required annual rate increase determined by the 2014 update was 8.5 percent per year for a 5-year period to adequately fund the activities of the City's Wastewater Division.

## 2.4 2015 Salinity Study

The 2015 Salinity Study provided guidance to the City with planning of short- and long-term mitigation alternatives to help meet NPDES permit requirements for RWQCP effluent TDS.

### 2.4.1 Purpose

The purpose of this study was to determine the most feasible and cost effective method for the City to achieve future reliable NPDES effluent permit compliance for TDS. This study reviewed groundwater supply, recharge projects, and discharger TDS contributions to the sewer collection system. Mitigation alternatives and recommendations were developed as discussed below.

### 2.4.2 Summary of Findings

1. The planning period for the Salinity Study was through the year 2025. However, the analyses in the study were performed in 2012 and were not updated for the 2015 Salinity Study. Such analyses included the development of a PWS model by Geoscience Support Services, Inc. to simulate the impact of future water supply projects on potable water TDS concentrations.

2. The most significant source of TDS in RWQCP effluent is the source water itself.
3. The initial groundwater supply analysis that was done for the report was completed in 2012. This analysis was based on a model developed to predict TDS levels in the groundwater that serves the majority of the potable water customers in the City. Many of the projects that the analysis was based on were in a state of flux, until the City decided to finalize the Salinity Study in 2015. To accommodate project uncertainties and the resulting TDS projections, it was decided, based on analyses by the City, that the future source water TDS would increase at a constant rate of 2 mg/L per year.
4. Based on the projected TDS increase of 2 mg/L per year, RWQCP effluent and water supply TDS concentrations were estimated. Using that assumption, the RWQCP effluent TDS limit was predicted to be exceeded by 2028.
5. Five TDS mitigation alternatives were developed and evaluated, each based on a TDS removal goal of 75 mg/L. The final recommended alternative was to apply RO technology to a portion of the RWQCP MBR treatment train (Plant 1) effluent in order to meet the effluent TDS limit. Implementation of this project could likely be delayed until 2021.
6. Other relevant findings related to TDS concentrations of the source water that is part of the influent flow to the RWQCP are shown below:
    a. Rubidoux CSD's potable water TDS concentration was expected to increase by 90 mg/L over the planning period.
    b. Jurupa CSD's potable water TDS concentration was expected to stay approximately the same over the planning period due to their desalter.
    c. Other outside sources of water to the RWQCP were not expected to show an increase in TDS concentration over the planning period.

## 2.5 2014 Influent Metering Flow Project

The PDR for this project was completed by URS Corporation in 2014. Although the construction and installation described by the PDR was not installed due to financial constraints at the time, the recommendations are presented in the following sections.

### 2.5.1 Purpose

The purpose of this project was to update the Influent Metering Study prepared by Carollo in 2005, prepare construction documents, and provide engineering services during construction for three new influent flow metering facilities for the RWQCP. This project sought to upgrade the flowmeters on three existing influent sewage Trunk lines entering the RWQCP to obtain more accurate and reliable flow measurements.

### 2.5.2 Summary of Findings

Influent flow is currently conveyed to the RWQCP via five influent trunk sewers which are metered as follows:

- Rubidoux trunk sewer: 14-inch ABB MagMaster electromagnetic flowmeter.
- Jurupa trunk sewer: Flo-Dar radar velocity and ultrasonic depth flowmeter.
- Acorn and Arlanza trunk sewers: Combined at a junction structure within the Plant and their combined flow is metered with a Flo-Dar unit.
- Riverside/Hillside trunk sewers: Combined at a junction structure upstream of the Plant and their combined flow is metered with a Flo-Dar unit.

Despite steady population growth in the RWQCP's service area between 2005 and 2013, influent flows were on a relatively flat or declining trend. The Acorn, Arlanza, and Santa Ana (Riverside/Hillside) influent Trunk lines were hydraulically modeled to determine if each line had sufficient capacity. Hydraulic calculations showed that surcharge or near surcharge conditions may occur under high flow scenarios for all of the RWQCP's influent lines under the existing conditions at the time. The PDR indicated that the proposed metering facilities would increase hydraulic losses in the lines, and may increase the likelihood of surcharge conditions. To combat this, the PDR recommended that a bypass channel should be installed at the headworks to prevent flooding at the metering facility.

The PDR recommended that all flowmeters be upgraded to electromagnetic flowmeters, including some considerations for various types. In the PDR flowmeter sizes were selected in order to achieve a self-cleaning velocity (2+ fps) at least once a day, while not exceeding the pipe scouring velocity of 7 fps. The recommended meter sizes listed in the PDR are duplicated below for reference.

Table 2.1 Recommended Flowmeter Sizes for Plant Influent Truck Sewers *(2014 Influent Flowmetering Project)*

| Influent Trunk Line and Existing Diameter | Metered Run Pipe Diameter and Flowmeter Diameter | Self-Cleaning Velocity *(min. velocity at the time of the PDR)* | Pipe Scouring Velocity *(max. velocity projected by the PDR)* |
|---|---|---|---|
| Acorn, 27 in | 16 in | 2.2 fps | 3.2 fps |
| Arlanza, 51 in | 36 in | 4.8 fps | 6.9 fps |
| Santa Ana (Riverside/Hillside), 48 in | 36 in | 3.8 fps | 7.2 fps |

Additionally, the PDR included sections for proposed design, site investigation, construction efforts, and preliminary construction costs. The proposed design includes recommendations for electromagnetic flowmeter selection, a layout for each metering facility, required construction materials, and manufacturers for the diversion structure sluice gates, design criteria for the meter vault structures, and pipe and valve selection criteria. The site investigation section discusses geotechnical study requirements such as borehole locations and standard methods to use. The site survey and potholing sections describe items to be identified.

## 2.6 Biosolids Handling Assessment Study

The 2017 Biosolids Handling Assessment Study by Lee & Ro, Inc. provided guidance to the City with planning of short- and long-term mitigation alternatives to address operational and maintenance deficiencies in the existing sludge dewatering facilities.

### 2.6.1 Purpose

The purpose of this study was to provide an analysis of problematic issues surrounding the biosolids handling equipment. Additionally, the study provided recommendations to improve overall system performance.

### 2.6.2 Summary of Findings

1. The City's existing dewatering pumps and piping system experience significant operational and maintenance issues that result in added costs to the City.

2. The dewatered sludge pumps were originally intended to operate 8 hrs/day to accommodate the plant's daily dewatered sludge production of 55 gpm.
3. Due to high pressure issues, mechanical failures, sludge bridging of the diverter gate, and inaccurate load cell readings, operations staff have reduced pump flow rates and increased runtime to constant operation (i.e., 24 hrs/day). Constant operations costs coupled with excessive maintenance and replacement costs reduced efficiency and increased overall operation costs to the City.
4. The following short-term recommendations were made with a total estimated construction cost of $133,000:
   a. Add check valves to the injection ring.
   b. Relocate, and add new, existing lubricated water injection ring to a point adjacent to pumps.
   c. Install an additional lubricating water injection ring at the piping nearest the silos.
   d. Increase pipe diameter (8 inches to 12 inches) after wye connection to Silo No. 2.
   e. Install a Teflon® cover on the existing diverter gate.
   f. Install a laser or an ultrasonic sensor for sludge detection (replaces load cells).
5. At the time of this study, the City was planning to add four additional screw presses to the existing sludge dewatering facility. However, the new dewatering equipment cannot function at full load production due to capacity limitations of the existing Seepex progressive cavity pumps.
6. To accommodate the ultimate capacity of 52 mgd, the City will require more dewatered sludge pump capacity.
7. A long-term recommendation was made to replace the existing Seepex progressive cavity pumps with higher capacity pumps capable of overcoming high-pressure conditions. Three options were evaluated by this study, which are listed below with their associated costs:
   a. Progressive cavity pump at $884,000 total estimated construction cost.
   b. Piston pump at $860,000 total estimated construction cost.
   c. Conveyor system at $908,000 total estimated construction cost.

## 2.7 MBR Plant Expansion

Phase I of the MBR Plant Expansion project was completed during the summer of 2017. This project has been described briefly in previous sections. Phase II of the MBR Plant Expansion project will be to increase the MBR capacity from 26 mgd to 32 mgd. The projected timeline and cost for this project will be included in the CIP portion of this update to the Master Plan. A series of TMs were created as the basis of design for this project. As part of this expansion, many of the existing facilities have been rehabilitated or repurposed. The final design criteria for this expansion and other pertinent details are discussed in Volume 4, Chapters 1 and 3 of this update to the Master Plan.

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 3

## Wastewater Treatment System

## Process Design and Reliability Criteria

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Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 3: PROCESS DESIGN AND RELIABILITY CRITERIA

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Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 3: PROCESS DESIGN AND RELIABILITY CRITERIA

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:15:38-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 3

# PROCESS DESIGN AND RELIABILITY CRITERIA

## 3.1 Purpose

The purpose of this Chapter is to summarize the process capacity analysis of the existing liquid treatment and solids handling processes at the RWQCP.

## 3.2 Conclusions and Recommendations

### 3.2.1 Conclusions

- Current plant influent constituent concentrations are higher than the MBR treatment train (Plant 1) Phase I Expansion design basis. Corresponding average influent TSS, BOD, and ammonia loads are 26, 25, and 14 percent greater, respectively, than the Phase I design basis for a given average flow condition. This is due to severe drought conditions and water conservation over the last decade that have reduced influent flow and significantly increased wastewater constituent concentrations.
- Current peak month influent flow and load peaking factors are also higher than the MBR treatment train (Plant 1) Phase I Expansion design basis. Corresponding peak month influent TSS, BOD, and ammonia loads are approximately 20 percent greater than the Phase I design basis for a given peak month condition. This is due to severe drought conditions and water conservation over the last decade that have reduced influent flow and significantly increased wastewater constituent concentrations.
- Reliable MBR treatment train (Plant 1) process capacity is 26.0 mgd ADF with one MBR tank out of service. The limiting process component is the MBR hydraulic flux.
- Reliable ACT treatment train (Plant 2) process capacity is 17.5 mgd ADF with one of the larger secondary clarifiers out of service.
- Overall plant treatment capacity is limited by grit removal and WAS thickening process capacity. Assuming that these limitations are removed, the reliable overall process capacity would be 39.7 mgd ADF with one of the MBR tanks out of service. Assuming that field oxygen transfer efficiency is greater than the conservative value used in the capacity analysis, the reliable overall process capacity could increase to 43.5 mgd ADF.

### 3.2.2 Recommendations

- Conduct DAFT stress tests to determine the maximum feasible SLR. This will determine when an alternate standby thickener, such as a trailer-mounted gravity belt thickener or rotary drum thickener, is needed for reliable WAS thickening operation (if one DAFT is taken out of service for rehabilitation or repair).
- Refurbish the unused small anaerobic digester (former Digester 3) as Digester 5 to provide adequate redundancy. The four larger digesters (Digesters 1 through 4) would provide adequate process capacity to handle peak month volatile solids loading conditions. To ensure that all larger digesters are available during peak sustained volatile solids loading conditions, the small digester would be used during average loading conditions so that

one of the larger digesters could be taken out of service for cleaning. No adjustment to influent flow rate would be necessary during this maintenance service.

## 3.3 Background

The existing Riverside RWQCP treatment processes include preliminary treatment with mechanical bar screens and vortex-type grit tanks, parallel liquid treatment trains with MBRs and activated sludge, WAS thickening with DAFTs, thickened sludge stabilization with anaerobic digesters, and digested sludge dewatering with a combination of centrifuges, BFPs, and screw presses. Process schematics of the liquid treatment and solids handling treatment trains are shown on Figures 3.1 and 3.2, respectively.

The MBR treatment train (Plant 1) includes primary clarifiers, aeration tanks with a pre-anoxic zone and internal mixed liquor recycle pumps, aeration air blowers, post-anoxic tanks, MBR tanks, deoxygenation tanks with RAS pumps, WAS pumps, and a chlorine contact tank. The primary clarifiers, new aeration tank, post-anoxic tanks, MBR tanks, and deoxygenation tanks were added as part of the MBR treatment train (Plant 1) Phase I Expansion. The former rectangular secondary clarifiers were modified and repurposed as the post-anoxic tanks, MBR tanks, and deoxygenation tanks.

The ACT treatment train (Plant 2) includes primary clarifiers, aeration tanks with a pre-anoxic zone and internal mixed liquor recycle pumps, aeration air blowers, secondary clarifiers, RAS pumps, WAS pumps, granular media filters, and two chlorine contact tanks operated in series.

Primary effluent equalization is provided to equalize diurnal DWF and WWF to reduce the design plant influent PWWF:ADF peaking factor of 2.2 to an equalized secondary influent peaking factor of 1.5. Primary effluent from the MBR treatment train (Plant 1) and ACT treatment train (Plant 2) primary clarifiers is commingled in the equalization basin, so a portion of the flow from the ACT treatment train (Plant 2) primary clarifiers can be directed to the MBR treatment train (Plant 1) aeration tanks and vice versa.

Additional detail on the existing liquid treatment processes is provided in Volume 4, Chapter 1, Existing Facilities.

Primary sludge is thickened in the primary clarifiers. WAS from the MBR treatment train (Plant 1) and ACT treatment train (Plant 2) is thickened in DAFT. Thickened primary and WAS is blended in a mixing tank for a consistent anaerobic digester feed quality. The blended sludge is stabilized in four anaerobic digesters. Two of the digesters were constructed as part of the 2011 Phase I Expansion project and two of the three existing digesters were modified as part of the project. Digested sludge is dewatered with a combination of centrifuges, and screw presses.

Waste backwash from the ACT treatment train (Plant 2) granular media filters, DAFT overflow, and sludge dewatering centrate/filtrate/pressate are returned to the plant headworks.

Additional detail on the existing solids handling treatment train is provided in Volume 5, Chapter 1, Existing Facilities.

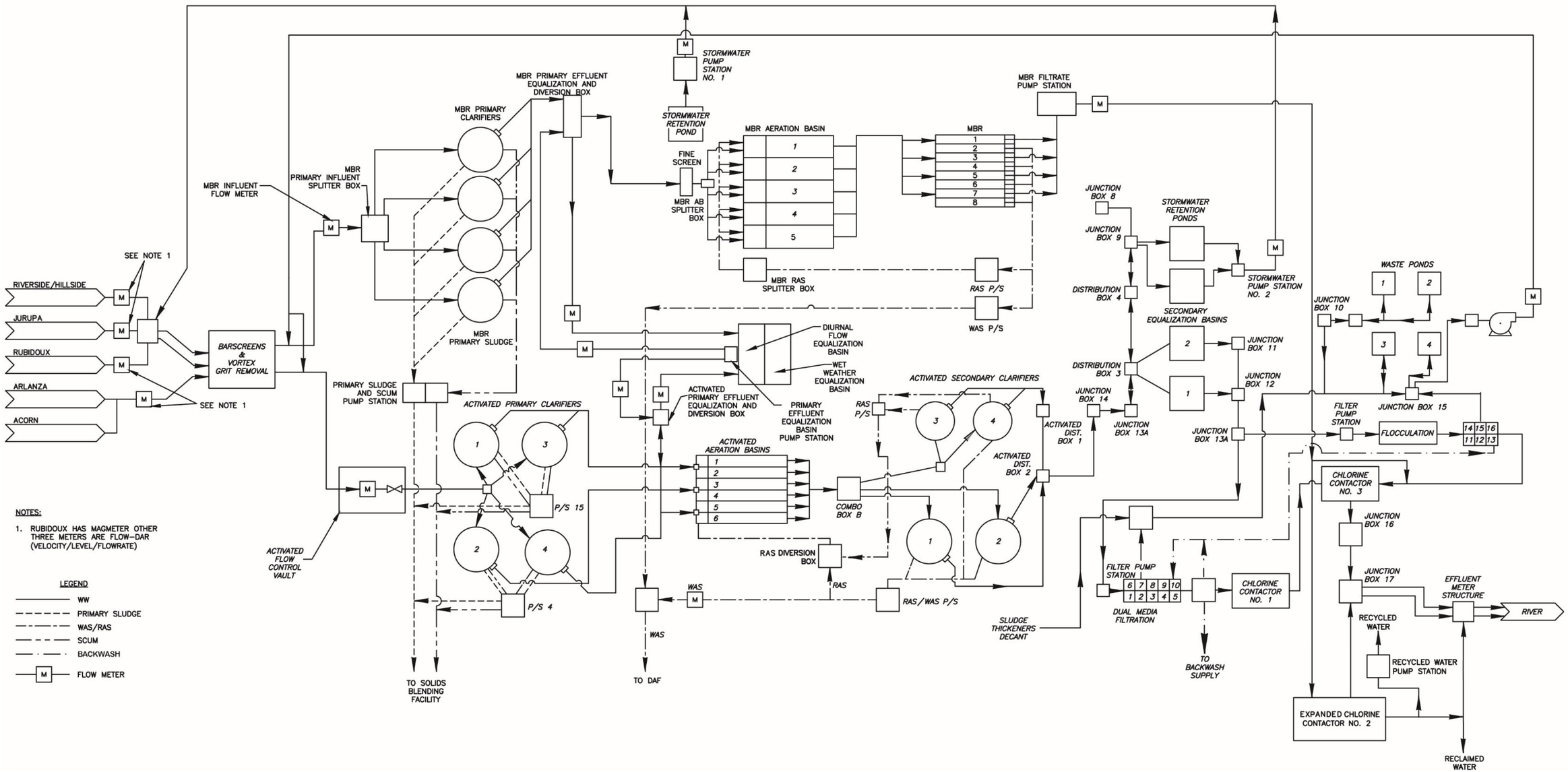



Figure 3.1 Process Flow Schematic

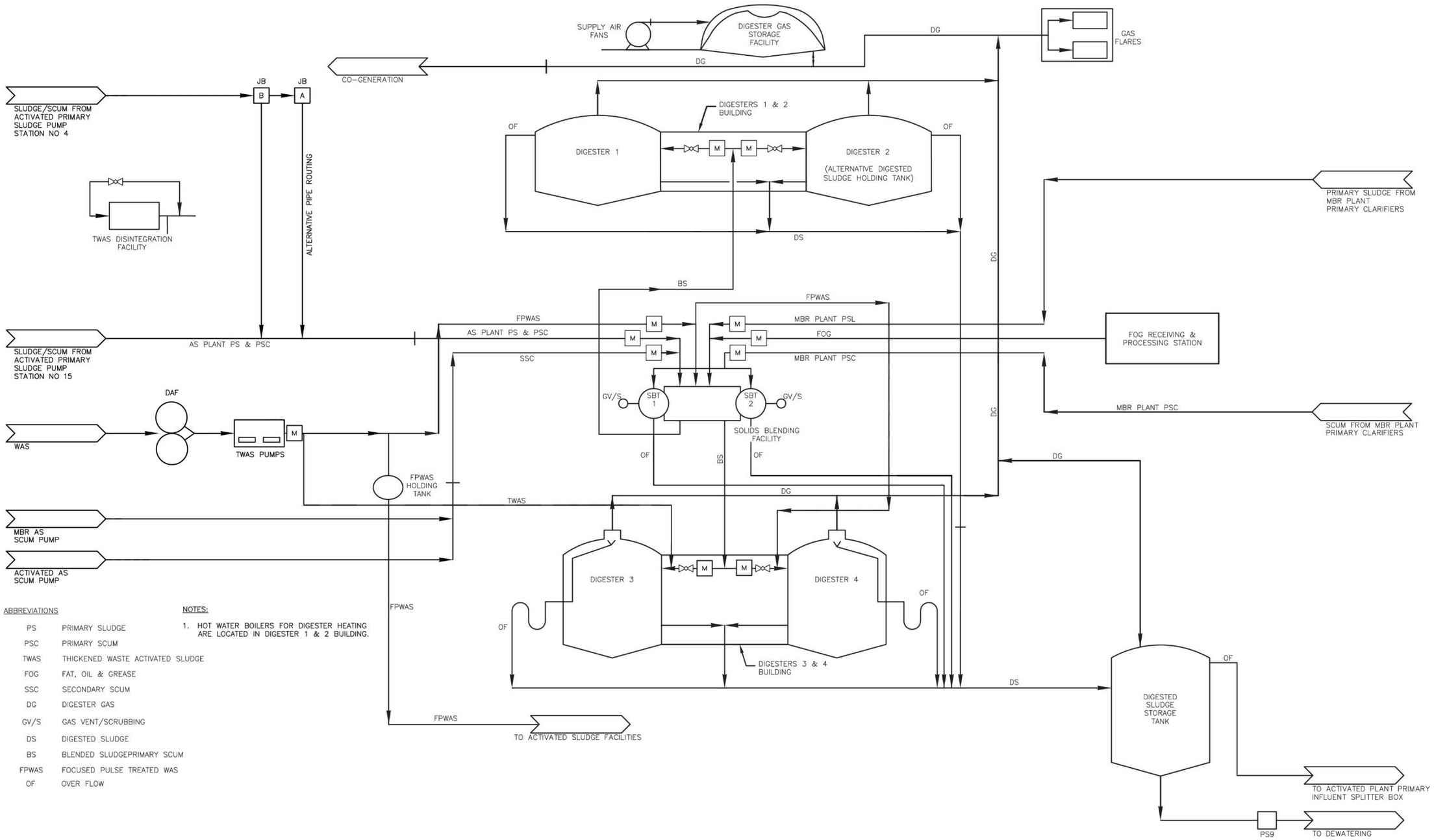



Figure 3.2 Process Flow Schematic

## 3.4 Historical Operations and Performance Data

Historical operations and performance data were analyzed to determine current wastewater characteristics, influent flow and loading peaking factors, typical unit process operating conditions, and typical unit process performance for use in process model configuration and calibration, described below. Time series plots of the relevant historical data are included in Appendix 3A.

### 3.4.1 Influent Flow and Loading

Plant influent constituent concentrations and loading patterns were determined for this update of the Master Plan by analyzing plant influent for the 5-year period of 2011 through 2016. As expected, recent influent constituent concentrations have increased relative to the MBR treatment train (Plant 1) Phase I Expansion design basis because of reduced water consumption within the service area. Reduced consumptive water use (i.e., the portion of potable water that enters the household sewer) provides less dilution of the organic and nutrient loads from residential and commercial sources. Current loading patterns (e.g., average day maximum month conditions) were determined by analyzing the recent calculated plant influent loads.

Table 3.1 summarizes the average and peak month plant influent conditions determined for this project together with the values from the Phase I Plant Expansion design data. Average annual influent concentrations increased between about 14 percent (COD, $NH_3$-N) and 25 percent (TSS, BOD) since the Phase I design data were developed. In addition, the influent volatile solids fraction increased by about 8 percent.

Table 3.1 Plant Influent Conditions

| | 2012 Phase I Design Criteria | | Update to the Master Plan | |
|---|---|---|---|---|
| | *Annual Average* | *Maximum Month* | *Annual Average* | *Maximum Month* |
| Flow rate, mgd | 46 | 46 | 46.0 | 52.9 |
| TSS, mg/L | 270 | 310 | 341 | 368 |
| VSS, mg/L | 224 | 257 | 305 | 330 |
| VSS:TSS | 83% | 83% | 89% | 90% |
| COD, mg/L | 660 | 760 | 744 | 804 |
| sCOD, mg/L | 178 | 205 | 257 | 278 |
| sCOD:COD | 27% | 27% | 35% | 35% |
| BOD, mg/L | 295 | 340 | 369 | 398 |
| sBOD, mg/L | 88.5 | 102 | 170 | 184 |
| sBOD:BOD | 30% | 30% | 46% | 46% |
| TKN, mgN/L | 40 | 46 | 48.3 | 49.8 |
| $NH_3$-N, mgN/L | 28 | 32 | 31.9 | 32.9 |
| $NH_3$-N:TKN | 70% | 70% | 66% | 66% |
| Alkalinity, mg/L as $CaCO_3$ | 305 | 305 | 305[(1)] | 305[(1)] |

Table 3.1 Plant Influent Conditions (continued)

| | 2012 Phase I Design Criteria | | Update to the Master Plan | |
|---|---|---|---|---|
| | *Annual Average* | *Maximum Month* | *Annual Average* | *Maximum Month* |
| Temperature, deg C | 24 | 20 (winter), 30 (summer) | 26.0(2) | 21.7 (minimum)(3), 29.7 (maximum)(4) |
| Influent loads | | | | |
| TSS load, klb/d | 104 | 119 | 131 | 162 |
| COD load, klb/d | 253 | 292 | 285 | 355 |
| BOD load, klb/d | 113 | 130 | 142 | 176 |
| $NH_3$-N load, klbN/d | 10.7 | 12.3 | 12.2 | 14.5 |

Notes:
(1) No plant data available, assumed 2012 Phase I Design Criteria value.
(2) Average of 2016 weekly effluent temperature from CWQIS database.
(3) Minimum 4 week rolling average of 2016 weekly effluent temperature from CWQIS database, occurred 1/14 - 2/3.
(4) Maximum 4 week rolling average of 2016 weekly effluent temperature from CWQIS database, occurred 7/27 - 8/17.

Current loading patterns show a peaking factor for peak month TSS, COD, and BOD influent loads of 1.25; and peak month $NH_3$-N influent load of 1.20. Current flow patterns show a peaking factor for peak month influent flows of 1.15. The Phase I Plant Expansion peaking factor for peak month influent loads was 1.15; a peaking factor for peak month influent flows was not included.

As shown in the table, current peak month influent loads, which will be used to determine current process capacity as described below, are between about 20 and 35 percent higher than the Phase I Plant Expansion design data values.

### 3.4.2 MBR Treatment Train (Plant 1)

Historical operation and performance data of the MBR treatment train (Plant 1) is summarized in Figures A-20 through A-27 in Appendix 3A.

### 3.4.3 ACT Treatment Train (Plant 2)

Historical operation and performance data of the ACT treatment train (Plant 2) is summarized in Figures A-28 through A-38 in Appendix 3A.

### 3.4.4 Solids Handling

Historical operation and performance data of the solids handling facilities is summarized in Figures A-39 through A-50 in Appendix 3A.

## 3.5 Process Model Configuration and Calibration

A "whole plant" process model was used for this project to evaluate plant performance under current and future flow and loads determine limiting conditions for each liquid treatment and solids handling unit process. A "whole plant" process model includes both liquid treatment and solids handling processes to incorporate the impact of liquid treatment residuals (i.e., primary sludge, WAS) on solids handling processes and vice versa. Configuration and calibration of the Riverside RWQCP process model are described in the following sections.

## 3.6 Process Model Configuration

A "whole plant" model of the Riverside RWQCP was developed based on the existing liquid treatment and solids handling facilities using BioWin 5.2, a commercially available wastewater

treatment process simulator (EnviroSim Associates Ltd., Hamilton, Ontario, Canada). This is the Industry's standard modeling software. The simulator was used to determine existing process capacity, evaluate alternatives to increase process capacity where necessary, and evaluate alternatives to meet more stringent final effluent nutrient discharge limits. Integrating the liquid treatment and solids handling processes enables simulation of digester gas energy production; dewatered sludge production; and solids handling nitrogen and phosphorus recycle loads.

The BioWin configuration shown in Figure 3.3 reflects four MBR treatment train (Plant 1) aeration tanks, four post-anoxic tanks, four MBR tanks, and two deoxygenation tanks that were in service between July 1 and September 30, 2017. Unit process physical data, such as primary clarifier surface area and sidewater depth, aeration tank volume and sidewater depth, aeration air diffuser density, secondary clarifier surface area and sidewater depth, membrane tank and post-anoxic tank volume and sidewater depth, and anaerobic digester volume, were taken from Phase I Plant Expansion design documents and shop drawing submittals.

## 3.7 Process Model Calibration

The Riverside RWQCP "whole plant" model was calibrated using the influent flow rate and constituent loads between July 1 and September 30, 2016. Influent wastewater characteristics are defined by COD concentration, TKN concentration, TP concentration, inorganic suspended solids, pH, and alkalinity. A set of wastewater fractions are used to calculate BOD from the biodegradable COD, VSS from the particulate COD, TSS from the particulate COD and inorganic suspended solids, and ammonia from the TKN.

Values for the following model input parameters were determined from operations data during this period:

- Wastewater temperature.
- Influent inorganic suspended solids.
- MBR treatment train (Plant 1) primary sludge flow rate.
- MBR treatment train (Plant 1) primary suspended solids removal.
- MBR treatment train (Plant 1) internal mixed liquor recycle flow rate.
- MBR treatment train (Plant 1) return sludge flow fraction.
- MBR treatment train (Plant 1) waste sludge flow rate.
- Activated sludge primary suspended solids removal.
- ACT treatment train (Plant 2) internal mixed liquor recycle flow rate.
- ACT treatment train (Plant 2) return sludge flow fraction.
- ACT treatment train (Plant 2) waste sludge flow rate.
- Secondary clarifier effluent TSS concentration.
- Granular media filter waste backwash flow rate.
- DAFT solids capture efficiency.
- Dewatered sludge TS concentration.
- Sludge dewatering solids capture efficiency.

Similar decreasing DO setpoints of 2.0 mg/L in diffuser grid 1, 1.5 mg/L in diffuser grid 2, and 1.0 mg/L in diffuser grid 3 were assumed to simulate oxygen transfer in the MBR treatment train (Plant 1) and the ACT treatment train (Plant 2) aeration tanks.

Preliminary calibration results showed high MBR treatment train (Plant 1) waste sludge production compared waste sludge flowmeter values as measured at the plant. This higher sludge

production resulted in higher digester feed volatile solids load and dewatered sludge total solids production. Assuming that the MBR treatment train (Plant 1) waste sludge flowmeter is incorrect, the corresponding model input flow rate was reduced by 30 percent. This adjustment reduced the difference between the simulated and reported digester feed solids load and dewatered sludge production to within 10 percent. This adjustment is reasonable because it corresponds to the measured amount of dewatered biosolids that is sent offsite for land application.

A table comparing the final calibration results to the corresponding plant operations and performance data is included in Appendix 3B. The difference between simulated and reported values for key parameters – MBR aeration tank suspended solids concentration, MBR aeration tank aeration air demand, MBR effluent nitrate concentration, activated sludge aeration tank suspended solids, activated sludge aeration tank air demand, activated sludge effluent nitrate concentration, anaerobic digester volatile solids load, digested sludge total solids load, and dewatered sludge solids production – was less than 10 percent. These results are within acceptance criteria recommended in the International Water Association Scientific & Technical Report 22, *Guidelines for Using Activated Sludge Models* (2013), and are very good given that a detailed influent wastewater sampling campaign was not conducted.

## 3.8 Process Capacity Evaluation

The calibrated process model was run under future plant influent flow and load conditions to identify the process capacity of each unit process.

## 3.9 Process Model Application

The calibrated process model was used to simulate Riverside RWQCP performance under corresponding peak month plant influent flow conditions at an average flow rate of 46.0 mgd. Peak month peak influent loads were based on current constituent concentrations and peaking factors.

The flow split between the MBR treatment train (Plant 1) and the ACT treatment train (Plant 2) was 1.3:1.0 (i.e., 26 mgd: 20 mgd), based on the Phase I design data. An aerobic SRT of 7.5 days was assumed for both the MBR treatment train (Plant 1) and ACT treatment train (Plant 2) to ensure reliable nitrification at the minimum wastewater temperature of 20 degree C.

### 3.9.1 Process Reliability Criteria

The process capacity evaluation assumed that one process unit from the MBR treatment train (Plant 1) would be out of service as required by Division 4, Chapter 3, Article 10; Environmental Health, Recycled Water Criteria, Reliability Requirements for Full Treatment; in Title 22 California Code of Regulations. The evaluation also assumed that one process unit (secondary clarifier) would be out of service. Our interpretation of Title 22 assumes that only one of the process units from either the MBR treatment train (Plant 1) or the ACT treatment train (Plant 2) would be out of service at a time, which seems reasonable. In this case, taking one of the MBR treatment train (Plant 1) process units out of service is the most restrictive capacity assumption for the RWQCP so that is what the secondary treatment capacity is based upon.

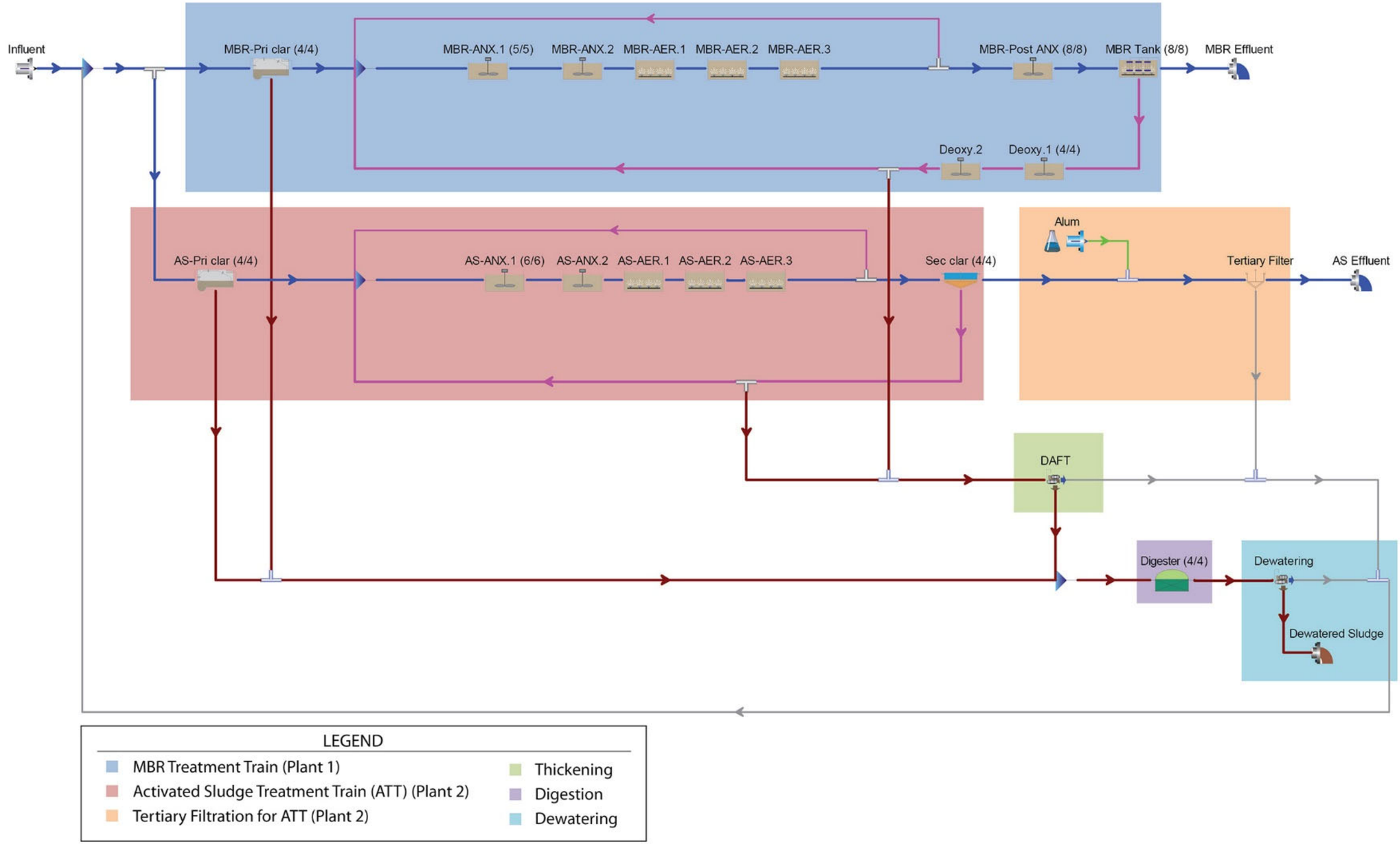



Figure 3.3 BioWin Configuration

## 3.10 Unit Process Capacity

### 3.10.1 MBR Treatment Train (Plant 1)

The process capacity of the MBR treatment train (Plant 1) is summarized in Figure 3.4. The MBR treatment train (Plant 1) process capacity is estimated using five criteria: primary clarifier surface overflow rate, MBR aeration tank air demand, MBR hydraulic flux, and MBR solids mass flux, and chlorine contact tank hydraulic residence time. The Figure indicates the limiting value for each criterion. The MBR hydraulic flux and solids mass flux are calculated assuming one MBR tank out of service. The CCB 2 has been approved to operate at a contact time of 29 min at a free chlorine residual of 1 mg/L (CT approximately 30 mg-min/L). This adjusts the capacity of CCB 2 to 46 mgd max flow. Letter dated May 6, 2019 from the SWRCB Division of Drinking Water is included in Appendix 3D for reference.

The Figure shows that the MBR hydraulic flux, solids flux, and the chlorine contact tank hydraulic residence time limits the MBR treatment train (Plant 1) capacity to 26.0 mgd ADF, with one MBR tank out of service. The next limitation is aeration, with a reliable aeration air blower capacity of 27,000 scfm that limits the MBR treatment train (Plant 1) capacity to 29.4 mgd. However, for aeration to be the bottleneck for MBR treatment capacity, the City must implement an MBR Phase II expansion project that provides sufficient MBR hydraulic flux and solids flux.

### 3.10.2 ACT Treatment Train (Plant 2)

Secondary clarifier SLR limits were calculated using state point analysis and a design sludge volume index of 120 mL/g based on historical operations and performance data. Figure 3.5 shows the benefit of using the design (i.e., 90th percentile) Sludge Volume Index of 120 mL/g to calculate clarifier capacity instead of the typical value of 150 mL/g. Figure 3.6 shows that the secondary clarifiers are critically loaded at 17.5 mgd ADF with one of the larger clarifiers out of service, when operating with an sludge volume index of 120 mL/g (residence time of 3.5 hours).

The process capacity of the ACT treatment train (Plant 2) is summarized in Figure 3.7. The ACT treatment train (Plant 2) process capacity is estimated using six criteria: primary clarifier SOR, activated sludge aeration tank air demand, secondary clarifier SOR, secondary clarifier SLR, filter hydraulic loading rate, chlorine contact tank hydraulic residence time. The Figure indicates the limiting value for each criterion. The secondary clarifier SOR and SLR are calculated assuming that one of the larger clarifiers is out of service. The maximum filter hydraulic loading rate of 3.33 gpm/sq ft is based on historical operation. The minimum chlorine contact tank hydraulic residence time of 120 minutes (calculated based on the entire tank volume) is based on the Title 22 minimum modal contact time of 90 minutes, assuming a conservative modal contact time: hydraulic residence time ratio of 0.75.

Figure 3.7 shows that the secondary clarifier SLR limits the ACT treatment train (Plant 2) capacity to 17.5 mgd ADF with one of the larger clarifiers out of service. This is closely followed by the aeration air capacity (approximately 20 mgd).

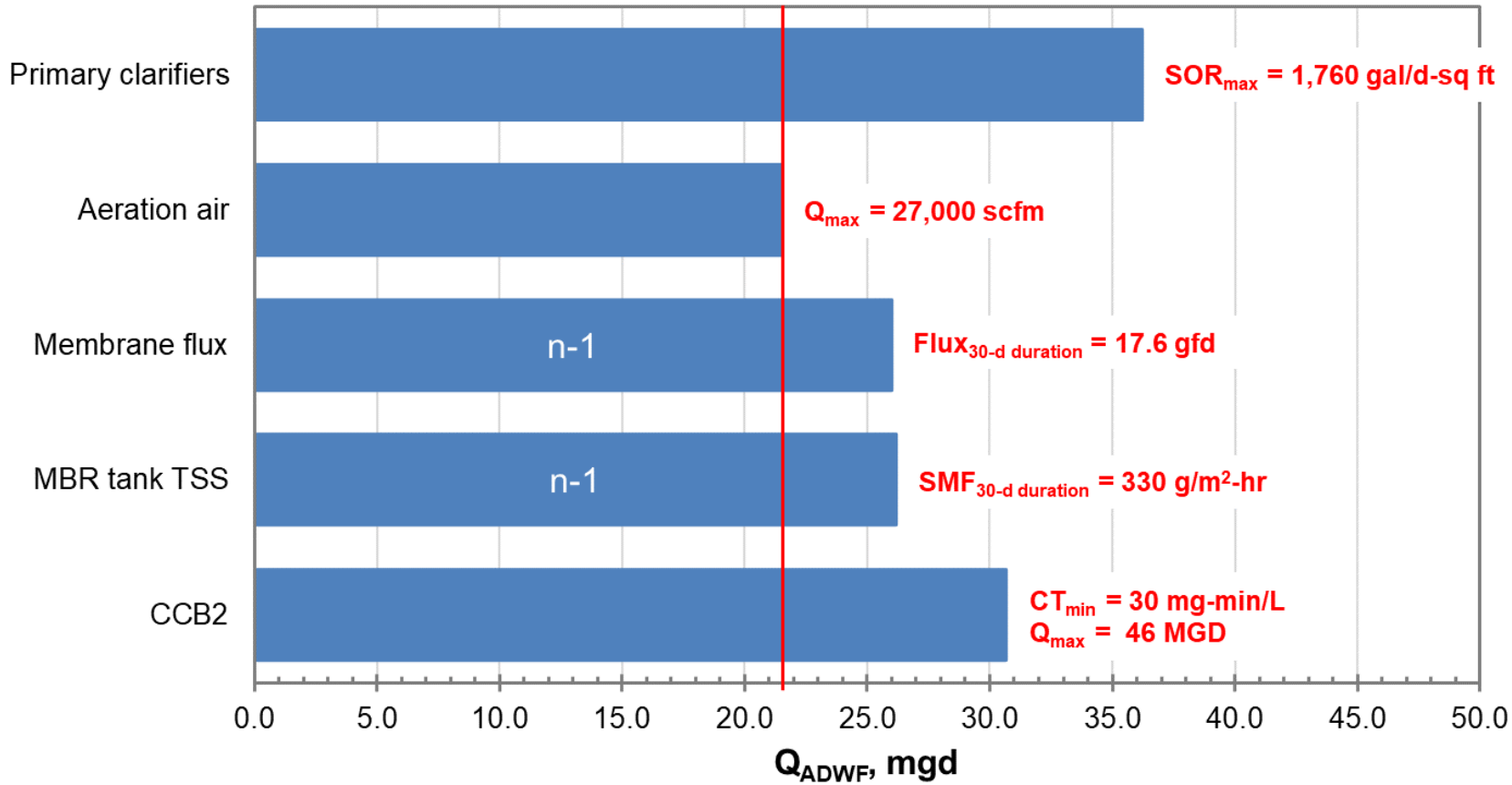



Figure 3.4 MBR Treatment Train (Plant 1) Process Capacity Summary

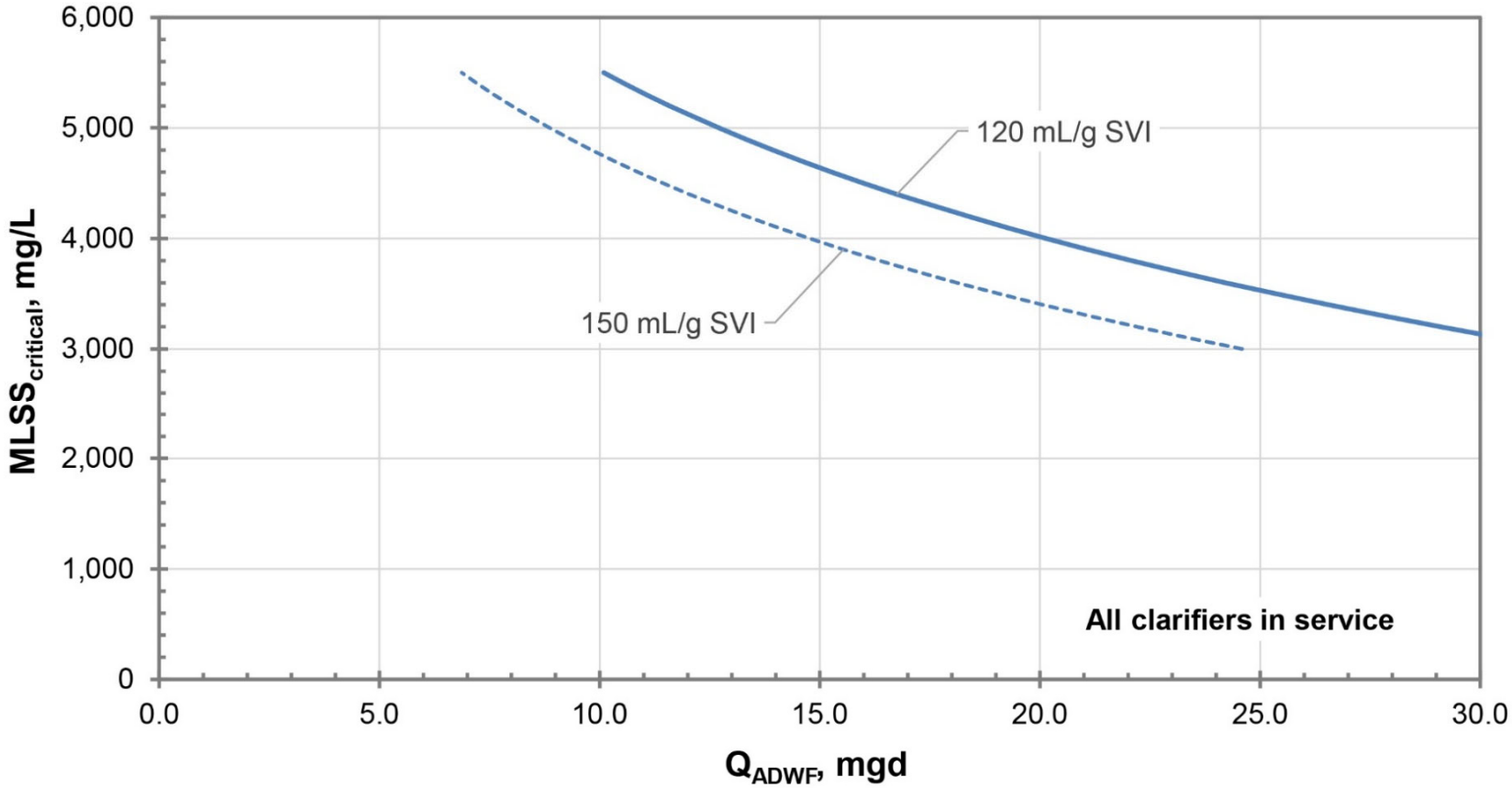



Figure 3.5 ACT Treatment Train (Plant 2) Critical MLSS Concentration, Sludge Volume Index = 120 and 150 mL/g

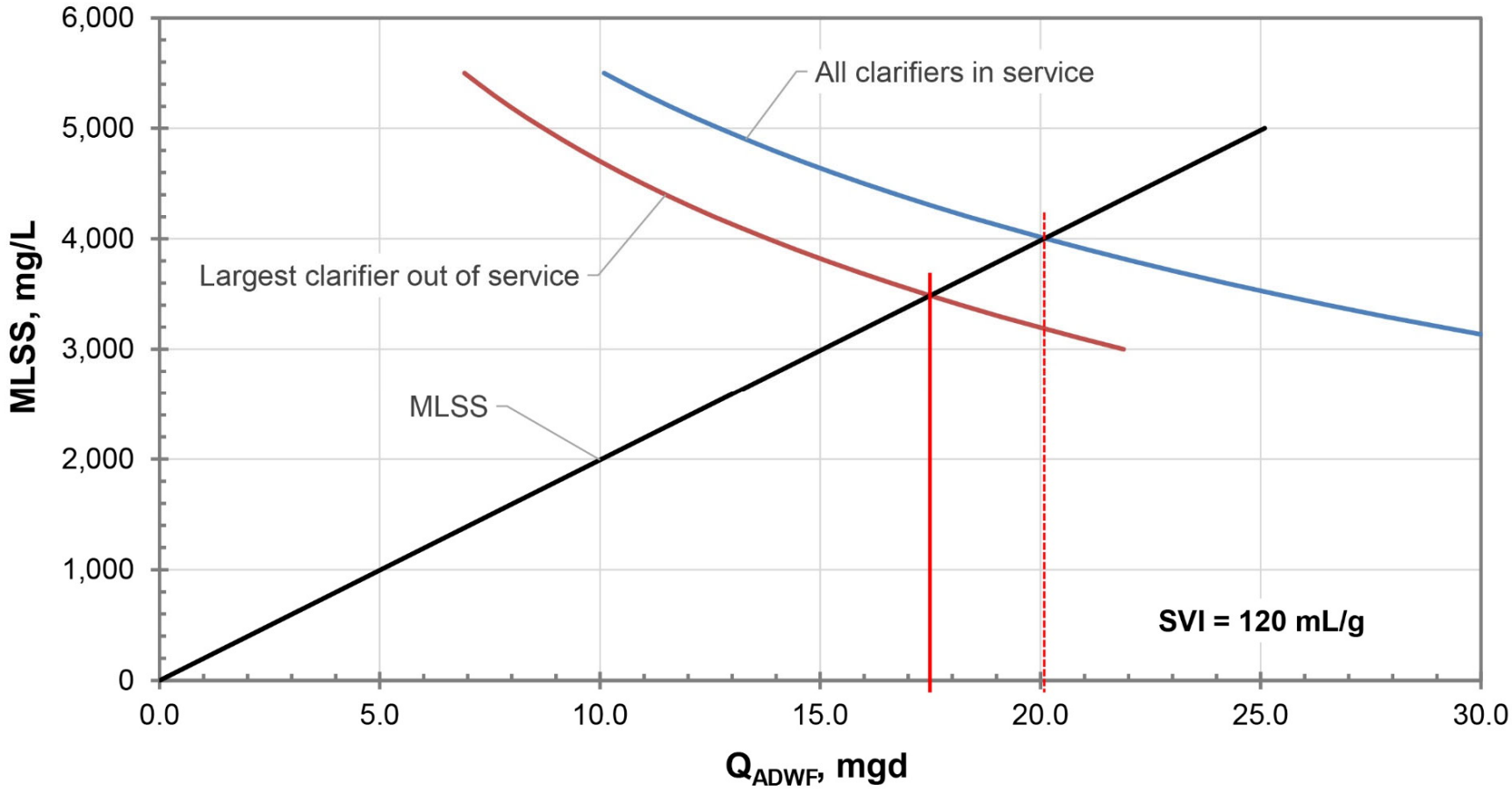



Figure 3.6 ACT Treatment Train (Plant 2) Secondary Clarifier Influent Critical Suspended Solids Concentration, Sludge Volume Index = 120 mL/g and Largest Clarifier Out of Service

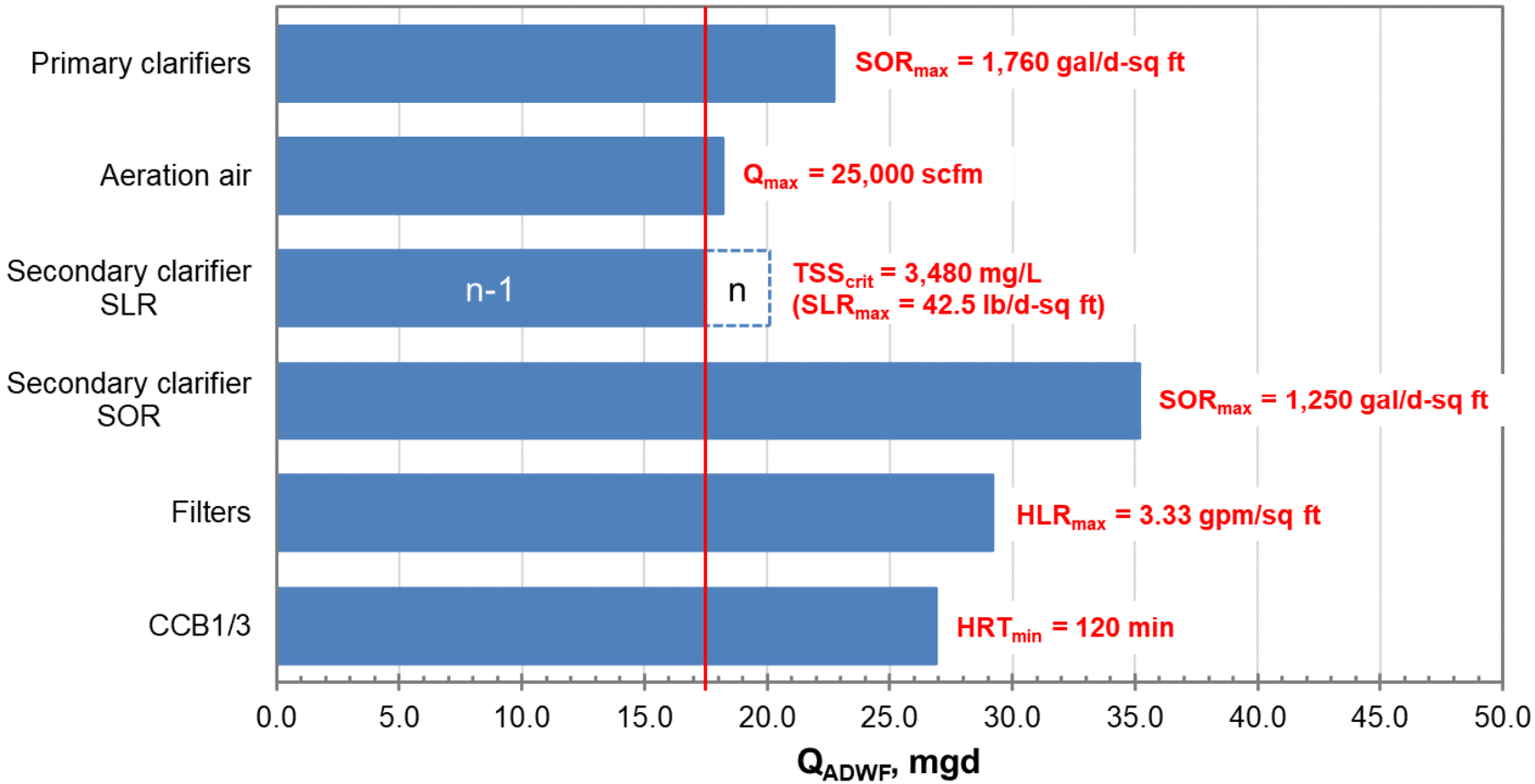



Figure 3.7 The Process Capacity of the ACT Treatment Train (Plant 2) Summary

### 3.10.3 Overall Process Capacity

The overall process capacity of the Riverside RWQCP is summarized in Figure 3.8, which includes preliminary treatment and solids handling process capacity together with the most conservative primary/secondary/tertiary treatment condition. Process capacity determination for preliminary treatment facilities, bar screens and grit tanks, is discussed in Volume 4, Chapter 4, Preliminary Treatment. The process capacity of the DAFTs was determined based on simulated waste sludge production and a peak SLR of 24.0 ppd/sq ft with both units in service. The process capacity of the anaerobic digesters was determined based on simulated blended sludge production, a peak volatile SLR of 0.15 ppd/cu ft, and a minimum hydraulic residence time of 15.0 days with Digesters 1 through 4 in service. The sludge dewatering process capacity was determined based on a screw press peak SLR of 700 pph per unit and a peak hydraulic loading rate of 55 gpm per unit with 6 screw presses in service, operating 24 hrs/day, 7 days/week.

Figure 3.8 shows that, aside from grit tank and thickening capacity, the overall process capacity of the Riverside RWQCP is 43.5 mgd ADF with one MBR tank and one of the larger activated sludge secondary clarifiers out of service concurrently (26.0 mgd for the MBR treatment train (Plant 1) and 17.5 mgd for the ACT treatment train (Plant 2)). Assuming that one MBR tank is out of service and all activated sludge secondary clarifiers are in service, the overall process capacity increases to 45.8 mgd ADF, as indicated by the dashed portion of the primary/secondary/tertiary treatment bar in the figure. MBR hydraulic flux and activated sludge secondary clarifier SLR are the limiting criteria for the two treatment trains in this scenario.

The capacity and evaluation of the grit chambers is described in Volume 4, Chapter 4. The capacity and evaluation of the thickening is described in Volume 5, Chapter 4, Solids Production and Thickening.

In order to take one of the larger anaerobic digesters out of service for maintenance and/or cleaning during dry weather, Digester 5 (former Digester 3) is required to be in operation to maintain an acceptable volatile SLR and hydraulic residence time.

It should be noted that the aeration air demands in each treatment train that were used to determine the plant capacity were simulated using the measured $\alpha F$ values from *in situ* offgas testing conducted by the City in March 2018. The full report on the offgas testing is included in Appendix 3C. The average $\alpha F$ for the ACT treatment train (Plant 2) and MBR treatment train (Plant 1) is 0.47 and 0.53, respectively. The difference between the average $\alpha F$ values probably reflects the membrane diffuser element age in each train, as well as the difference in mixed liquor concentration. City Staff stated that the diffuser elements in the AS train aeration tanks have not been replaced since they were originally installed in the early 2000s.

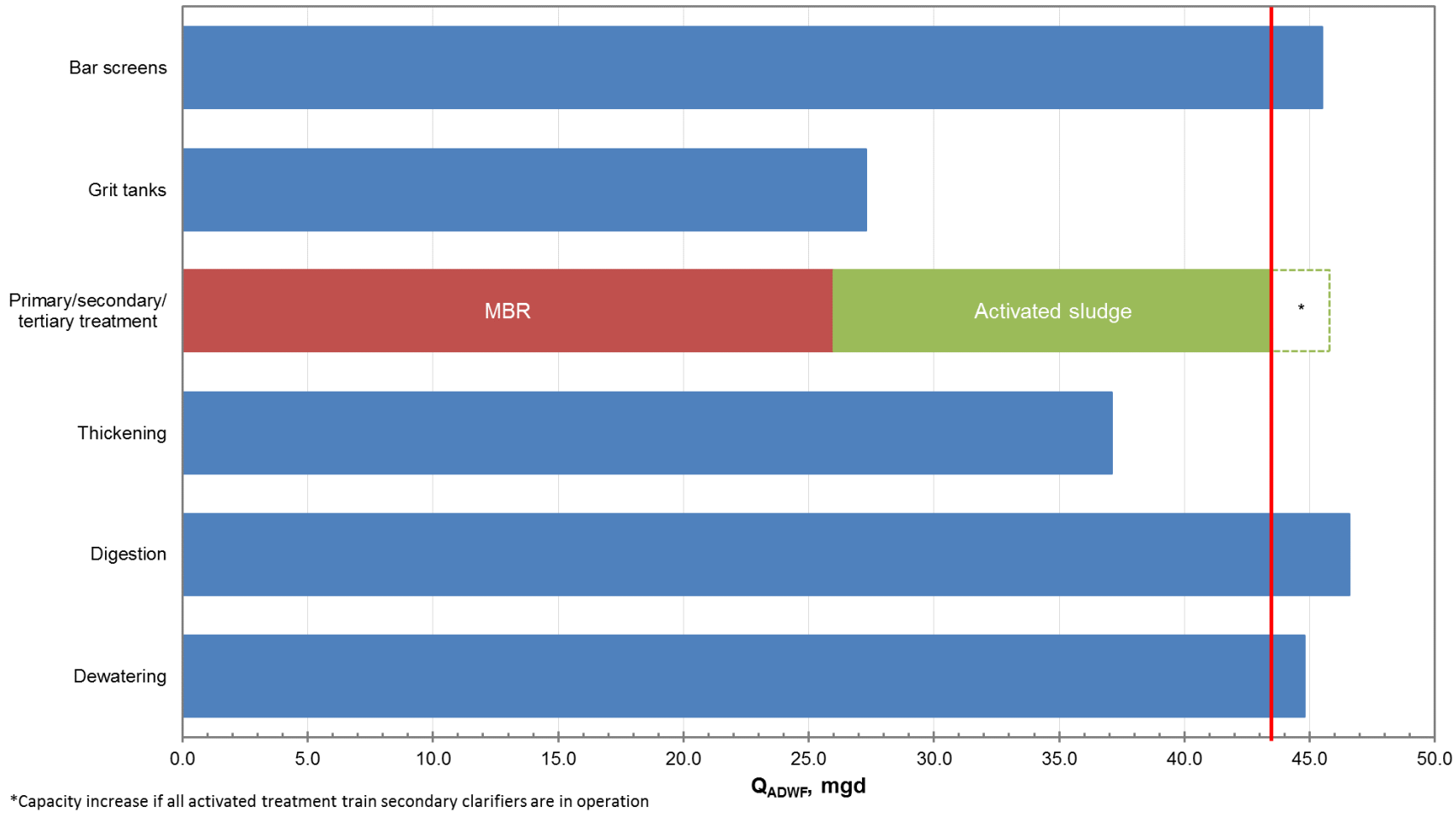



Figure 3.8 The Overall Process Capacity of the Riverside RWQCP Summary

Appendix 3A

# HISTORICAL OPERATIONS AND PERFORMANCE DATA

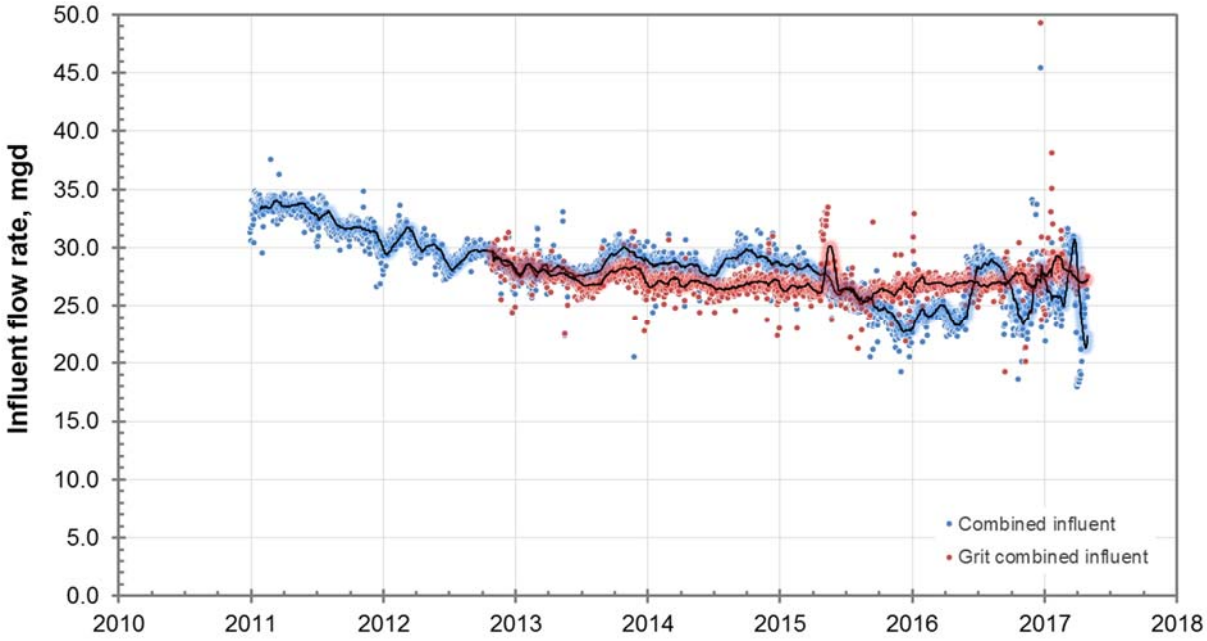



Figure A-1. Influent flow rate, January 2011 – April 2017

*The highlighted trendlines represent a 28-day moving average.*

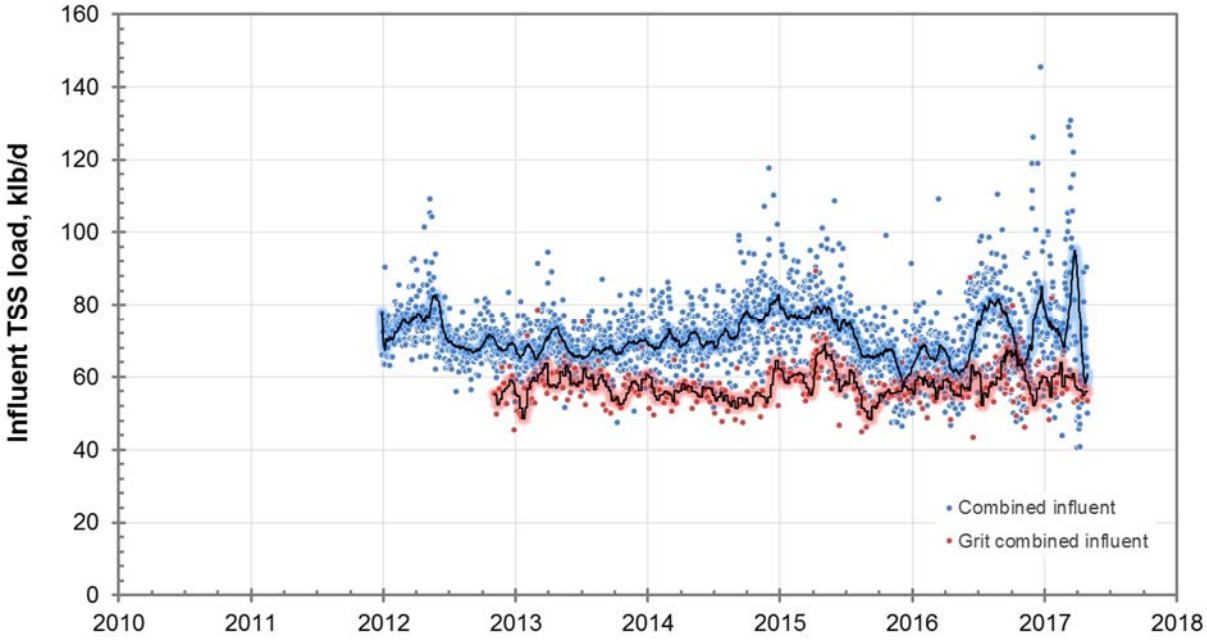



Figure A-2. Influent flow rate, January 2011 – April 2017

*The combined influent load is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and TSS concentrations. The highlighted trendlines represent a 28-day moving average.*

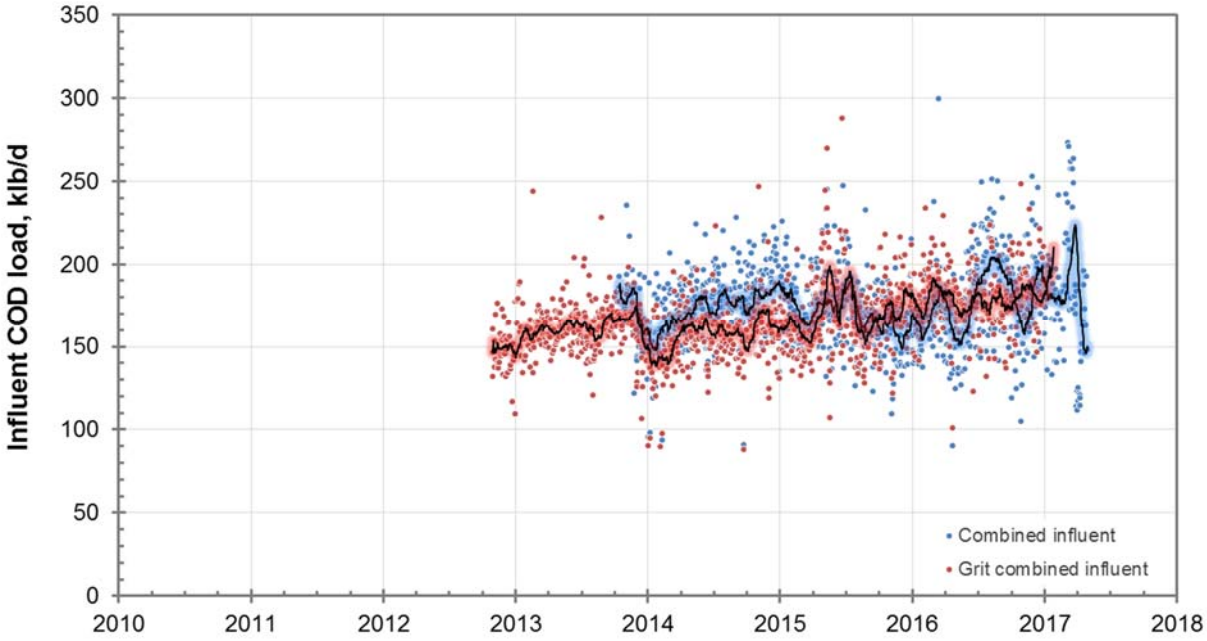



Figure A-3. Influent flow rate, January 2011 – April 2017

*The combined influent load is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and COD concentrations. The highlighted trendlines represent a 28-day moving average.*

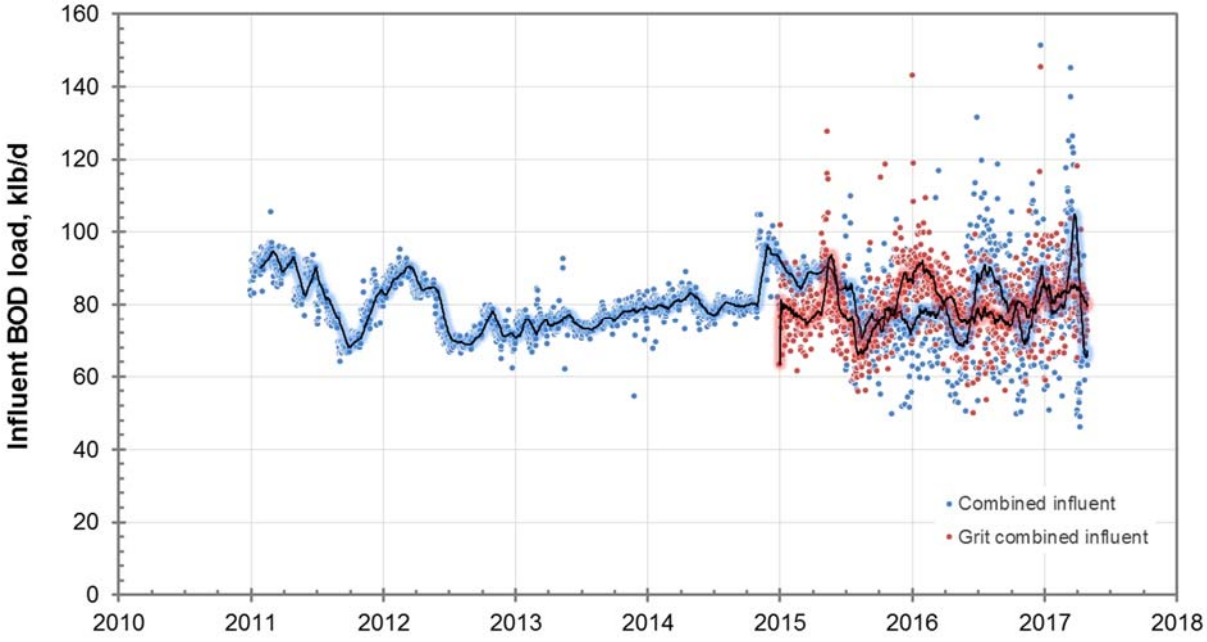



Figure A-4. Influent flow rate, January 2011 – April 2017

*The combined influent load is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and BOD concentrations. The highlighted trendlines represent a 28-day moving average.*

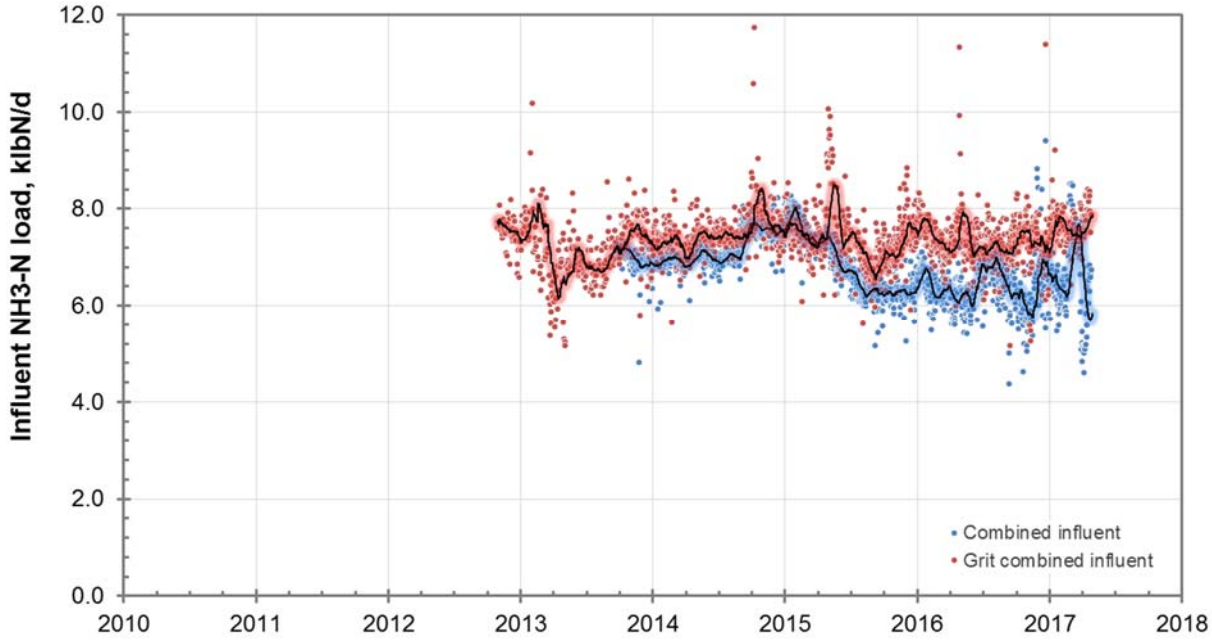



Figure A-5. Influent flow rate, January 2011 – April 2017

*The combined influent load is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and NH3-N concentrations. The highlighted trendlines represent a 28-day moving average.*

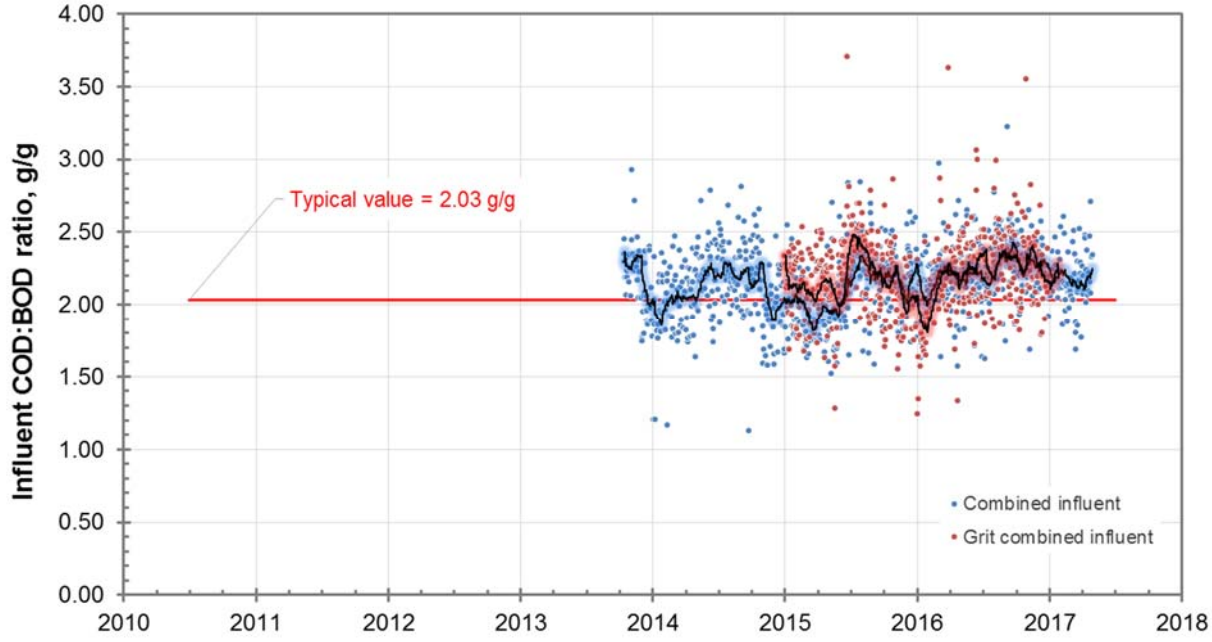



Figure A-6. Influent flow rate, January 2011 – April 2017

*The COD:BOD ratio of 2.03 g/g is based on "typical" COD and BOD concentrations of 500 and 246 mg/L, respectively. The combined influent concentrations are calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and COD and BOD concentrations. The highlighted trendlines represent a 28-day moving average.*

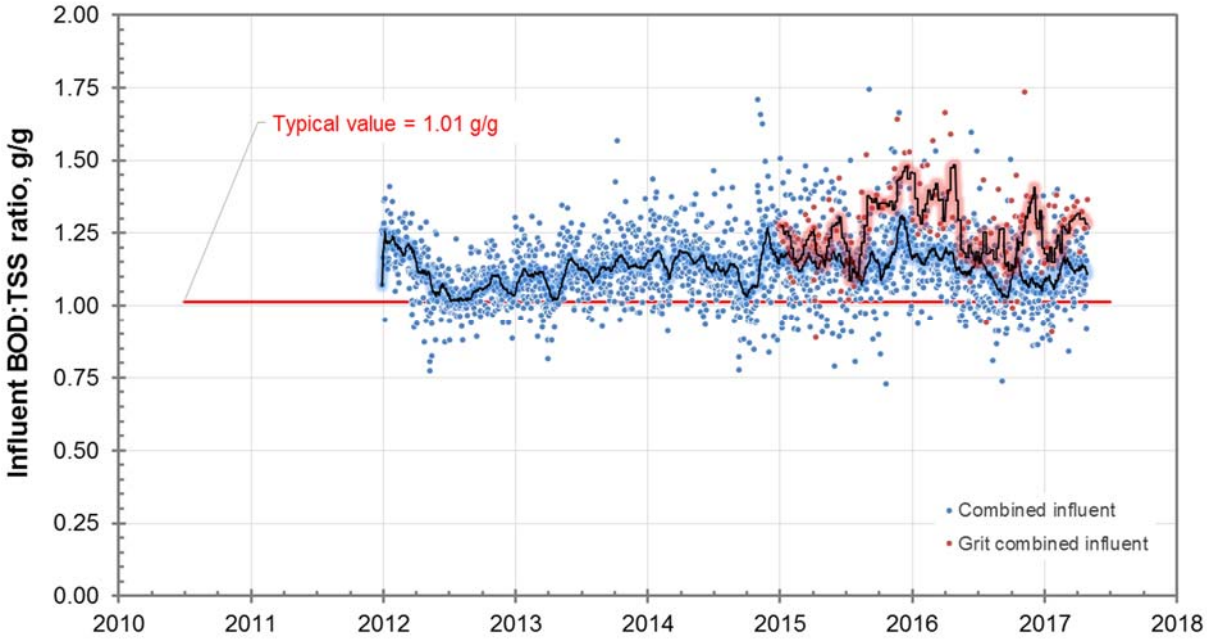



Figure A-7. Influent flow rate, January 2011 – April 2017

*The BOD:TSS ratio of 1.01 g/g is based on "typical" BOD and TSS concentrations of 246 and 243 mg/L, respectively. The combined influent concentrations are calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and BOD and TSS concentrations. The highlighted trendlines represent a 28-day moving average.*

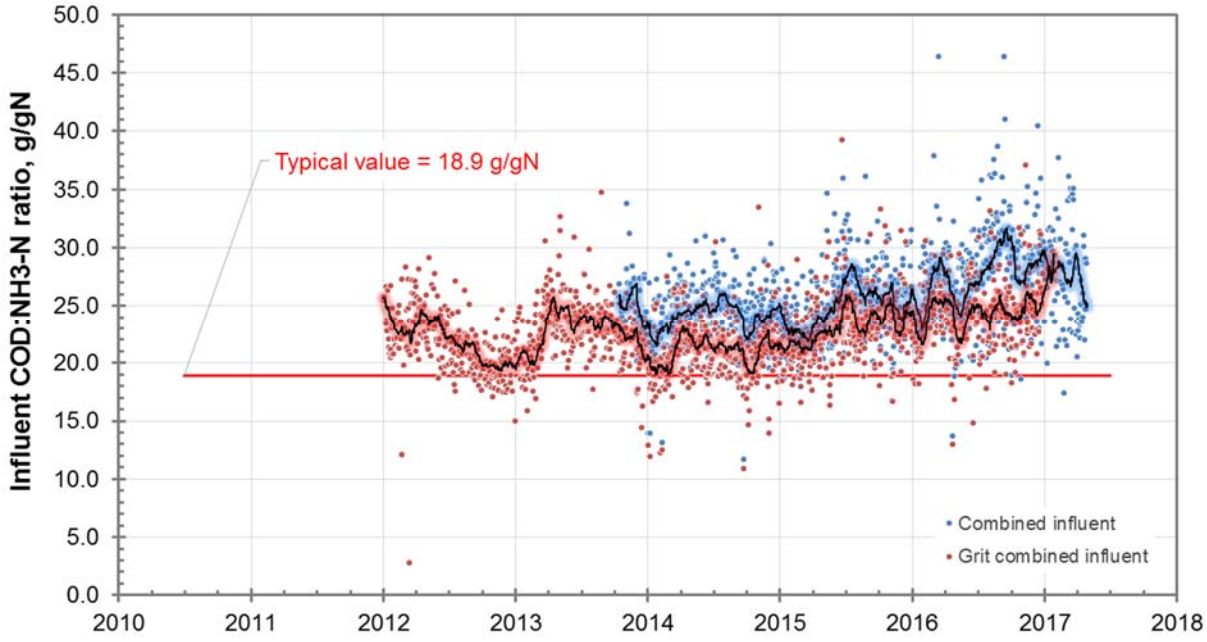



Figure A-8. Influent flow rate, January 2011 – April 2017

*The COD:NH3-N ratio of 18.9 g/gN is based on "typical" COD and NH3-N concentrations of 500 and 26.4 mg/L, respectively. The combined influent concentrations are calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and COD and NH3-N concentrations. The highlighted trendlines represent a 28-day moving average.*

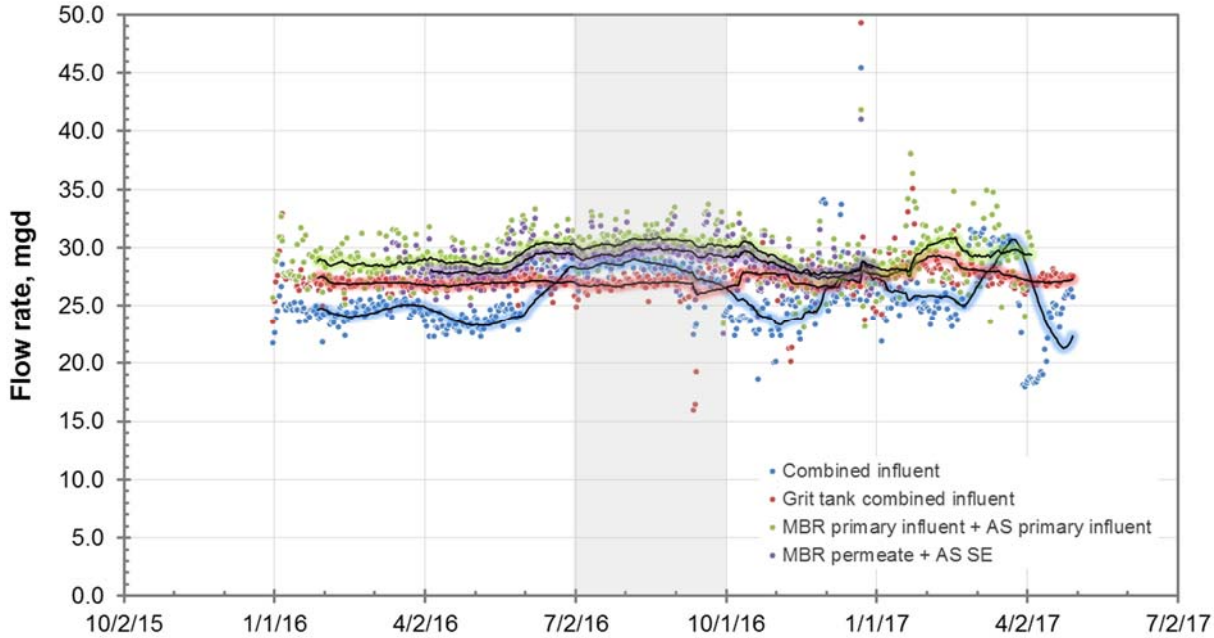



Figure A-9. Influent flow rate, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

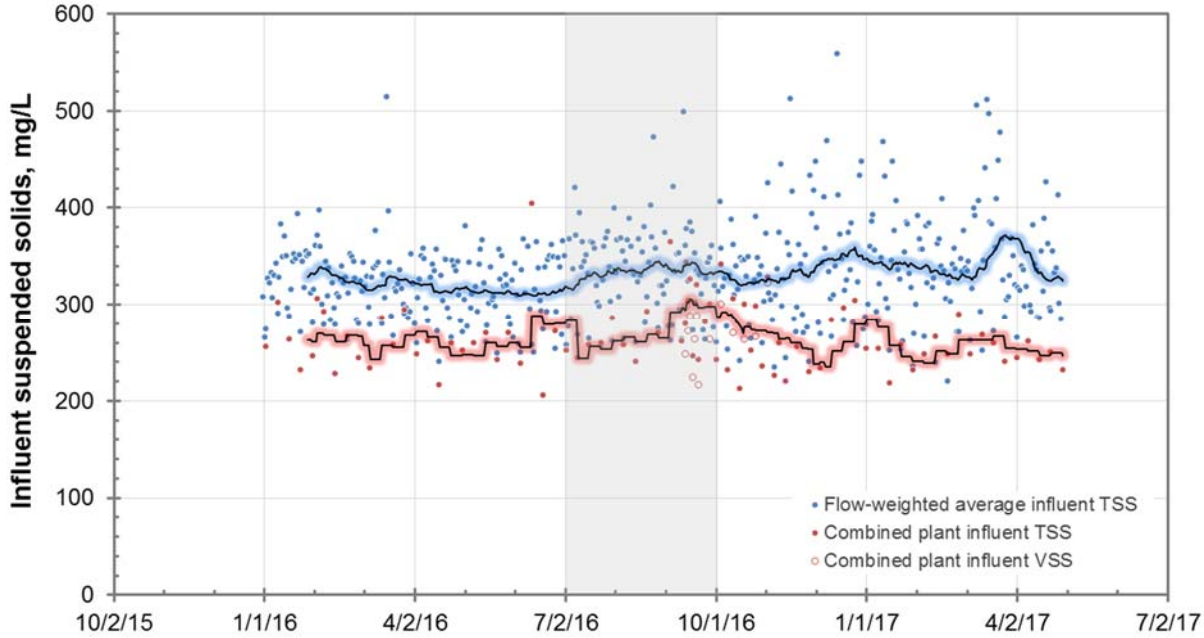



Figure A-10. Influent suspended solids concentration, January 2016 – April 2017

*The flow-weighted average influent concentration is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and TSS concentrations. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

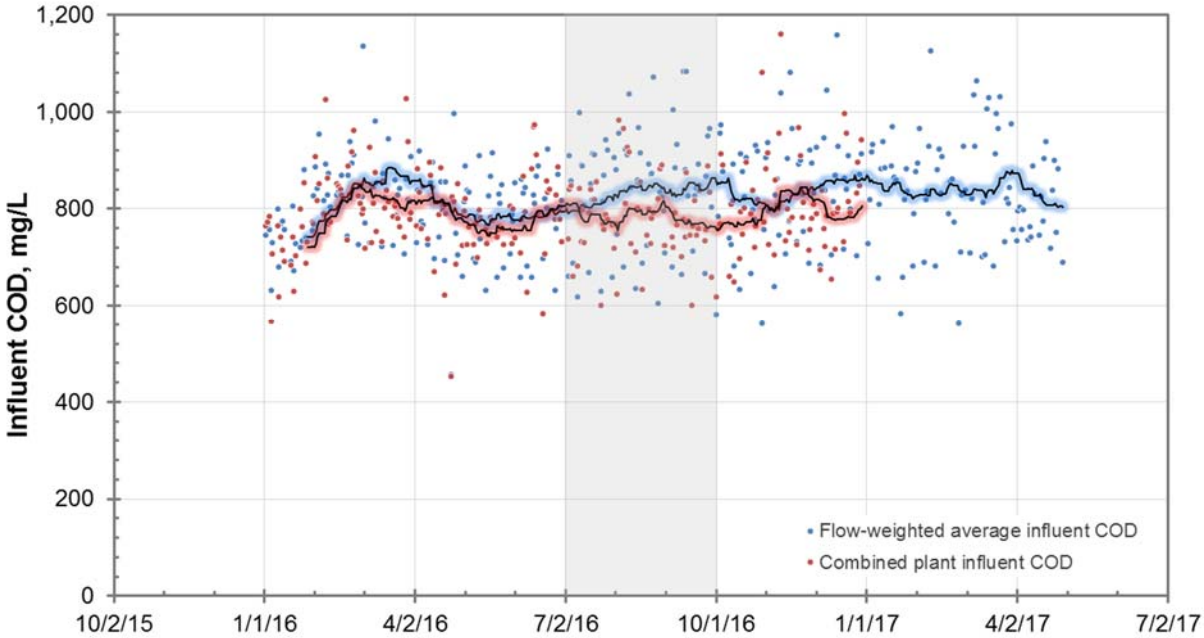



Figure A-11. Influent COD concentration, January 2016 – April 2017

*The flow-weighted average influent concentration is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and COD concentrations. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

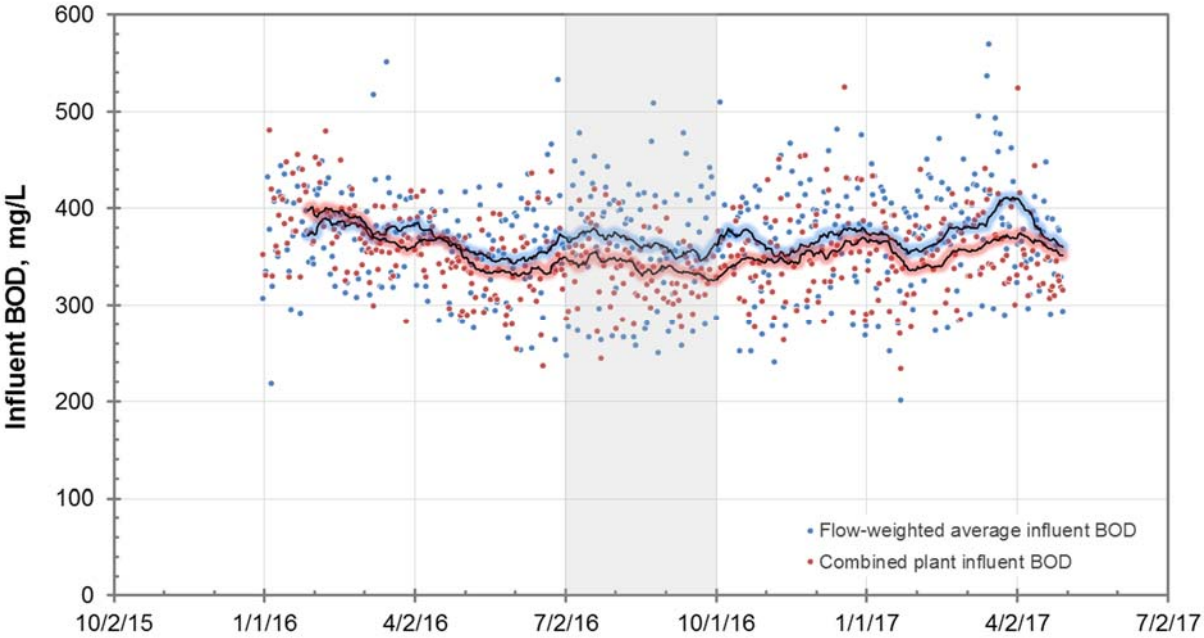



Figure A-12. Influent BOD concentration, January 2016 – April 2017

*The flow-weighted average influent concentration is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and BOD concentrations. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

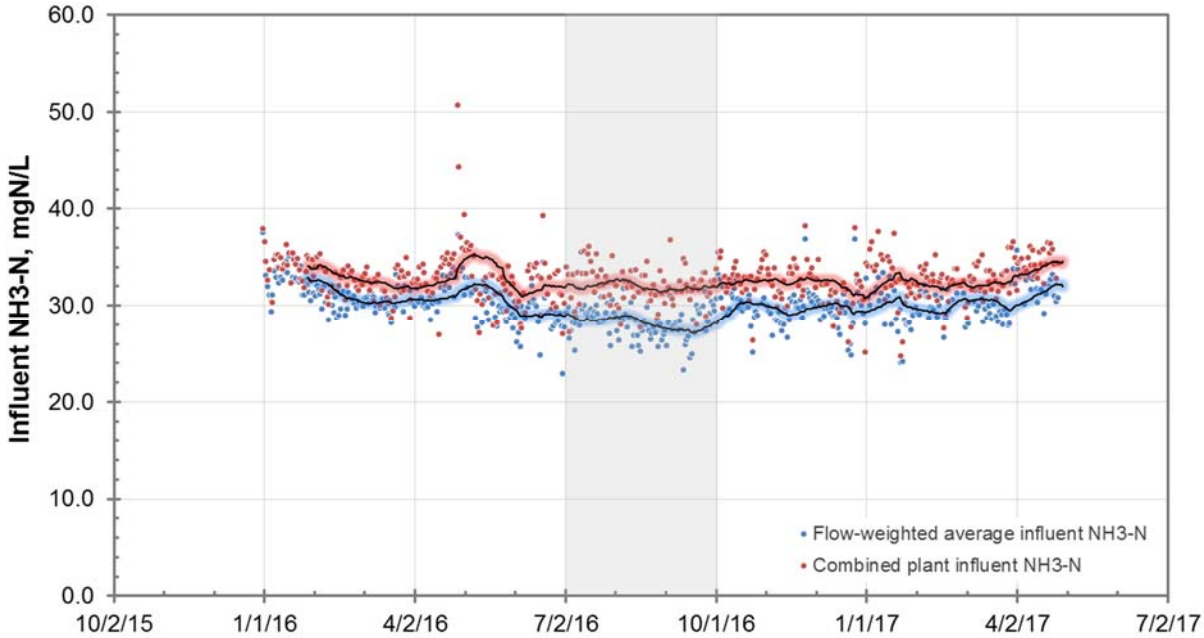



Figure A-13. Influent NH3-N concentration, January 2016 – April 2017

*The flow-weighted average influent concentration is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and NH3-N concentrations. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

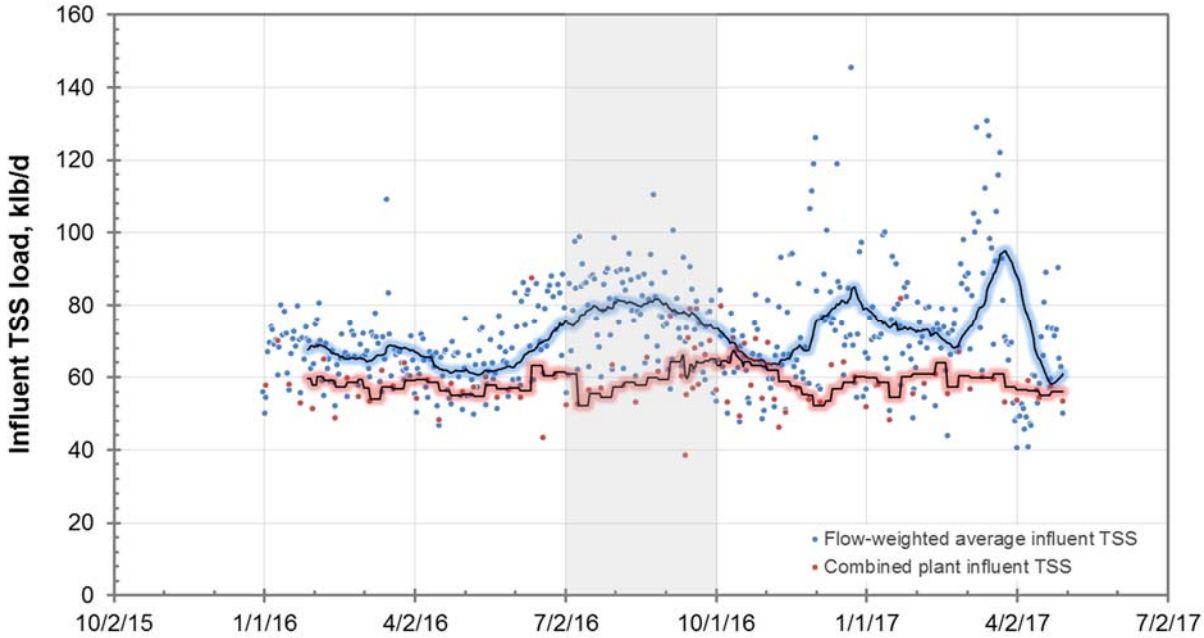



Figure A-14. Influent TSS load, January 2016 – April 2017

*The flow-weighted average influent load is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and TSS concentrations. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

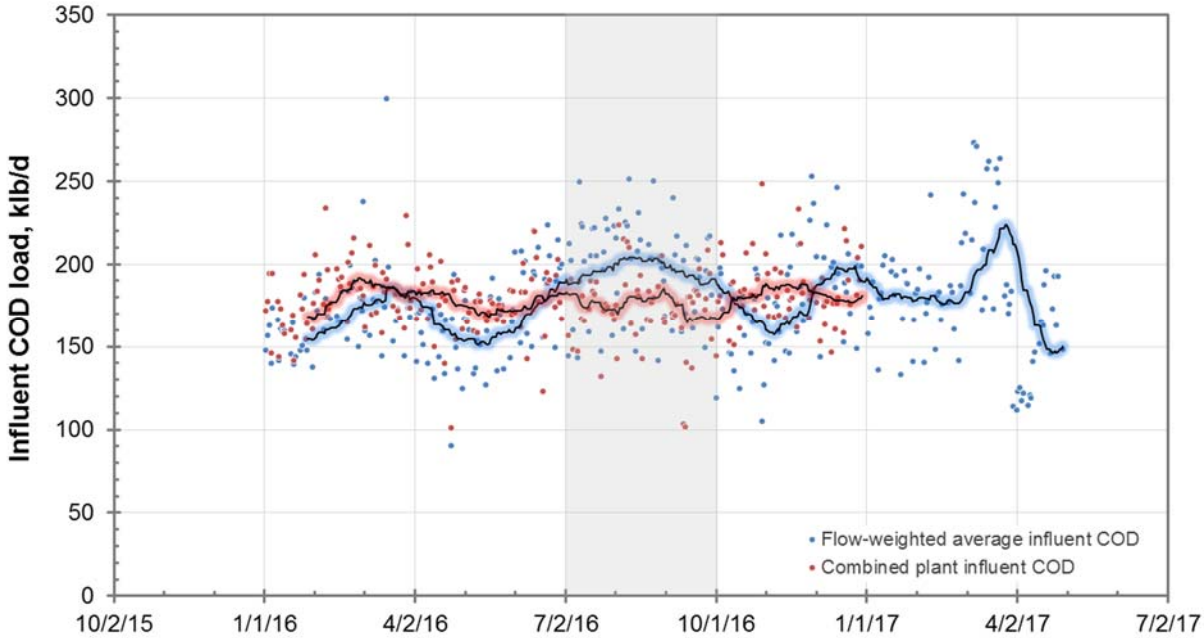



Figure A-15. Influent COD load, January 2016 – April 2017

*The flow-weighted average influent load is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and COD concentrations. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

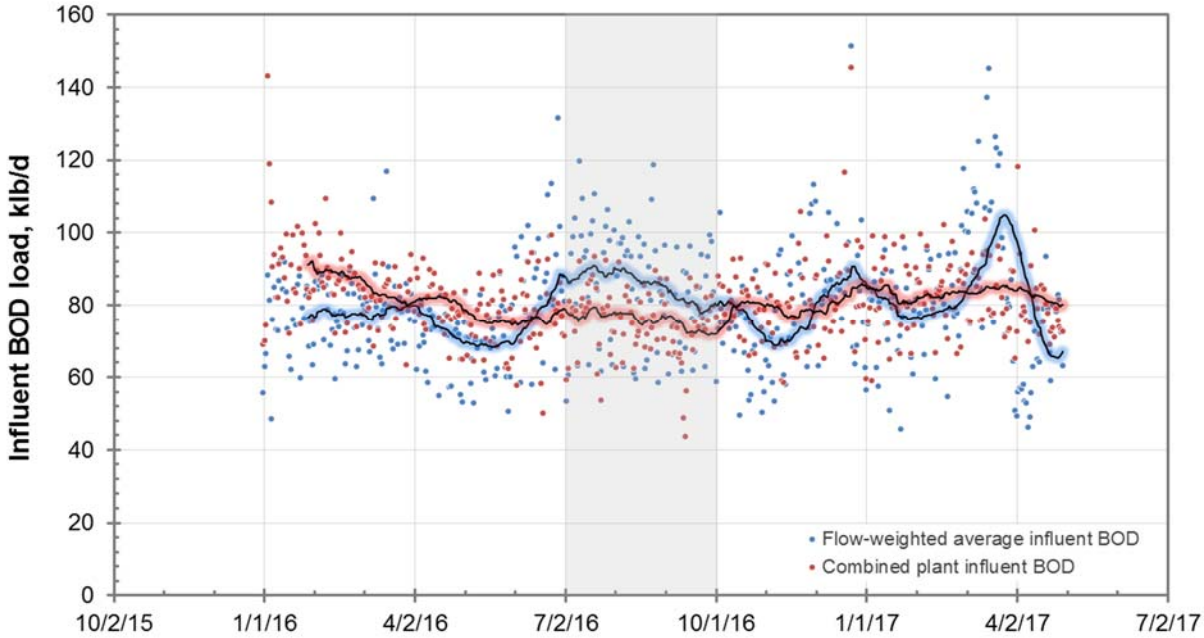



Figure A-16. Influent BOD load, January 2016 – April 2017

*The flow-weighted average influent load is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and BOD concentrations. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

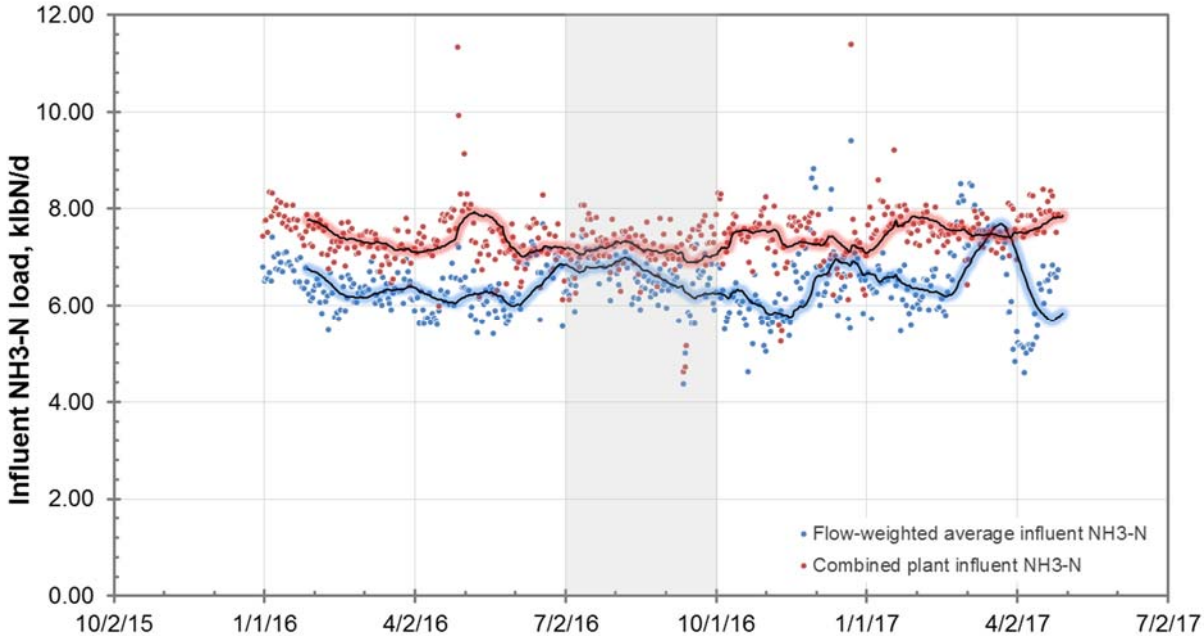



Figure A-17. Influent NH3-N load, January 2016 – April 2017

*The flow-weighted average influent load is calculated using the reported Acorn/Arlanza, Riverside/Hillside, Jurupa, and Rubidoux flow rates and NH3-N concentrations. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

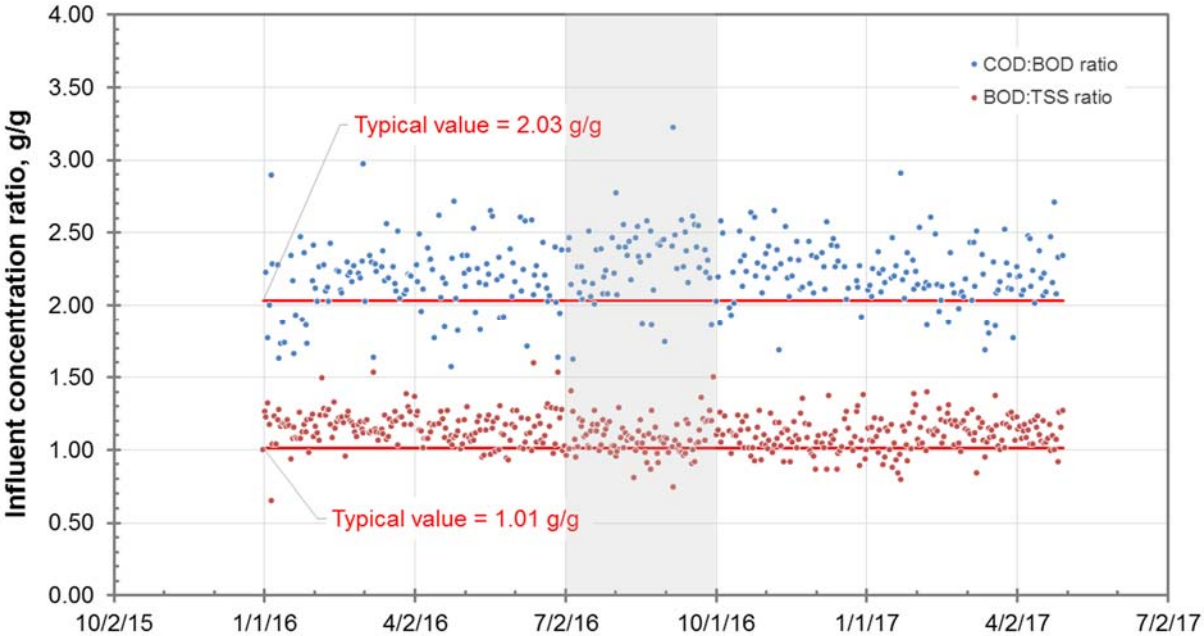



Figure A-18. Influent TSS, COD, and BOD concentration ratios, January 2016 – April 2017

*Ratios are calculated using the flow-weighted average influent concentrations. The COD:BOD ratio of 2.03 g/g and BOD:TSS ratio of 1.01 g/g is based on "typical" COD, BOD, and TSS concentrations of 500, 246, and 243 mg/L, respectively. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

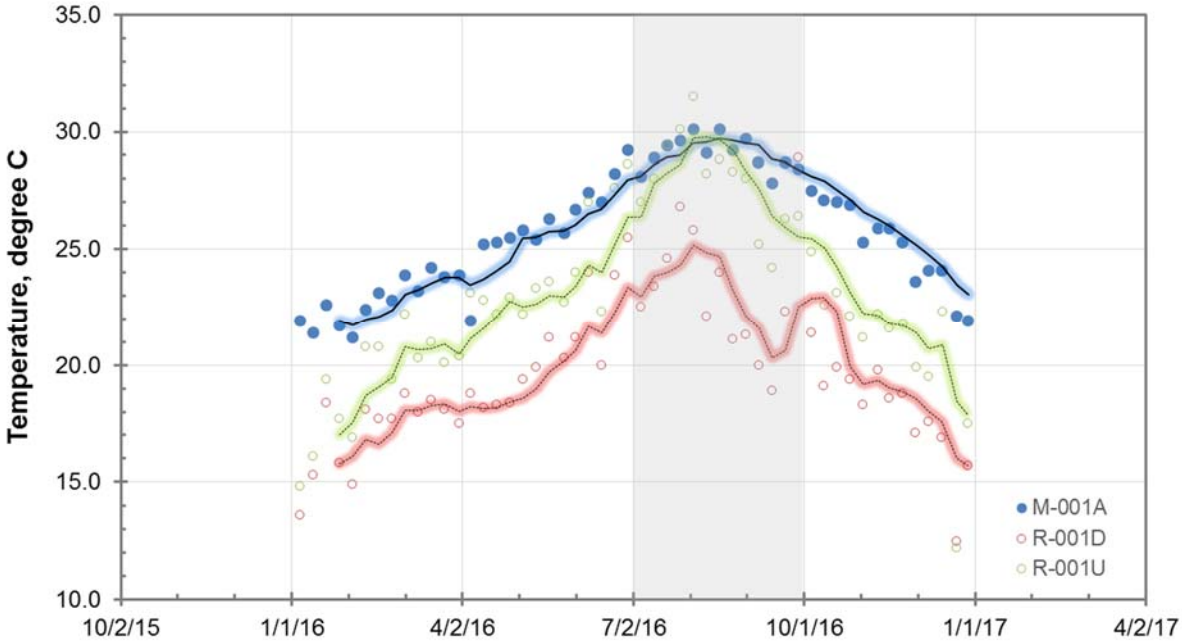



Figure A-19. Effluent temperature, January – December 2016

*M-001A is measured at the end of the effluent pipeline. R-001U and R-001D are measured in the Santa Ana River upstream and downstream of the plant discharge. The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

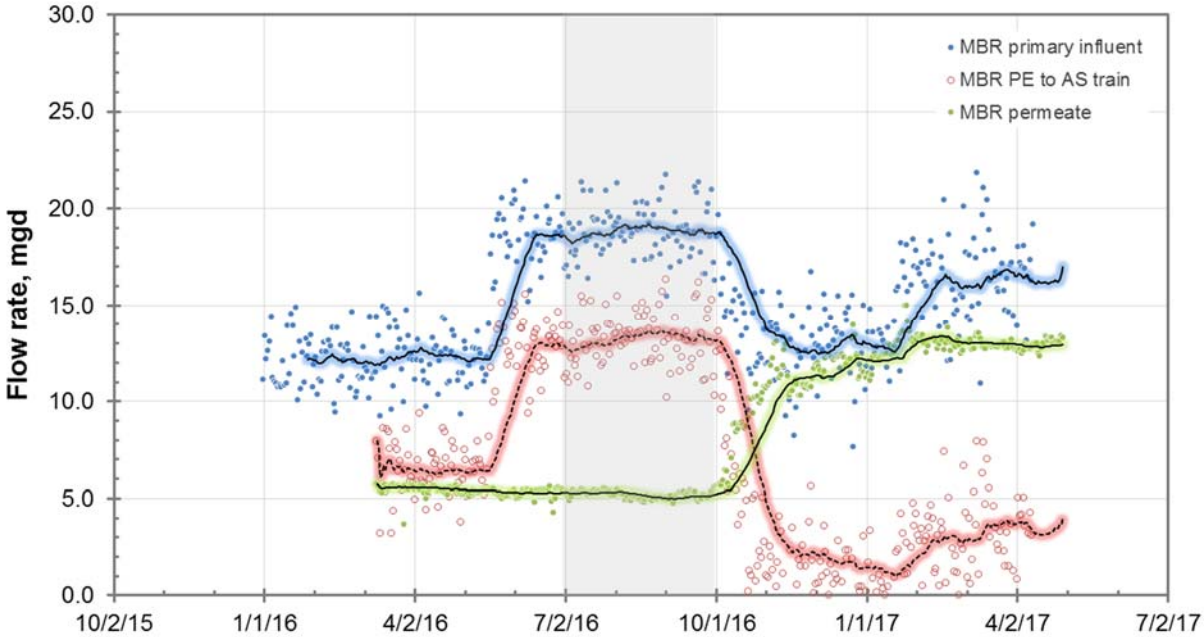



Figure A-20. MBR treatment train flow rates, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

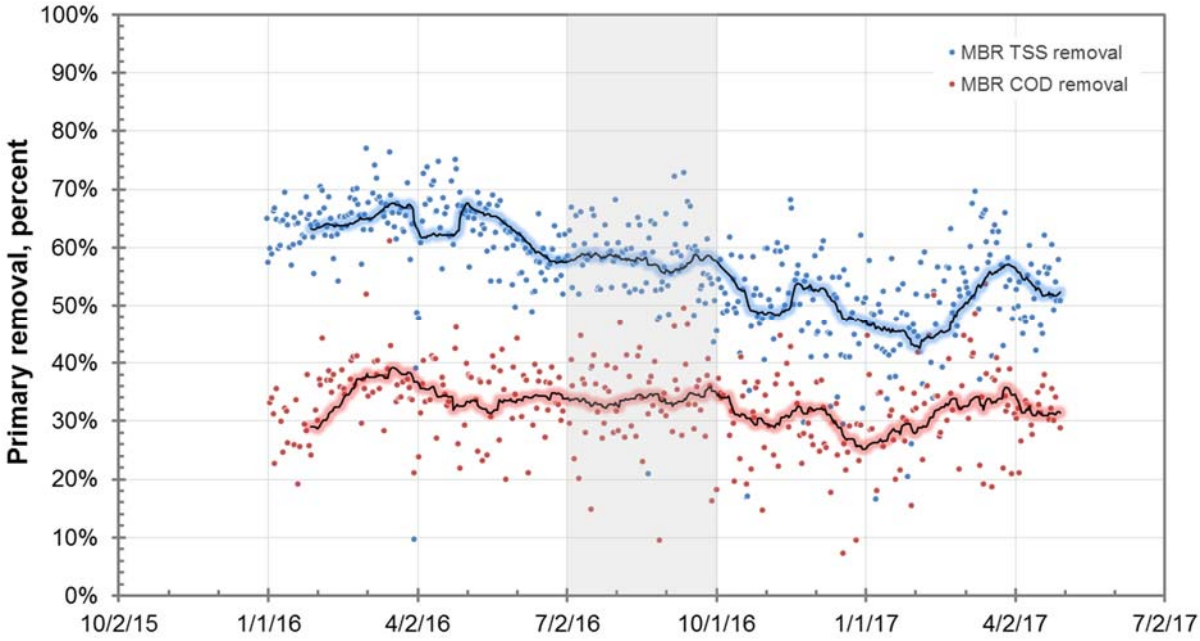



Figure A-21. MBR treatment train primary removals, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

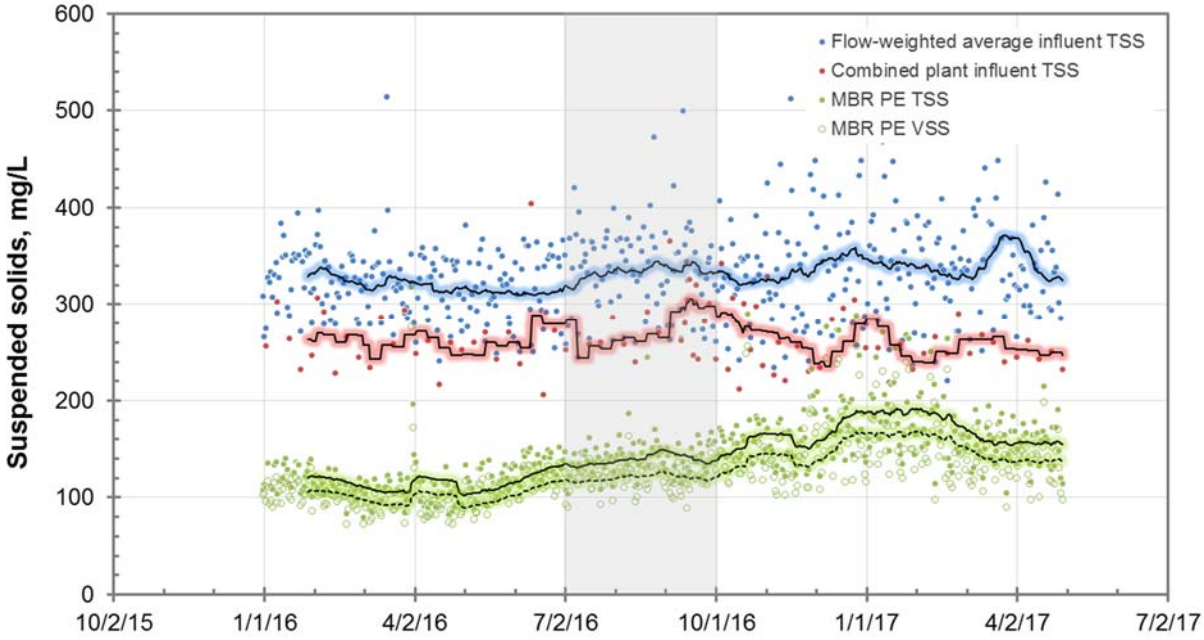



Figure A-22. MBR treatment train suspended solids concentration profile, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

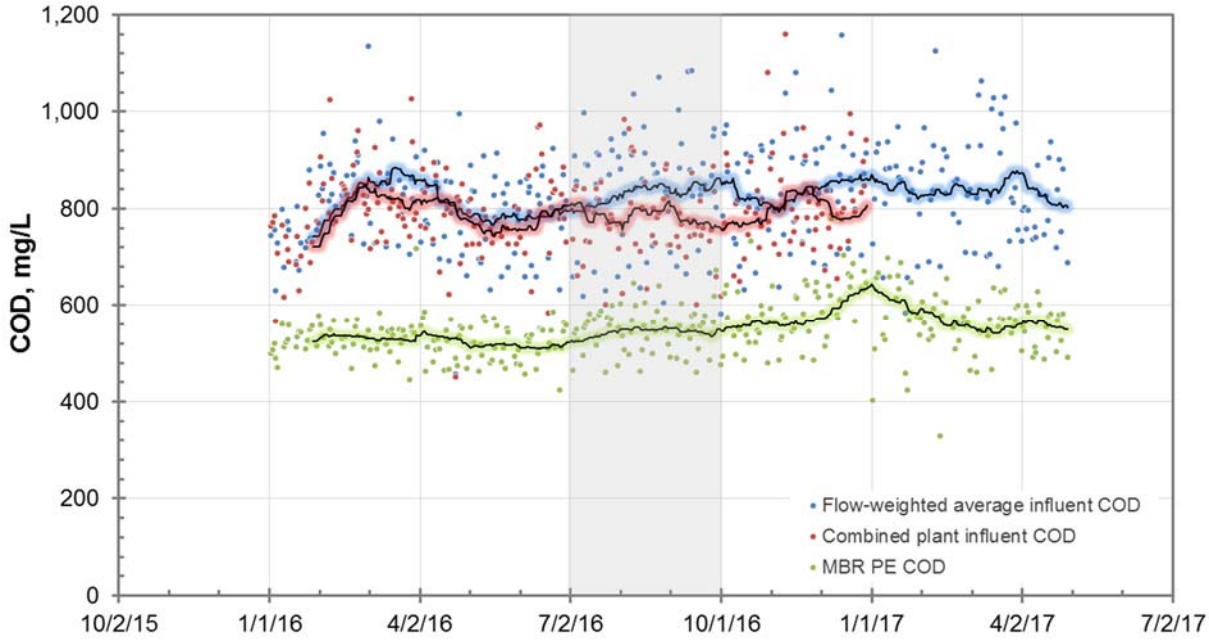



Figure A-23. MBR treatment train COD concentration profile, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

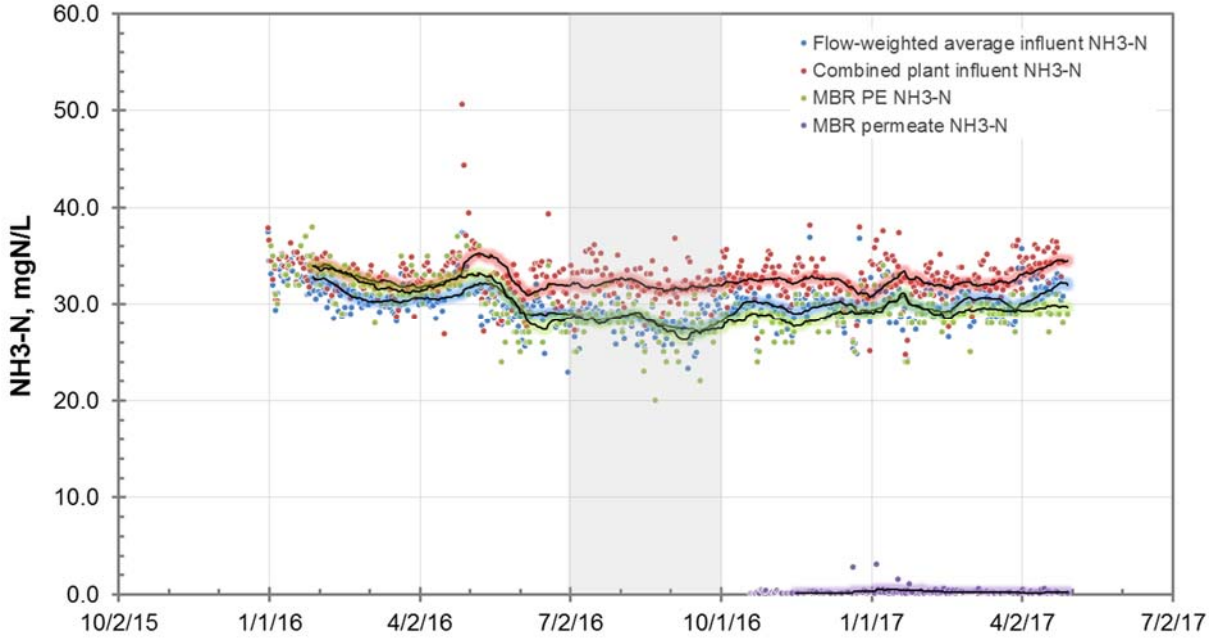



Figure A-24. MBR treatment train NH3-N concentration profile, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

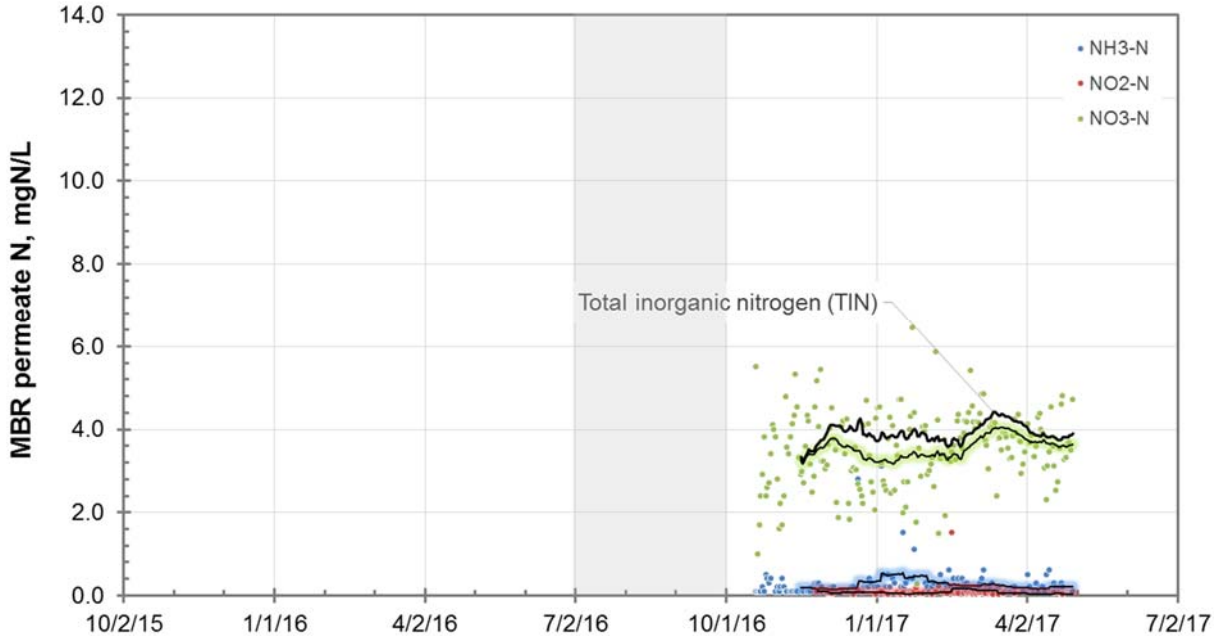



Figure A-25. MBR permeate nitrogen concentration, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

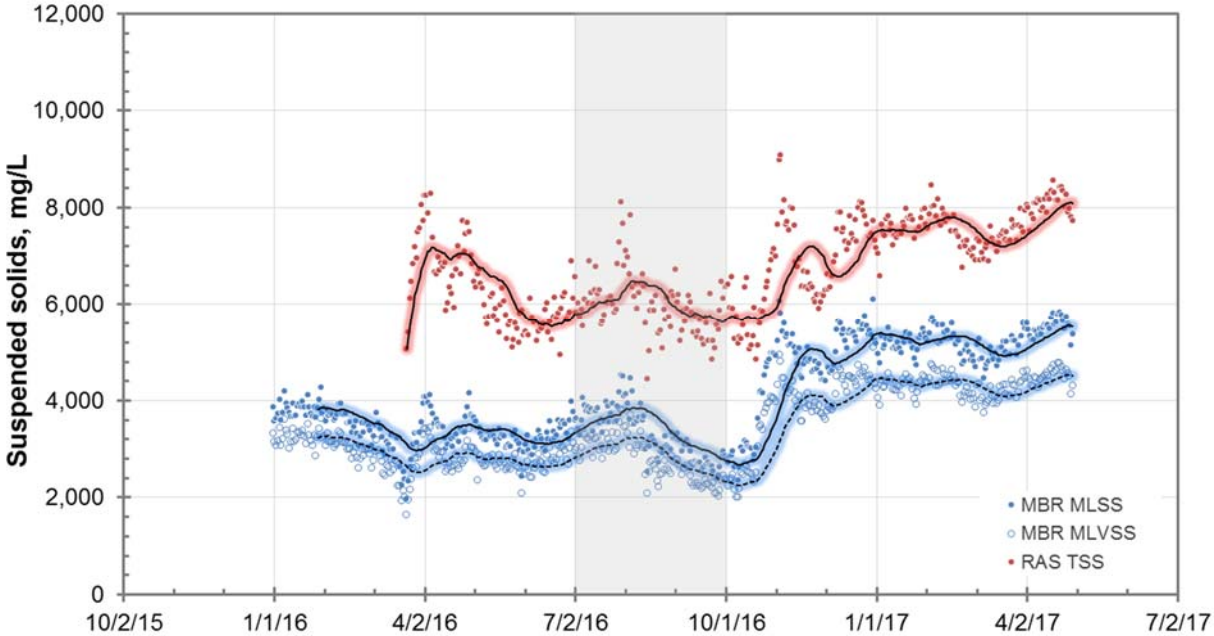



Figure A-26. MBR treatment train mixed liquor and return sludge suspended solids, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

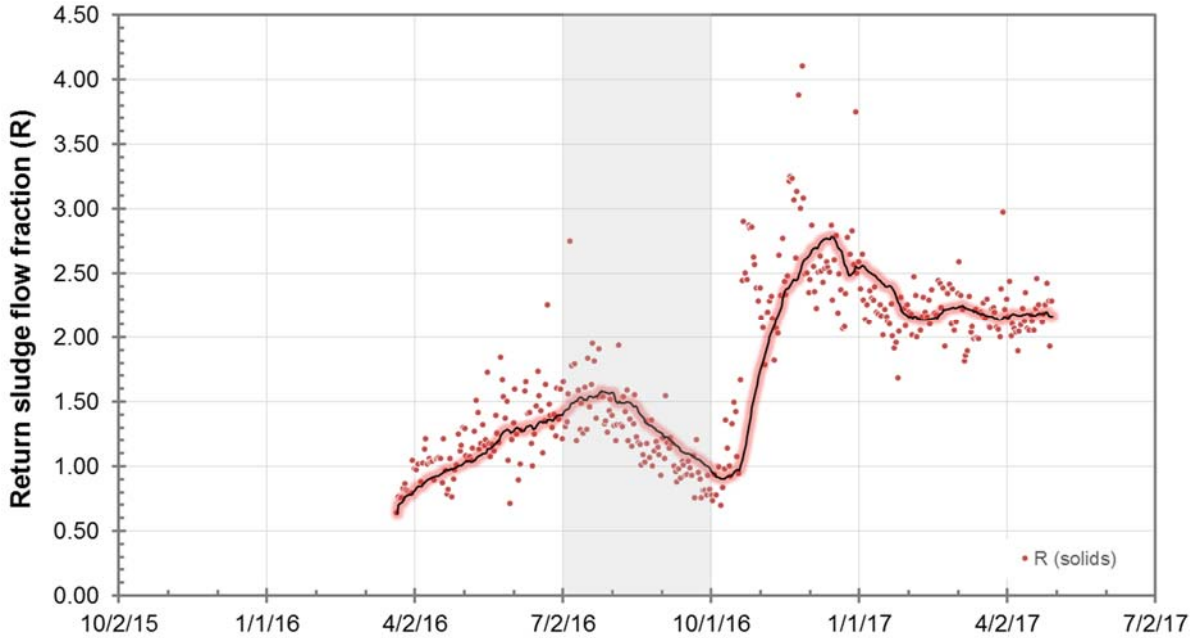



Figure A-27. MBR treatment train return sludge flow fraction, January 2016 – April 2017

*The ratio of the return sludge and mixed liquor TSS concentrations is used to calculate "R (solids)". The highlighted trendline represents a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

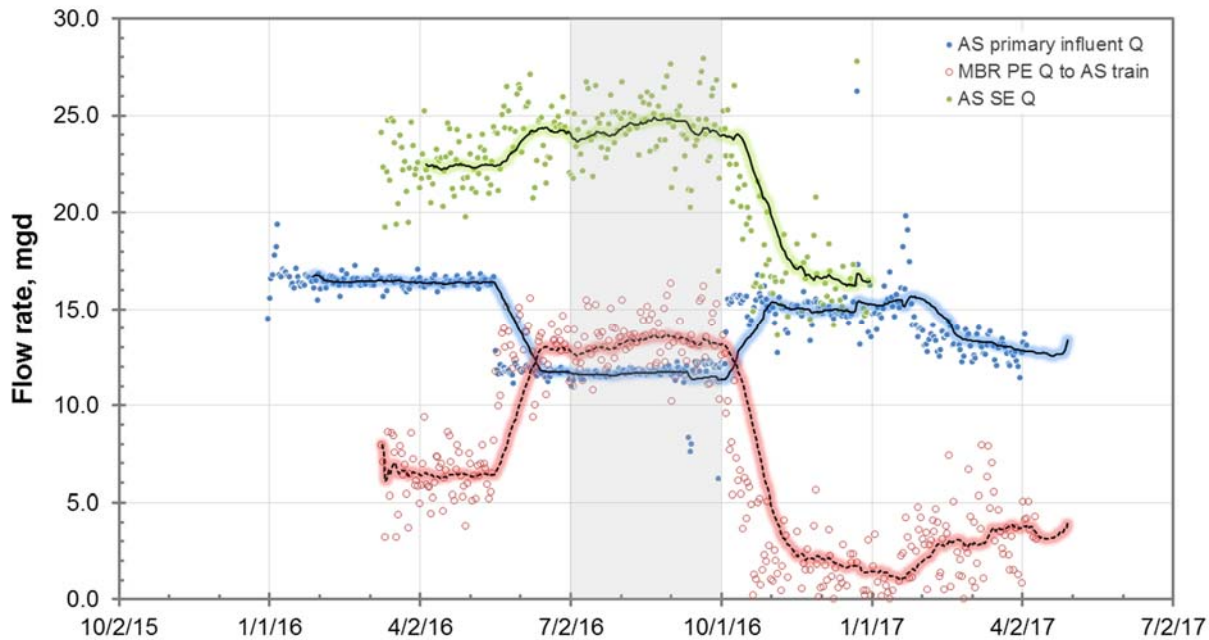



Figure A-28. Activated sludge treatment train flow rates, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

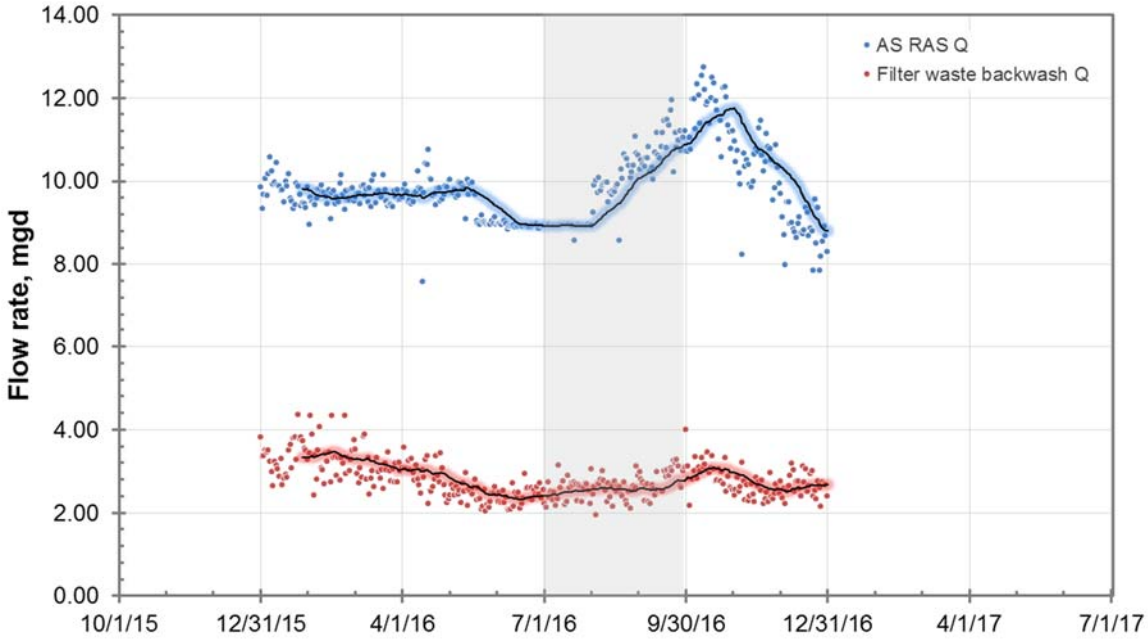



Figure A-29. Activated sludge treatment train return sludge and filter waste backwash flow rates, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

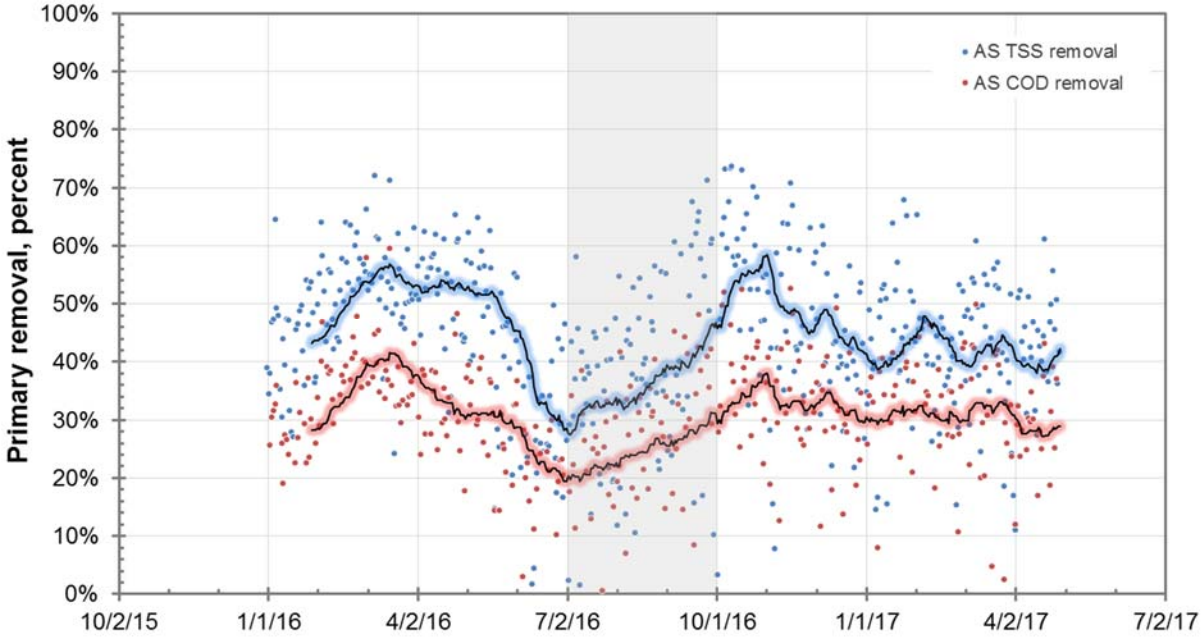



Figure A-30. Activated sludge treatment train primary removals, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

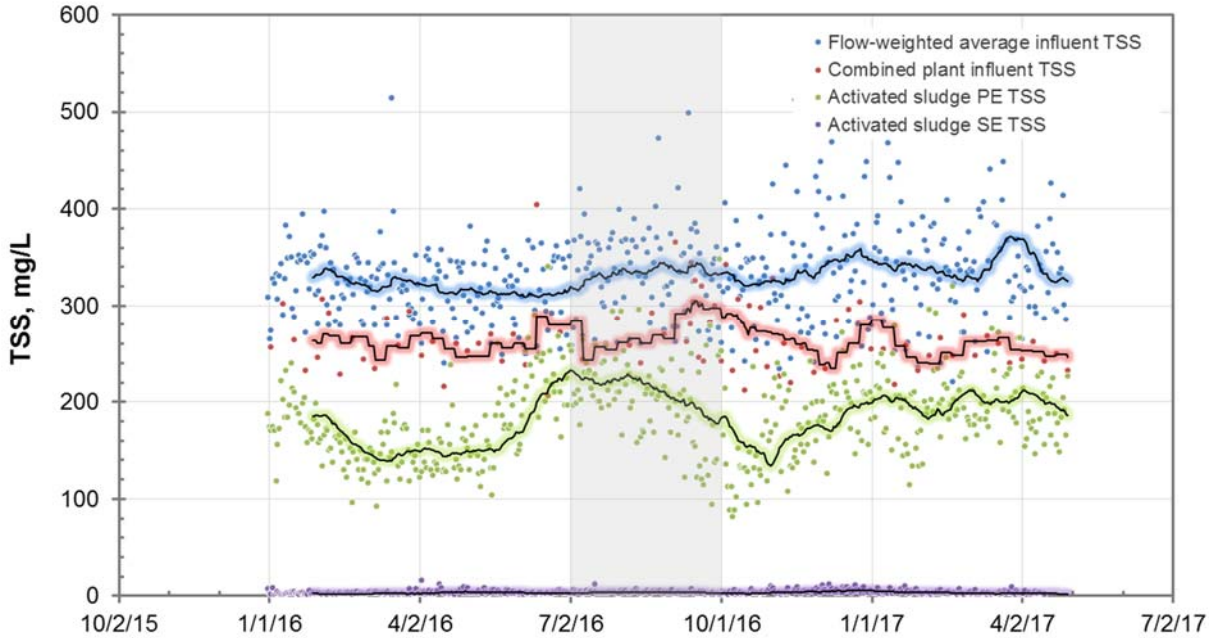



Figure A-31. Activated sludge treatment train TSS concentration profile, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

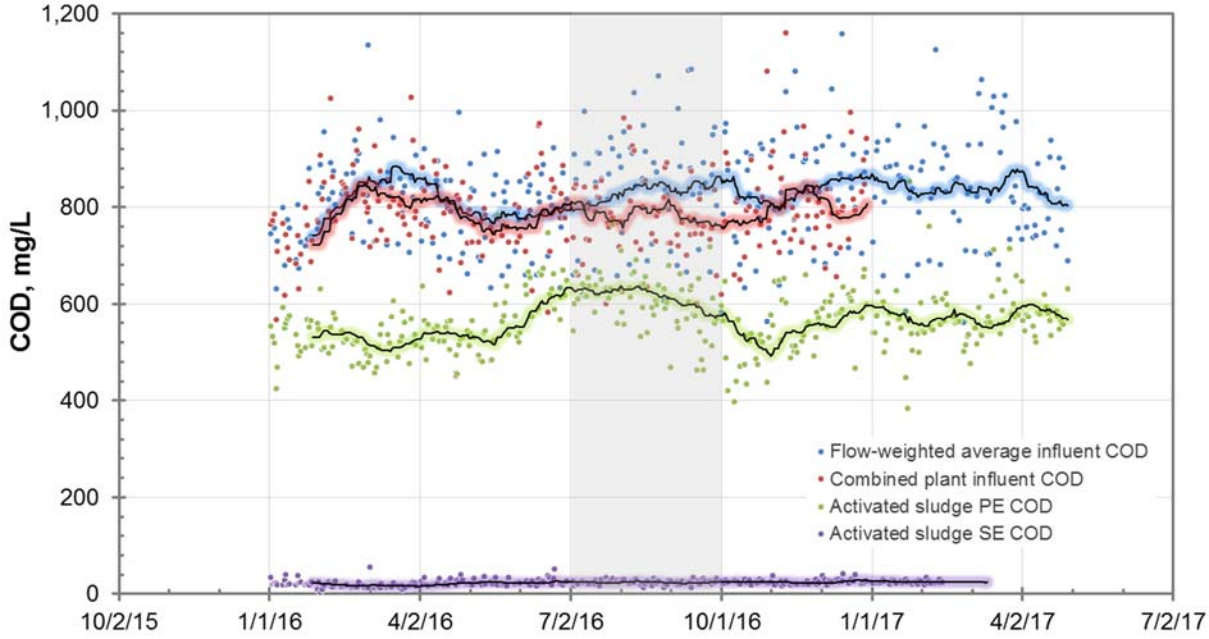



Figure A-32. Activated sludge treatment train COD concentration profile, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

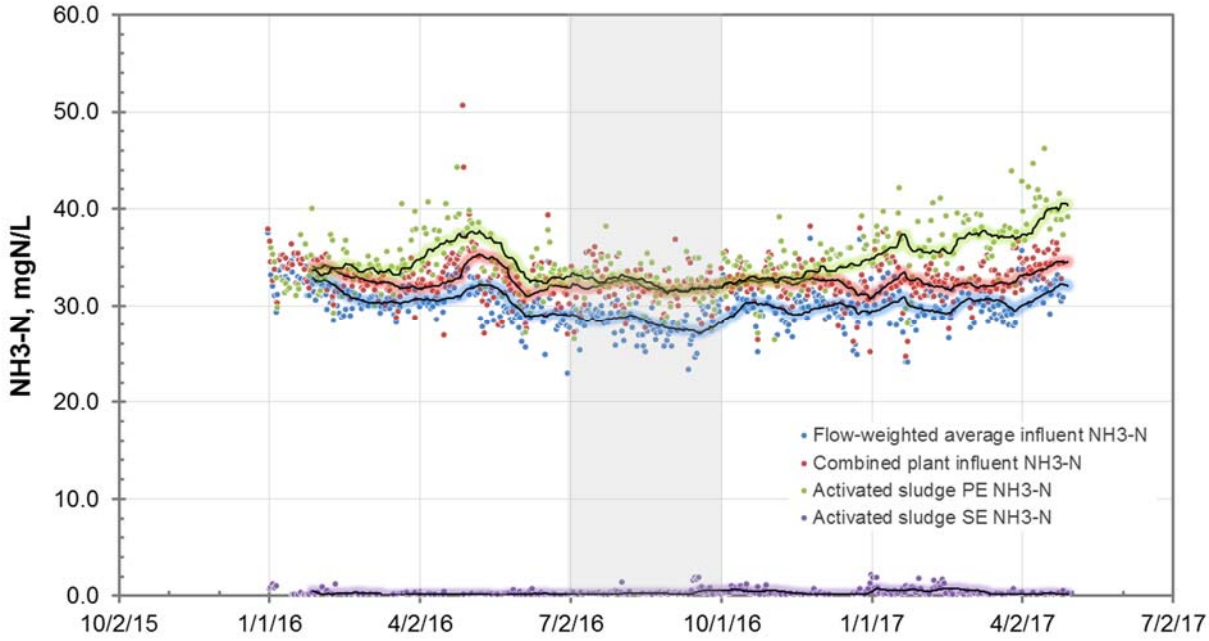



Figure A-33. Activated sludge treatment train NH3-N concentration profile, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

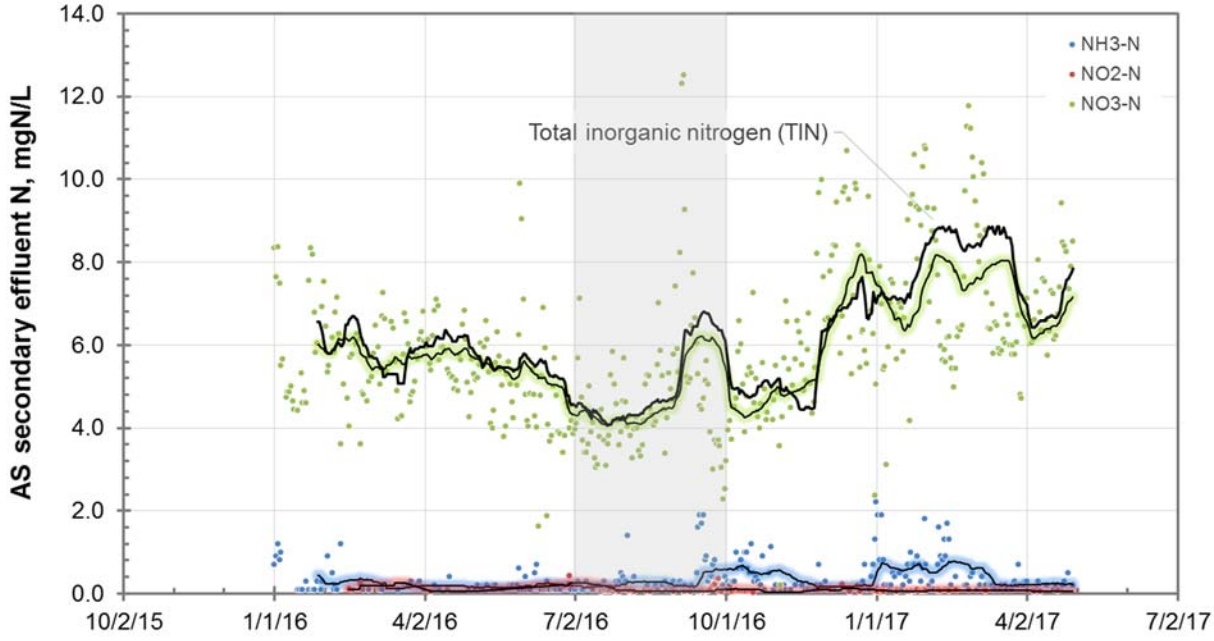



Figure A-34. Activated sludge treatment train secondary effluent nitrogen concentration, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

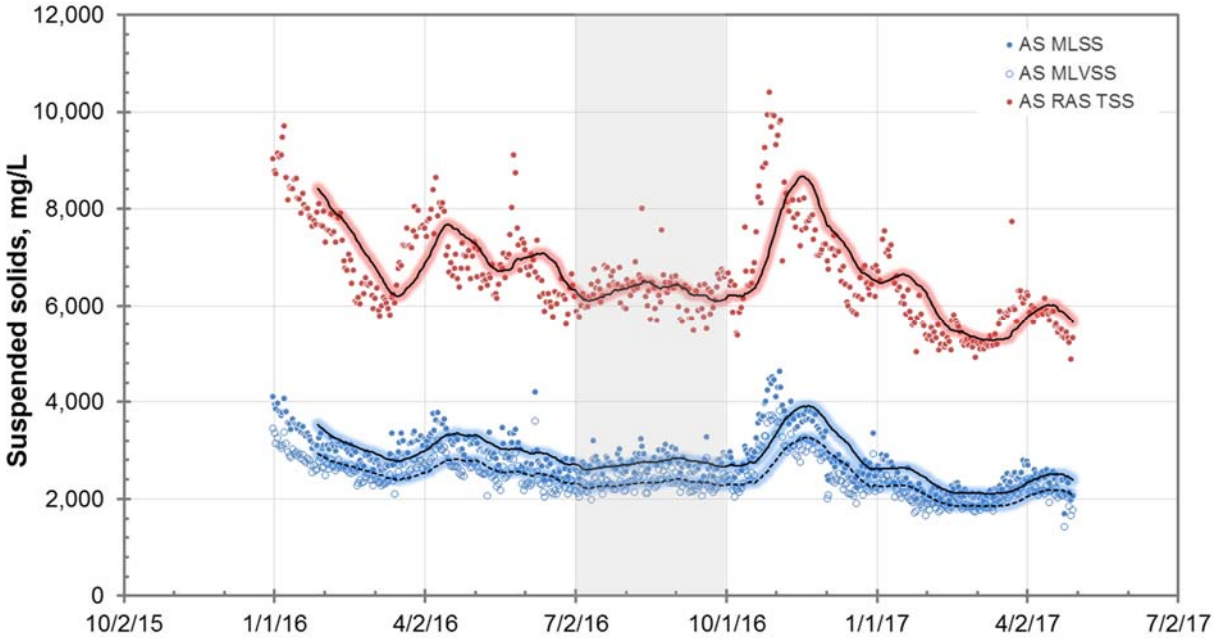



Figure A-35. Activated sludge treatment train mixed liquor and return sludge suspended solids concentration, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

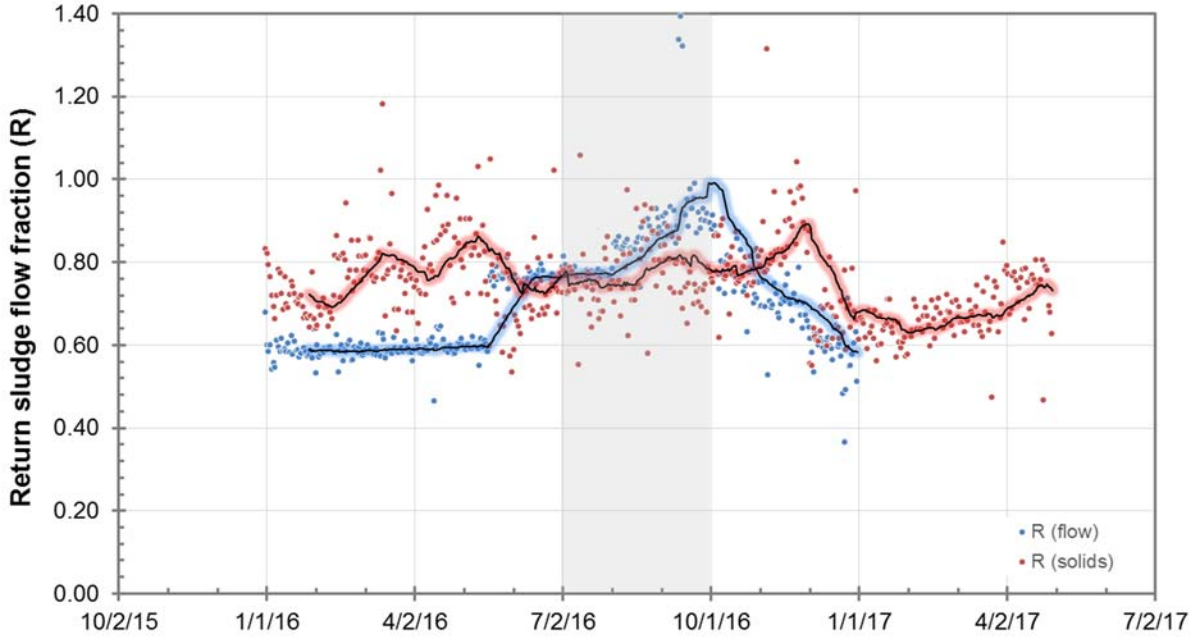



Figure A-36. Activated sludge treatment train return sludge flow fraction, January 2016 – April 2017

*The ratio of the return sludge and mixed liquor TSS concentrations is used to calculate "R (solids)". The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

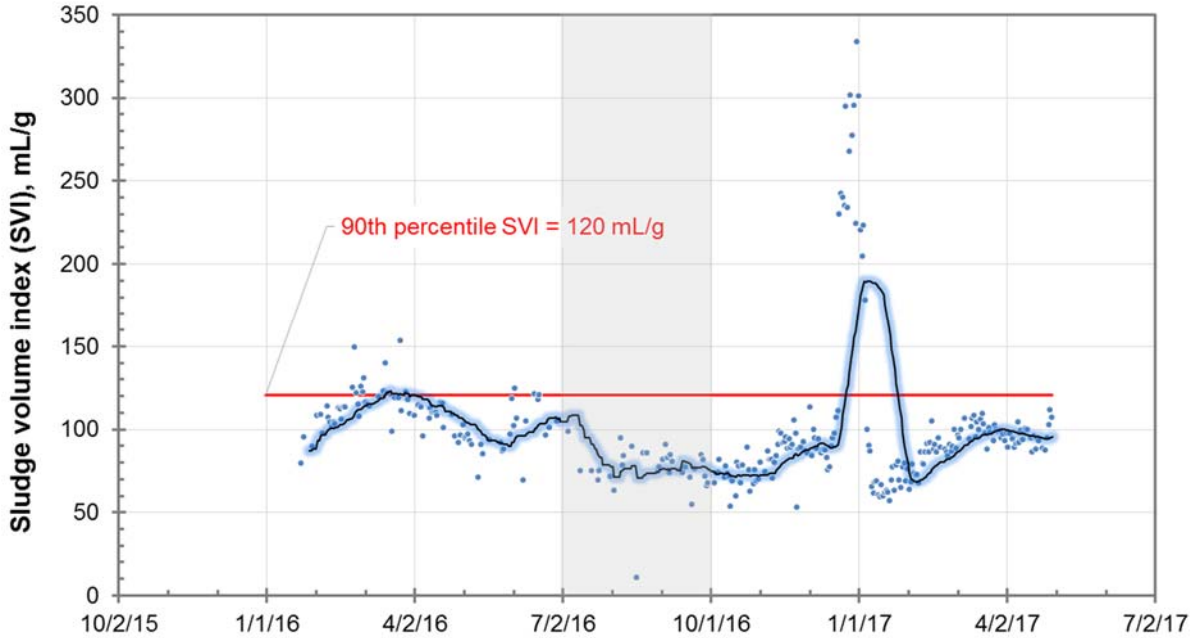



Figure A-37. Activated sludge treatment train sludge volume index, January 2016 – April 2017

*The highlighted trendline represents a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

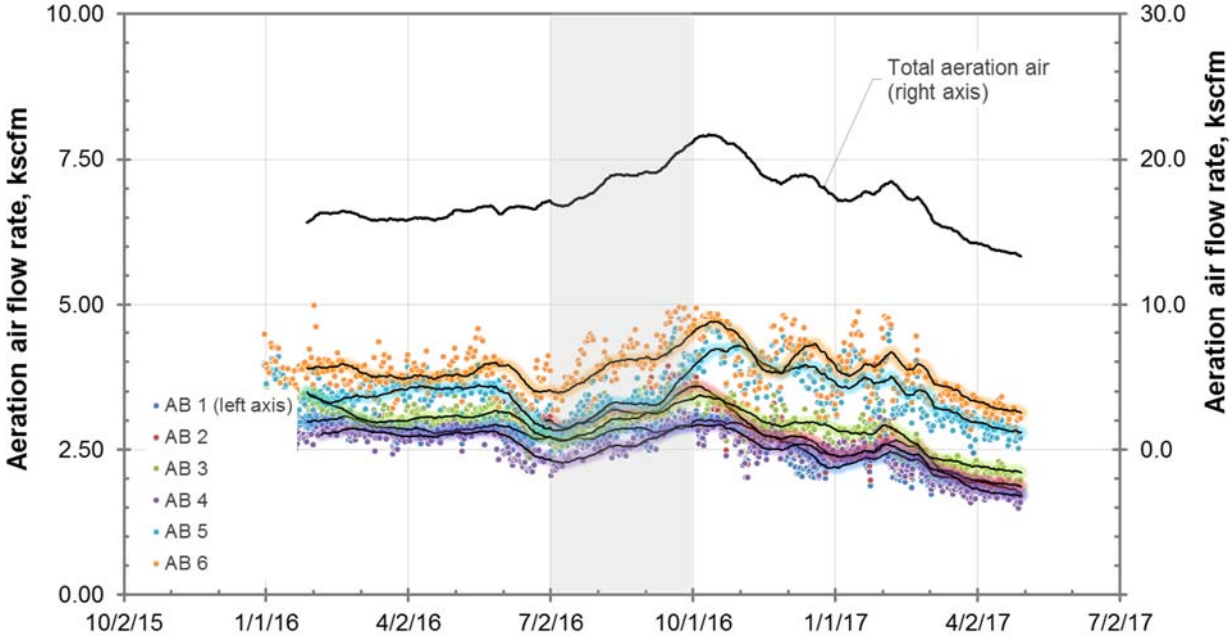



Figure A-38. Activated sludge treatment train aeration air flow rate, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30). All six aeration basins were in service during this period.*

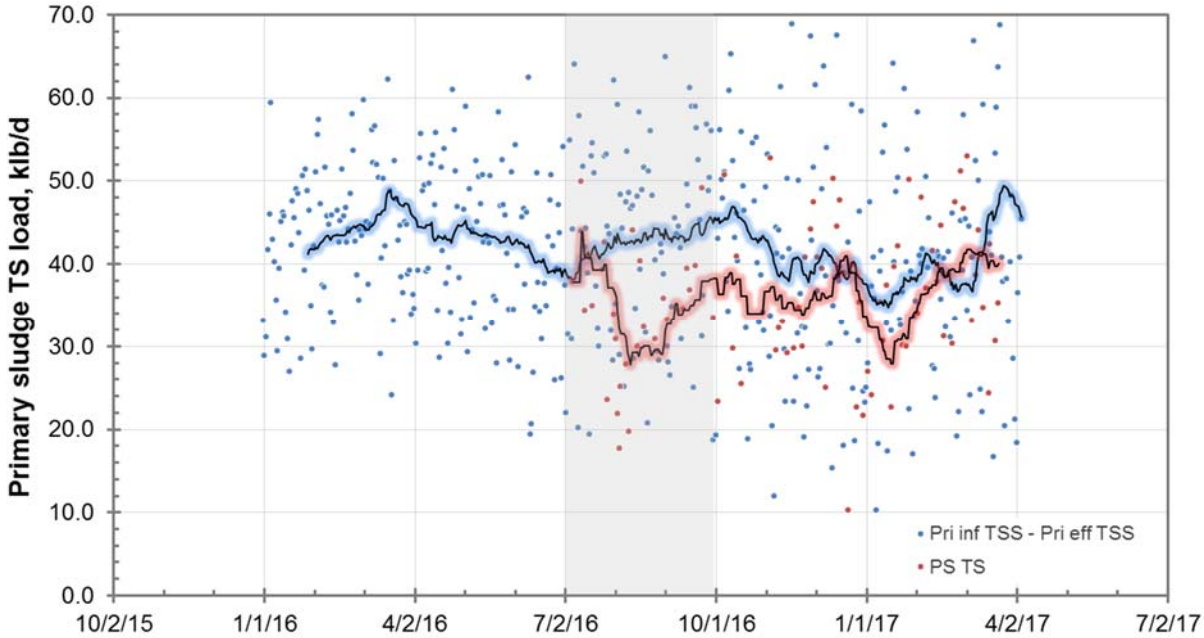



Figure A-39. Primary sludge TS load, January 2016 – April 2017

*The digester feed TS load minus the TWAS TS load is represented as "PS TS". The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

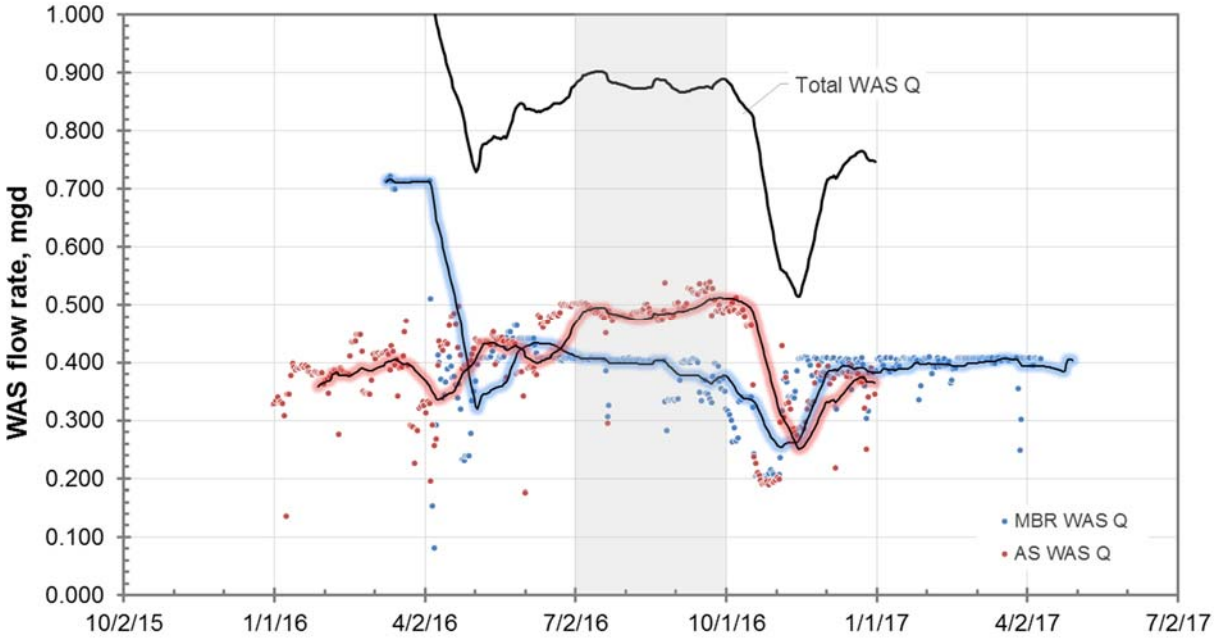



Figure A-40. WAS flow rate, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

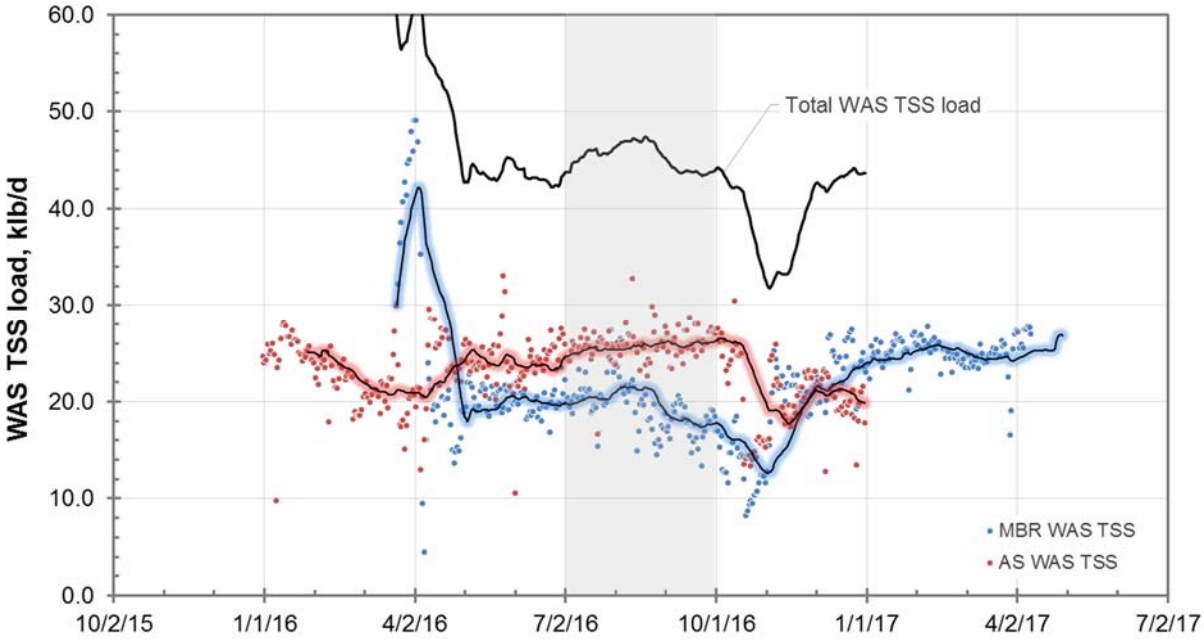



Figure A-41. WAS TSS load, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

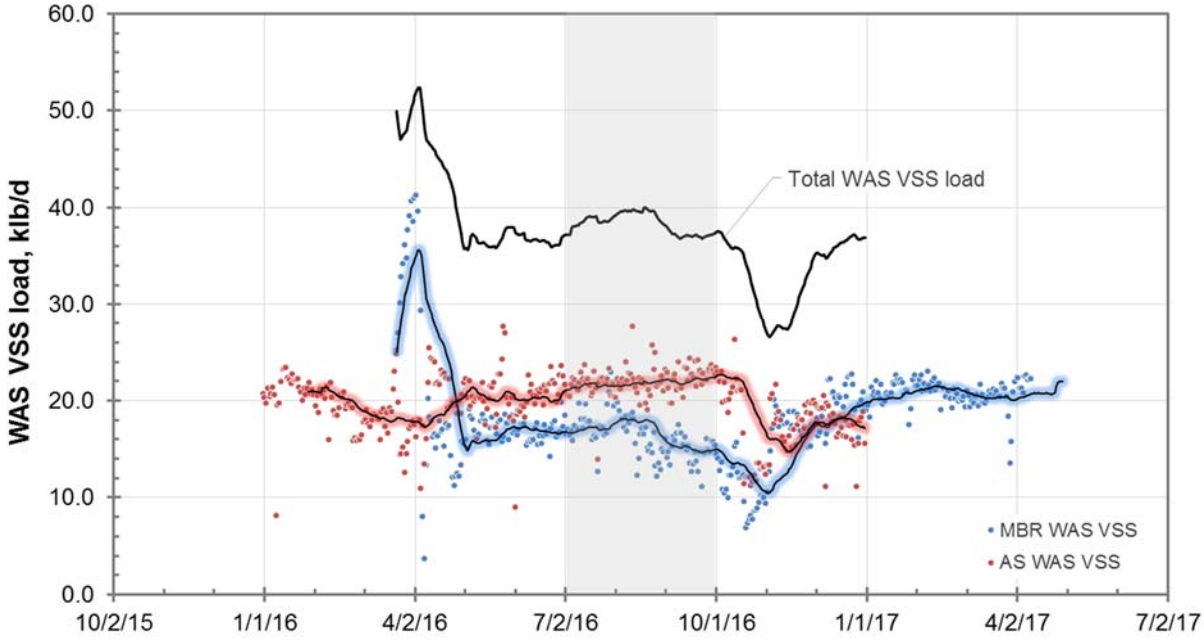



Figure A-42. WAS VSS load, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

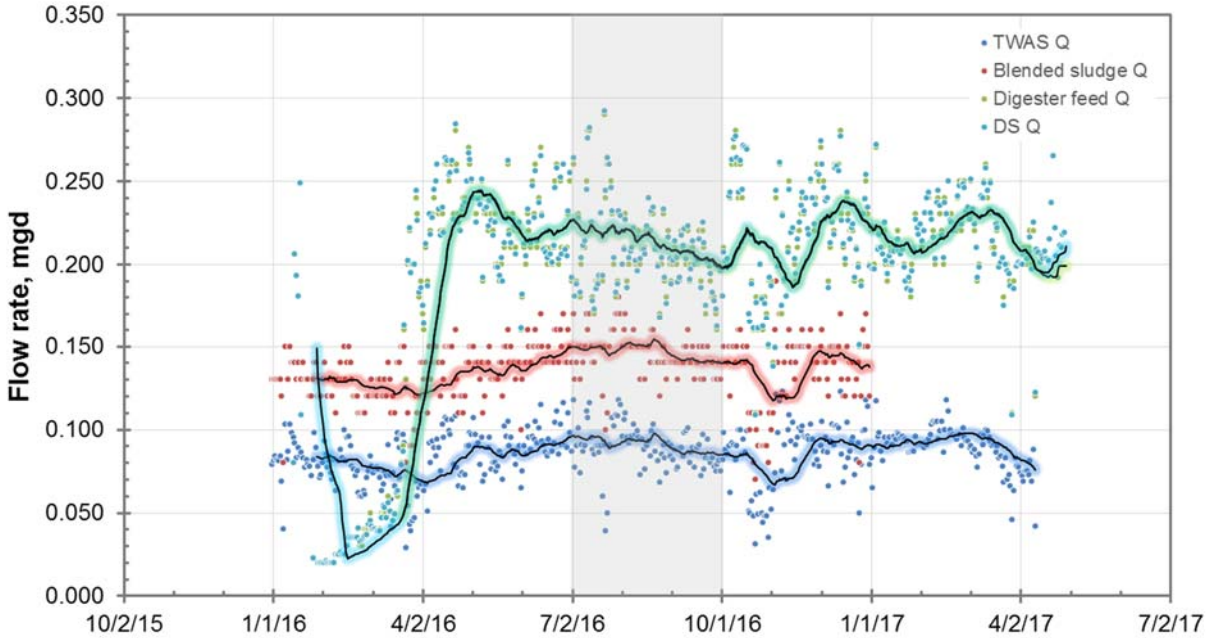



Figure A-43. Solids handling flow rates, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

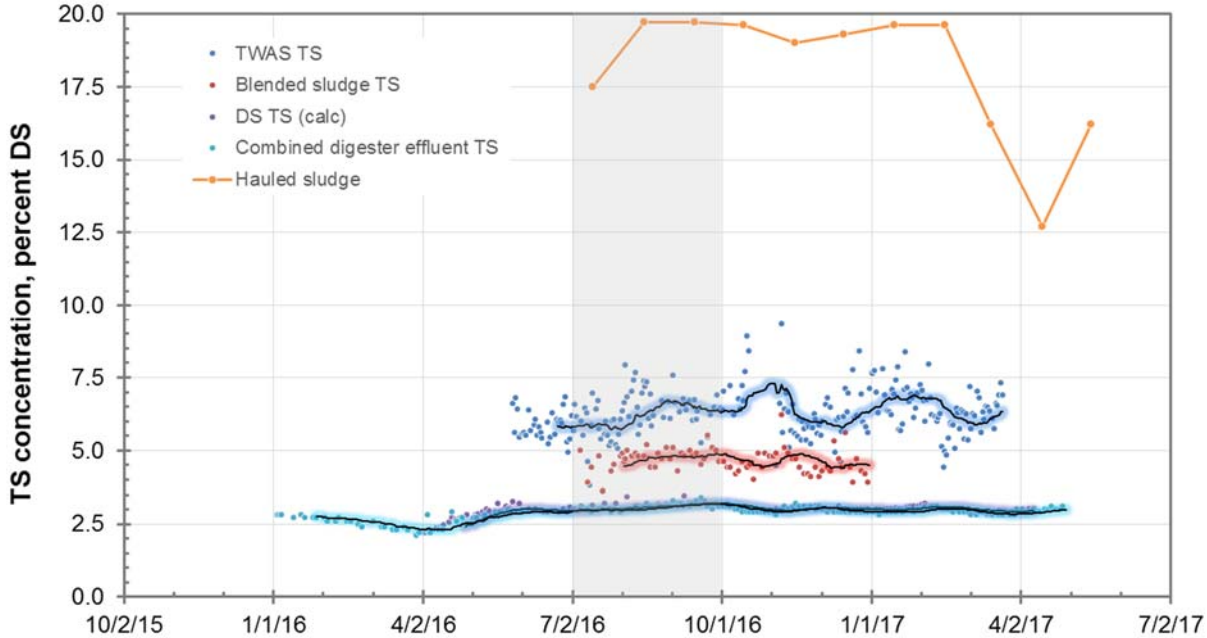



Figure A-44. Solids handling total solids concentration, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

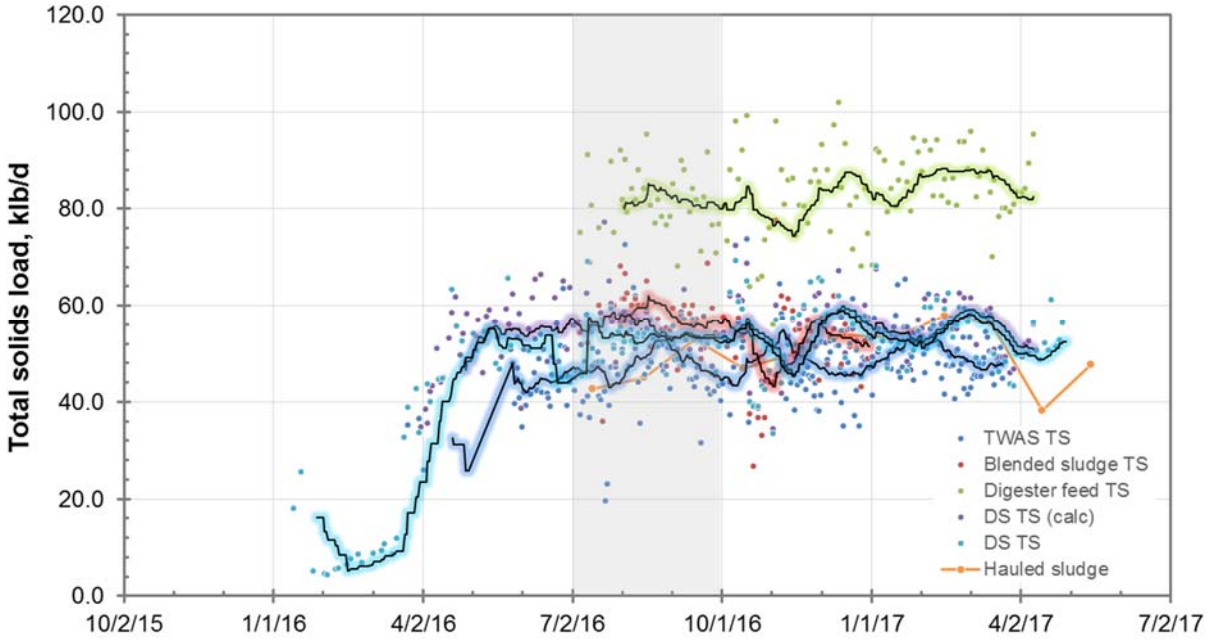



Figure A-45. Solids handling total solids load, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

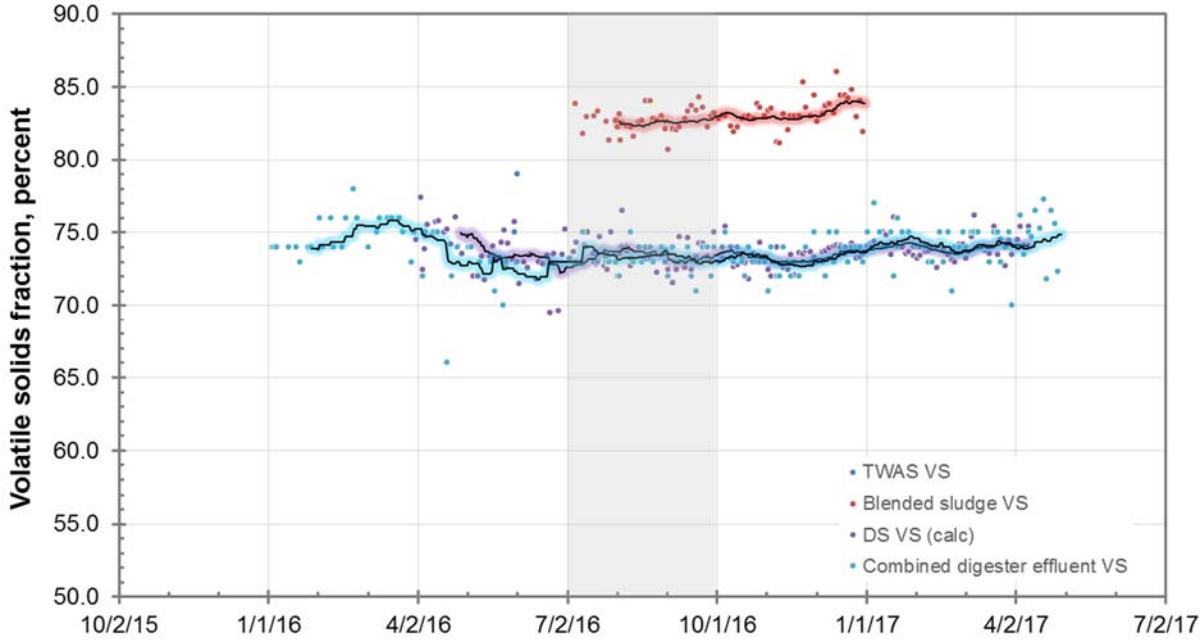



Figure A-46. Solids handling volatile solids fraction, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

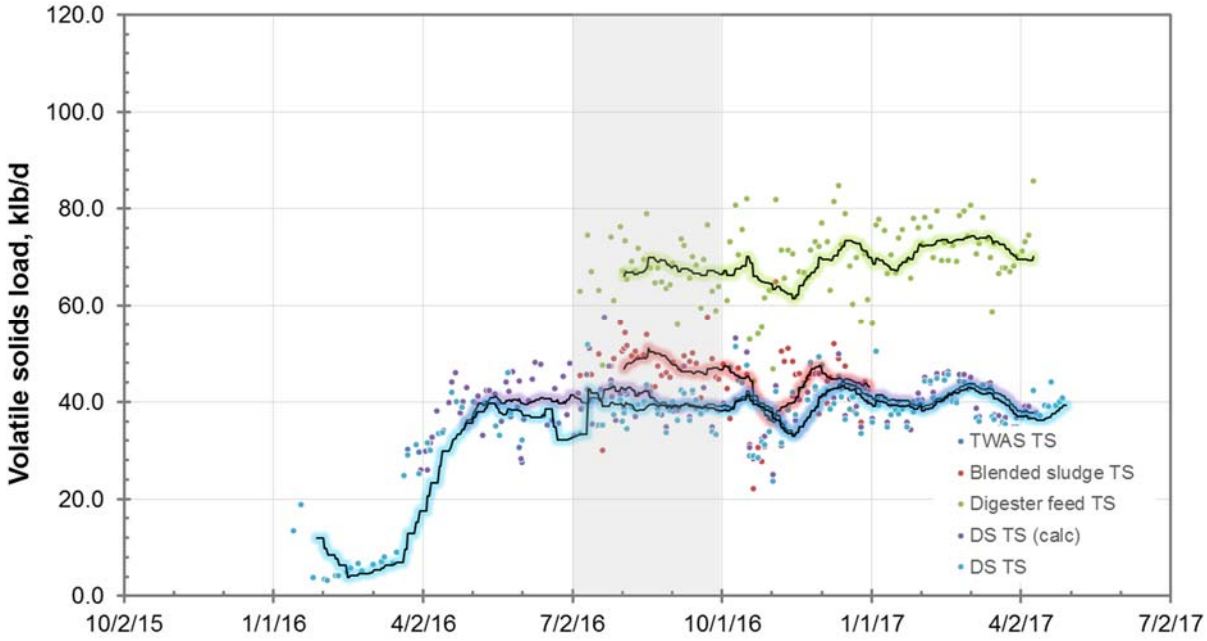



Figure A-47. Solids handling volatile solids load, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average. The shaded area indicates data used for process model calibration (July 1 – September 30).*

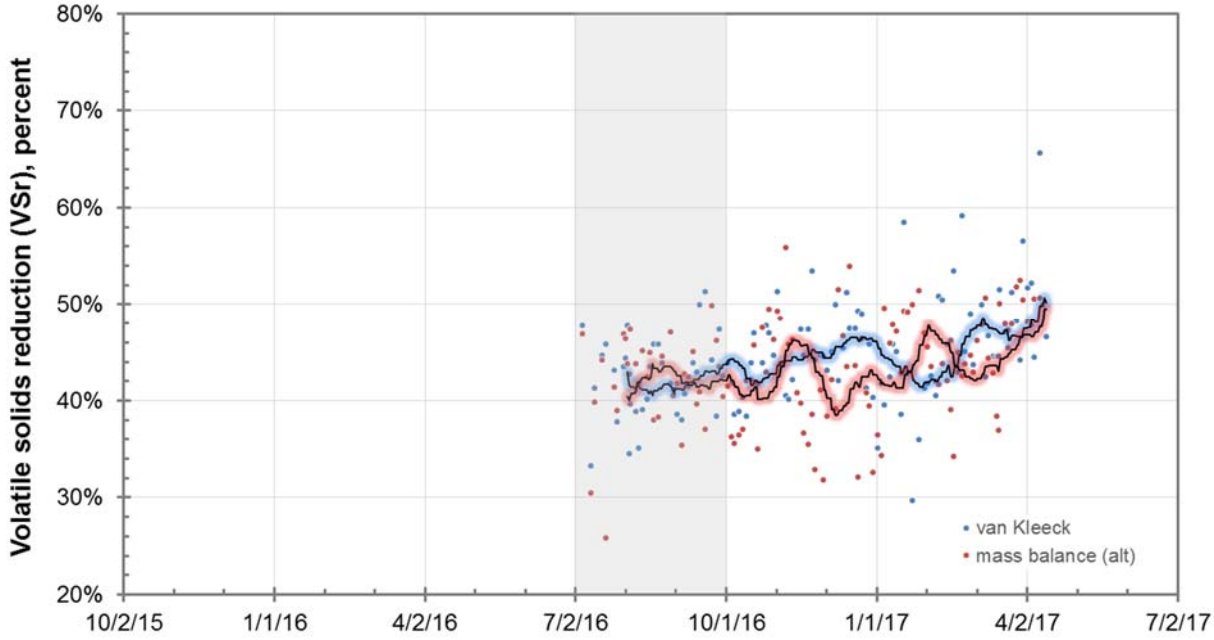



Figure A-48. Anaerobic digester volatile solids reduction, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

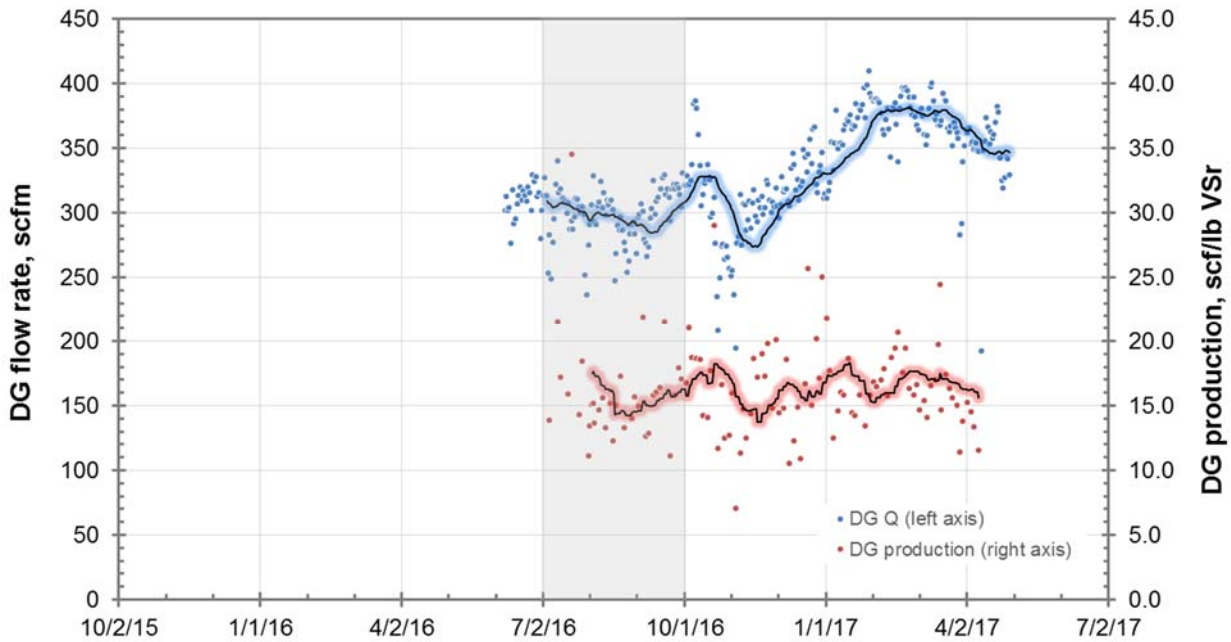



Figure A-49. Digester gas production, January 2016 – April 2017

*The highlighted trendlines represent a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

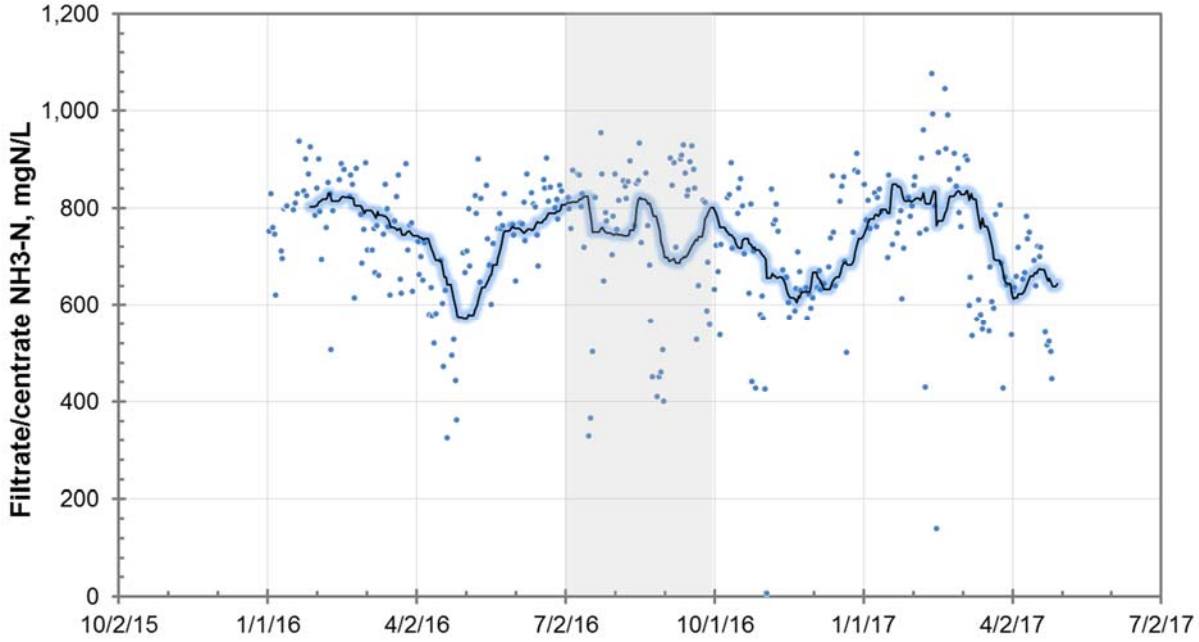



Figure A-50. Solids dewatering recycle ammonia concentration, January 2016 – April 2017

*The highlighted trendline represents a 28-day moving average.*
*The shaded area indicates data used for process model calibration (July 1 – September 30).*

Appendix 3B

# PROCESS MODEL CALIBRATION SUMMARY

**Table B-1. BioWin calibration summary**

| | RWQCP data[1] | BioWin calibration[2] | Difference | Notes |
|---|---|---|---|---|
| **Plant influent** | | | | |
| Flow rate, mgd | 27.9 | 27.9 | --[3] | *Influent flow rate set to reported operations and performance data* |
| TSS, mg/L | 336 | 341 | 1% | |
| VSS, mg/L | -- | 305 | -- | |
| ISS[4], mg/L | -- | 35 | --[3] | *Influent ISS concentration set to match reported MBR and activated sludge treatment train MLVSS fractions* |
| COD, mg/L | 839 | 744 | -11% | *Influent COD concentration set to match reported influent BOD concentration* |
| BOD, mg/L | 363 | 369 | 2% | |
| NH3, mgN/L | 31.9 | 31.9 | 0% | |
| TSS load, klb/d | 78.1 | 79.4 | 2% | |
| COD load, klb/d | 196 | 173 | -12% | |
| BOD load, klb/d | 84.7 | 85.8 | 1% | |
| NH3 load, klbN/d | 7.12 | 7.42 | 4% | |
| Temperature, degC | 29.1 | 29.1 | --[3] | *Temperature data from California Integrated Water Quality System (CIWQS) SMR database* |
| **Plant recycles** | | | | |
| Flow rate, mgd | -- | 3.51 | -- | |
| **Primary influent** | | | | |
| Flow rate, mgd | -- | 31.4 | -- | |
| **MBR treatment train** | | | | |
| Primary influent flow rate, mgd | 18.8 | 18.8 | --[3] | *MBR treatment train primary influent flow rate set to reported operations and performance data* |
| Primary TSS removal, percent | 58.0 | 53.6 | -8% | *Primary clarifier solids capture set to match reported PS solids load* |
| Equalized PE flow rate, mgd | 12.0 | 12.0 | 0% | |
| PE TSS, mg/L | 140 | 154 | 10% | |
| PE VSS, mg/L | 121 | 137 | 13% | |
| PE COD, mg/L | 548 | 449 | -18% | |
| PE NH3, mgN/L | 27.8 | 32.4 | 17% | |
| MLSS, mg/L | 3,330 | 3,230 | -3% | |
| MLVSS, mg/L | 2,800 | 2,660 | -5% | |
| MLVSS fraction, percent | 84.1 | 82.4 | -2% | |
| RAS TSS, mg/L | 6,000 | 5,739 | -4% | |
| R[5] | 1.28 | 1.28 | --[3] | *RAS flow fraction calculated based on reported MLSS and RAS TSS concentrations* |
| Mixed liquor recycle flow rate, mgd | -- | 34.6 | -- | *Assumed 6,000 gpm (8.65 mgd) for each aeration tank in service* |
| Total SRT, d | -- | 9.06 | -- | |
| Aerobic SRT[6], d | -- | 7.10 | -- | |
| Total aeration air flow rate, kscfm | -- | 11.3 | -- | |

**Table B-1. BioWin calibration summary**

| | RWQCP data[1] | BioWin calibration[2] | Difference | Notes |
|---|---|---|---|---|
| **Activated sludge treatment train** | | | | |
| Primary influent flow rate, mgd | 11.5 | 12.6 | 10% | |
| Primary TSS removal, percent | 38.0 | 39.3 | 3% | *Primary clarifier solids capture set to match reported PS solids load* |
| Equalized PE flow rate, mgd | 18.3 | 19.3 | 5% | |
| PE TSS, mg/L | 204 | 201 | -1% | |
| PE VSS, mg/L | -- | 179 | -- | |
| PE COD, mg/L | 604 | 515 | -15% | |
| PE NH3, mgN/L | 32.0 | 32.4 | 1% | |
| MLSS, mg/L | 2,730 | 2,720 | 0% | |
| MLVSS, mg/L | 2,320 | 2,220 | -4% | |
| MLVSS fraction, percent | 85.0 | 81.6 | -4% | |
| RAS TSS, mg/L | 6,290 | 6,240 | -1% | |
| R[5] | 0.77 | 0.77 | --[3] | *RAS flow fraction calculated based on reported MLSS and RAS TSS concentrations* |
| Mixed liquor recycle flow rate, mgd | -- | 51.9 | -- | *Assumed 6,000 gpm (8.65 mgd) for each aeration tank in service* |
| Total SRT, d | -- | 7.41 | -- | |
| Aerobic SRT[6], d | -- | 5.58 | -- | |
| Total aeration air flow rate, kscfm | 19.2 | 19.3 | 1% | |
| SE TSS, mg/L | 3.5 | 3.5 | 0% | *SE TSS set by adjusting clarifier solids capture efficiency* |
| SE VSS, mg/L | 3.0 | 2.8 | -7% | |
| SE COD, mg/L | 24.0 | 24.0 | 0% | |
| SE NH3, mgN/L | 0.37 | 0.04 | --[3] | |
| SE NO2, mgN/L | 0.09 | 0.01 | --[3] | |
| SE NO3, mgN/L | 4.79 | 5.01 | 5% | |
| Filter waste backwash flow rate, mgd | 2.63 | 2.63 | --[3] | *Granular media filter waste backwash flow rate set to reported operations and performance data* |
| **Primary sludge** | | | | |
| MBR PS TSS load, klb/d | 30.7 | 27.8 | -9% | |
| Activated sludge PS TSS load, klb/d | 12.9 | 13.7 | 6% | |
| Combined PS TSS load, klb/d | 43.5 | 41.5 | -5% | |

**Table B-1. BioWin calibration summary**

| | RWQCP data[1] | BioWin calibration[2] | Difference | Notes |
|---|---|---|---|---|
| **Waste activated sludge** | | | | |
| MBR treatment train | | | | |
| WAS Q, mgd | 0.387 | 0.271 | -30% | *MBR WAS flow rate set at 70 percent of reported value to match reported MLSS and MLVSS concentrations* |
| WAS TSS load, klb/d | 19.4 | 13.0 | -33% | |
| WAS VSS load, klb/d | 16.3 | 10.7 | -34% | |
| Activated sludge treatment train | | | | |
| WAS Q, mgd | 0.493 | 0.493 | --[1] | *Activated sludge WAS flow rate set to reported operations and performance data* |
| WAS TSS load, klb/d | 25.8 | 25.7 | 0% | |
| WAS VSS load, klb/d | 22.0 | 21.0 | -5% | |
| Combined WAS stream | | | | |
| WAS Q, mgd | 0.880 | 0.764 | -13% | |
| WAS TSS, percent | 0.62 | 0.61 | -2% | |
| WAS TSS load, klb/d | 45.2 | 39.1 | -13% | |
| WAS VSS load, klb/d | 38.3 | 31.8 | -17% | |
| **Thickened waste activated sludge** | | | | |
| TWAS Q, mgd | 0.089 | 0.071 | -20% | |
| Solids capture efficiency, percent | -- | 95.0 | -- | *Assumed DAFT solids capture* |
| TWAS TS, percent | 6.23 | 6.30 | 1% | |
| TWAS VS fraction, percent | -- | 81.4 | -- | |
| TWAS TS load, klb/d | 47.4 | 37.1 | -22% | |
| TWAS VS load, klb/d | -- | 30.2 | -- | |
| **Sludge blending tank** | | | | |
| BS Q, mgd | 0.145 | 0.199 | 37% | |
| BS TS, percent | 4.73 | 4.74 | --[3] | *BS TS concentration set by adjusting primary clarifier underflow flow fraction, which varies the PS TS concentration* |
| BS VS fraction, percent | 82.7 | 85.2 | 3% | |
| BS TS load, klb/d | 57.3 | 78.6 | 37% | |
| BS VS load, klb/d | 47.3 | 67.0 | 42% | |
| **Digester feed** | | | | |
| BS Q, mgd | 0.209 | 0.199 | -5% | |
| BS TS, percent | 4.73 | 4.74 | --[3] | *Sludge blending tank TS concentration* |
| BS VS fraction, percent | 82.7 | 85.2 | 3% | *Sludge blending tank VS fraction* |
| BS TS load, klb/d | 81.1 | 78.6 | -3% | |
| BS VS load, klb/d | 67.1 | 67.0 | 0% | |

**Table B-1. BioWin calibration summary**

| | RWQCP data[1] | BioWin calibration[2] | Difference | Notes |
|---|---|---|---|---|
| **Anaerobic digester 3 & 4** | | | | |
| DS Q, mgd | 0.209 | 0.214 | 2% | |
| DS TS, percent | 3.10 | 2.98 | -4% | *Flow-weighted average of reported AD3 & AD4 TS concentrations* |
| DS VS fraction, percent | 73.6 | 77.7 | 6% | *Flow-weighted average of reported AD3 & AD4 VS fractions* |
| DS TS load, klb/d | 54.9 | 49.3 | -10% | |
| DS VS load, klb/d | 40.4 | 38.3 | -5% | |
| **Digested sludge** | | | | |
| DS Q, mgd | 0.210 | 0.214 | 2% | |
| DS TS, percent | 3.10 | 2.98 | -4% | |
| DS VS fraction, percent | 73.3 | 77.7 | 6% | |
| DS TS load, klb/d | 53.0 | 49.3 | -7% | |
| DS VS load, klb/d | 38.8 | 38.3 | -1% | |
| $VSr_{Van\ Kleeck}$, percent | 42.0 | 39.7 | -5% | |
| $VSr_{mass\ balance}$, percent | 42.0 | 42.8 | 2% | |
| Digester gas flow rate, scfm | 298 | 302 | 1% | |
| **Dewatered sludge** | | | | |
| TS, percent | 19.0 | 19.0 | --[1] | *Data from G. Perez* |
| VS fraction, percent | 71.2 | 77.6 | 9% | |
| TS load, klb/d | 46.9 | 44.4 | -5% | *Data from G. Perez* |
| Solids capture efficiency, percent | 88.5 | 90.0 | 2% | *Dewatering solids capture efficiency set to match reported dewatered sludge production* |
| Centrate/filtrate NH3, mgN/L | 752 | 745 | -1% | |

**Notes:**
(1) - Arithmetic average of reported operations and performance data from 7/1 through 9/30/2016
(2) - User-specified values are indicated by shaded cells, BioWin-calculated values are shown in blue text
(3) - User-specified value equal to reported operations and performance data
(4) - Inorganic suspended solids (i.e., TSS - VSS)
(5) - RAS flow fraction, R, is the ratio of the RAS flow rate to the wastewater flow rate
(6) - Calculated using aerated fraction of aeration tank only

Appendix 3C

# RIVERSIDE WATER QUALITY CONTROL PLANT - REPORT OF FULL SCALE OFFGAS ANALYSIS OF MEMBRANE GRID AERATION SYSTEM

REDMON
ENGINEERING
COMPANY

*Consulting Engineers*

*PO Box 044258*

*Racine, Wisconsin 53404-7005*

*4142)46718993*

May 4, 2018

Carollo Engineers
Attn.: Ron Appleton, Jr., P.E.
72700 Ygnacio Valley Road, Suite 300
Walnut Creek, CA 94598

Re: Riverside Water Quality Control Plant - Report of Full Scale Offgas Analysis of Membrane Grid Aeration System

Dear Ron,

Attached please find my report of the full-scale offgas tests conducted in March of this year; on the Environmental Dynamics Incorporated (EDI) fine pore membrane tube (Mini-Panel) aeration system at the at the Riverside WQCP.

Following your review, should you have any questions, please let me know.

Best regards,

**REDMON ENGINEERING COMPANY**

David T. Redmon, P.E.

REDMON
ENGINEERING COMPANY

# FULL SCALE OFFGAS ANALYSIS OF THE EDI FINE PORE MEMBRANE TUBE AERATION SYSTEM AT THE RIVERSIDE WATER QUALITY CONTROL PLANT IN RIVERSIDE, CALIFORNIA

**March 2018**

## INTRODUCTION

Redmon Engineering Company was engaged as a subcontractor to Carollo Engineers to conduct an offgas evaluation of the Environmental Dynamics Incorporated (EDI) fine pore membrane grid aeration system installed in the aeration basins at the Riverside Water Quality Plant in Riverside, California. The purpose of the tests was to measure the oxygen transfer efficiency of the existing membrane tube diffusers (Mini-Panels) under process water conditions

The objective of the offgas evaluation was to provide site-specific measurements of oxygen transfer efficiency, alpha and oxygen transfer rate of the membrane tube aeration system that was installed in both the Conventional Activated Sludge aeration basins and the Membrane Bio-Reactor (MBR) aeration basins .

On March 13, 14, and 15, 2018, David Redmon of Redmon Engineering Company, with the assistance of plant personnel, conducted offgas tests on the aeration system at the Riverside WQCP. The results of this offgas evaluation are presented in this report.

PO Box 044258 Racine, Wisconsin 53404-7005 (414) 467-8993

**BACKGROUND**

The full-scale test involves placing a floating offgas collection device on the liquid surface of the basin(s) in question at various locations and to analyze the exiting gas for the partial pressure of oxygen compared to that of ambient air. In addition, the rate of offgas evolution is typically measured for each offgas collection hood sampling position employed and each test condition. These data are analyzed according to the procedures described in the paper, "Oxygen Transfer Efficiency Measurements in Mixed Liquor Using Offgas Techniques," by Redmon, et al. (WPCF November, 1983) and the ASCE “Standard Guidelines for In-Process Oxygen Transfer Testing,” (ASCE-18-96). The offgas paper is contained in Appendix I for the reader’s reference.

**Aeration System**

The aeration system tested at the Riverside WQCP consists of two batteries aeration basins. The first battery was constructed several years ago and consists of six (6) aeration basins each of these basins is 40 feet wide by about 250 feet in length and having a side water depth of 16.8 feet. The diffuser submergence is approximately 15.8 feet. These basins are operated as Conventional Activated Sludge systems. The second battery of aeration basins also consists of six (6) basins, each 40 feet wide by 200 feet in length and having a side water depth of 16.8 feet. These basins are referred to as the MBR (Membrane Bio-reactor) aeration basins. This portion of the plant uses

membranes for solids separation instead of secondary clarifiers used in the Conventional Activated Sludge portion of the plant. The first fifty feet of each aeration basin is mixed, but not aerated. In the Conventional Activated Sludge basins there are four more aerobic zones (each approximately 50 feet in length). The last zone in these basins are aerated with only enough air to mix that section of the basin.

In the MBR Aeration Basins there are three aerobic zones following the initial selector zone. Again, each of these three zones in operated aerobically. Figure 1 is a plan view drawing indicating the general layout of the MBR Aeration Basin 5 and the diffuser layout. The Conventional Activated Sludge basins have a similar layout. Drawings for these basins were not available and are therefore not included here.

In the case of MBR Aeration Basins there are about 206 Mini-Panel assemblies in Zone 1; 99 assemblies in Zone 2; and 80 assemblies in Zone 3. Each assembly consists of two membrane tube diffusers, each with a total perforated area of 2.64 square feet. The Conventional Activated Sludge aeration basins each have about 102 diffuser assemblies in Zone 1, 51 assemblies in Zone 2; and 30 assemblies in Zone 3.

**Test Parameters**

The full-scale tests were conducted to measure oxygen transfer efficiency, alpha factor, and oxygen transfer rates under actual operating conditions.

Manufacturers of aeration systems typically quote performance based on clean water oxygen transfer test results. To compare data, the tests should best be conducted in large-scale tanks in accordance with standard procedures (ASCE Clean Water Test Standard, 1992). For a given basin geometry, diffuser type and layout, aeration equipment manufacturers can provide acceptable estimates of clean water standard oxygen transfer efficiency (SOTE) and equilibrium dissolved oxygen (DO) concentration at standard conditions as time approaches infinity ($C^*_{\infty 20}$). Standard conditions of temperature and pressure are 20°C and 1.0 atmosphere of pressure (29.92 in Hg or 760 mm Hg), respectively.

To estimate the oxygen transfer efficiency in the process water under actual operating conditions, the following equation is used (ASCE, 1992):

$$OTE_F = \alpha(SOTE)(\Theta^{T-20})(Y\Omega\beta C^*_{\infty 20} - C) / C^*_{\infty 20}$$

Where:

| | | |
|---|---|---|
| $OTE_F$ | = | Process water oxygen transfer efficiency, mass fraction of oxygen transferred per unit of oxygen supplied, decimal fraction. |
| $\alpha$ | = | Alpha, the ratio of mass transfer coefficients, process water to clean water, decimal fraction. |

$\Theta$ = Mass transfer coefficient temperature correction factor, generally taken to be 1.024, dimensionless.

T = Temperature of the process water, °C.

Y = Temperature correction factor ($C^*_{bST}/C_{*b20}$) of the steady state DO saturation concentration, dimensionless.

Where:

$C^*_{bST}$ = Tabulated DO surface saturation value at temperature T, taken from Standard Methods, mg/l.

$C^*_{b20}$ = Tabulated DO surface saturation value at 20°C taken from Standard Methods, mg/l.

$\beta$ = Ratio of steady state DO saturation concentration in process and clean water, dimensionless (basis total dissolved solids).

$\Omega$ = Pressure correction factor ($P_b/P_s$) for the steady state DO saturation concentration, dimensionless.

Where:

$P_b$ = Local barometric pressure for the site, in Hg.

$P_s$ = Standard barometric pressure, 29.92 in Hg (101.3 k Pa).

C = Dissolved oxygen concentration averaged over process water volume being evaluated, mg/l.

All of the factors involved in the conversion from clean water to process water, except alpha, and the fouling factor can be reasonably estimated from published or assumed values. The field studies were conducted at the Riverside Water Quality Control Plant in an effort to provide site-specific estimates of OTE and alpha(F), as well as OUR, for use in assessing the aeration performance of the aeration system under process water conditions.

## RESULTS

### General

The results of the full-scale offgas evaluation is summarized as Tables 1 to 3. Figure 2 is a plan view of both an MBR basin and a Conventional aeration basin, showing the offgas sampling plan used. The offgas collection hood used was two feet wide by eight feet in length, thus having a total capture area of 16 square feet.

Table 1 summarizes the offgas results obtained on March 13, 2018 for MBR Aeration Basin 5. The first several columns of this table, including time, sampling station designation, mixed liquor temperature, gas-phase sensor output (Mog and Mr), DO concentrations (C), and offgas flow rate are obtained from the field data sheets. Knowing the dissolved oxygen (DO) saturation value from clean water testing of the equipment in question ($C^*_{20}$), the field saturation value ($C^*_f$) can be estimated by applying corrections for local atmospheric pressure, mixed liquor temperature and total dissolved solids, which are reflected in the beta factor. The column headed $C^*_f$-DO (Column 7) represents the DO driving force (saturation minus the DO concentration) at that sampling station.

Columns 8 and 9 are the float heights (in millimeters) for the two rotameters measuring offgas flow rate. Column 10 is the collection area of the offgas collection hood. Column 11 lists the measured offgas flux for the sample location in question. The offgas flux is determined by dividing the offgas flow rate by the offgas collection hood area (Column 10).

Column 12 is the calculated airflow per diffuser and is determined by dividing the offgas flux (scfm per square foot) by the diffuser density (the number of diffusers per square foot) beneath the hood.

Column 13 lists the total airflow to each of the test zones in the basin in question. In each zone the average offgas flux times the surface area of the zone in question

yields an estimate of the total airflow to the zone in question. The total airflow for the basin overall is obtained by summing the estimated airflow in each zone.

The gas-phase oxygen transfer efficiency under process conditions is given by the columns headed $OTE_F$ (Column 14), $OTE_{SP20}$ (Column 15) and SOTEpw (Column 16). The field oxygen transfer efficiency ($OTE_F$) is the actual gas-phase transfer, as a decimal, under existing field conditions of DO concentration, barometric pressure, total dissolved solids, mixed liquor temperature and prevailing operating mode. $OTE_{SP20}$ is the transfer efficiency per each mg/l of driving force, corrected to a 20°C mixed liquor temperature. SOTEpw is the oxygen transfer efficiency in process water corrected to standard conditions of one atmosphere of pressure, zero DO concentration and 20°C. Column 17, SOTEcw, is an estimate of the clean water oxygen transfer efficiency based on the data submitted by EDI.

Column 18 is the ratio of SOTEpw to SOTEcw. When diffusers are new this ratio is known as alpha. In this case, the aeration system had been in operation for several years so the ratio of SOTEpw to SOTEcw is known as alpha(F), where F is a fouling factor. The fouling factor accounts for changes in diffuser oxygen transfer efficiency due to fouling and changes in the membrane properties. When the diffusers are new the fouling factor is unity (1.00). Column 19 is the computed oxygen uptake rate (OUR) for each hood location based on a gas-phase mass balance. The mass balance calculation procedure used to calculate the OUR is presented in Appendix II.

Listed beneath each block of data (one block of data for each zone) is the overall average values of DO concentration, offgas flux, diffuser air flow, alpha and oxygen uptake rate along with the total air flow and the mean weighted average $OTE_F$ and SOTEpw values for the entire basin. These values are shown in red. At the very bottom of each table is the overall average performance. The same parameters listed for each block of data are presented in blue.

**Test Results**

The first set of offgas data was obtained on Tuesday, March 13th. Table 1 summarizes of the offgas results for the first day of testing. Looking at the first line of data, it is seen that the first sample location (5.1.1, hood location Basin 5, Zone 1, hood position #1) was tested at 1127 hours. The mixed liquor temperature was 24.6 degrees Celsius and the dissolved oxygen concentration was 1.65 mg/l. The offgas flux for this location was observed to be 1.319 scfm per square foot of surface area, which is equivalent to an airflow rate of approximately 6.4 scfm per diffuser. The field oxygen transfer efficiency for sample location 5.1.1 is 11.96% when corrected to standard conditions is 14.94%. The alpha(F) value for this location is computed to be 0.46 and the oxygen uptake rate (OUR), based on the gas-phase mass balance, is 156 mg/l/hr. The results for each of the remaining test locations are presented in a similar matter.

As mentioned earlier, at the bottom of each block of data is listed the average DO concentration, offgas flux, airflow per diffuser average transfer efficiency, alpha(F), and average oxygen uptake rate in red.

The overall average performance is shown at the bottom of the table. In the case of MBR Aeration Basin 5 the overall average dissolved oxygen concentration is 1.4 mg/l, the average offgas flux is 0.734 scfm per square foot, the average airflow per diffuser is 5.5 scfm. The total airflow based on the offgas data is 4,293 scfm. The overall mean weighted average field oxygen transfer efficiency (OTEf) is 14.35% and the standard oxygen transfer efficiency in process water (SOTEpw), also known as alpha(SOTE), is 16.86%. The overall computed value of alpha(F) is 0.53 and the average oxygen uptake rate (OUR) is 100 mg/l/hr. The approximate mixed liquor suspended solids concentration was 7,300 mg/l. The writer does not know what the Mean Cell Residence Time (MCRT) was for this basin at the time it was tested.

The second set of offgas data was obtained on March 14$^{th}$ and is presented in Tables 2. This table presents the data for Conventional Activated Sludge Aeration Basin 6. The offgas data for this basin was acquired between about 1000 and 1530 hours. During this period the total airflow to the test basin was about 2,600 scfm and the mixed liquor suspended solids concentration was about 1,900 mg/l. Once again, the MCRT for this basin at the time of test is unknown to the writer.

The overall average performance values are listed at the bottom of the table. The overall average DO concentration was 1.578 mg/l, the average offgas flux was 0.427 scfm/ft$^2$, and the average airflow per diffuser was 3.16 scfm. The overall average field oxygen transfer efficiency was observed to be 13.2% and when corrected to standard conditions, the overall SOTEpw was 15.9%. Comparing this value to the standard oxygen transfer efficiency in clean water, yields an alpha(F) of 0.47. The overall oxygen uptake rate is computed to be 54.3 mg/l/hr.

On the final day of testing (March 15$^{th}$) MBR Aeration Basin 4 was tested. The offgas test results are summarized in Table 3. The mixed liquor suspended solids concentration for this basin was about 6,800 mg/l, based on plant data available at the time. The overall airflow rate to this basin was about 3,400 scfm based on the offgas flux measurements.

The overall performance values for MBR Basin 4 are quite similar to those from MBR Aeration Basin 5. In the case of MBR Aeration Basin 4 the overall average DO concentration was observed to be 1.7 mg/l, the average offgas flux was 0.579 scfm/ft$^2$, and the average airflow per diffuser was about 4.1 scfm. The overall field oxygen transfer efficiency (OTEf) was observed to be 14.65%, and when corrected to standard conditions (SOTEpw) was 17.9%. The overall average alpha(F) for Basin 4 is 0.53 and the average OUR is 76 mg/l/hr.

**DISCUSSION:**

The primary purpose of the offgas evaluation of the aeration system at the Riverside WQCP was to verify the oxygen transfer performance of the aeration equipment currently installed at the plant. Of greatest interest was the determination of the alpha(F) factor associated with both the Conventional Activated Sludge part of the plant and the MBR aeration equipment.

A review of Tables 1 and 3 (Basins 5 and 4) show an overall alpha(F) of 0.53 for both MBR Aeration Basins tested. Looking at the individual aerobic zones, it is seen that the alpha(F) in the first aerobic zone is about 20% lower than aerobic zones 2 and 3. In both cases the alpha(F) for Zone 1 is 0.47. The remaining alpha(F) values range between 0.52 and 0.60. Looking at Tables 1 and 3, one does not see a progressive increase in alpha(F) from inlet to outfall of the basins. This may be due to the fact that both aeration trains are using the Modified Ludzack-Ettlinger (MLE) configuration for denitrification. When using the MLE Process, there is internal recycle from the end of the aeration basin back to the inlet to the anoxic zone. Typically the internal recycle rates are in the range of about 200% to 400%. With this much recycle one would expect a more homogenous mixture of biosolids and wastewater and less variation in the oxygen transfer inhibiting substances from a spatial point of view.

A review of the offgas data from the Conventional Activated Sludge Aeration Basin 6 (Table 2) shows similar spatial distribution regarding the observed alpha(F)

values. Once again the alpha(F) for the first aerobic zone is about 20% lower than the alpha(F) values observed for aerobic Zones 2 and 3. The overall alpha(F) for this basin is 0.47. The overall alpha(F) values for two MBR Aeration Basins are approximately 12% higher than the overall alpha for the Conventional Activated Sludge basin tested.

Once again, it should be pointed out that the MLSS concentration in the two MBR Aeration Basins was in the range of about 6,800 to 7,300 mg/l, while the MLSS concentration in the Conventional Activated Sludge basin was about 1,900 mg/l. The writer does not know what the MCRT was for the three aeration basins at the time they were tested. Generally speaking, alpha is expected to be higher in aeration systems with long MCRT's than it would in systems with low MCRT's.

Another interesting aspect of the data is that despite the fact that the oxygen uptake rates (OUR) are 50% to 100% higher in the MBR Aeration Basins compared to the Conventional Activated Sludge basin, that the alpha(F) are quite similar to one another. As mentioned earlier the Conventional basin's alpha(F) value is only about 12% lower than the alpha(F) values for the MBR basins.

## SYMBOLS AND NOMENCLATURE

| | | |
|---|---|---|
| DO | = | Dissolved Oxygen |
| C | = | DO concentration, mg/l |
| $C^*_F$ | = | DO saturation value applicable for equipment in use and existing conditions, mg/l |
| $C^*_F$-C | = | DO driving force or effective DO deficit, mg/l |
| $C^*_{\infty 20}$ | = | DO saturation value in clean water for system tested at standard conditions as time approaches infinity |
| $C^*_{ST}$ | = | Tabulated DO surface saturation value at temperature T, taken from Standard Methods, mg/l |
| $C^*_{20}$ | = | Tabulated DO surface saturation value at 20 C taken from standard Methods, mg/l |
| EPDM | = | E-Ethylene, P-propylene, D-Diene comoners, M-polyMethylene backbone; synthetic rubber |
| AOTR | = | Actual Oxygen Transfer Rate in process water at existing conditions |
| fpm | = | Feet per minute |
| gpm | = | Gallons per minute |
| Hg | = | Mercury |
| Hood Area | = | Offgas Hood Collection Area, square feet |

Symbols and Nomenclature
Page 2

| | | |
|---|---|---|
| $K_La$ | = | Apparent volumetric mass transfer coefficient of oxygen in clean water and/or process water |
| MLSS | = | Mixed Liquor Suspended Solids, mg/l |
| MLT | = | Mixed Liquor Temperature, °C |
| M(og) | = | Gas phase oxygen sensor output in millivolts for offgas stream |
| M(r) | = | Gas phase oxygen sensor output in millivolts for reference stream |
| MWA | = | Mean weighted average |
| Offgas Flux Rate | = | Rate of offgas evolution per square foot of collection area as measured by offgas rotameters, scfm/sq ft |
| $OTE_F$ | = | Process water oxygen transfer efficiency, mass fraction of oxygen transferred per unit of oxygen supplied, decimal fraction |
| $OTE_{SP20}$ | = | Oxygen Transfer efficiency per each mg/l of driving force under Standard Conditions |
| OUR | = | Oxygen Uptake Rate by mixed liquor, mg/l/hr |
| $P_b$ | = | Local barometric pressure for the site, in Hg |
| $P_s$ | = | Standard barometric pressure, 29.92 in Hg |
| Rmm 1 & Rmm 2 | = | Float Height in millimeters, from scale, for rotameters 1 and 2 in offgas analyzer |

| | | |
|---|---|---|
| scfm | = | Air flow rate, Standard cubic feet per minute |
| SOTE | = | Standard Oxygen Transfer efficiency at 20°C and zero DO |
| SOTEcw | = | Standard Oxygen Transfer efficiency at Standard Conditions and zero DO in clean water |
| SOTEpw | = | Standard Oxygen Transfer efficiency at Standard Conditions and zero DO in process water |
| SOTR | = | Standard Oxygen Transfer Rate in clean water at 20°C and zero DO |
| SRT | = | Solids Retention Time or Sludge Age, days |
| Standard Conditions | = | Barometric Pressure of 29.92 in Hg and 20°C |
| Submergence | = | Height of liquid above diffusers, feet |
| T | = | Temperature, °C |
| TDS | = | Total Dissolved Solids in mixed liquor, mg/l |
| wg | = | Water gauge |
| $\alpha$ | = | Alpha, the ratio of mass transfer coefficients (KLa), or standard oxygen transfer efficiency, process water to clean water, decimal fraction |
| $\beta$ | = | Beta, the ratio of steady state DO saturation concentration in process and clean water, dimensionless (basis total dissolved solids) |

| | | |
|---|---|---|
| $\Omega$ | = | Pressure correction factor ($P_b/P_s$) for the steady state DO saturation concentration, dimensionless |
| $\Theta$ | = | Mass transfer coefficient temperature correction factor, generally taken to be 1.024, dimensionless |
| Y | = | Temperature correction factor ($C^*_{ST}/C^*_{20}$) for the steady state DO saturation concentration, dimensionless |

# FULL SCALE OFFGAS ANALYSIS
## OF THE
# EDI FINE PORE MEMBRANE GRID AERATION SYSTEM
## AT THE
# RIVERSIDE WQCP

## RIVERSIDE, CALIFORNIA

**PERFORMED ON BEHALF OF:**

**CAROLLO ENGINEERS**

**CONDUCTED**

**March 2018**

**PERFORMED BY:**


**REDMON ENGINEERING COMPANY**

**PO Box 044258**
**Racine, Wisconsin 53404**

**(414) 467-8993**

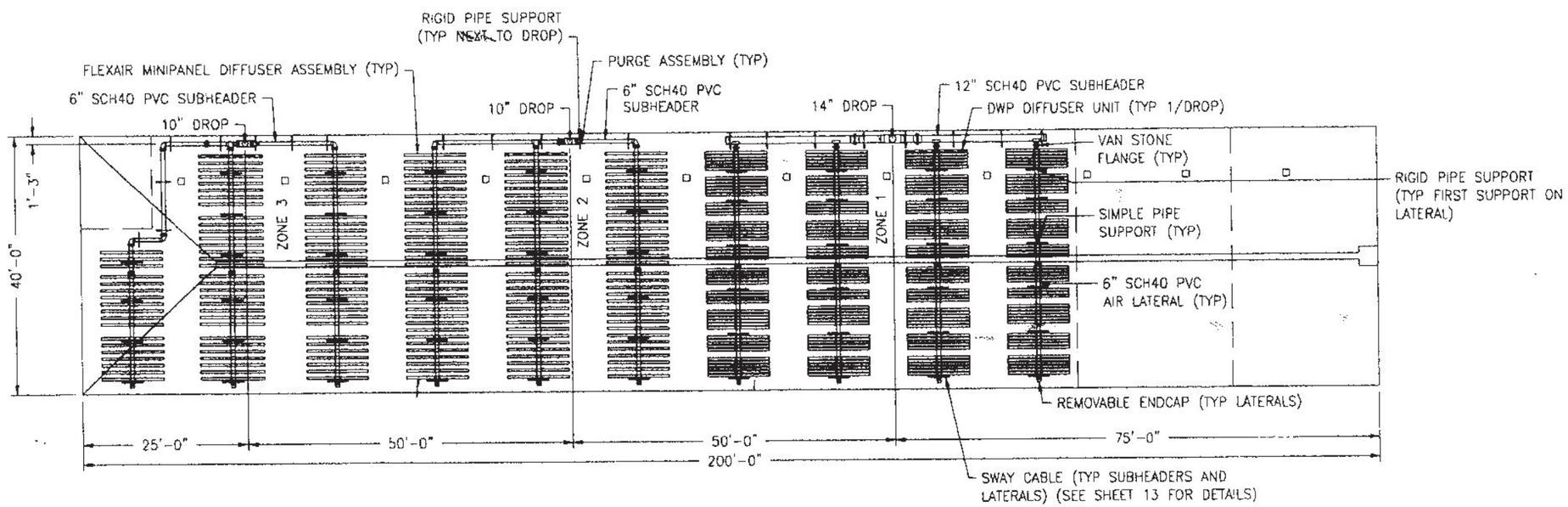
# FIGURE 1 RIVERSIDE WQCP - MBR AERATION BASIN LAYOUT



aeratipp for life™

REVISION HISTORY

| REV | DESCRIPTION | DATE | APPROVED |
|---|---|---|---|
| A | INITIAL RELEASE | 2/19/13 | LDH |
| B | CALLOUT CLARIFICATIONS | 7/16/13 | LDH |




| | # OF DIFFUSERS |
|---|---|
| ZONE 1 | 206 |
| ZONE 2 | 99 |
| ZONE 3 | 80 |

NOTES:

1. AERATION-MIXING BY EDI FLEXAIR MINIPANEL DIFFUSER ASSEMBLIES.
2. LIQUID DEPTH IS 16.80 FT. DIFFUSER DEPTH IS 15.80 FT.
3. TOTAL = 385 FLEXAIR MINIPANEL DIFFUSER ASSEMBLIES.
4. SEE SHEETS 6-8 FOR ZONE DETAILS.
5. SEE SHEET 13 FOR SWAY BRACE DETAILS.

PROJECT NAME: RIVERSIDE, CA

PROJECT DESCRIPTION: NEW BASIN 5 LAYOUT

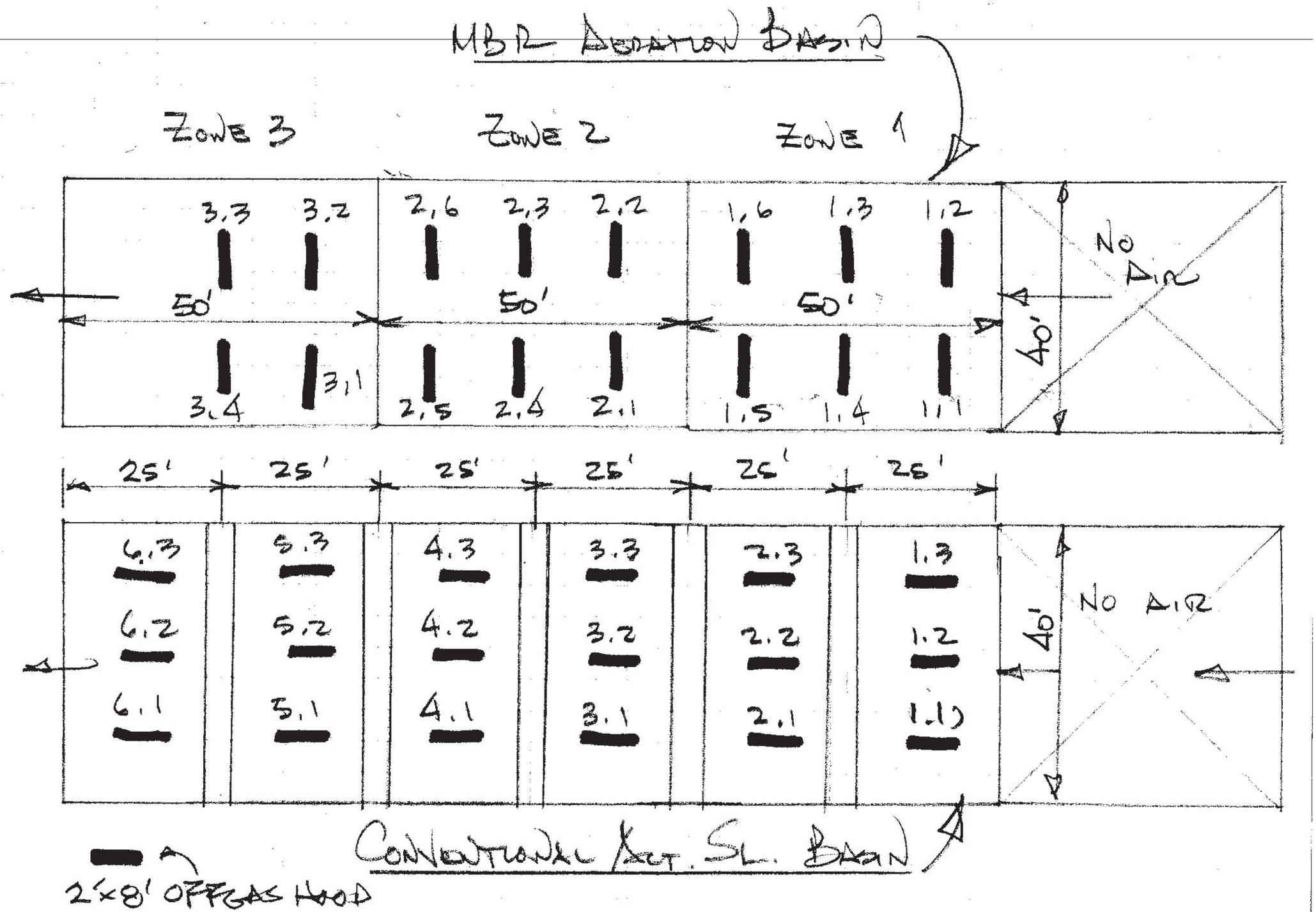
EDI FLEXAIR® AERATION-MIXING SYSTEM

| FOR: | BY: | DATE: | SCALE: |
|---|---|---|---|
| LDH | TSP | 1/16/13 | 1/16" = 1' |

Environmental Dynamics International
P: +1 573.474.9456
F: +1 573.474.6988
wastewater.com
Worldwide Headquarters:
5603 Paris Road
Columbia, MO USA 65202

| PROJECT NUMBER | SHEET NUMBER | DWG NUMBER |
|---|---|---|
| 20160 | 5 OF 13 | 107255 |

**FIGURE 2 - OFFGAS SAMPLING PLAN**



# TABLE 1
## SUMMARY DATA SHEET - FULL SCALE OFFGAS TESTS

DATE: March 13, 2018

SITE: RIVERSIDE CA
SYSTEM: EDI MINI PANELS
SUBMERGENCE: 15.90 FT.
SWD: 16.80 FT.
DIFFUSERS/BASIN: 770

MLSS: 7,250 MG/L
MLVSS: MG/L
TDS: 1,000 MG/L (ASSUMED)
SRT: DAYS
TOTAL AIR RATE: 4,293 SCFM

LOCAL BAROMETER: 29.30 in. Hg.
BETA: 0.97
$C^*_{20}$: 10.70 MG/L
$C^*_F$: 9.30 MG/L

$H_R$: 0.00
$H_{OG}$: 0.00 LB $H_2O$/LB B.D. AIR
$CO_2$: 0.00

### MBR AERATION BASIN # 5

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| TIME | STATION | ML TEMP °C | M(og) (mv) | M(r) (mv) | C (mg/l) | $C^*_F$-C (mg/l) | Rmm 1 | Rmm 2 | HOOD AREA (sq ft) | OFFGAS FLUX (scfm/sq ft) | SCFM PER DIFFUSER | TOTAL AIR FLOW (cfm) | $OTE_F$ (decimal) | $OTE_{SP20}$ (decimal) | SOTEpw | SOTEcw | ALPHA(F) | OUR (mg/l/hr) |
| 1127 | 5.1.1 | 24.6 | 911 | 1009 | 1.85 | 7.45 | 0 | 210 | 16 | 1.319 | 6.39 | 439 | 0.1196 | 0.01440 | 0.1494 | 0.322 | 0.46 | 156.0 |
| 1138 | 5.1.2 | 24.6 | 908 | 1008 | 1.95 | 7.35 | 0 | 205 | 16 | 1.289 | 6.25 | 429 | 0.1223 | 0.01492 | 0.1548 | 0.323 | 0.48 | 155.8 |
| 1155 | 5.1.3 | 24.6 | 913 | 1013 | 1.90 | 7.40 | 0 | 165 | 16 | 1.041 | 5.05 | 347 | 0.1219 | 0.01477 | 0.1533 | 0.332 | 0.46 | 125.5 |
| 1208 | 5.1.4 | 24.6 | 918 | 1013 | 2.00 | 7.30 | 0 | 180 | 16 | 1.134 | 5.50 | 378 | 0.1155 | 0.01418 | 0.1472 | 0.329 | 0.45 | 129.5 |
| 1225 | 5.1.5 | 24.6 | 908 | 1013 | 1.95 | 7.35 | 0 | 165 | 16 | 1.041 | 5.05 | 347 | 0.1276 | 0.01556 | 0.1615 | 0.332 | 0.49 | 131.3 |
| 1236 | 5.1.6 | 24.6 | 908 | 1012 | 1.90 | 7.40 | 0 | 165 | 16 | 1.041 | 5.05 | 347 | 0.1264 | 0.01532 | 0.1590 | 0.332 | 0.48 | 130.1 |
| | | | | | $\bar{X}$ = 1.93 | | | | | $\bar{X}$ = 1.144 | $\bar{X}$ = 5.55 | Σ = 2,286 | MWA = 0.1220 | | MWA = 0.1539 | $\bar{X}$ = 0.328 | $\bar{X}$ = 0.47 | $\bar{X}$ = 138.0 |
| 1325 | 5.2.1 | 24.7 | 872 | 1006 | 1.60 | 7.70 | 0 | 105 | 16 | 0.694 | 7.00 | 231 | 0.1626 | 0.01889 | 0.1961 | 0.310 | 0.63 | 111.6 |
| 1336 | 5.2.2 | 24.7 | 893 | 1007 | 1.60 | 7.70 | 0 | 115 | 16 | 0.750 | 7.57 | 250 | 0.1389 | 0.01613 | 0.1674 | 0.314 | 0.53 | 103.0 |
| 1348 | 5.2.3 | 24.8 | 862 | 1009 | 1.40 | 7.90 | 0 | 95 | 16 | 0.638 | 6.44 | 212 | 0.1774 | 0.02004 | 0.2079 | 0.313 | 0.66 | 111.9 |
| 1358 | 5.2.4 | 24.8 | 876 | 1010 | 1.35 | 7.95 | 0 | 90 | 16 | 0.611 | 6.17 | 203 | 0.1623 | 0.01821 | 0.1890 | 0.315 | 0.60 | 98.0 |
| 1410 | 5.2.5 | 24.8 | 877 | 1012 | 1.10 | 8.20 | 0 | 80 | 16 | 0.556 | 5.61 | 185 | 0.1626 | 0.01770 | 0.1837 | 0.320 | 0.57 | 89.4 |
| 1420 | 5.2.6 | 24.8 | 879 | 1011 | 1.15 | 8.15 | 0 | 65 | 16 | 0.474 | 4.78 | 158 | 0.1600 | 0.01752 | 0.1818 | 0.321 | 0.57 | 75.0 |
| | | | | | $\bar{X}$ = 1.37 | | | | | $\bar{X}$ = 0.621 | $\bar{X}$ = 6.26 | Σ = 1,240 | MWA = 0.1600 | | MWA = 0.1875 | $\bar{X}$ = 0.316 | $\bar{X}$ = 0.60 | $\bar{X}$ = 98.1 |
| 1447 | 5.3.1 | 24.8 | 881 | 1013 | 0.95 | 8.35 | 0 | 62 | 16 | 0.458 | 4.93 | 201 | 0.1593 | 0.01702 | 0.1767 | 0.320 | 0.55 | 72.1 |
| 1457 | 5.3.2 | 24.8 | 882 | 1013 | 0.90 | 8.40 | 0 | 60 | 16 | 0.447 | 4.81 | 196 | 0.1581 | 0.01680 | 0.1743 | 0.320 | 0.54 | 69.9 |
| 1510 | 5.3.3 | 24.8 | 876 | 1015 | 1.05 | 8.25 | 165 | 0 | 16 | 0.262 | 2.82 | 115 | 0.1672 | 0.01808 | 0.1877 | 0.345 | 0.54 | 43.3 |
| 1522 | 5.3.4 | 24.8 | 917 | 1020 | 0.85 | 8.45 | 0 | 85 | 16 | 0.584 | 6.29 | 256 | 0.1245 | 0.01315 | 0.1365 | 0.309 | 0.44 | 71.9 |
| | | | | | $\bar{X}$ = 0.94 | | | | | $\bar{X}$ = 0.438 | $\bar{X}$ = 4.71 | Σ = 767 | MWA = 0.1486 | | MWA = 0.1643 | $\bar{X}$ = 0.324 | $\bar{X}$ = 0.52 | $\bar{X}$ = 64.3 |
| | | OVERALL | | | $\bar{X}$ = 1.41 | | | | | $\bar{X}$ = 0.734 | $\bar{X}$ = 5.51 | Σ = 4,293 | MWA = 0.1435 | | MWA = 0.1686 | $\bar{X}$ = 0.322 | $\bar{X}$ = 0.53 | $\bar{X}$ = 100.2 |

# TABLE 2
## SUMMARY DATA SHEET - FULL SCALE OFFGAS TESTS

DATE: March 14, 2018

SITE: RIVERSIDE CA
SYSTEM: EDI MINI PANELS
SUBMERGENCE: 15.90 FT.
SWD: 16.80 FT.
DIFFUSERS/BASIN: 420

MLSS: 1,900 MG/L
MLVSS: MG/L
TDS: 1,000 MG/L (ASSUMED)
SRT: DAYS
TOTAL AIR RATE: 2,558 SCFM

LOCAL BAROMETER: 29.27 in. Hg.
BETA: 0.97
$C^*_{20}$: 10.70 MG/L
$C^*_F$: 9.30 MG/L

$H_R$: 0.00
$H_{OG}$: 0.00 LB $H_2O$/LB B.D. AIR
$CO_2$: 0.00

### CONVENTIONAL ACT SL BASIN #6

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| TIME | STATION | ML TEMP °C | M(og) (mv) | M(r) (mv) | C (mg/l) | $C^*_F$-C (mg/l) | Rmm 1 | Rmm 2 | HOOD AREA (sq ft) | OFFGAS FLUX (scfm/sq ft) | SCFM PER DIFFUSER | TOTAL AIR FLOW (cfm) | $OTE_F$ (decimal) | $OTE_{SP20}$ (decimal) | SOTEpw | SOTEcw | ALPHA(F) | OUR (mg/l/hr) |
| 1024 | 6.1.1 | 24.6 | 900 | 1004 | 1.25 | 8.05 | 0 | 70 | 16 | 0.502 | 2.43 | 167 | 0.1276 | 0.01421 | 0.1475 | 0.352 | 0.42 | 63.3 |
| 1040 | 6.1.2 | 24.6 | 907 | 1008 | 1.25 | 8.05 | 0 | 68 | 16 | 0.491 | 2.38 | 164 | 0.1234 | 0.01375 | 0.1427 | 0.352 | 0.41 | 59.9 |
| 1054 | 6.1.3 | 24.6 | 918 | 1009 | 1.20 | 8.10 | 0 | 72 | 16 | 0.513 | 2.49 | 171 | 0.1113 | 0.01232 | 0.1279 | 0.352 | 0.36 | 56.5 |
| 1133 | 6.2.1 | 24.6 | 927 | 1005 | 2.10 | 7.20 | 0 | 90 | 16 | 0.611 | 2.96 | 203 | 0.0962 | 0.01198 | 0.1244 | 0.344 | 0.36 | 58.1 |
| 1145 | 6.2.2 | 24.6 | 917 | 1005 | 2.00 | 7.30 | 0 | 92 | 16 | 0.622 | 3.02 | 207 | 0.1079 | 0.01326 | 0.1376 | 0.342 | 0.40 | 66.4 |
| 1202 | 6.2.3 | 24.7 | 912 | 1005 | 2.15 | 7.15 | 0 | 92 | 16 | 0.622 | 3.02 | 207 | 0.1143 | 0.01431 | 0.1485 | 0.342 | 0.43 | 70.3 |
| | | | | | $\bar{X}$ = 1.66 | | | | | $\bar{X}$ = 0.560 | $\bar{X}$ = 2.72 | Σ = 1,119 | MWA = 0.1127 | | MWA = 0.1379 | $\bar{X}$ = 0.347 | $\bar{X}$ = 0.40 | $\bar{X}$ = 62.4 |
| 1226 | 6.3.1 | 24.7 | 913 | 1002 | 2.30 | 7.00 | 0 | 71 | 16 | 0.507 | 5.12 | 169 | 0.1098 | 0.01403 | 0.1456 | 0.314 | 0.46 | 55.0 |
| 1236 | 6.3.2 | 24.7 | 904 | 1001 | 2.30 | 7.00 | 0 | 68 | 16 | 0.491 | 4.95 | 164 | 0.1196 | 0.01529 | 0.1587 | 0.315 | 0.50 | 58.1 |
| 1249 | 6.3.3 | 24.7 | 907 | 1002 | 2.30 | 7.00 | 0 | 68 | 16 | 0.491 | 4.95 | 164 | 0.1170 | 0.01495 | 0.1552 | 0.315 | 0.49 | 56.8 |
| 1323 | 6.4.1 | 24.7 | 885 | 1005 | 1.70 | 7.60 | 0 | 58 | 16 | 0.436 | 4.40 | 145 | 0.1464 | 0.01723 | 0.1789 | 0.320 | 0.56 | 63.1 |
| 1334 | 6.4.2 | 24.7 | 891 | 1005 | 1.45 | 7.85 | 0 | 54 | 16 | 0.415 | 4.19 | 138 | 0.1396 | 0.01591 | 0.1651 | 0.322 | 0.51 | 57.3 |
| 1343 | 6.4.3 | 24.7 | 902 | 1008 | 1.75 | 7.55 | 0 | 53 | 16 | 0.410 | 4.14 | 137 | 0.1291 | 0.01529 | 0.1587 | 0.322 | 0.49 | 52.3 |
| | | | | | $\bar{X}$ = 1.97 | | | | | $\bar{X}$ = 0.458 | $\bar{X}$ = 4.62 | Σ = 916 | MWA = 0.1260 | | MWA = 0.1598 | $\bar{X}$ = 0.318 | $\bar{X}$ = 0.50 | $\bar{X}$ = 57.1 |
| 1407 | 6.5.1 | 24.7 | 872 | 1004 | 1.35 | 7.95 | 180 | 0 | 16 | 0.284 | 2.32 | 95 | 0.1604 | 0.01805 | 0.1873 | 0.347 | 0.54 | 45.0 |
| 1416 | 6.5.2 | 24.7 | 874 | 1004 | 1.35 | 7.95 | 195 | 0 | 16 | 0.308 | 2.52 | 103 | 0.1581 | 0.01779 | 0.1846 | 0.345 | 0.54 | 48.2 |
| 1426 | 6.5.3 | 24.7 | 878 | 1002 | 1.35 | 7.95 | 154 | 0 | 16 | 0.246 | 2.01 | 82 | 0.1513 | 0.01703 | 0.1767 | 0.355 | 0.50 | 36.8 |
| 1458 | 6.6.1 | 24.7 | 874 | 1004 | 0.70 | 8.60 | 163 | 0 | 16 | 0.259 | 2.12 | 86 | 0.1581 | 0.01645 | 0.1707 | 0.353 | 0.48 | 40.5 |
| 1510 | 6.6.2 | 24.7 | 875 | 1004 | 0.85 | 8.45 | 152 | 0 | 16 | 0.244 | 2.00 | 81 | 0.1570 | 0.01662 | 0.1725 | 0.350 | 0.49 | 37.9 |
| 1522 | 6.6.3 | 24.7 | 872 | 1003 | 0.85 | 8.45 | 144 | 0 | 16 | 0.231 | 1.89 | 77 | 0.1600 | 0.01694 | 0.1758 | 0.357 | 0.49 | 36.5 |
| | | | | | $\bar{X}$ = 1.08 | | | | | $\bar{X}$ = 0.262 | $\bar{X}$ = 2.14 | Σ = 523 | MWA = 0.1576 | | MWA = 0.1784 | $\bar{X}$ = 0.351 | $\bar{X}$ = 0.51 | $\bar{X}$ = 40.8 |
| | | OVERALL | | | $\bar{X}$ = 1.57 | | | | | $\bar{X}$ = 0.427 | $\bar{X}$ = 3.16 | Σ = 2,558 | MWA = 0.1321 | | MWA = 0.1587 | $\bar{X}$ = 0.339 | $\bar{X}$ = 0.47 | $\bar{X}$ = 53.4 |

# TABLE 3
## SUMMARY DATA SHEET - FULL SCALE OFFGAS TESTS

DATE: March 15, 2018

SITE: RIVERSIDE CA
SYSTEM: EDI MINI PANELS
SUBMERGENCE: 15.90 FT.
SWD: 16.80 FT.
DIFFUSERS/BASIN: 770

MLSS: 6,830 MG/L
MLVSS: MG/L
TDS: 1,000 MG/L (ASSUMED)
SRT: DAYS
TOTAL AIR RATE: 3,418 SCFM

LOCAL BAROMETER: 29.26 in. Hg.
BETA: 0.97
$C^*_{20}$: 10.70 MG/L
$C^*_F$: 9.30 MG/L

$H_R$: 0.00
$H_{OG}$: 0.00 LB $H_2O$/LB B.D. AIR
$CO_2$: 0.00

### MBR AERATION BASIN #4

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| TIME | STATION | ML TEMP °C | M(og) (mv) | M(r) (mv) | C (mg/l) | $C^*_F$-C (mg/l) | Rmm 1 | Rmm 2 | HOOD AREA (sq ft) | OFFGAS FLUX (scfm/sq ft) | SCFM PER DIFFUSER | TOTAL AIR FLOW (cfm) | $OTE_F$ (decimal) | $OTE_{SP20}$ (decimal) | SOTEpw | SOTEcw | ALPHA(F) | OUR (mg/l/hr) |
| 1026 | 4.1.1 | 24.4 | 934 | 1005 | 2.40 | 6.90 | 0 | 215 | 16 | 1.348 | 6.53 | 449 | 0.0876 | 0.01143 | 0.1186 | 0.322 | 0.37 | 116.7 |
| 1039 | 4.1.2 | 24.4 | 918 | 1000 | 2.35 | 6.95 | 195 | 136 | 16 | 1.181 | 5.72 | 393 | 0.1015 | 0.01316 | 0.1366 | 0.327 | 0.42 | 118.5 |
| 1050 | 4.1.3 | 24.4 | 902 | 1005 | 2.15 | 7.15 | 0 | 178 | 16 | 1.122 | 5.44 | 374 | 0.1264 | 0.01593 | 0.1653 | 0.330 | 0.50 | 140.2 |
| 1100 | 4.1.4 | 24.4 | 901 | 1006 | 2.20 | 7.10 | 0 | 150 | 16 | 0.953 | 4.62 | 317 | 0.1287 | 0.01633 | 0.1695 | 0.337 | 0.50 | 121.2 |
| 1114 | 4.1.5 | 24.4 | 909 | 1007 | 2.25 | 7.05 | 0 | 130 | 16 | 0.838 | 4.06 | 279 | 0.1200 | 0.01534 | 0.1592 | 0.342 | 0.47 | 99.4 |
| 1125 | 4.1.6 | 24.4 | 885 | 1007 | 2.05 | 7.25 | 0 | 130 | 16 | 0.838 | 4.06 | 279 | 0.1487 | 0.01848 | 0.1918 | 0.342 | 0.56 | 123.2 |
| | | | | | $\bar{X}$ = 2.23 | | | | | $\bar{X}$ = 1.047 | $\bar{X}$ = 5.07 | Σ = 2,091 | MWA = 0.1158 | | MWA = 0.1532 | $\bar{X}$ = 0.333 | $\bar{X}$ = 0.47 | $\bar{X}$ = 119.9 |
| 1206 | 4.2.1 | 24.4 | 890 | 1008 | 1.40 | 7.90 | 0 | 78 | 16 | 0.546 | 5.51 | 182 | 0.1434 | 0.01635 | 0.1697 | 0.321 | 0.53 | 77.4 |
| 1217 | 4.2.2 | 24.4 | 846 | 1004 | 1.35 | 7.95 | 0 | 77 | 16 | 0.540 | 5.45 | 180 | 0.1913 | 0.02168 | 0.2250 | 0.321 | 0.70 | 102.1 |
| 1304 | 4.2.3 | 24.5 | 888 | 1005 | 1.70 | 7.60 | 0 | 75 | 16 | 0.529 | 5.34 | 176 | 0.1430 | 0.01691 | 0.1755 | 0.322 | 0.55 | 74.8 |
| 1315 | 4.2.4 | 24.5 | 887 | 1005 | 1.90 | 7.40 | 0 | 62 | 16 | 0.458 | 4.62 | 153 | 0.1442 | 0.01751 | 0.1817 | 0.329 | 0.55 | 65.3 |
| 1325 | 4.2.5 | 24.5 | 885 | 1004 | 2.10 | 7.20 | 250 | 0 | 16 | 0.391 | 3.95 | 130 | 0.1453 | 0.01814 | 0.1882 | 0.337 | 0.56 | 56.2 |
| 1336 | 4.2.6 | 24.5 | 881 | 1005 | 1.85 | 7.45 | 220 | 0 | 16 | 0.347 | 3.50 | 116 | 0.1509 | 0.01821 | 0.1890 | 0.343 | 0.55 | 51.8 |
| | | | | | $\bar{X}$ = 1.72 | | | | | $\bar{X}$ = 0.469 | $\bar{X}$ = 4.73 | Σ = 936 | MWA = 0.1538 | | MWA = 0.1884 | $\bar{X}$ = 0.329 | $\bar{X}$ = 0.57 | $\bar{X}$ = 71.3 |
| 1404 | 4.3.1 | 24.5 | 847 | 1005 | 1.55 | 7.75 | 130 | 0 | 16 | 0.211 | 2.27 | 92 | 0.1906 | 0.02210 | 0.2294 | 0.355 | 0.65 | 39.8 |
| 1415 | 4.3.2 | 24.5 | 870 | 1006 | 1.05 | 8.25 | 172 | 0 | 16 | 0.273 | 2.94 | 120 | 0.1649 | 0.01797 | 0.1865 | 0.344 | 0.54 | 44.5 |
| 1426 | 4.3.3 | 24.5 | 881 | 1007 | 1.00 | 8.30 | 170 | 0 | 16 | 0.269 | 2.90 | 118 | 0.1532 | 0.01659 | 0.1722 | 0.344 | 0.50 | 40.8 |
| 1437 | 4.3.4 | 24.5 | 858 | 1007 | 1.10 | 8.20 | 80 | 0 | 16 | 0.138 | 1.49 | 60 | 0.1800 | 0.01973 | 0.2048 | 0.370 | 0.55 | 24.6 |
| | | | | | $\bar{X}$ = 1.18 | | | | | $\bar{X}$ = 0.223 | $\bar{X}$ = 2.40 | Σ = 390 | MWA = 0.1698 | | MWA = 0.1952 | $\bar{X}$ = 0.353 | $\bar{X}$ = 0.56 | $\bar{X}$ = 37.4 |
| | | OVERALL | | | $\bar{X}$ = 1.71 | | | | | $\bar{X}$ = 0.579 | $\bar{X}$ = 4.07 | Σ = 3,418 | MWA = 0.1465 | | MWA = 0.1789 | $\bar{X}$ = 0.338 | $\bar{X}$ = 0.53 | $\bar{X}$ = 76.2 |

# A P P E N D I X  I

## 1983 OFFGAS PAPER
## BY
## REDMON, ET. AL.

# Oxygen transfer efficiency measurements in mixed liquor using off-gas techniques

David Redmon, William C. Boyle, Lloyd Ewing

It has been reported that approximately 1.75 million hp of aeration equipment is currently in place on the North American continent, in both municipal and industrial treatment facilities.[1] These facilities are being operated at a power cost exceeding $0.6 billion per year. Evidence suggests that the overall oxygen transfer efficiency for this equipment is low and the cost of power could be reduced by as much as 50% by improved design and operation.[1] Although there are many reasons for imperfect application of oxygen transfer devices in wastewater, one basic cause has been the unavailability of, or failure to use, optimal methods for the measurement of oxygen transfer.

**The off-gas measurement technique may be a tool for obtaining more useful design data for aeration systems.**

Consensus procedures for testing oxygen transfer devices in clean water are being developed.[2-4] Adequate test procedures for the assessment of aeration equipment under actual process conditions are less developed at this time. Several methods have been employed over the years to evaluate oxygen transfer in suspended growth systems. In a detailed review of these dirty water test procedures. the American Society of Civil Engineers (ASCE) Committee on Oxygen Transfer Standards outlined the assumptions required for these methods and the limitations for each procedure.[5,6] Table 1 briefly summarizes some of these constraints.

Most field test procedures may be classified according to two criteria: the presence or absence of wastewater flow (continuous versus batch tests); the rate of change of dissolved oxygen (DO) in the test volume (steady state versus unsteady state tests). In general, steady state tests are simpler to perform than unsteady state tests, but they do not provide an estimate of the effective DO saturation value in submerged aeration systems. Both procedures require an accurate determination of oxygen uptake rate (OUR) and test volume dissolved oxygen (DO) concentrations that are constant and greater than zero. In an effort to improve the accuracy of the OUR measurement and to ensure steady state conditions within the test cell with respect to DO, flow, and OUR, batch endogenous tests may be performed. These tests, however, often do not realistically project operating conditions.

Several other test procedures for field oxygen transfer are proposed to overcome some of these limitations. Tracer test methods have been used in both clean and dirty water oxygen transfer tests.[6-8] Although the procedure is very extensive and costly, the results obtained with this method are very precise and presumably accurate. The method does not require complete mixing in the test volume or aeration tank DO values greater than zero. In submerged aeration systems, however, this method suffers the same disadvantage as other steady state tests, which is the inability to estimate the effective DO saturation value.

The performance of a mass balance on oxygen in the gas phase under process conditions has been referred to as the off-gas method. This procedure offers a number of advantages over more traditional techniques currently used for this purpose. This paper describes this method, discusses its limitations, and provides data on recently conducted field studies.

## HISTORICAL PERSPECTIVE

The use of off-gas measurements in biological reactors is not a new concept. Initially, off-gas analyses were performed to estimate the respiratory demand of biological cultures. As early as 1939, Sawyer and Nichols[9] described a volumetric method used in a closed system to determine the *in situ* oxygen uptake of activated sludge in the laboratory. Hoover *et al.*[10] in 1954 described a method of aeration control in a fermentation system using a paramagnetic oxygen analyzer developed earlier by Pauling *et al.*[11] Pirt and Callow[12] also used *in situ* respiratory demand measurements in studies on the continuous production of butanediol. Both oxygen and $CO_2$

*OFFGAS PAPER*

**ERRATA: Page 1339**

* In Equation (1), substitute $V\rho' \frac{dC}{dt}$ for $V\rho \frac{d\bar{Y}}{dt}$

* Substitute " = " for " − " between $Y_{og}$ and $K_La$

* Add Definition of $\rho'$ = density of liquid
  after $\rho$ = density of oxygen

**Table 1—Assumptions and limitations for dirty water oxygen transfer testing.**

| Major assumptions | Limitations |
|---|---|
| Test cell DO constant (spatial/temporal) | Estimate of OUR |
| Influent flow to cell constant | Estimate of effective DO saturation value |
| Influent DO to cell constant | DO must be greater than zero |
| Test cell OUR constant | Test performed under true process conditions |
| Effective $K_L^a$ in cell constant | |

were monitored by Orsat analysis. Some of the first field studies that were reported on the use of off-gas techniques to evaluate aeration devices under process conditions were outlined by Downing[13] and Downing *et al.*[14] In this method, oxygen was captured with a light hood covering all or a portion of the aeration tank [capture areas varied from 2.3 to 13.8 $m^2$ (25 to 149 sq ft)] and the captured oxygen was determined with a paramagnetic oxygen analyzer. These authors calculated the effective overall transfer rate ($\alpha K_L a$), therefore, both captured gas flow rate and equilibrium saturation DO had to be estimated. Gas flow rate was measured by $CO_2$ injection and material balance calculation. The DO saturation value for these diffused air systems was calculated by a mid-depth correction. Barker *et al.*[15] described an off-gas method used to estimate the oxygen transfer efficiency of a turbine aerator under process conditions. An inverted 0.2-$m^3$ (55-gal) drum was used to capture the off-gas. Oxygen was determined by a paramagnetic oxygen analyzer and transfer was expressed as percent oxygen transfer efficiency (OTE). Off gas methods were also described by Conway and Kumke[16] for analyzing a sparged turbine in clean water. Similar to the method of Barker *et al.*, off-gas was captured with a 0.2-$m^3$ (55-gal) container. Gas analyses were performed by both a mass spectrometer and a direct reading oxygen analyzer. Results were reported as percent OTE. Leary *et al.*[17,18] conducted extensive off-gas analyses of the Milwaukee Jones Island aeration tanks from 1967 to 1968. A 46-cm (18-in.) diameter hood was used to collect off-gas, and analyses were performed using both Orsat and gas chromatographic techniques. Data was reported as both percent OTE and $\alpha K_L a$. More recently, off-gas techniques have again been proposed as a procedure for aeration control in field installations[19,20] completing the cycle initiated by Hoover *et al.* in 1954.

## THEORETICAL DEVELOPMENT

The oxygen transfer capability of a submerged air device may be estimated by means of a gas phase mass balance over the aerated volume. A number of assumptions may be made to simplify this analysis. These include the following:

- Inerts, including nitrogen are conservative; that is, there is no net absorption or desorption of the constituents in question;
- There is negligible denitrification at the test location;
- The air flow rate to the basin is constant during the test;
- The barometric pressure is constant during the test,
- The off-gas humidity is equivalent to the saturated value at mixed liquor temperature where the latter is less than the instrument inlet temperature, though in other cases it will be equivalent to the saturated value at the instrument inlet temperature; and
- Negligible oxygen transfer is taking place at the liquid surface.

Referring to Figure 1, a gas phase mass balance over the liquid volume, $V$, may be written

$$V\rho \frac{d\bar{Y}}{dt} = \rho q_i Y_R - \rho q_o Y_{og} - K_L a(C^* - C)V \qquad (1)$$

where

$\rho$ = density of oxygen at temperature and pressure at which gas flow is expressed ($M/L^3$),
$q_i$, $q_o$ = total gas volume flow rates of inlet and outlet gases ($L_3/t$),
$Y_R$, $Y_{og}$ = mole fractions (or volumetric fractions) of oxygen gas in inlet and outlet gases,
$K_L a$ = overall oxygen mass transfer coefficient, ($1/t$)
$C^*$ = saturation concentration of oxygen in test liquid in equilibrium with exit gas ($M/L^3$),
$C$ = equilibrium concentration of oxygen in test liquid, ($M/L^3$), and
$V$ = test cell volume ($L^3$)

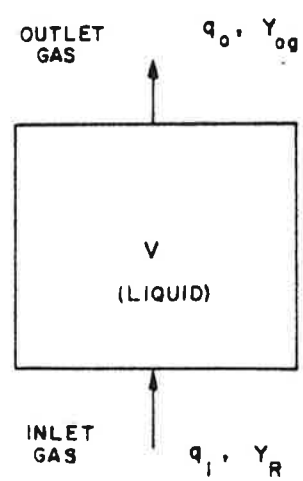



**Figure 1—Gas phase mass balance.**

At steady state

$$\frac{\rho}{V}(q_i Y_R - q_o Y_{og}) = K_L a(C^* - C) \quad (2)$$

If one assumes that the volume of $CO_2$ produced and imparted to the gas stream just equals that of oxygen absorbed, and that nitrogen is conservative, this equation reduces to

$$K_L a = \frac{\rho}{V} q \frac{(Y_R - Y_{og})}{(C^* - C)} \quad (3)$$

where $q = q_i = q_o$

The value of $K_L a$ may be estimated from Equation 2 or 3 provided that measurements are made of $Y_R$ and $Y_{og}$, the inlet and outlet mole fractions of oxygen ($q$), the total gas flow rate, and $C$. In addition, an estimate must be made of $C^*$ under test conditions.

Another method of reporting oxygen transfer is by the calculation of OTE expressed as a fraction.

$$\text{OTE} = \frac{\rho q_i Y_R - \rho q_o Y_{og}}{\rho q_i Y_R} \quad (4)$$

Again, if one assumes $CO_2$ evolution is equivalent to oxygen absorption, this equation reduces to

$$\text{OTE} = \frac{Y_R - Y_{og}}{Y_R} \quad (5)$$

Equations 4 and 5 simplify the computation of oxygen transfer in a given system because no estimate of $C^*$ is required, although gas flow rates must be accurately monitored if a correction for $CO_2$ evolution is to be accounted for in Equation 4.

Gas flow measurements may be omitted from Equation 4 by using molar ratios of inlet and outlet oxygen to the inert gas fractions as given below:

$$\text{OTE} = \frac{\text{mass } O_2 \text{ in} - \text{mass } O_2 \text{ out}}{\text{mass } O_2 \text{ in}}$$

$$= \frac{G_i(M_o/M_i)MR_{o/i} - G_i(M_o/M_i)MR_{og/i}}{G_i(M_o/M_i)MR_{o/i}}$$

and

$$\text{OTE} = \frac{MR_{o/i} - MR_{og/i}}{MR_{o/i}} \quad (6)$$

where

$G_i$ = mass rate of inerts (including nitrogen & argon) ($M/t$),
$M_o$, $M_i$ = molecular weights of oxygen and inerts, and
$MR_{o/i}$, $MR_{og/i}$ = mole ratio of oxygen to inerts in inlet and in off-gas.

The mole ratio of oxygen to inerts may be expressed by Equations 7 and 8 as

$$MR_{o/i} = \frac{Y_R}{1 - Y_R - Y_{CO_2(R)} - Y_{W(R)}} \quad (7)$$

and

$$MR_{og/i} = \frac{Y_{og}}{1 - Y_{og} - Y_{CO_2(og)} - Y_{W(og)}} \quad (8)$$

where

$Y_{CO_2(R)}$, $Y_{CO_2(og)}$ = mole fractions of $CO_2$ in inlet gas ($R$) or off-gas ($og$), and
$Y_{W(R)}$, $Y_{W(og)}$ = mole fractions of water vapor in inlet gas ($R$) or off-gas ($og$).

Equations 7 and 8 may be substituted into Equation 6 to estimate OTE. It may be noted in the rare case that the mole fraction of $CO_2$ produced just equals that of oxygen absorbed, Equations 6, 7 and 8 reduce to Equation 5.

Finally, the value of $Y_{og}$ and $Y_R$ may be calculated as follows:

$$Y_R = 0.2095\,(1 - Y_{W(R)}) \quad (9)$$

and

$$Y_{og} = \left(\frac{MV_{(og)}}{MV_{(R)}}\right) Y_R \quad (10)$$

where a sensor with linear response of millivolts to partial pressure of oxygen is used and where $MV_{(og)}$, $MV_{(R)}$ are the millivolt output readings of the oxygen sensor, which have been corrected as required for absolute sensor cell pressures and temperatures.

Additional refinements in these equations for nitrogen solution or volatilization may also be made, but preliminary calculations indicate that this correction is minor and may normally be omitted from the calculations.

## INSTRUMENTATION

There seems to be general agreement among investigators that off-gas techniques offer substantial advantages over most other procedures for determining oxygen transfer efficiencies for submerged aeration systems under process condition. The greatest drawbacks have been related to the instrumentation. Two practical problems had to be overcome to make the method more acceptable. The first was the need for a gas collection device that was light and easy to handle, but large enough to collect a representative off-gas sample. The early work reported by Downing[13] indicated that the smaller the hood the more variable was the measurement of $K_L a$. The second problem was the selection of an oxygen sensor that could precisely detect small differences in the partial pressure of oxygen, and be adaptable to *in situ* measurements. Over the years investigators have employed gas chromatography, paramagnetic oxygen analyzers and, more recently, polarographic probes and electrochemical galvanic cells.

The four major components of the off-gas equipment used in this investigation were a floating hood to capture the gas, a hose connecting the hood to the analytical

circuit, an analytical circuit for monitoring off-gas composition, temperature, pressure, and gas flow rate, and a vacuum source to draw gas from the hood through the analytical circuit.

The hood used for these studies was a section of a 0.6-m (2-ft) diameter pipe cut longitudinally along the pipe diameter to a length of 2.67 m (8.75 ft), to provide a surface capture area of 1.62 $m^2$ (17.5 sq ft). The hood was provided with ballast tanks to ensure that it remained stable within a given sampling cross section. A 38-mm (1.5-in.) diameter connecting host carried the exhaust gases to an analytical circuit. Pressure under the hood was monitored by means of plastic tubing leading from a port on the hood to the analytical circuit. Suction of exhaust gas from the hood was provided by a vacuum cleaner. The suction line was valved to maintain a small, but constant negative or positive pressure (±0.2 in.) under the hood. A slight but constant vacuum on the order of −4.0 to −6.0 in. water was maintained in the analytical circuit when off-gas and reference air measurements were made.

The analytical circuit is depicted in Figure 2. A polarographic DO probe was used to measure oxygen partial pressure in the off-gas and reference air samples. Later, during the investigations, an electrochemical galvantic cell was used in series with the probe. Carbon dioxide was monitored batchwise by bleeding off gas flowing through the circuit to a volumetric $CO_2$ analyzer.

The gas was analyzed in this circuit by passing a small portion of the test gas through a flowmeter and past the

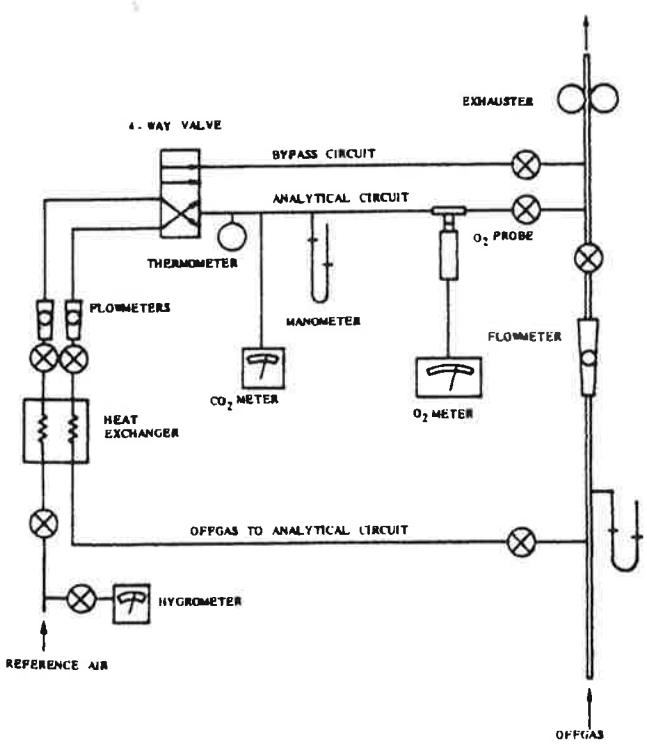



**Figure 2—Schematic diagram of off-gas analyzer circuit.**

oxygen probe. Gas temperature, humidity, and pressure were monitored and controlled within the circuit so that the difference in partial pressure of oxygen between the reference air and off-gas could be precisely obtained.

## CONDUCT OF THE TEST

Prior to the field test, the DO sensor was checked daily for accuracy with gases of certified composition. Pure nitrogen was also passed through the analytical circuit under test conditions to ensure that no leaks existed. These tests were normally conducted several times during the day.

The linearity of the probe was checked periodically by drawing the reference air past the sensor under various levels of reduced pressure while keeping the gas temperature constant. Under these circumstances the partial pressure of oxygen was directly proportional to the total absolute pressure. The criterion applied was that the ratio of absolute pressures for the two conditions divided by the ratio of the meter outputs fell in the range of 0.995 to 1.005.

Using this test for linearity, it was found that the calibration setting for probe output (in millivolts) was not critical, and essentially the same relative change in the voltage output occurred regardless of the setting with reference at ambient conditions. To obtain the maximum sensitivity, the reference output was set as close to full scale as was practical, because probe error is normally a fixed fraction of the full-scale reading. It should be emphasized that this procedure required precise measurement of the difference in the two gas streams and, therefore, it was not necessary to have an accurate determination of the absolute value of the partial pressure of oxygen in either stream.

Once the analytical circuit was checked, the gas collection hood was fixed in place at a predetermined location and a vacuum drawn at the instrument discharge. Reference air was first drawn into the analytical circuit and, a series of observations of temperature, humidity, pressure, and sensor millivolt readings were recorded over a period of about 5 minutes. A portion of the off-gas was diverted through the analytical circuit for a typical period of 5 to 10 minutes. During this time adjustments to the volume of off-gas drawn from the collection hood were made to obtain an equilibrium pressure condition beneath the hood at near ambient pressure (±0.2-in. water). Parallel measurements were recorded for off-gas temperature, humidity, pressure, $CO_2$ concentration, and sensor millivolt reading. Total off-gas flow rate was also recorded and used later in calculation of a bulk OTE for the entire tank volume. In addition to these measurements, mixed liquor temperature, DO, and local barometric pressure were also recorded. The hood was then moved to a new location and reference gas was again drawn through the circuit and measurements recorded. The reference gas

check between each off-gas measurement provided a means for simple probe drift correction if required. This technique reduced errors in measuring the differences between off-gas and reference streams. The entire procedure for one determination required approximately 15 minutes.

The method involved sampling a sufficient number of locations within a given basin to obtain a representative sample of off-gas from the tank or tank element. A reasonable agreement between the applied and collected air flow rates was used to indicate a representative sampling layout. Initial field studies were conducted by drawing reference air directly from the plant air line. However, experience at a number of plants indicated that ambient air drawn directly within the vicinity of the analytical system yielded nearly identical readings to plant air. This approach greatly simplified the test procedure, and was adopted as a standard procedure thereafter.

The value of OTE for each sampling point was calculated using Equations 6, 7, 8, 9, and 10. Temperature corrections were made using the expression:

$$OTE_{20°} = OTE_T \, 1.024^{(20-T)} \tag{11}$$

In order to estimate a weighted bulk average OTE for an entire tank or tank element, the following equation was used.

$$OTE_w = \frac{\sum_0^n OTE_n q_{an}}{\sum q_{an}} \tag{12}$$

where

$OTE_w$ = weighted OTE
$OTE_n$ = OTE value at sampling point, $n$, and
$q_{an}$ = the collected gas flow rate per unit surface area at sampling point, $n$, $(L^3/tL^2)$.

To provide comparisons between values of OTE measured for different systems, one may normalize the measured OTE by dividing it by $(C^* - C)$,

$$OTE_{sp} = \frac{OTE}{C^* - C} \tag{13}$$

In this equation, $C^*$ is the calculated saturation value of oxygen in the wastewater. In diffused air systems $C^*$ may be estimated by

$$C^* = C^*_{20} \frac{P_b}{P_s} \frac{C_{sT}}{C^*_{s20}} \beta \tag{14}$$

where

$C^*_{20}$ = clean water saturation value in the same geometry at comparable air flow rate and at standard conditions $(M/L^3)$,
$P_b$ = atmospheric pressure during the test $(f/L^2)$,
$P_s$ = standard atmospheric pressure (usually 1.00 atm at 100% relative humidity $(f/L^2)$,
$C_{sT}$ = surface saturation book value for dissolved oxygen at the test temperature $(M/L^3)$,
$C^*_{s20}$ = the surface saturation book value for dissolved oxygen at 20°C $(M/L^3)$, and
$\beta$ = the ratio of process wastewater saturation value to that of clean water.

The value of $C^*_{20}$ is often available from clean water tests of the diffuser system under the appropriate process configurations and gas flows. If it is not available it must be calculated using a gas-side correction model assuming some effective saturation depth.[2]

The data presented in this paper are uncorrected field measured OTE values except where subscripted as $OTE_{20°}$ or $OTE_{sp}$.

## RESULTS

Off-gas field studies have been conducted during the past 3 years at over nine treatment facilities, including two industrial plants, and a variety of diffused air systems. Early tests were conducted at Whittier Narrows, California, Madison, Wisconsin, Brandon, Wisconsin, Ridgewood, New Jersey, and a DuPont facility. These tests were conducted in parallel with other field test procedures for comparative purposes. Other off-gas studies were conducted for a variety of purposes including:

- Evaluation of diffuser clogging problems,
- Evaluation of effectiveness of diffuser cleaning procedures,
- Evaluation of several diffuser types in side-by-side tests under process conditions, and
- Evaluation of aeration system control procedures.

**Comparative test results.** Table 2 presents the results of comparative tests conducted at four field sites. In general, the comparisons of the off-gas procedure with other currently-used field techniques are good. These tests reported were conducted under ideal conditions where the comparative test methods were applicable.

For example, field tests at Sites A and B were conducted at plants where mixed liquor DO values generally exceeded 0.5 mg/L and where approximate steady state conditions were achieved. Site A used both tapered aeration and step aeration thereby producing a wide variation in point values of OTE as measured by off-gas methods. Point values of steady state respirometeric tests versus off-gas data were, of course, not applicable, but overall bulk average values were comparable. At Site B, the small aeration tank was almost completely mixed with respect to DO. Point values of off-gas OTE varied substantially along test cross sections depending on off-gas flow rates (discussed later), but comparisons between tests were reasonable.

Results at Site C, another municipal plant, were also very favorable. At this plant, a non-steady state method with hydrogen peroxide was used for comparison. This

**Table 2—Selected field test results off-gas versus other field methods.**

| Site | Diffuser system | Off gas analyses OTE-% (avg) | Off gas analyses OTE-% (range) | DO-mg/L (range) | OTE-% (comparison test) | Comparison method |
|---|---|---|---|---|---|---|
| A | Floor coverage ceramic | 11.85 | 8.0–16.7 | 0.9–3.9 | 12.68 | Respir. rate steady state |
| A | Dual roll coarse bubble | 6.16 | 5.1–6.8 | 2.0–4.1 | 6.29 | Respir. rate steady state |
| B | Jet aeration along longitud. wall (q = 67 L/s) | 5.34 | 4.5–7.7 | 3.8[a] | 5.31 | Respir. rate steady state |
| | (q = 18 L/s) | 9.58 | 8.7–11.1 | 1.1[a] | 12.5 | Respir. rate steady state |
| C | Spiral roll coarse bubble | 3.2 | — | 1.8–4.1 | 3.3 | $H_2O_2$/non-steady state |
| C | Spiral roll coarse bubble | 1.1 | — | 6.8–7.2 | 1.7 | $H_2O_2$/Non-steady state |
| D | Floor coverage coarse bubble | 7.7 | 5.5–11.1 | 0 | 7.3–7.8[b] | Radioactive tracer |

[a] Average DO in approximately CSTR system.
[b] Range of values depending on actual air flow rate.
[c] All OTE values calculated under field conditions and not corrected to 20°C.

method, described by Kayser,[5,21] must be performed under steady state conditions. Such conditions were established at Site C by diverting a substantial amount of the wastewater flow from the test system. Of special note, here, was the excellent agreement achieved between these two methods at an OTE value of 3.3%. The apparent poor agreement at the second test condition was not surprising considering the extremely low operating efficiency at that condition. The effectiveness of off-gas methods at low OTE values was of great concern to the investigators owing to the extremely small differences between reference air and off-gas oxygen concentrations. The excellent comparability of these methods at Site C was further reinforced by nonradioactive tracer tests performed at this site.

The tests conducted at Site D were reported by Campbell.[6] Here, the radioactive tracer technique using Krypton-85 and tritium was compared against the off-gas procedure. Excellent agreement was reported between the two methods in this highly loaded, single aeration tank where complete-mix conditions were approximated. The wide range of off-gas OTE point values measured at this facility was primarily the result of the variations in off-gas flow rates [3.4 to 4.5 L/m$^2$/s (0.67 to 0.88 cfm/ft$^2$)]. Even though the coarse bubble diffusers were distributed over the entire tank floor, there was a substantial localized boiling along the tank surface.

Important to note in all of these comparative tests, is the method of computation used to convert $\alpha K_L a$ values, which were estimated by all the other field test procedures, to OTE values measured by the off-gas procedure. The conversion was executed by the following general relationship,

$$\mathrm{OTE} = \frac{\alpha K_L a (C^* - C) V}{q\rho} \qquad (15)$$

To achieve this comparison, it was necessary to have good estimates of $C^*$ for the diffuser system geometry in the process wastewater at the appropriate temperature, pressure, and $q$, the gas flow rate of the diffuser system.

**Other test results.** The value of off-gas measurements goes far beyond the ability to estimate bulk OTE values for a given system. No other field test for oxygen transfer can provide discrete point information for a given system. The analysis of oxygen transfer at a specified location can greatly benefit the design engineer and the operator. A few examples of this information are presented to illustrate this extra benefit of the method.

Figure 3 presents the results of OTE and off-gas flow rate measurements that were observed at twelve cross-sections along the aeration tank at Site A. Floor coverage ceramic diffusers were used at this site. Tapered aeration was employed and primary effluent was discharged in equal amounts at three points along this folded tank. Tapered air was achieved in these tanks by varying diffuser density and, to some extent, by throttling gas flow rates to the diffuser headers. Aeration control was accomplished through DO monitoring and manual shut-down or start-up of blowers. Figure 3 indicates that during this period, DO control appeared to be effective. Because OTE for ceramic diffusers are relatively insensitive to air flow rates per diffuser, it seemed that increases in OTE along each tank section were the result of increases in alpha values.

A similar plot for the cross roll configuration at this same site appears as Figure 4. These folded tanks were equipped with fine bubble tubes in the first bay and coarse bubble units in the last two bays. All diffusers were uniformly spaced and primary effluent was evenly split to three points. This data suggests that gas flow rates were not well distributed, perhaps because of ineffective throttling of air control valves. The decline in OTE values along the tank length was not expected and could not be

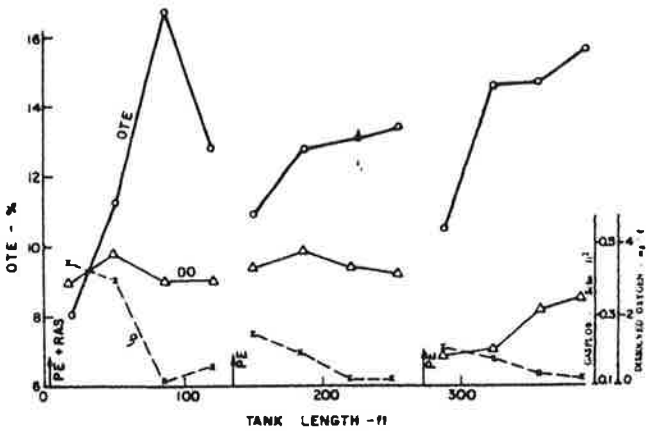



**Figure 3—Gas transfer analyses along tank length—Site A. Floor coverage, ceramic domes, and tapered air.**

entirely attributed to changes in gas flow rates. It is clear that operational corrections in this system were necessary.

An additional study was conducted near the discharge end of a three-pass cross roll system with various types of wide band tubular diffusers. A plan view of the tank indicating the header locations, diffuser locations and sampling plan is shown in Figure 5. The test results are presented in Table 3. Of particular interest was the apparent utility of the gas phase analysis which permits evaluation of several aerator types within a relatively small portion of a single tank. With tracers or liquid phase methods, such analysis is not possible.

The used units had been operating at the test plant for about 3 years. Comparing new and used Type C devices, there was a reduction in performance over that period. A similar reduction had most likely occurred with Type D diffusers, however, new Type D units were not available for test at that time.

It should be noted that the discharge end of the basin was selected to minimize potential alpha variations in the test region. The diffuser layout was such that the

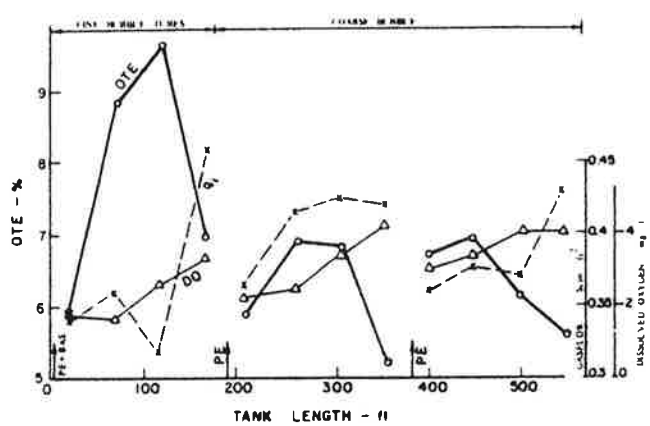



**Figure 4—Gas transfer analyses along tank length—Site A. Cross roll, fine, and coarse bubble.**

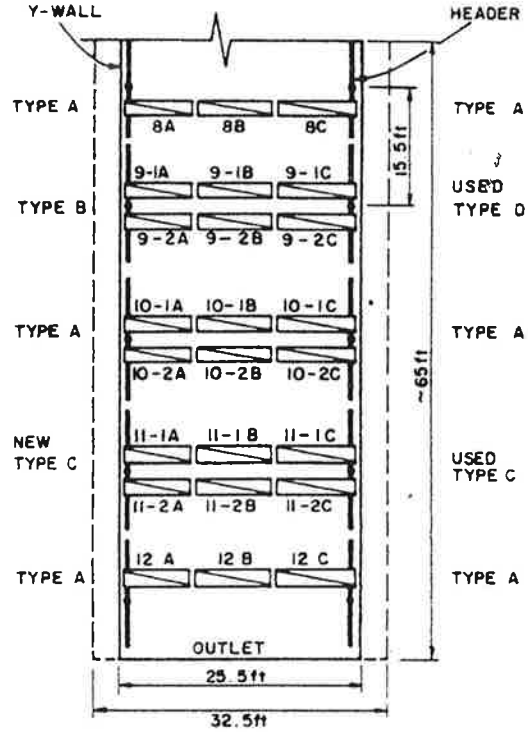



**Figure 5—Aerator and sample site layout for comparative analysis of diffusers.**

reference system, Type A, was on both ends of the test zone to pick up the relative apparent change in alpha across the test boundary. The data substantiated a constant alpha for the test region. Based on the above results, this method can be a useful tool in aeration system retrofitting consideration. It can measure the dirty water performance characteristics of various systems at a particular site under actual field conditions, with modest expenditure of time and effort. However, these systems should not be com-

**Table 3—In-process tubular diffusers comparisons with off gas procedures.**

| Diffuser | Location[a] | OTE-%[b] | DO-mg/L | $OTE_{20°0}$[c] |
|---|---|---|---|---|
| Type A | Station 8 | 7.95 | 0.9 | 8.29 |
| Type B | Station 9 | 11.62 | 1.7 | 14.18 |
| Used | | | | |
| Type D | Station 9 | 9.17 | 1.7 | 11.33 |
| Type A | Station 10 | 7.96 | 2.0 | 10.28 |
| New | | | | |
| Type C | Station 11 | 12.94 | 1.7 | 15.96 |
| Used | | | | |
| Type C | Station 11 | 9.05 | 1.7 | 11.00 |
| Type A | Station 12 | 6.99 | 1.2 | 8.29 |

[a] See Figure 5.
[b] Field OTE.
[c] OTE corrected 20°C at 0 DO (Equations 11 and 13).

pared solely on the basis of OTE measurements when new, because other factors including back pressure, power, available driving force, and maintenance should also be considered.

Sample point selection in evaluating oxygen transfer data for a given system is critical to proper assessment of the system. Figure 6 presents a layout of the aeration tank at Site B that was equipped with jet aeration along one longitudinal wall directed across the tank. Although DO values were uniform in this small basin, off-gas flow rates and OTE values were not (Table 4). As would be expected, OTE values were generally higher over points of lower gas flow discharge. To estimate overall basin transfer efficiency using Equation 12, total captured gas flows were compared against measured values. Accordance between the two (±15%) indicates a reasonable sample point selection. In this particular study, gas capture flow rates were much higher than the rated capacity of the blowers. Further evaluation of the rating curves by the manufacturer revealed that the blower capacity had been seriously underestimated. No gas flow metering devices were available at Site B.

## DISCUSSION

The results of 3 years of field experience with the off-gas procedure have been very encouraging. The procedure is relatively simple and straightforward, and the equipment required for precise and accurate *in situ* measurement is available. Hood designs will continue

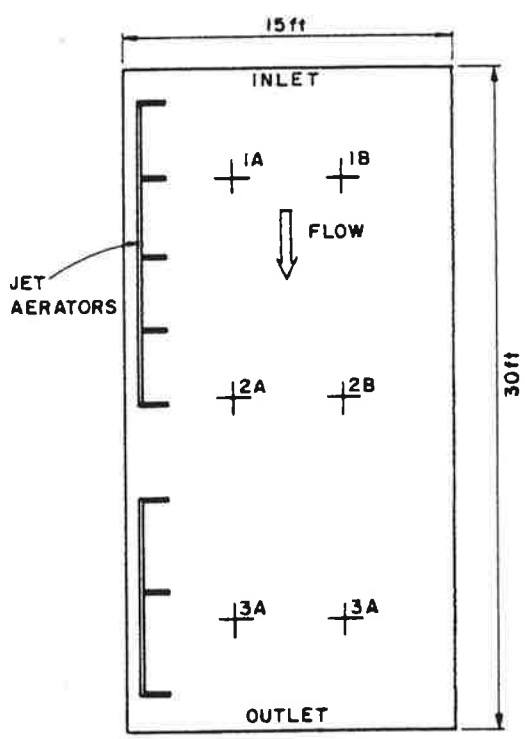



**Figure 6—Aerator and sample site layout at Site B.**

**Table 4—Off gas analysis—Site B[a]**

| Station | $q_A$-L/m²/s | OTE-% |
|---|---|---|
| 1A | 0.34 | 11.0 |
| 1B | 0.80 | 9.3 |
| 2A | 0.79 | 6.9 |
| 2B | 0.75 | 8.8 |
| 3A | 0.73 | 10.2 |
| 3B | 0.40 | 11.1 |
| Weighted Average | — | (9.58) |
| 1A | 2.21 | 4.5 |
| 1B | 0.66 | 7.7 |
| 2A | 2.27 | 4.9 |
| 2B | 2.09 | 6.2 |
| 3A | 2.61 | 4.4 |
| 3B | 0.31 | 11.9 |
| Weighted Average | — | (5.34) |

[a] See Figure 6.

to improve so that they are lighter and more portable than the ones used in this study.

There is currently no way to effectively evaluate the precision of this method under process conditions because it is difficult to achieve constant performance of aeration equipment under field conditions. Under the most uniform aeration conditions, reproducibility seems to be well within acceptable ranges for this type of field measurement (less than ±10%). It should be noted that the estimate of the reproducibility of the method may be primarily the result of changing conditions at a given sampling point rather than the precision of the analytical system.

The accuracy of this technique cannot be determined because, to date, there is no standard against which to compare field OTE measurements. Many researchers feel that the radioactive tracer procedure described by Neal *et al.*[7] represents the state-of-the art available today for oxygen transfer rate measurement under process conditions. The results of the study at Site D were very encouraging in this regard. It should be remembered, however, that the tracer measurements provided a bulk average $\alpha K_L a$, whereas the off-gas measurements represented an average of local oxygen transfer efficiencies. It is also pertinent that where the objective of the test is to measure efficiency or predict air rate requirements, the off-gas method is significantly less subject to errors resulting from air rate measurement.

The accuracy of the off gas-measurement for prediction of local OTE is dependent on a number of measured variables including gas phase oxygen concentration, gas phase carbon dioxide concentration, gas phase humidity, gas phase temperature, gas phase pressure, and rate of off-gas flow. The accuracy of the oxygen sensor was continuously checked using certified gases or reduced pressure as described earlier. Results with the sensors used in this study were excellent, which indicates a high degree of precision and accuracy. Most sensors are temperature

compensated. The effectiveness of this compensation varies, therefore each device should be carefully checked. Gas phase $CO_2$ and humidity seem to be the other major variables that influence computation of OTE. Off-gas flow rate measurement is used in Equation 12 primarily to weight OTE values. Its accuracy, therefore, is of secondary interest. On the other hand, if Equation 4 is used to estimate OTE values, a very accurate assessment of off-gas (and inlet gas) rates is required. Downing[13] employed $CO_2$ injection in the outlet gas with subsequent $CO_2$ analysis in the gas stream to ensure accurate estimates of gas flow. A sensitivity analysis of the parameters influencing the calculation of OTE will be published in the near future.

In evaluating the off-gas procedure against other methods currently available, several advantages and disadvantages may be enumerated. Some off-gas methods are particularly suitable because they:

- Measure local performance,
- Yield OTE directly and are relatively insensitive to the precision of air flow measurement,
- Are applicable in tanks where spatial variation of gas flow rate, loading, DO, and alpha exist,
- Seem to have exceptionally good precision and accuracy as compared to other conventional methods,
- Produce relatively fast and inexpensively,
- Provide a simple and reliable means of measuring alpha values in aeration systems of known clean water performance,
- Provide a means for simultaneous side-by-side comparisons of different aeration systems under process conditions,
- Are applicable in anoxic tanks, and
- Do not require process interruption.

Off-gas methods can be disadvantageous because:

- Technique is not applicable to mechanical aeration systems;
- Tanks must be accessible to personnel,
- Severe foaming may complicate gas sampling,
- Severe turbulence may cause difficulty in hood placement, and
- Method requires accurate measurements of $CO_2$ and humidity of reference air and off-gas.

Additional field research continues with the off-gas procedure. The method has a wide list of applications that extend beyond the routine measurement of bulk transfer efficiency. As indicated previously, off-gas provides air distribution and transfer efficiency profiles that can be used to evaluate system operation and maintenance requirements. Furthermore off-gas may be used to monitor temporal changes in oxygen transfer caused by diffuser clogging, alpha variations, and gas flow adjustments. Recently the off-gas method has been used to evaluate a variety of diffuser devices in side-by-side tests under process conditions.

The translation of clean water oxygen transfer data to process conditions continues to produce a significant amount of uncertainty in the design of aeration systems. Through the use of field measuring techniques, a compilation of useful data will eventually be accumulated to provide the design engineer with better scale-up data. Hopefully, the off-gas technique will provide an additional tool to achieve that end.

## ACKNOWLEDGMENTS

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**Authors.** David Redmon and Lloyd Ewing are engineers at Ewing Engineering Co., Milwaukee, Wis. William Boyle is a professor of civil and environmental engineering at the University of Wisconsin-Madison. Correspondence should be addressed to William Boyle at 3230 Engineering Bldg., University of Wisconsin-Madison, Madison, WI 53706.

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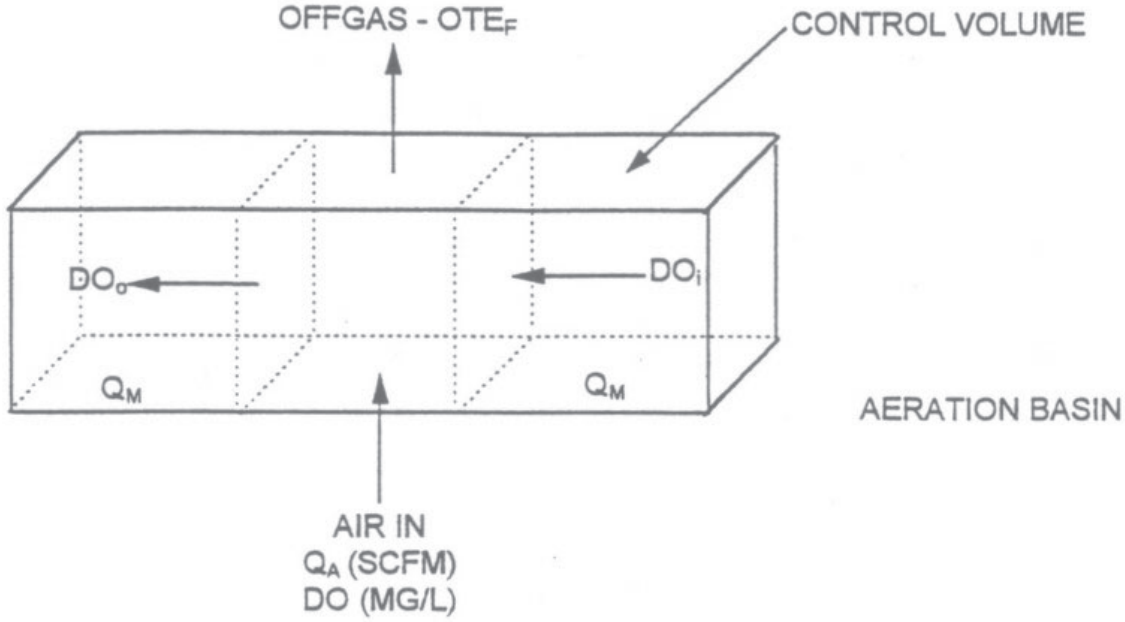
# A P P E N D I X  I I

## MASS BALANCE PROCEDURE
## TO CALCULATE
## OXYGEN UPTAKE RATE

# OUR BY GAS-PHASE MASS BALANCE




## MASS BALANCE

$$OUR = \frac{\text{OTR - DO TRANSPORT}}{\text{VOLUME OF LIQUID IN CONTROL ZONE}} - \frac{\Delta \text{ DO CONCENTRATION}}{\text{TIME}}$$

WHERE:

OUR = OXYGEN UPTAKE RATE {MG/L/HR}

OTR = OXYGEN TRANSFER RATE {MG/HR}

= $Q_A$ (1.036)($OTE_F$)(454,000)

$Q_A$ = AIR RATE TO CONTROL VOLUME IN SCFM

1.036 = LBS OF OXYGEN PER HOUR PER 1 SCFM

$OTE_F$ = GAS PHASE OXYGEN TRANSFER EFFICIENCY UNDER PREVAILING CONDITIONS BY OFFGAS ANALYSIS LBS OXYGEN TRANSFERED/LBS OXYGEN SUPPLIED

454,000 = MG/LB

DO TRANSPORT = $(DO_{OUT} - DO_{IN})(Q_M)(60)(3.785)$ {MG/LHR}

$Q_M$ = LIQUID FLOW RATE THROUGH CONTROL VOLUME IN GAL/MIN

3.785 = LITERS/GAL

VOLUME OF CONTROL ZONE = LITERS

$\frac{\Delta \text{ DO CONCENTRATION}}{\text{TIME}}$ = END OF OBSERVATION PERIOD DIVIDED BY OBSERVATION TIME IN HOURS. AT STEADY STATE CONDITIONS THIS TERM IS ZERO SINCE DO IS ZERO.

Appendix 3D

# CONDITIONAL ACCEPTANCE OF FREE CHLORINE DISINFECTION

CALIFORNIA
Water Boards

GAVIN NEWSOM
GOVERNOR

JARED BLUMENFELD
SECRETARY FOR
ENVIRONMENTAL PROTECTION

**State Water Resources Control Board**
Division of Drinking Water

May 6, 2019

Edward Filadelfia
Technical and Compliance Manager
City of Riverside Public Works Department
3900 Main Street
Riverside, CA 92522

Subject: Conditional Acceptance of Free Chlorine Disinfection at City of Riverside to comply with California Water Recycling Criteria (3390031-729)

Dear Mr. Filadelfia,

The Division of Drinking Water's (Division) Recycled Water Unit has reviewed a request, dated January 9, 2019, from Carollo Engineering to consider free chlorine as an alternative disinfection technology for compliance with the California Water Recycling Criteria (Title 22), Section 60320.5. Accompanying the request was a report entitled, "City of Riverside, Chlorine CT and Modal Contact Time Study, Contact Time Testing and CT Demonstration Results", dated November 2018. The report outlines findings from a study conducted at the Riverside Regional Water Quality Control Plant (Riverside) located in the City of Riverside, CA using Riverside's full-scale chlorine contact basin #2. The purpose of the study is to show Riverside can use free chlorine disinfection to meet Title 22 requirements, thus allowing a shorter contact time in their chlorine contact basin #2. This will allow Riverside to reduce disinfection byproducts and improve overall operational efficiency.

Demonstration studies conducted using free chlorine at Riverside have satisfactorily shown an equal degree of treatment and reliability as those technologies listed in Title 22. Based on a review of the materials submitted, the Division grants conditional acceptance of free chlorine as an alternative treatment technology for recycled water disinfection at Riverside's chlorine contact basin #2 only, subject to the following conditions:

1. Riverside must deliver a minimum free chlorine residual contact time (FCRCT) of 30 mg-min/L at all times to meet both the virus and total coliform reduction requirements of Title 22, section 60301.230.

2. Riverside must meet a minimum free chlorine residual (FCR) of 1.0 mg/L at all times at the chlorine contact basin #2 sample point #2.

3. Riverside must maintain a minimum free chlorine modal contact time at the chlorine contact basin #2 sample point #2 of 29-minutes at all times, based on a peak flow of 46-mgd.

4. Riverside must monitor for ammonia prior to the free chlorine addition and optimize the secondary treatment system to prevent ammonia breakthrough.

5. The free chlorine disinfection process must meet Title 22, Section 60301.230(b) requirements for total coliform.

6. Riverside must be operated with a built-in automatic reliability feature that must be triggered when the minimum FCRCT or FCR are below the targets. The reliability features shall be approved in a new or updated Title 22 Engineering Report for Riverside.

7. Ammonia analyzers, free chlorine analyzers, and flow meters must be installed and properly calibrated to ensure proper disinfection at all times.

8. Ammonia and free chlorine analyzers must be inspected and checked against a reference bench-top unit routinely to determine accuracy. If an on-line analyzer reading varies from the bench-top reading, the on-line analyzer must be recalibrated by a procedure recommended by the manufacturer.

9. Flow meters must be inspected and checked against other flow determination methods routinely to determine accuracy.

10. Riverside should be operated in accordance with an approved operations plan, which specifies clearly the operational limits and responses required for critical alarms. The operations plan should be submitted and approved prior to issuance of the permit. A copy of the approved operations plan should be maintained at the treatment plant and be readily available to operations personnel and regulatory agencies. A quick reference plant operations data sheet should be posted at the treatment plant and include the following information:
    a. The alarm set points for tertiary turbidity, high flow, low FCR, low contact time, and low FCRCT.
    b. The values of high tertiary turbidity, high flow, low FCR, and low FCRCT when recycled water flow must be diverted.
    c. The required frequency of calibration for critical online instruments; such as online turbidity meters, ammonia analyzers, and chlorine analyzers.

11. Riverside should maintain records of compliance with these conditions, made available to DDW or the Regional Water Quality Control Board (RWQCB) upon request. In addition, Riverside should report a summary of compliance with an approved operations plant as part of its annual reporting to the RWQCB.

12. These conditions shall be incorporated into the final permit for Riverside.

Conditional acceptance is specific to the free chlorine disinfection process at Riverside's chlorine contact basin #2 only, as detailed in the report dated November 2018 described above. The Division must review in advance any future proposed changes made in the design or

operation of this treatment technology to determine whether the modifications will require additional testing to ensure Title 22 equivalency.

Review and approval of free chlorine disinfection at Riverside is provided through the RWQCB's Water Reclamation permitting process. Prior to Riverside being permitted by the RWQCB, an approved Title 22 Engineering Report for the project is required. The engineering report shall include how Riverside meets the requirements of Title 22 and this letter. The Division's local district offices will also review the free chlorine disinfection process at Riverside to confirm full compliance with all applicable treatment and reliability features required by Title 22.

If you have any questions regarding this letter, please contact Randy Barnard at (619) 525-4022 or via email at Randy.Barnard@waterboards.ca.gov.

Sincerely,

Randy Barnard, P.E.
Recycled Water Unit Chief
Recycled Water Unit
Division of Drinking Water
State Water Resources Control Board
1350 Front St., Rm. 2050
San Diego, CA 92101

cc:

1. Hope Smythe, Executive Officer, RWQCB 8 (via email)
2. Steve Williams, P.E., District Engineer, DDW-District 20 (via email)
3. Robert Eland, City of Riverside, RWQCP, 5950 Acorn Street, Riverside, CA 92504

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 4

## Wastewater Treatment System

## Preliminary Treatment

Engineers...Working Wonders With Water®

City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 4: PRELIMINARY TREATMENT

FINAL | June 2019

City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 4: PRELIMINARY TREATMENT

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:17:02-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 4

# PRELIMINARY TREATMENT

## 4.1 Purpose

The purpose of this chapter is to describe upgrades to the existing headworks facility and upstream flow metering facilities, as well as estimate the overall capital cost of improvements through 2037. Alternatives for six major components were evaluated and are discussed in subsequent sections. They are organized by order of importance: influent metering, bypass channel, grit chambers, conveyors, screening, screenings grinding/washing/compacting, and grit washing/compacting. This chapter includes recommendations based on initial investigations. However, final decisions about specific equipment types or layouts should be determined during the preliminary and final design stages of each associated project.

## 4.2 Conclusions and Recommendations

- Influent metering solutions are discussed in Appendix 4A and a summary of the conclusions and recommendation are presented below:
  - A single, centralized influent flow meter (master meter) to measure the combined influent flow rate of all wastewater entering the plant was found to be infeasible due to hydraulic limitations at the headworks, which would cost between $50 and $70 million to be corrected.
  - A single flow metering structure for the A/A trunk sewer was also deemed infeasible due to operational constraints and negative hydraulic characteristics, such as reverse flow through the inverted siphon and backwater conditions at the headworks.
  - The Santa Ana trunk sewer flowmeter location proposed by URS was confirmed as feasible through hydraulic modeling.
  - Assuming construction of the influent metering structures proposed by URS, the total estimated project cost is approximately $6.3 million. This cost was used in the CIP for the influent metering modifications and improvements.
- The existing headworks facility has a capacity of 52 mgd on an ADF basis. This is sufficient for the estimated capacity of 39 mgd in 2037.
- A headworks bypass channel tying the influent pipelines directly into the grit basin bypass channel is recommended.
- Existing grit chambers have a treatment capacity of 60 mgd and should be rehabilitated in lieu of a costly capacity upgrade.
- Existing conveyance system in the Screenings room should be replaced in kind.
- Climber-type and multi-rake-type bar screens are two alternatives for bar screens. During preliminary design, special consideration should be given to methods of over-coming capacity limitations of multi-rake bar screens.
- Bar screen spacing should not be less than 3/8-inch.
- Existing screenings washer/compactor should be replaced with a shaftless auger compactor with increased washing capabilities.
- Grit washing technology should be selected during preliminary design.

## 4.3 Background

The headworks facilities include screening and grit removal facilities for the protection of downstream equipment and processes. Both screenings and grit are washed, dewatered, and sent to a sanitary landfill. The facility was constructed in 1999 and has not undergone any major upgrades since that time. Table 4.1 lists equipment included in the existing headworks facilities. Currently, there is no bar screen bypass channel or standby grit handling system (grit chamber, pumping, or washing system). However, flow can be routed to bypass the grit chambers when a unit is out of service. Furthermore, the existing grit chambers have a combined manufacturer's hydraulic capacity rating of 100 mgd. However, vortex grit chambers typically do not meet grit removal requirements at these high flows. Based on Carollo's past experience with other WWTPs, the treatment capacity of the City's vortex grit chambers (capacity at which grit removal meets stated manufacturer requirements) is likely around 60 mgd (30 mgd per grit chamber). However, due to variations in water quality and installation at different WWTPs, grit removal testing is recommended to verify current performance levels.

As stated in Volume 2, Chapter 3, Population, Flow, and Loading Projections, RWQCP average daily influent flow is projected to be 39 mgd. Furthermore, previous master plans and the MBR Plant Expansion Drawings used a wet weather peaking factor of 2.2 for all processes upstream of the Primary Effluent Equalization Basins. This update to the Master Plan did not re-evaluate this peaking factor; however, the results of the flow monitoring program as described in Volume 3, Chapter 3, Flow Monitoring Program, indicate a similar wet weather peaking factor of 2.4 for a single wet weather event. For the purposes of this update to the Master Plan, the wet weather peaking factor of 2.2 will be used in accordance with previous master plans. Similarly, the dry weather peaking factor used in this update to the Master Plan is 1.4.

Table 4.1 Existing Headworks Facilities

| Description | Value |
|---|---|
| Influent Screenings Generation, cu. yds./day[1] | |
| Average (wet screenings) | 15 |
| Average (washed/compacted) | 7.5 |
| Maximum (wet screenings) | 22.5 |
| Maximum (washed/compacted) | 11 |
| Grit Production, cu. yds./day[1] | |
| Average | 4 |
| Maximum | 8 |
| Influent Bar Screens | |
| Number | 4 duty |
| Width, ft. | 3.5 |
| Clear Openings Between Bars, in. | 0.5 |
| Type | Climber |
| Wet Screenings Conveyor | |
| Number | 2 duty |
| Type | Shaftless Screw |

Table 4.1 Existing Headworks Facilities (continued)

| Description | Value |
|---|---|
| Screenings Washing Compactor Unit | |
| Number | 2 duty |
| Dry Screenings Conveyor[2] | |
| Number | 2 duty |
| Type | Shaftless Screw |
| Vortex Grit Removal Units | |
| Number | 2 duty |
| Diameter, ft. | 20 |
| Hydraulic Capacity, each (total), mgd | 50 (100)[3] |
| Removal Capacity, each (total), mgd | 30 (60)[4] |
| Grit Removal Efficiency, percent | |
| >50 Mesh | 95 |
| >100 Mesh | 65 |
| Grit Pumps | |
| Number per tank | 1 |
| Capacity, gpm | 250 |
| Head, ft. | 45 |
| Type | Centrifugal Recessed Impeller |
| Grit Classifiers (Teacup) | |
| Number | 2 duty |
| Type | Hydraulic Vortex |
| Diameter, in. | 42 |
| Capacity, gpm/each | 250 |
| Grit Dewatering Equipment (Snail) | |
| Number | 2 duty |
| Belt Width, in. | 18 |

Notes:
(1) Based on a design ADF of 50 mgd and a peaking factor of 2.0.
(2) Compacted screenings and washed grit are conveyed together to the roll-away basin.
(3) Based on manufacturer's hydraulic rated capacity.
(4) De-rated removal capacity (60 percent of manufacturer's hydraulic rated capacity) based on Carollo experience.
(5) Source: 2008 City of Riverside Wastewater Collection and Treatment Facilities Integrated Master Plan.

## 4.4 Influent Metering Structure

As mentioned in Volume 4, Chapter 2, Summary of Planning Studies, the City commissioned URS to complete a PDR addressing the concerns regarding influent metering limitations. URS proposed new metering facilities and upgrading to electromagnetic flowmeters. In addition to the work completed by URS, Carollo summarized evaluation options for installing centralized electromagnetic flowmeters (a) for the combined influent wastewater flow (master meter), (b) for the A/A Trunk Sewer, and (c) the Santa Ana Trunk Sewer as detailed in Appendix 4A. However, the combined influent flow meter and the A/A trunk sewer combined influent flowmeter alternatives were both deemed to be not feasible for hydraulic, operational, and cost

reasons. The location of the Santa Ana Trunk Sewer flow meter as proposed by URS was hydraulically confirmed as feasible and is recommended. For planning purposes, this update to the Master Plan assumes the flow metering structures recommended by URS in the PDR are to be constructed. Therefore, the costs URS prepared in the PDR were updated per Volume 2, Chapter 4, Basis of Cost Estimate, and used for the influent flow metering improvements as part of the CIP.

## 4.5 Bypass Channel

Under the current method of operation, all four bar screens operate continuously. The conformed 2012 Phase 1 Plant Expansion drawings for the MBR treatment train (Plant 1) expansion were developed using the ADF of 52 mgd and a peaking factor of 2.2 upstream of the primary EQ basins. According to the revised hydraulic profile provided in the conformed 2012 drawings, there is less than a foot of freeboard in the bar screen channels at a peak flow of 114.4 mgd. Furthermore, experience has shown that when one bar screen blinds (i.e., becomes clogged by debris), the remaining screens blind shortly afterward leading to a flooded headworks. Based on Volume 6, Chapter 1, Plant 1 Condition Assessment Results a headworks bypass channel is recommended for near-term planning. Such a channel should be configured to bypass the screens and grit basins entirely because unscreened influent can clog the grit pipes.

Multiple layout alternatives were evaluated and briefly discussed in the following paragraphs.

### 4.5.1 Bypass Channel in Screenings Room

Installing a bypass channel in the existing Screenings room involves relocating all of the screenings and grit handling equipment, extending the influent box into part of the blower room, and adding a channel as shown on Figure 4.1. On Figure 4.1, blinded screens are shown in red with blocked flow paths shown in orange shading. Blue shading shows the flow path during bypass. Additionally, five automatic gates (shown as green lines) would need to be installed to properly automate the process. Table 4.2 shows the summary of the advantages and disadvantages for this alternative. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $2.4 million.

Table 4.2 Summary of Bypass Channel Alternative Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Fairly simple solution. | Relocation of existing screenings and grit handling equipment. |
| | May result in grit settling in the channel. |
| | Structural modifications to Screenings and Blower Rooms. |

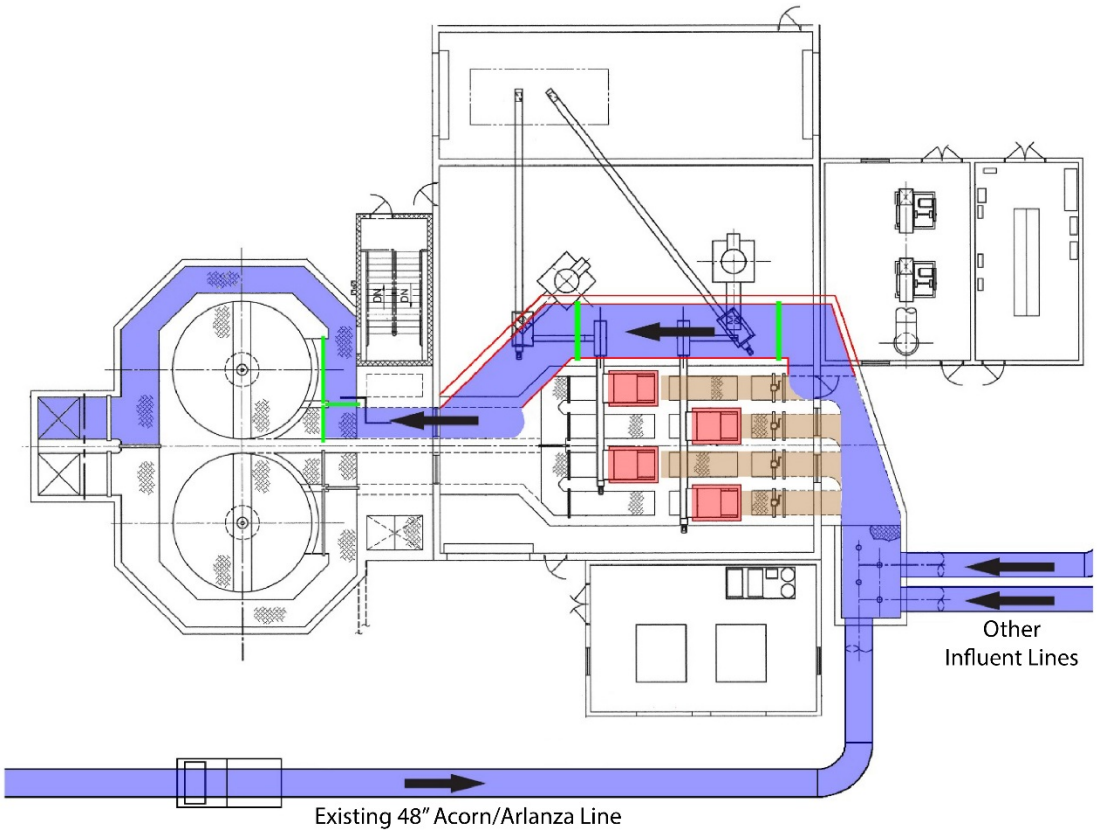


Figure 4.1 Bypass Channel in Screenings Room

### 4.5.2 Bar Screen Bypass Gates in Screenings Room

The second alternative involves installing automatic gates between bar screen channels to bypass blinded screens. Gates would be installed between staggered bar screens in the available space. However, during the June 28, 2017 workshop, this alternative was immediately dismissed as too complicated and too congested. It will not be discussed further.

### 4.5.3 Bypass Structure Adjacent to Headworks Structure

The third alternative allows for the installation of a bypass structure outside of the existing Headworks facility. As shown on Figure 4.2, a new bypass structure would be installed at the same location of the existing Acorn/Arlanza flowmeter and sampling structure intercepting the Acorn/Arlanza influent line. This bypass channel would then tie-in to the effluent discharge side of the bottom grit chamber bypassing the entire headworks. Again, automatic gates would be required to facilitate bypass operation. Table 4.3 shows the summary of the advantages and disadvantages for this alternative. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $1.1 million.

Table 4.3 Summary of Bypass Structure Alternative Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Direct bypass capabilities | Requires relocation of the Acorn/Arlanza flowmeter and sampling station. |
| Low impact to operation during construction | |
| Does not increase congestion in the Screening Room | |

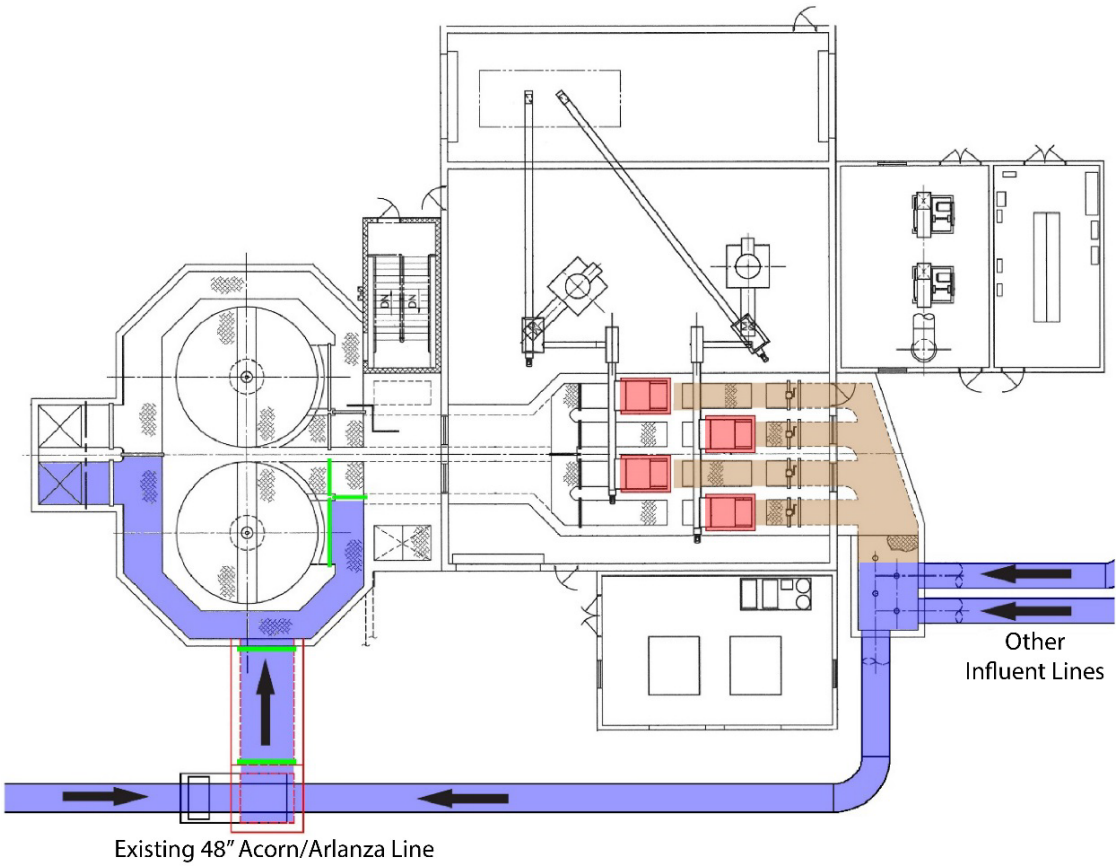


Figure 4.2 Bypass Structure

### 4.5.4 Alternatives Comparison and Recommendation

Table 4.4 shows a comparison of the evaluated alternatives. During the June 28, 2017 workshop, the third alternative, bypass structure adjacent to headworks structure (Figure 4.2), was selected as the primary recommendation moving forward. However, both alternatives can be considered during preliminary design.

Table 4.4 Comparison of Bypass Alternatives

| Parameter | Bypass in Screenings Room | Bypass Structure Adjacent to Headworks |
|---|---|---|
| Screenings room space impact | Relocate Screenings and Grit Equipment | No impact to Headworks Building |
| Impact during construction | Headworks Process Congestion During Construction | No Process Impact During Construction |
| Construction constraints | Marginal Construction Difficulty | Construction Occurs Around Online Processes Without Impact |
| Bypass capacity | Provides Full Bypass | Provides Full Bypass |
| Project Cost | $2.4 Million | $1.1 Million |

## 4.6 Grit Chamber

The two installed vortex grit chambers are 20-foot diameter units with a manufacturer rated (i.e., hydraulic) capacity of 50 mgd each. However, the City does not see sufficient grit removal at such flows. In Carollo's experience, vortex grit chambers do not adequately removal grit at the hydraulic rated capacity and as a result typically de-rate the capacity by 40 percent resulting in a combined grit removal capacity of 60 mgd for the installed units. Based on projected flows presented in Volume 2, Chapter 3 Population, Flow, and Loading Projections, the ADF of 39 mgd with a wet weather peaking factor of 2.2 and dry weather peaking factor of 1.4 are used in this update to the Master Plan. This corresponds to a predicted PWWF of 85.8 mgd and a peak DWF of 54.6 mgd. Additionally, based on the 1996 drawings for the RWQCP Headworks Project by Brown and Caldwell, the vortex grit chambers are designed to have a hydraulic capacity of 50 mgd with an associated grit removal efficiency of 95 percent for grit larger than 50 mesh and 65 percent for grit larger than 100 mesh.

### 4.6.1 Design Treatment Capacity Considerations

Some considerations need to be given when determining the necessary grit chamber capacity. In the design of a new grit chamber layout, the chamber would be sized and designed based on PWWF. This is due in part to the fact that the majority of grit that enters a treatment plant is usually carried in during wet weather events. Additionally, a new grit chamber layout would include redundancy by providing a standby grit chamber. However, in the case of the RWQCP additional factors need to be considered.

As noted above, this update to the Master Plan uses a design flow in 2037 of 39 mgd, a peak dry weather design flow is 54.6 mgd and the peak wet weather design flow is 85.8 mgd. Therefore, the City must decide if full grit removal is required during peak events and if a standby grit basin is required.

### 4.6.2 Rehabilitate Existing Grit Chambers or “Do-Nothing" Alternative

In this alternative, existing grit chambers would be rehabilitated as necessary to maintain current operation. The existing headworks facilities were built in 1999. Typically, concrete structures are expected to last 50 years and mechanical components are expected to last 20 years, although this can be extended through proper maintenance and is dependent upon operating conditions. Immediate replacement of the structure or various concrete elements may be unnecessary and excessive. Additionally, the condition assessment completed as part of this update to the Master Plan was limited to a visual inspection of processes and equipment without any confined space entry. For this reason, the grit chambers and mechanical equipment therein were not assessed. A detailed investigation should be conducted to determine condition, remaining useful life, and a cost allocation associated with rehabilitation. For planning purposes, $0.34 million will be allocated for the study and rehabilitation of the existing grit chambers.

The existing grit chambers have a combined hydraulic rated capacity of 100 mgd and a combined de-rated removal capacity of 60 mgd. This means that flows in excess of 60 mgd up to 100 mgd can pass through the existing grit chambers, but will likely not meet grit removal requirements, as stated above, meaning that more grit will escape the grit chambers and pass into downstream processes during peak wet weather events. During peak DWF, the grit chambers should be able to meet the manufacturer’s stated grit removal efficiencies. Existing grit chambers can treat the peak dry weather design flow and hydraulically pass the peak wet

weather design flow at a lower grit removal efficiency, but will require both units in service. For full grit removal and/or redundancy under all flow conditions, this alternative is insufficient.

### 4.6.3 Retrofitting Existing With New Technology

The primary advantage of vortex grit chamber technologies is that they provide adequate grit removal at low head losses. Other grit removal technologies such as HeadCell® provide better grit removal but at the cost of higher head loss. As stated in Section 4.4, the bar screen channels have less than 1 foot of head loss at peak wet weather conditions. As a result, alternative technologies are not feasible within the existing hydraulic limitations.

### 4.6.4 Add a 20-Foot Vortex Grit Chamber

In this alternative, another 20-foot vortex grit chamber would be installed in available space adjacent to the existing grit chambers. This will increase grit removal capacity to 90 mgd. However, it is critical that flow and grit be adequately split between all in-service grit chambers for efficient grit removal. Given the existing layout, there is no place a new 20-foot vortex grit chamber could be installed without significant redesign of the existing bar screen effluent channels and grit chamber influent channels to provide the desired hydraulic flow split conditions. For this reason, this alternative was immediately dismissed as infeasible.

### 4.6.5 Replace Existing Grit Chambers with Two 32-Foot Vortex Grit Chambers

This alternative involves the sequential demolition of existing grit chambers and construction of larger replacement vortex grit chambers. During construction of this alternative, existing grit pumps would likely need to be relocated to keep one grit chamber in operation while the other is demolished and replaced. The installed 32-foot vortex grit chambers each have a 100 mgd hydraulic capacity and a 60-mgd removal capacity. This solution would allow one unit to be out of service during peak DWFs, but require both units in service for PWWFs. Figure 4.3 shows a proposed layout of this alternative. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $19.4 million.

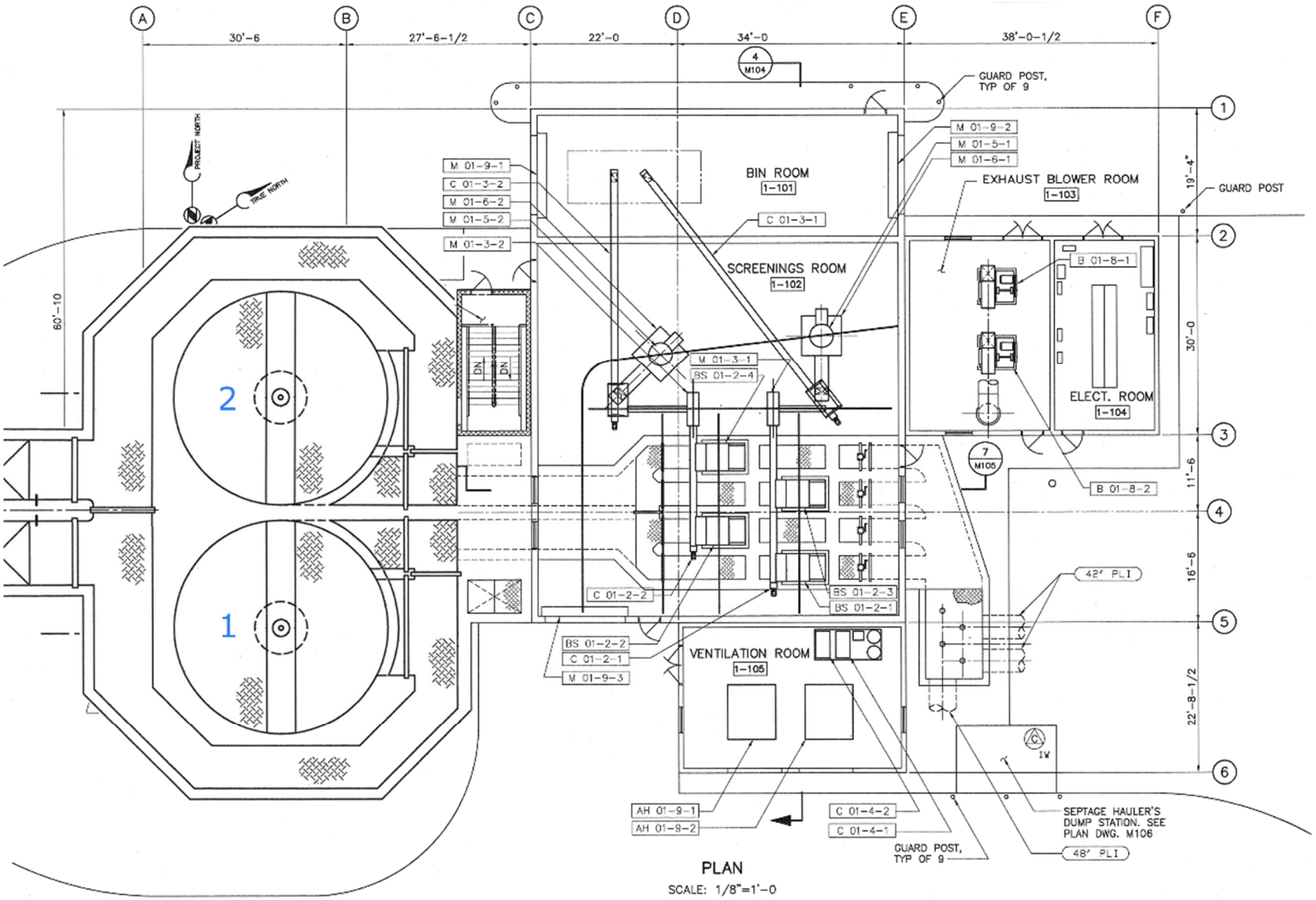


Figure 4.3 Two 32-Foot Grit Chamber Layout

### 4.6.6 Replace Existing Grit Chambers with Four 24-Foot Vortex Grit Chambers

In this alternative, the existing grit chambers would be demolished immediately and a new grit chamber layout with four 24-foot vortex grit chambers would be constructed. Each 24-foot grit chamber has a hydraulic capacity of 70 mgd and a removal capacity of 42 mgd. This solution would allow one unit to be out of service during a peak wet weather event and would provide sufficient grit removal under any flow conditions. However, significant modifications would need to be made to existing facilities including bar screen effluent channels, ACT treatment train (Plant 2)/MBR treatment train (Plant 1) flow diversion structure, and other downstream piping. Figure 4.4 shows a suggested layout of a four grit basin configuration. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $30.8 million.

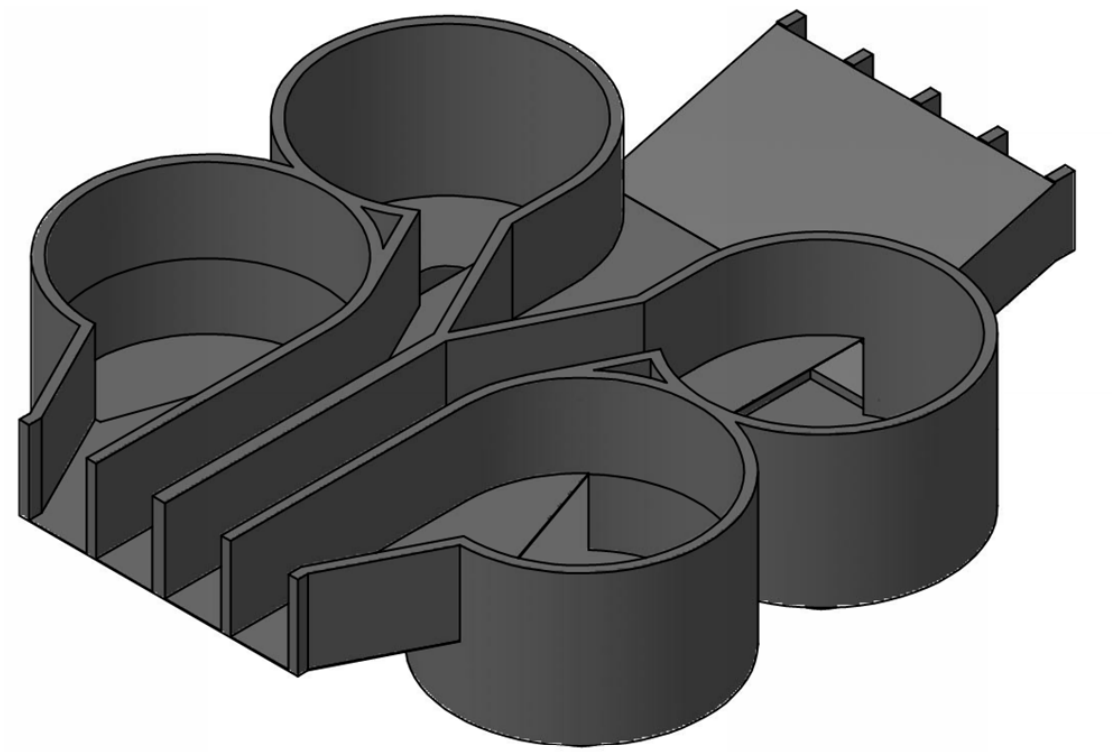

Figure 4.4 Four 24-Foot Grit Chamber Layout

### 4.6.7 Alternatives Comparison and Recommendation

Table 4.5 shows a comparison of the evaluated alternatives. In the June 28, 2017 meeting, the City decided that the existing grit chamber system, or the "Do-Nothing" alternative was sufficient. Per the RWQCP Condition Assessment presented in Volume 6, Chapter 1, Plant 1 Condition Assessment Results, the grit chambers themselves demonstrated no immediate need for structural rehabilitation. However, the condition assessment was limited to a visual assessment of readily accessible assets only. A detailed investigation of the grit chambers and

the mechanical components therein should be conducted to determine if rehabilitation is required.

Table 4.5 Comparison of Grit Chamber Alternatives

| Parameter | Rehab Existing or "Do Nothing" | Two 32-ft | Four 24-ft |
|---|---|---|---|
| Upstream Impact | No Change | Marginal Increase in Headloss | Marginal Increase in Headloss |
| Layout Impact | No Change | Cleans Up Existing ACT treatment train (Plant 2) to MBR treatment train (Plant 1) Flow Split | Relocates Several Facilities and Impacts the Road |
| Construction/Operations Impact | No Change | Some Bypass Pumping During Construction | Full Grit Chamber Bypass Required |
| Treatment Redundancy | No Redundancy | No Redundancy | 3 duty + 1 standby |
| Sufficient at Peak DWF | Meets Removal Requirements | Meets Removal Requirements | Meets Removal Requirements with Redundancy |
| Sufficient at PWWF | Does Not Meet Removal Requirements | Meets Removal Requirements | Meets Removal Requirements with Redundancy |
| Project Cost | $0.34 million | $19.4 million | $30.8 million |

## 4.7 Screenings/Grit Conveyance

As part of the condition assessment discussed in Volume 6, Chapter 1, Plant 1 Condition Assessment Results, the existing shaftless screw conveyors were identified for replacement due to excessive corrosion and general poor condition. For this analysis, two alternative technologies were considered: belt conveyors and shaftless screw conveyors.

### 4.7.1 Belt Conveyors

Belt conveyors offer some advantages but suffer significant disadvantages that should be considered. The strongest advantages of belt conveyors is the ability to convey large objects or abrasive materials without issue, but the numerous rollers and idlers require regular maintenance and also the conveyors can be messy. Figure 4.5 shows a roller and idler with screenings build-up. Table 4.6 shows the advantages and disadvantages of this type of conveyor.

Planning level cost estimates indicate this alternative would have an associated project cost of approximately $1 million.

Figure 4.5 Belt Conveyors can Accumulate Screenings

Table 4.6 Summary of Belt Conveyor Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Long lengths possible | Can be messy (significant housekeeping requirements) |
| Can accommodate multiple drop-off points with plow system (e.g., bypass compactor) | Not well suited for wet material (with free water) |
| Can handle both screenings and grit without increased wear | Spillage/carryover can generate additional odors |
| Can convey large objects | Difficult to cover for odor control |
| | Intermediate drop-off points require conveyor to be flat |
| | Large number of rollers and idlers require maintenance which is time consuming |

### 4.7.2 Shaftless Screw Conveyors

The City currently has shaftless screw conveyors (Figure 4.6) and is pleased with the technology. The conveyors can get blocked by large debris, but are easy to use and do not require excessive maintenance. Table 4.7 shows the advantages and disadvantages of this type of conveyor.

Planning level cost estimates indicate this alternative would have an associated project cost of approximately $0.9 million.

Figure 4.6 Existing Screenings and Grit Shaftless Screw Conveyor

Table 4.7 Summary of Shaftless Screw Conveyor Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Less susceptible to spills | Length is limited to approximately 100 ft without intermediate drive shafts |
| Suitable for wet material (with free water) | Conveying abrasive material reduces liner life |
| Easy to cover for odor control | Cannot convey large objects |
| Can accommodate multiple drop-off points with slide gate system (e.g., bypass compactor) | |
| Fewer components compared to belt conveyors | |

### 4.7.3 Screenings Washer/Compactor Bypass

The existing screenings screw conveyor discharges directly into the screenings washer/compactor unit. The City has indicated that the existing screenings washer/compactors encounter numerous challenges (discussed in Section 4.8). To prevent excessive build-up in the screenings screw conveyor, Carollo often recommends a method of bypassing the screenings washer/compactor by extending the conveyance equipment beyond the washer/compactor inlet and discharging directly to a bin/can. In this fashion, screenings can continue to flow along the

conveyor without causing problems upstream when the screenings washer/compactor is clogged and/or out of service.

### 4.7.4 Alternatives Comparison and Recommendation

Table 4.8 shows a comparison of the evaluated alternatives. In the June 28, 2017 meeting, the City decided to continue using shaftless screw conveyors in the future with preferred manufacturers including Keystone, Huber, and Spirac. It is recommended to include a screenings washer/compactor bypass option in the design of the next shaftless screw conveyor replacement. It may also be worthwhile to connect odor control ducting to the shaftless screw conveyor covers for improved odor control in the Screenings room.

Table 4.8 Comparison of Conveyance Alternatives

| Parameter | Belt Conveyor | Shaftless Screw Conveyor |
|---|---|---|
| Current Installation | Not Used | Currently Installed |
| Cleanliness | Screenings Carryover can be Messy | Fully Enclosed |
| Multiple drop-off points | Designs for Multiple Discharge Locations | Designs for Multiple Discharge Locations |
| Odor Control | Uncovered; No Direct Control | Fully Enclosed Allows for Direct Pull Off of Air |
| Suitable for abrasive materials | Marginal Impact | More Susceptible |
| Maintenance requirements | More Stators and Rollers to Maintain | Standard Maintenance |
| Project Costs | $1 million | $0.9 million |

## 4.8 Bar Screens

As part of the condition assessment discussed in Volume 6, Chapter 1, Plant 1 Condition Assessment Results, the existing bar screens were identified as a regular maintenance item. The City would also like to consider a smaller screen opening size to increase capture and protect downstream processes and equipment. For this analysis two alternative technologies were considered: climber screens and multi-rake screens.

### 4.8.1 Climber Screens

The existing bar screens are climber screens shown on Figure 4.7. In the City's experience, large objects tend to jam the rake arm causing it to break and fall into the channel. There have been significant issues with blinded screens in the past. Newer models of climber screens have a rake arm capable of pivoting around large objects preventing the arm from breaking and falling into the channel. The advantages and disadvantages of this type of screen are presented in Table 4.9 below. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $2.8 million.

Figure 4.7 Existing Climber Bar Screens

Table 4.9 Summary of Climber Screen Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| RWQCP operating experience | Taller equipment above operating floor limiting access to limit switches |
| No moving parts in the water | Screenings loading to conveyor/compactor is in larger loads |
| All routine maintenance can be done above the channel - no channel entry required | Single rake limits screenings removal rate especially for deep channels (longer rake cycle time) |
| In newer models, the rake arm is able to pivot around large objects | |

### 4.8.2 Multi-Rake Bar Screens

Multi-rake bar screens are rapidly gaining popularity in the wastewater industry for a number of reasons. One of the biggest advantages is that multi-rake screens remove screenings on a more frequent basis, thereby minimizing chances of blinding the screens. However, there are some significant disadvantages that will need to be considered during preliminary design. Multi-rake bar screens have a frame which takes up 6 in. on each side of the channel. The existing channels are 3 ft, 6 in. wide meaning the effective screening width would be reduced to 2 ft, 6 in. This may adversely affect upstream head loss. It has been suggested that a greater angle of inclination could compensate for this increase in head loss, but this would need to be investigated during preliminary design. Figure 4.8 shows a diagram of a multi-rake screen in use. Table 4.10 shows the advantages and disadvantages of this type of bar screen. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $2.5 million.

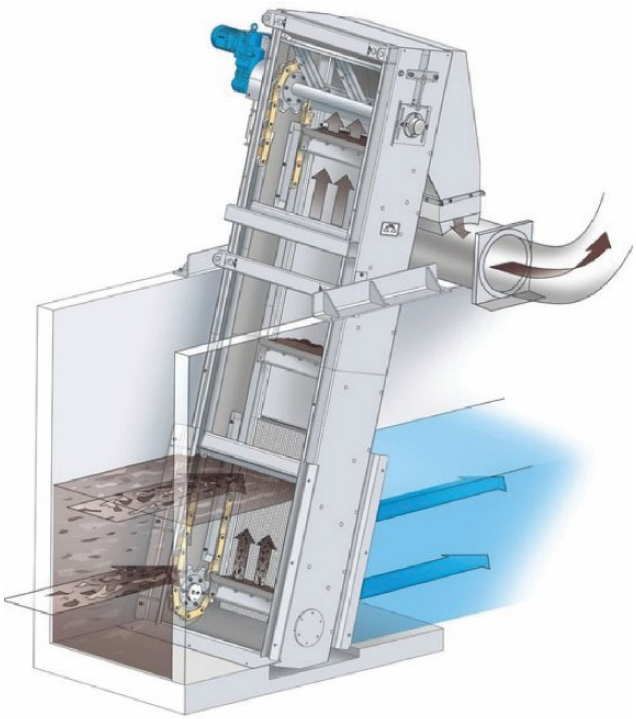

Figure 4.8 Multi-rake Screen Operational Diagram

Table 4.10 Summary of Multi-Rake Screen Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Shorter equipment above operating floor - easier maintenance access | Moving parts in the water |
| Most maintenance can be done above the channel | Bottom sprockets require annual inspection and maintenance (channel entry) |
| Multiple rakes and two speeds increase screenings removal rate | Lose 1 foot of screen field width for side frame |
| Screenings loading to conveyor/compactor is more uniform - small loads | New technology for RWQCP |
| Easy to cover for odor control; flat plates can be mounted to the side frame and do not require entry or removal for most screen maintenance | |
| Rake can be reversed to try to dislodge jams | |

### 4.8.3 Minimum Bar Screen Spacing

Bar screen spacing impacts both the screenings capture efficiency as well as head loss through the screens. While a smaller screen spacing increases screenings capture, head loss increases due to a decrease in effective flow path area. Table 4.11 discusses the advantages and disadvantages to consider when comparing two screens with different spacing. At the June 28, 2017 workshop,

the City determined that the minimum acceptable bar screen spacing that should be considered for future installation should not be less than 3/8 in.

Table 4.11 Summary of Smaller Bar Screen Spacing Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Higher removal | Removal of more organic material |
| Reduce downstream maintenance of the digesters | Increased disposal cost due to increased screenings capture |
| | Faster screen blinding |
| | More frequent operation/maintenance |
| | Reduce screen capacity |
| | Higher head loss and negative impact on collection system hydraulics |

### 4.8.4 Alternatives Comparison and Recommendation

Climber screens are the currently installed technology and have caused problems for maintenance staff in the past. Newer technology may be able to compensate for these problems. However, the City is interested in exploring new technology such as the multi-rake screen. It would be valuable to complete a detailed evaluation during preliminary design to determine if an atypical angle of inclination can compensate for limited effective screen area caused by the multi-rake frame. Regardless of the technology selected, the minimum recommended bar screen spacing is 3/8 in. Overall, available technologies and bar screen spacing should undergo a detailed preliminary design analysis before a decision is made. Table 4.12 shows a comparison of the two alternate technologies.

Table 4.12 Comparison of Bar Screen Alternatives

| Parameter | Climber | Multi-Rake |
|---|---|---|
| Effective Channel Width | Full Channel Width Flow | Frame Takes 6 in. on each side |
| Reliability | New Models are Typically Very Reliable | New Models are Typically Very Reliable |
| Screen/Room Height | Taller than Multi-Rake Units | More Compact |
| Moving Parts in Water | No Submerged Bearings | Submerged Bearings Require Confined Space Entry for Maintenance |
| Hydraulic Impact | No Change | May Increase Headloss Depending on Design |
| Ease of Covering and Odor Control | No Change | Can Pull Foul Air Directly From Enclosure |
| Screenings Removal Rate | No Change | Increased Removal Rate |
| Screenings Loading Rate to Compactors | No Change | More Frequent Removal |
| RWQCP Experience | Currently Installed | New Technology |
| Project Cost | $2.8 million | $2.5 million |

## 4.9 Screenings Grinding/Washing/Compacting

As part of the condition assessment discussed in Volume 6, Chapter 1, Plant 1 Condition Assessment Results, the existing screenings grinding/washing/compacting units were identified as a regular maintenance item with expensive repair costs. The original equipment was gutted

and replaced in 2004 with a Muffin Monster proprietary grinder/washer/compactor system. The City would consider alternative technologies with better performance and less maintenance requirements. For this analysis two alternative technologies were considered: shafted auger with some washing capability and shafted auger with increased washing capability.

### 4.9.1 Shafted Auger with Some Washing Capability

Figure 4.9 Typical Shafted Auger with Some Washing Capability

This technology injects some water with mild agitation to moderately clean screenings and compress them into a slug of relatively dry, screened mass for discharge. This is a common technology with multiple manufacturers. The washer can handle larger slugs than the alternative that provides a higher level of washing, discussed below. The advantages and disadvantages of this type of screenings washer/compacter are presented in Table 4.13, while Figure 4.9 shows a typical shafted auger that has some washing capability. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $0.8 million.

Table 4.13 Summary of Shafted Auger with Some Washing Capability Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| There are fewer parts to maintain | Moderate washing effectiveness |
| Washer can handle various slug sizes | Moderate dewatering effectiveness |
| Multiple manufacturers | Odor control is required |

### 4.9.2 Shafted Auger with Increased Washing Capability

Different to the shafted auger discussed above, this technology includes a kind of clothes washer agitation cycle and drum that provides a higher degree of washing for the screenings. The top portion of the unit has a wash basin in which the screenings are loaded. When the wash cycle initiates, the basin is filled with water and agitated to separate organics from the screenings. Organics are carried off in the washwater and the screenings are compressed by the shafted auger for discharge into the conveyor. While the unit is washing, it cannot accept any additional screenings loads.

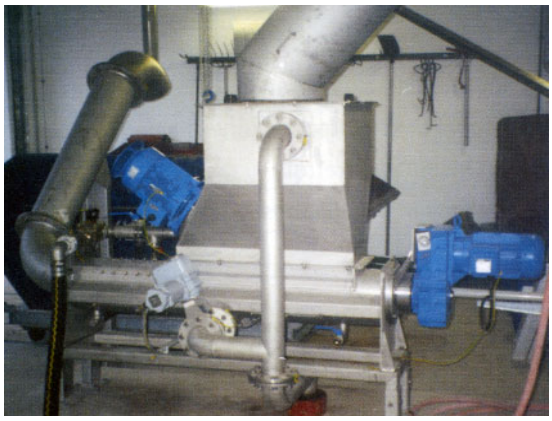

Figure 4.10 Typical Shafted Auger with Increased Washing Capability

Methodologies need to be considered during preliminary design to store screenings while the unit is in backwash to prevent the screenings screw conveyors from becoming clogged. Table 4.14 shows the advantages and disadvantages of this type of screenings washer/compacter while Figure 4.10 shows a typical equipment arrangement. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $0.7 million.

Table 4.14 Summary of Shafted Auger with Increased Washing Capability Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Good organics removal, dewatering, and washing efficiency | Washer cannot accept screenings during washing cycle |
| Odor control is not required | Sole-source may be required |

### 4.9.3 Alternatives Comparison and Recommendation

As regulatory changes impact organics disposal limitations, increased removal of organic material from screenings offers valuable benefits. Additionally, decreasing organics concentration in dewatered screenings reduces odor in the bin room and the screenings room. For these reasons, serious consideration should be given to the shafted auger that included increased washing capabilities in the future. Table 4.15 shows a comparison of the two alternate technologies. In the project meeting on June 28, 2017, City staff selected the shafted auger that included increased washing capabilities. The City plans to replace these units as a maintenance project sometime in the next 5 years.

Table 4.15 Comparison of Screenings Washing Alternatives

| Parameter | Shafted Auger With Some Washing Capability | Shafted Auger With Increased Washing Capability |
|---|---|---|
| O&M Requirements | Less Moving Parts | More Moving Parts |
| Washing Effectiveness | Some Washing | Better Washing |
| Dewatering | Squeezes Water On Discharge | Has a Dewatering Step |
| Ability to Handle Large Slugs of Solids | Continuous Input | Batched Process |
| Number of Manufacturers | Common Technology | Currently Sole-Sourced |
| Project Cost | $0.8 million | $0.7 million |

## 4.10 Grit Washing/Compacting

During the condition assessment discussed in Volume 6, Chapter 1, Plant 1 Condition Assessment Results, no issues or complaints were reported for the grit washing/dewatering equipment. For this reason, this update to the Master Plan presents technology alternatives for end-of-life planning purposes. For this analysis four alternative technologies were considered: traditional cyclone classifier, slurry cup and snail, turbo-washer, and COANDA.

### 4.10.1 Traditional Cyclone Classifier

Figure 4.11 Typical Cyclone Grit Classifier

Traditional cyclone classifiers were the industry standard grit washing technology for years. They provide adequate product at a wide range of hydraulic capacity for a moderate capital cost. However, cyclone classifiers have long suffered limited washing efficiency necessitating the use of odor control. Table 4.16 shows the advantages and disadvantages of this type of grit washing/dewatering technology. Figure 4.11 shows a typical cyclone classifier installation. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $1 million.

Table 4.16 Summary of Traditional Cyclone Classifiers Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Highly successful in similar applications | Provides limited washing of grit |
| Wide range of hydraulic capacity | Requires odor control |
| Produces adequate product | |
| Moderate capital cost | |

### 4.10.2 Slurry Cup and Snail

The City currently has slurry cup and snails installed and are happy with them. They work well and are not a source of concern for operations or maintenance staff. The advantages and disadvantages of this type of grit washing are presented in Table 4.17 below. Figure 4.12 shows the existing slurry cup and snail installation. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $1.6 million.

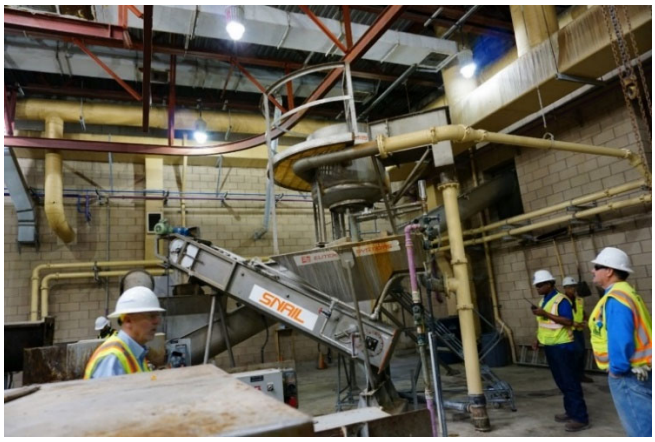


Figure 4.12 Existing Slurry Cup and Snail Installation

Table 4.17 Summary of Slurry Cup and Snail Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| RWQCP operating experience | Frequent/more intense maintenance required |
| Fair removal of organics and water | Poor load variation adaptability |
| No complaints from City staff | Requires odor control of grit |

### 4.10.3 Turbo-Washer by Smith and Loveless

Smith and Loveless began production of this technology in the US in 2011 and has more than 20 installations to date. The technology reportedly provides intense water flushing and high air infusion for organics removal. Organics are lifted from the grit as it settles and are flushed with the wash water while the cleaned grit is slowly transported up a patented screw providing increased washing and dewatering. The advantages and disadvantages of this type of grit washer are presented in Table 4.18. Figure 4.13 shows a typical Turbo-Washer installation. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $0.7 million.

Figure 4.13 Typical Turbo-Washer Installation

Table 4.18 Summary of Turbo-Washer Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Cyclones can handle a wide range of flow (250 and 1,000 gpm) | Manual drain valve to flush organics |
| Fairly simplistic operation and maintenance | Limited installations in the U.S. |
| Good removal of organics, water, and grit | Requires odor control of bin room |

### 4.10.4 COANDA by Huber

The COANDA grit washing/dewatering system was developed in and for municipalities in the U.K. due to increased organics disposal limitations in the region. Since its development, there have been several installations throughout the U.S. The COANDA unit has three rotating horizontal discs which create layers of settleable materials. Grit settles first and forms the bottom layer. Lighter organics remain in a suspended second layer with the third and highest layer being cleaner water. Jets of fluidizing water are used to create and maintain these separate layers.

Figure 4.14 Typical COANDA Installation

Similar to the Turbo-washer, cleaned grit is transported out of the washer by a screw conveyor. Table 4.19 shows the advantages and disadvantages of this type of grit washer. Figure 4.14 shows a typical COANDA installation. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $2 million.

Table 4.19 Summary of COANDA Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Produces cleaner, drier, less odorous product (no odor control) | Sole-source may be required |
| Reduces hauling cost | |

### 4.10.5 Alternatives Comparison and Recommendation

Compared to traditional cyclone classifiers, newer grit technologies remove more organics and water resulting in less odor and lower disposal costs. Table 4.20 presents manufacturer stated performance characteristics for each of the four technologies presented. Figure 4.15 shows a comparison between a typical cyclone classifier product and product from a COANDA unit.

Table 4.20 Manufacturer Stated Performance Characteristics of Grit Washing/Dewatering Alternatives

| Parameter | Cyclone Classifier | Slurry and Snail | Turbo Washer | COANDA |
|---|---|---|---|---|
| Volatile Solids | 15-20% | 15% | 5% | 3-5% |
| Water Content | 35-45% | 40% | 10% | 10% |
| Grit Capture (≥ 150 µ) | 90% | 95% | 95% | 95% |

Figure 4.15 Cyclone Classifier Product Grit (Left) vs. COANDA Product Grit (Right)

Table 4.21 shows a comparison of the four alternate technologies discussed above. The existing grit washing/dewatering equipment currently is in good condition and is expected for replacement at the end of its useful life, which is anticipated to be beyond 2037. Therefore, by the time the equipment needs to be replaced, different technologies or new information may be available about each of the presented technologies above. A detailed preliminary analysis should be done at the time replacement.

Table 4.21 Comparison of Grit Washing/Dewatering Alternatives

| Parameter | Cyclone Classifier | Slurry and Snail | Turbo Washer | COANDA |
|---|---|---|---|---|
| Site Space Impact | Similar In Size and Layout | Current Installation | Similar in Size; Different Layout | Similar in Size; Different Layout |
| Operational Complexity | Very Simple | Very Simple | More Complex | More Complex |
| Level of Maintenance Required | Simple and Infrequent Maintenance | Simple but More Frequent Maintenance | Simple and Infrequent Maintenance | Simple and Infrequent Maintenance |

Table 4.21 Comparison of Grit Washing/Dewatering Alternatives (continued)

| Parameter | Cyclone Classifier | Slurry and Snail | Turbo Washer | COANDA |
|---|---|---|---|---|
| Adaptability to Future Regulatory Changes | Not a Substantial Source of Organics Removal | Not a Substantial Source of Organics Removal | Good Organics Removal from Screenings | Good Organics Removal from Screenings |
| US Experience | Common In the USA | Common In the USA | Fairly New in the USA | Common In the USA |
| RWQCP Experience | Not Installed | Currently Installed | Not Installed | Not Installed |
| Project Cost | $1 million | $1.6 million | $0.7 million | $2 million |

Appendix 4A

# RWQCP INFLUENT FLOWMETERS

City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

## CHAPTER 4: PRELIMINARY TREATMENT

# APPENDIX 4A: RWQCP INFLUENT FLOWMETERS

FINAL | June 2019

City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

VOLUME 4: WASTEWATER TREATMENT SYSTEMS

CHAPTER 4: PRELIMINARY TREATMENT

# APPENDIX 4A: RWQCP INFLUENT FLOWMETERS

Carollo Project No. 10495A00

Digitally signed by Timothy J. Loper
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 06:27:24-07'00'

REGISTERED PROFESSIONAL ENGINEER
TIMOTHY J. LOPER
No. 70847
CIVIL
STATE OF CALIFORNIA

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Section 1

# PURPOSE

The purpose of this report is to summarize the evaluation of options for installing centralized magnetic flowmeters for the Acorn, Arlanza, and Santa Ana Trunk Sewers, as well as considering a single combined master meter to measure all flow entering the plant. The following was included in the analysis:

- An option for installing a centralized flow metering structure for all the influent flow to the RWQCP, a master meter.
- Options for installing a common magnetic flowmeter for the Acorn and Arlanza Trunk Sewers.
- Confirmation of the proposed location for the Santa Trunk Sewer magnetic flowmeter.
- Developing a preliminary layout for the two metering structures, including the location of automatic water quality sampling equipment.
- Evaluation of hydraulics to identify impacts to upstream trunk sewers as well as existing downstream facilities such as the headworks, primary clarifiers, and PE EQ basins.
- Developing a preliminary construction cost estimate for the most viable metering project to be included as a project in the Headworks area in the CIP.

Section 2

# BACKGROUND

The City's Public Works Department operates a comprehensive wastewater treatment and disposal system that serves most of the City, as well as the CSDs of Jurupa, Rubidoux, and Edgemont, and the community of Highgrove. The City added an evaluation of options to improve the influent flow metering to RWQCP to the scope of this update to the Master Plan.

The influent flow to the RWQCP is routed through five trunk sewers and is metered as follows:

- Rubidoux trunk sewer: 14-inch ABB MagMaster electromagnetic flowmeter.
- Jurupa trunk sewer: Flo-Dar radar velocity and ultrasonic depth flowmeter.
- Acorn and Arlanza trunk sewers: Combined at a junction structure within the Plant and their combined flow is metered with a Flo-Dar unit.
- Santa Ana River (Riverside/Hillside) trunk sewers: Combined at a junction structure upstream of the Plant and their combined flow is also metered with a Flo-Dar unit.

These four meters are at different locations and have varying accuracy. A study commissioned by the City and completed by URS proposed adding three magnetic flowmeters on the influent lines along Acorn, Arlanza, and Santa Ana trunk sewers. A copy of URS's 2014 report is included in Attachment A.

Section 3

# DESIGN CONSIDERATIONS AND EVALUATION CRITERIA

The following sections summarize the design considerations for the magnetic flowmeter facilities as well as the criteria used to evaluate the magnetic flowmeter structures. These criteria include those discussed in Volume 3, Chapter 5, Planning Criteria and Design Flows, in this update to the Master Plan as well as design velocities and slopes specific to the flowmeter implementation requirements. Specifically, major facility design requirements and considerations that will ultimately impact the magnetic flowmeter site layout and hydraulics are included in the following sections.

## 3.1 Magnetic Flowmeters Design

Typical magnetic flowmeters have straight pipe requirements specified by the manufacturer. Straight pipe requirements are the minimum length of pipe required upstream and downstream of the flowmeter to ensure accuracy. Typically, straight pipe requirements are expressed as a multiple of the flowmeter's diameter (e.g., 5D meaning five times the pipe's diameter). URS's 2014 report reviewed four commercial vendors' straight pipe requirements. For conservative planning purposes preliminary layouts will use the longest straight pipe requirements of the four venders and is summarized below:

- Upstream straight pipe requirements is five times the pipe diameter.
- Downstream straight pipe requirements is three times the pipe diameter.

Typical magnetic flowmeters operate under full pipe flow conditions only. There are two methods to achieve such conditions in a gravity sewer system. First, you can reduce a section of pipe to a smaller diameter thus ensuring the smaller section of pipe is always full. The magnetic flowmeter would then be installed in this smaller section of pipe. However, due to diurnal low flow conditions, reducing the pipe to a diameter small enough to be full under low flow conditions would result in velocities that far exceed the acceptable maximum of 5 fps during average and peak conditions. Therefore, the second alternative is the inverted siphon. In this configuration, flows are diverted from the gravity sewer into a pipe segment that drops in elevation, hence the name "inverted siphon." The magnetic flowmeter is installed in the lowest portion of the pipe and is full at all times under all flow conditions. Recent developments in magnetic flow meter technology allow for partially full pipe operation, even for sewer lines. Because this approach is relatively new, there are not many examples of where it has been used. Therefore, for the purposes of this Master Plan Update, the inverted siphon approach has been assumed. The option to use magnetic flow meters that can operate with partially full pipes can be considered during preliminary design. Design considerations for inverted siphons are discussed in more detail in the following sections.

### 3.2 Inverted Siphon Design

An inverted siphon is a pipe that dips below the HGL meaning that it is always under pressurized conditions even at low flow. The following are things to consider when designing an inverted siphon:

- Solids depositions under low flow conditions (Metcalf & Eddy, 1981).
- Clogging from large objects caught on bends (Metcalf & Eddy, 1981).
- Frequent and challenging cleaning requirements (Metcalf & Eddy, 1981).
- Foul odors caused by positive pressures developed in the sewer atmosphere upstream from an inverted siphon because of the downstream movement of air induced by the sewer flow (ASCE No. 60).

The objective when sizing inverted siphons is to provide adequate self-cleaning velocities under a wide range of conditions. ASCE Manuals and Reports on Engineering Practice No. 60 outlines the following design considerations when designing inverted siphons.

- A velocity of 2 to 3 fps must be reached at least once each day.
- Some agencies limit the rising slope to 15 percent, but slopes as great as 50 percent are used in some places.
- There should be no change of pipe diameter within the length of a siphon to reduce issues with cleaning operations.

### 3.3 Evaluation Criteria

Inverted siphon velocity criteria was developed for minimum DWF, ADWF, and PWWF conditions, and is summarized in Table 1.

Table 1 Design Velocities for Given Flow Conditions

| Flow Condition | Minimum Velocity (fps) | Maximum Velocity (fps) |
|---|---|---|
| Minimum DWF | >2 fps at least once every 24-hours | <10 fps |
| ADWF | >3 fps once every 24-hours and<br>Average velocity >2 fps | <10 fps |
| PWWF | Average velocity >3 fps | <10 fps |

Notes:
(1) ASCE Manual and Reports on Engineering Practice No. 60, Second Edition.

Section 4

# CENTRALIZED MASTER FLOW METERING STRUCTURE

In order to address the issues related with existing flowmeters listed above, the first alternative considered was a centralized flow metering structure (master meter) to measure combined flow to the RWQCP. Figure 1 shows a preliminary layout and potential location of the centralized flow metering structure over the existing odor control biofilter. Influent flow from the Rubidoux,

Jurupa, and Santa Ana trunk sewers junction box north of the headworks, would be re-routed to the upstream side of the new metering structure. The pipe carrying flow from the A/A trunk sewer line would be extended so that it can also flow into the upstream side of the centralized metering structure.

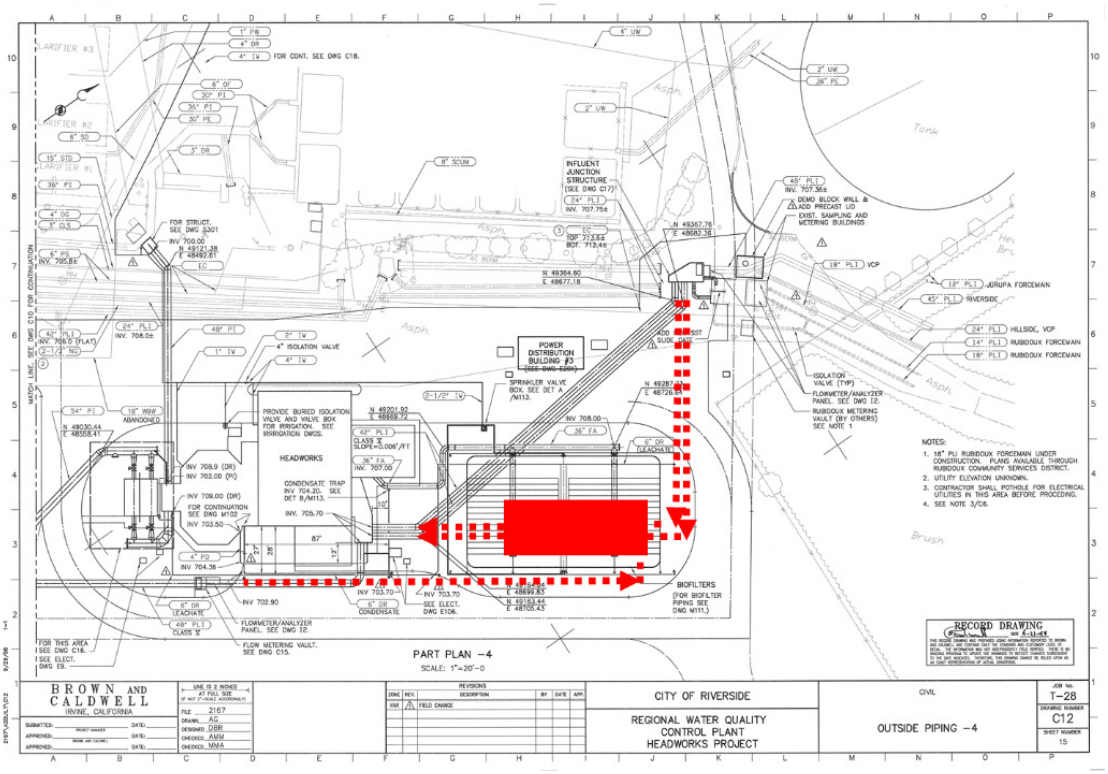


Figure 1 Centralized Flow Metering Structure Conceptual Layout

A preliminary hydraulics evaluation for the schematic layout in Figure 1 showed that about 4 feet of head is required for the flow metering structure to be operational. An earlier analysis of the capacity of the A/A Trunk Sewer, detailed in Volume 3, Chapter 7, Appendix 7A, Acorn/Arlanza Trunk Sewer Analysis, showed that this sewer is currently experiencing backwater conditions due to flow restrictions at the headworks. Modifications to the headworks would be needed to reduce the HGL by approximately 1 foot to prevent the backwater conditions.

Accordingly, to accommodate a reduction of 4-feet would require major modifications to the existing screenings channels and grit basins at the RWQCP, in order to gain the additional head for the flow metering structure. Some of the major modifications include:

1. Installing one new screenings channel.
2. Installing two new grit basins.
3. Relocating the odor control facility.

The approximate rehabilitation cost for such a headworks hydraulics project will be in the range of $50 to $70 million, including the cost for the new centralized metering structure and relocation of the existing odor control facility.

Because of the high cost of this project and the complexity of construction, this alternative has not been evaluated any further.

Section 5

# HYDRAULIC ANALYSIS FOR SANTA ANA AND A/A TRUNK SEWERS

In this section, consideration is given to combining the Acorn and Arlanza meters into a single magnetic flowmeter to reduce the total number of influent meters, and the location of the proposed Santa Ana trunk sewer flowmeter is evaluated.

The hydraulic model developed as part of Volume 3, Chapter 7, Capacity Evaluation and Proposed Improvements, was updated to include details of the proposed flowmeter locations. One of the challenges is properly sizing the inverted siphon to meet evaluation criteria under varying conditions. The following sections summarize the hydraulic model results for the Santa Ana and A/A Trunk Sewer flowmeter structures.

## 5.1 Santa Ana Flowmeter

Figure 2 shows the modeled layout of the Santa Ana Flowmeter Facility. The facility site was chosen because it is downstream of the Santa Ana Trunk parallel sewer lines and avoids interfering with existing facilities. The facility consists of a diversion structure that will divert flow to the inverted siphon that is parallel to the existing trunk sewer. The existing trunk sewer will be converted to a bypass line when the inverted siphon requires maintenance. Table 2, Table 3, and Table 4 summarize the modeled velocities under various flow conditions, for 24-inch, 30-inch, and 36-inch diameter siphons, respectively.

As the data in the tables indicates, no single inverted siphon diameter is able to meet velocity evaluation criteria for both existing and build-out conditions. A 24-inch diameter meter would work well for existing conditions, but would be too small once build-out is reached. A 30-inch diameter meter would also work for existing conditions, but would be too small for the build-out peak flow conditions. A 36-inch diameter meter would be too big for existing conditions, but suitable for build-out.

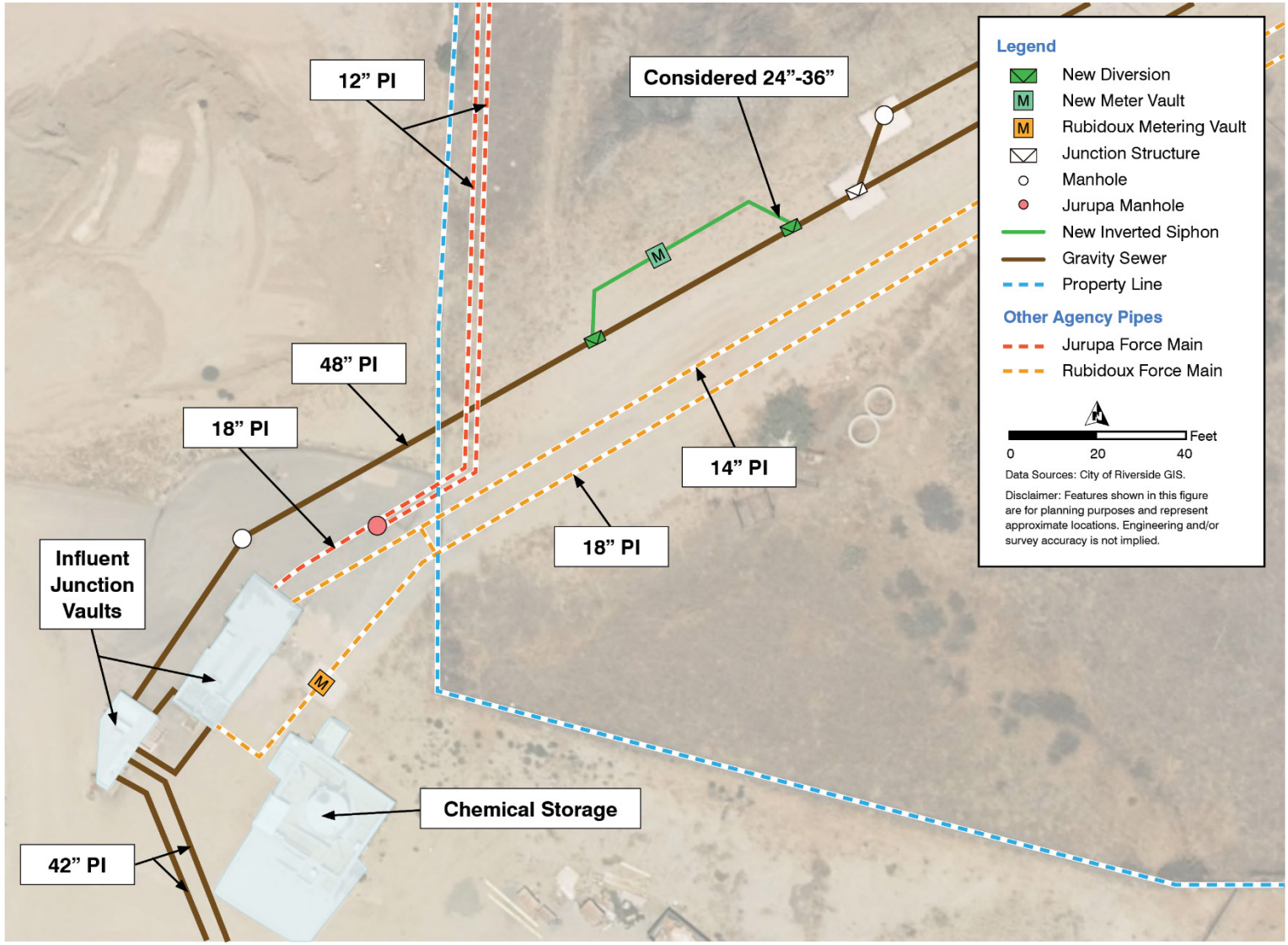


Figure 2 Modeled Santa Ana Trunk Flowmeter Facility

Table 2 Santa Ana 24-Inch Inverted Siphon - Velocity Summary

| Velocity (fps) | Existing Minimum DWF(1) | Existing ADWF | Existing PWWF(2) | Build-Out ADWF | Build-Out PWWF(2) |
|---|---|---|---|---|---|
| Average | 4.61 | 7.59 | 7.64 | 9.20 | 12.30 |
| Maximum | 5.66 | 9.36 | 9.34 | 11.53 | 15.01 |
| Minimum | 2.62 | 4.40 | 4.32 | 5.12 | 7.24 |

Notes:
(1) Minimum DWF = 34% ADWF.
(2) Assumes 10-year 24-hour storm event.

Table 3 Santa Ana 30-Inch Inverted Siphon - Velocity Summary

| Velocity (fps) | Existing Minimum DWF(1) | Existing ADWF | Existing PWWF(2) | Build-Out ADWF | Build-Out PWWF(2) |
|---|---|---|---|---|---|
| Average | 2.66 | 4.77 | 4.79 | 6.07 | 8.51 |
| Maximum | 3.37 | 5.99 | 5.98 | 7.53 | 12.97 |
| Minimum | 1.34 | 2.48 | 2.43 | 3.54 | 4.95 |

Notes:
(1) Minimum DWF = 34% ADWF.
(2) Assumes 10-year 24-hour storm event.

Table 4 Santa Ana 36-Inch Inverted Siphon - Velocity Summary

| Velocity (fps) | Existing Minimum DWF(1) | Existing ADWF | Existing PWWF(2) | Build-Out ADWF | Build-Out PWWF(2) |
|---|---|---|---|---|---|
| Average | 1.85 | 3.31 | 3.33 | 4.46 | 6.03 |
| Maximum | 2.33 | 4.16 | 4.15 | 5.51 | 9.07 |
| Minimum | 0.94 | 1.72 | 1.69 | 2.62 | 3.65 |

Notes:
(1) Minimum DWF = 34% ADWF.
(2) Assumes 10-year 24-hour storm event.

Accordingly, if the City were to construct this flowmeter facility, it is recommended that it is constructed so that the inverted siphons can be swapped with a larger-diameter pipeline in the future. It is recommended that the City initially install a 30-inch diameter inverted siphon to handle existing conditions. Eventually the City will replace this with a 36-inch diameter inverted siphon as flow rates increase.

It is hard to predict when the inverted siphon would need to be upsized because future RDII is unknown. For the purpose of this analysis, it is assumed that any upsize to the inverted siphon will occur when the diversion structure upstream of the siphon surcharges within 3 feet of the rim. The

hydraulic analysis shows that this condition will occur with the 30-inch diameter inverted siphon when flow reaches approximately 32 mgd in the Santa Ana trunk sewer, as shown in Figure 3.

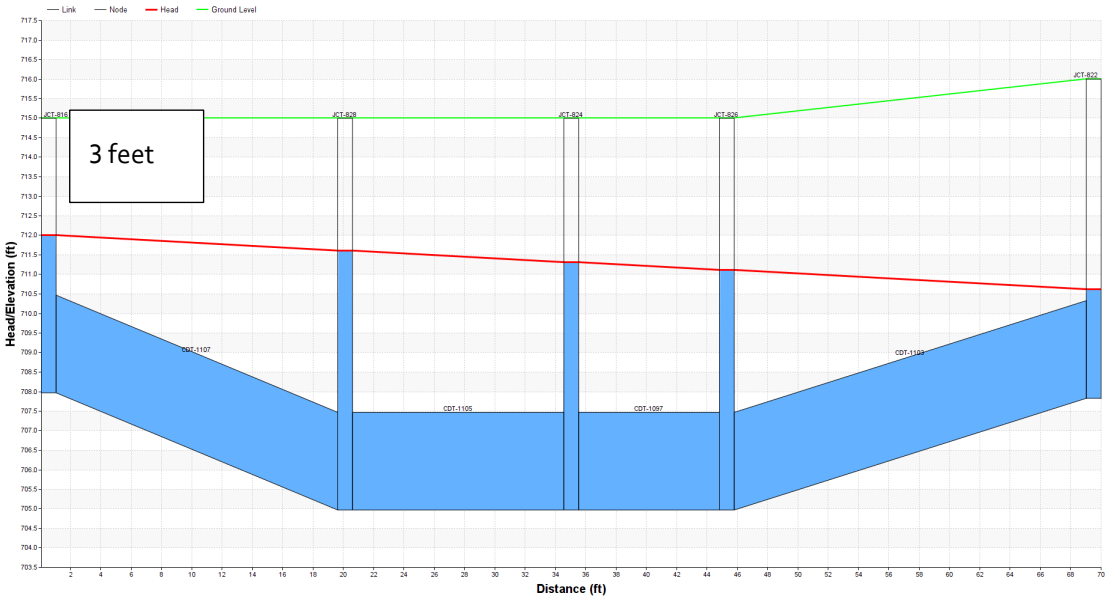


Figure 3 Santa Ana - 30-Inch Diameter Inverted Siphon HGL at 32 mgd

Figure 4 and Figure 5 show the HGL for the recommended 30-inch diameter and 36-inch diameter inverted siphons under existing and build-out PWWF conditions, respectively. The figures show that the recommended inverted siphon diameters will not exceed the surcharge criteria.

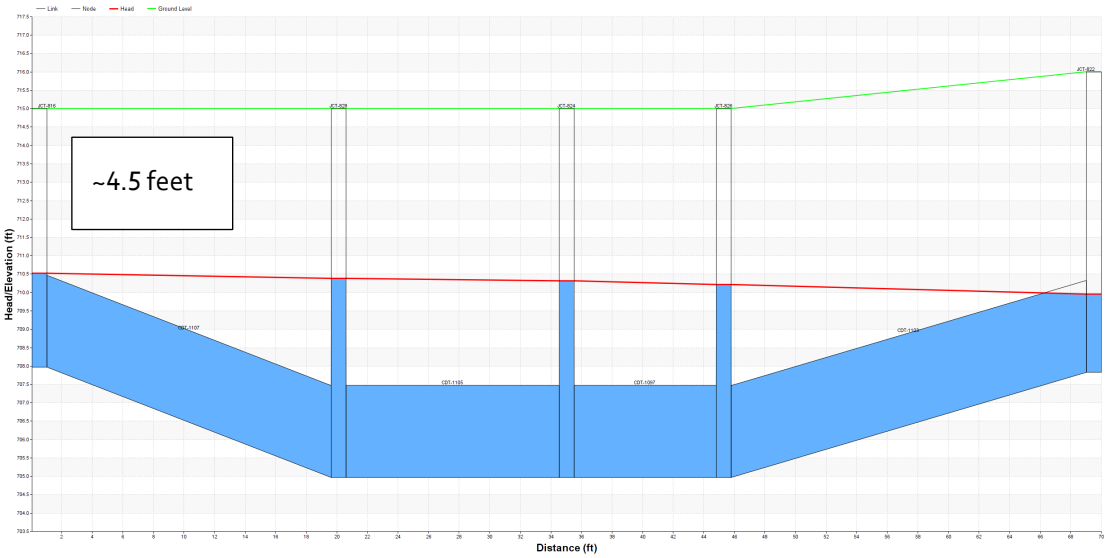


Figure 4 Santa Ana - 30-Inch Diameter Inverted Siphon HGL Under Existing PWWF Conditions

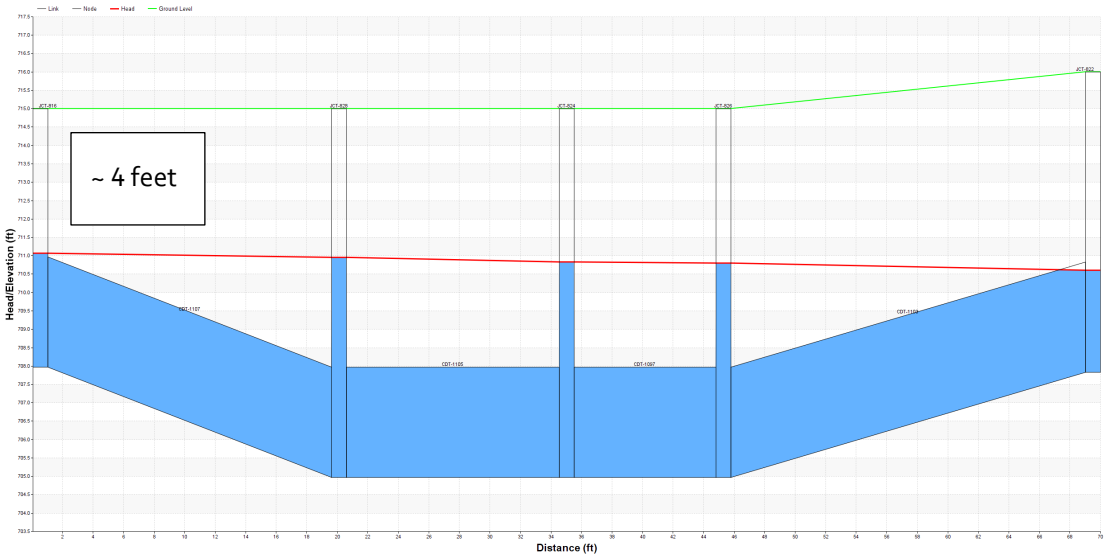


Figure 5 Santa Ana - 36-Inch Diameter Inverted Siphon HGL Under Build-Out PWWF Conditions

## 5.2 Acorn/Arlanza Combined Flowmeter

Previous studies recommend separate magnetic metering structures along the A/A Trunk Sewer. The City is interested in combining the flowmeter structures along the A/A Trunk Sewer to reduce costs. An earlier hydraulic analysis, as part of the Master Plan process, was completed to further understand the hydraulics of the A/A Trunk Sewer. A copy of this analysis can be found in Volume 3, Chapter 7, Appendix 7A, Acorn/Arlanza Trunk Sewer Analysis, of this update to the Master Plan. The hydraulic analysis showed that the A/A Trunk Sewer has sufficient capacity to convey existing PWWF conditions. Additionally, the analysis indicates that any surcharge conditions are caused by backwater effects created by hydraulic boundary conditions at the RWQCP's headworks.

For the purpose of this update to the Master plan, it is assumed that the combined flowmeter structure along the A/A Trunk Sewer would consist of a diversion structure to route flow to the inverted siphon parallel to the existing trunk sewer. The existing trunk sewer would be converted to a bypass line to be used when the inverted siphon requires maintenance.

Figure 6 shows the layout of the modeled A/A Trunk Sewer Flowmeter Facility. The hydraulic model showed that, when the metering site was moved further upstream, it caused backwater effects along the Arlanza trunk sewer. The site location on the A/A Trunk Sewer, on the north edge of the car park, was selected in order to minimize the impacts upstream and to be outside of the maintenance yard.

Figure 6 Modeled A/A Trunk Sewer Flowmeter Facility

Table 5, Table 6, and Table 7 summarize the modeled velocities under various flow conditions, for 24-inch, 30-inch, and 36-inch diameter siphons, respectively. According to Table 5, a 24-inch diameter inverted siphon satisfies the velocity criteria for both existing and build-out conditions. Figure 7 compares the minimum DWF conditions for the three different meter sizes and shows that a 24-inch diameter inverted siphon is able to maintain velocities above 2 fps for a majority of the day. On the other hand, the profiles for the 30-inch and 36-inch diameter flowmeters do not meet the veliocity criteria. A 30-inch diameter inverted siphon would only meet the velocity criteria of 2 fps only for a brief time period.

Further analysis of the 24-inch diameter inverted siphon (Figure 8) shows that this unit approaches the surcharge criteria under existing PWWF conditions. Figure 8 shows the HGL is almost 3 feet below the rim at the A/A Trunk Sewer junction structure for a 24-inch diameter inverted siphon. While the surcharge criteria is nearly exceeded, the more critical concern is the backwater conditions. Figure 8 also shows that the A/A Trunk Sewer downstream of the meter structure experiences backwater conditions that occur for various conditions, mainly RWQCP headworks boundary conditions. The only way to correct this is by improving the hydraulics at the headworks.

Table 5 A/A Trunk Sewer 24-Inch Inverted Siphon - Velocity Summary

| Velocity (fps) | Existing Minimum DWF(1) | Existing ADWF | Existing PWWF(2) | Build-Out ADWF | Build-Out PWWF(2) |
|---|---|---|---|---|---|
| Average | 2.60 | 4.91 | 4.95 | 5.84 | 6.93 |
| Maximum | 3.36 | 6.45 | 6.46 | 7.65 | 8.90 |
| Minimum | 1.30 | 2.47 | 2.43 | 2.75 | 4.18 |

Notes:
(1) Minimum DWF = 53% ADWF.
(2) Assumes 10-year 24-hour storm event.

Table 6 A/A Trunk Sewer 30-Inch Inverted Siphon - Velocity Summary

| Velocity (fps) | Existing Minimum DWF(1) | Existing ADWF | Existing PWWF(2) | Build-Out ADWF | Build-Out PWWF(2) |
|---|---|---|---|---|---|
| Average | 1.65 | 3.15 | 3.19 | 3.75 | 4.44 |
| Maximum | 2.15 | 4.13 | 4.14 | 4.89 | 5.68 |
| Minimum | 0.84 | 1.60 | 1.62 | 1.91 | 2.69 |

Notes:
(1) Minimum DWF = 53% ADWF.
(2) Assumes 10-year 24-hour storm event.

Table 7 A/A Trunk Sewer 36-Inch Inverted Siphon - Velocity Summary

| Velocity (fps) | Existing Minimum DWF(1) | Existing ADWF | Existing PWWF(2) | Build-Out ADWF | Build-Out PWWF(2) |
|---|---|---|---|---|---|
| Average | 1.15 | 2.19 | 2.21 | 2.59 | 3.08 |
| Maximum | 1.49 | 2.86 | 2.88 | 3.38 | 3.94 |
| Minimum | 0.57 | 1.16 | 1.14 | 1.28 | 1.88 |

Notes:
(1) Minimum DWF = 53% ADWF.
(2) Assumes 10-year 24-hour storm event.

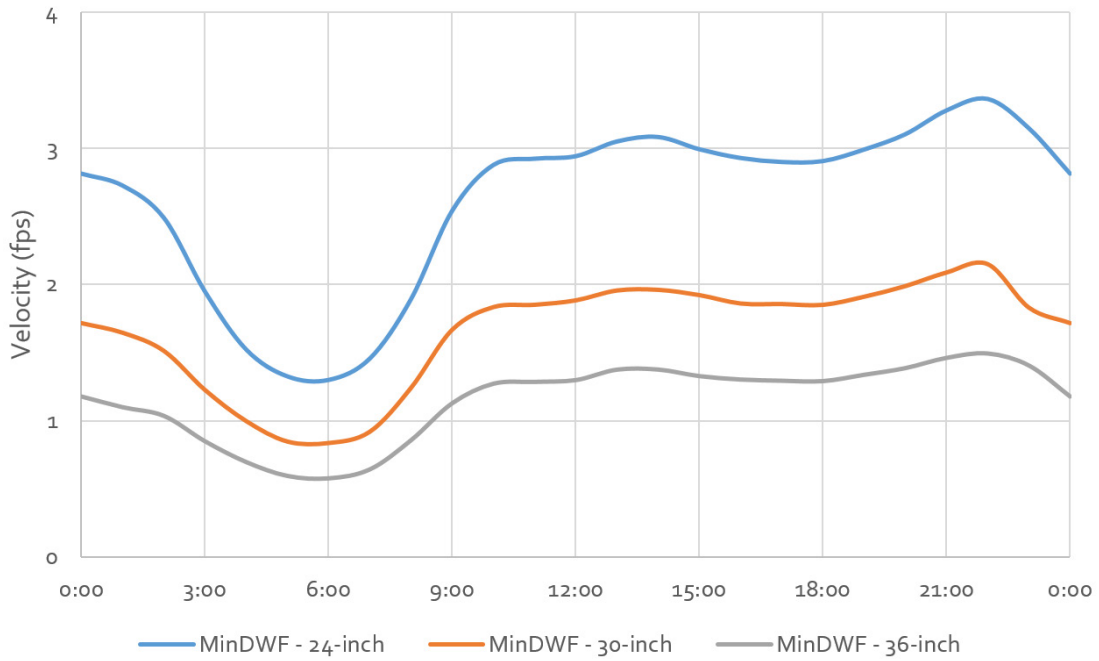


Figure 7 A/A Trunk Sewer - Velocity Under Existing Minimum DWF Conditions

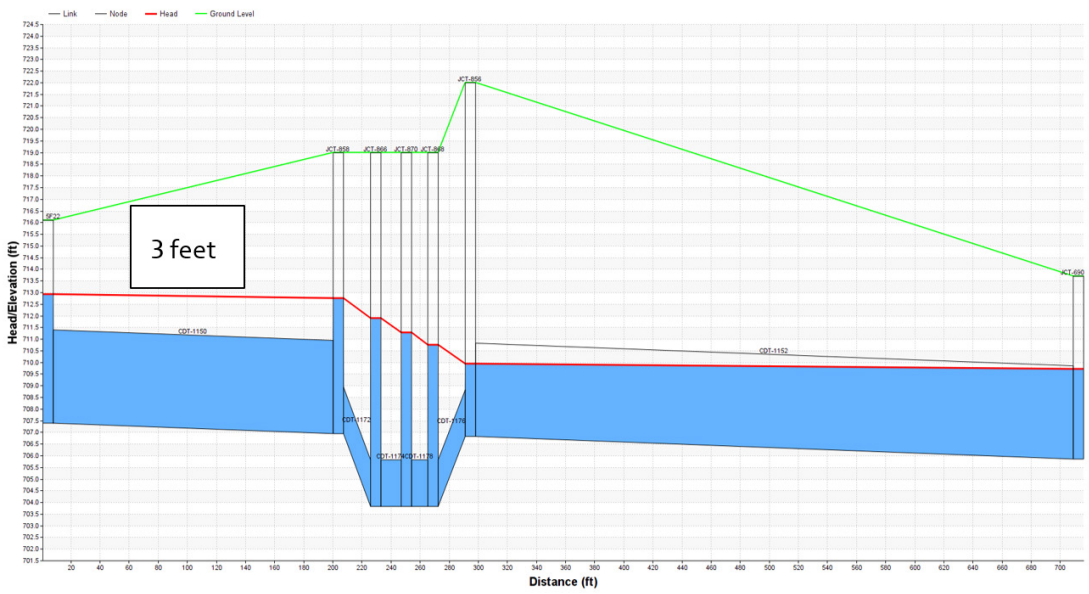


Figure 8 A/A Trunk Sewer - 24-Inch Inverted Siphon HGL Under Existing PWWF Conditions

Figure 9 and Figure 10 show the HGL under existing PWWF conditions for 30-inch and 36-inch diameter inverted siphons, respectively. These diameters meet the allowable surcharge conditions under existing PWWF conditions. However, a single inverted siphon diameter struggles to meet the design criteria under varying conditions.

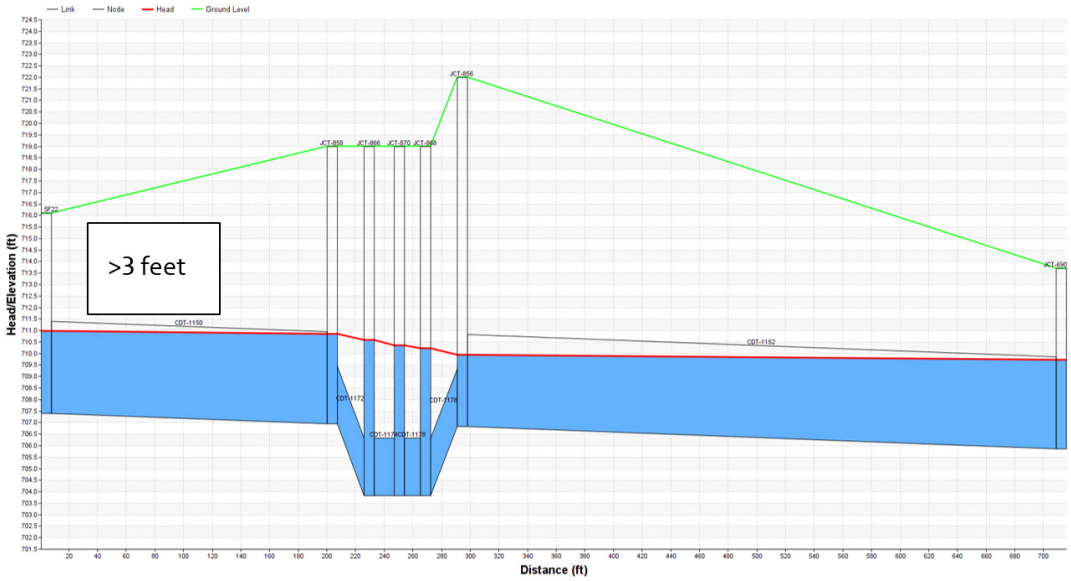


Figure 9 A/A Trunk Sewer - 30-Inch Inverted Siphon HGL Under Existing PWWF Conditions

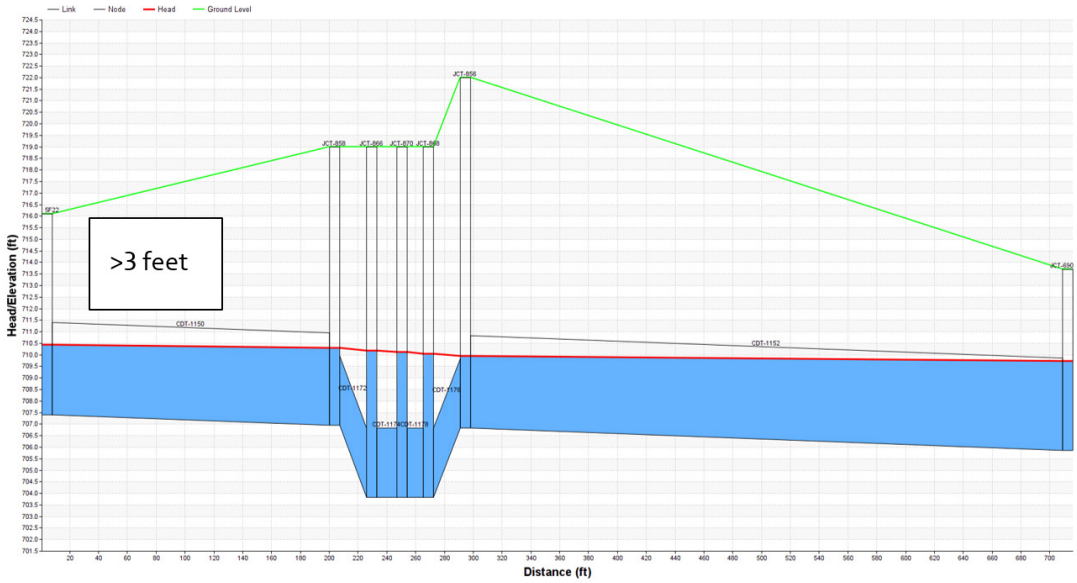


Figure 10 A/A Trunk Sewer - 36-Inch Inverted Siphon HGL Under Existing PWWF Conditions

The hydraulic analysis shows that having a single magnetic flowmeter along the A/A Trunk Sewer is not feasible and is not recommended for the following reasons:

- Pipe diameters are not operationally flexible.
- The impacts that backwater conditions at the headworks will have on the magnetic flowmeter are uncertain.
- The hydraulic model shows reverse flow through the inverted siphon, which could complicate operations and flowmeter performance.
- Increased operation and maintenance would be required to clean out settled solids through the bottom of the inverted siphon due to backwater conditions.

Accordingly, it is recommended that the City install two separate metering structures on the A/A Trunk Sewer line, as presented in URS's 2014 Study. A copy of this study is included in Attachment A.

## Section 6

# COSTS

This report confirms the proposed site layouts presented by the URS study in 2014, which can be referenced in Attachment A. The updated cost are presented below and have been escalated to 2018 dollar amounts. A more detailed cost estimate can be referenced in Attachment B. Note that for the Santa Ana metering system, the new meter may be located outside the plant boundary, but probably within the City's right-of-way. It was assumed that if the City has to purchase a small piece of land to accommodate construction, this would be less than $10,000.

Table 8 Summary of Costs for Influent Meter Project

| Description | Cost |
|---|---|
| **Metering System** | |
| Santa Ana (Riverside-Hillside) Metering System(1) | $887,000 |
| Acorn Metering System | $613,000 |
| Arlanza Metering System | $886,000 |
| Start-Up Testing and Training | $22,000 |
| Prepare and Maintain Record Drawings | $11,000 |
| Prepare Operation and Maintenance Manuals | $11,000 |
| Allowances(2) | $608,000 |
| Subtotal | $3,038,000 |
| Construction Cost Subtotal(3) | $4,800,000 |
| Total Project Cost(4) | $6,227,000 |

Notes:
(1) Assumes any land purchase to accommodate meter system would be less than $10,000.
(2) Allowances includes construction difficulty which is estimated at 25% of total construction cost.
(3) Construction cost includes 30% contingency, 10% General Conditions, 15% overhead, escalation to 2018 dollars, and sales tax.
(4) Project cost includes engineering, management, and legal, which is estimated at 30%.

Section 7

# SUMMARY

As mentioned in Volume 4, Chapter 2, Summary of Planning Studies, the City commissioned URS to complete a Preliminary Design Report to address concerns regarding influent metering limitations. URS proposed new metering facilities and upgrades to electromagnetic flowmeters. In addition to the work completed by URS, the City asked Carollo to evaluate the potential of installing a single centralized master meter to measure the combined flow rate of all wastewater entering the plant. This alternative was found to be not feasible due to the hydraulic limitations at the headworks. The hydraulic losses through the headworks would need to be lowered by about 4 feet to make this alternative hydraulically feasible. This was estimated to cost between $50 and $70 million.

Centralized electromagnetic flowmeter for the A/A Trunk Sewer was also evaluated, and the location of the Santa Ana Trunk Sewer electromagnetic flowmeter structure was reviewed. The hydraulic analysis showed that a centralized electromagnetic flowmeter for the A/A Trunk Sewer is not feasible, but the selected location for the Santa Ana trunk sewer flowmeter was confirmed.

For planning purposes, this report includes the updated costs of three electromagnetic metering structures prepared by URS in the PDR as part of the CIP. The costs were updated to reflect the criteria laid out in Volume 2, Chapter 4, Basis of Cost Estimates.

Attachment A

# 2014 URS PDR FOR INFLUENT FLOW METERING PROJECT

PRELIMINARY DESIGN REPORT

# INFLUENT FLOW METERING PROJECT
# RIVERSIDE REGIONAL WATER QUALITY CONTROL PLANT

Prepared for:

**City of Riverside**
**Public Works Department**
**Water Quality Control Plant Engineering Section**
5950 Acorn Street
Riverside, California 92504

October 31, 2014

Prepared by:

URS Corporation
2020 East First Street, Suite 400
Santa Ana, California 92705
(714) 835-6886  Fax: (714) 433-7701
www.urs.com

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# Acronyms and Abbreviations

| | |
|---|---|
| AACE | American Association of Cost Engineering |
| AADF | Annual Average Daily Flow |
| ac | Acre |
| ACI | American Concrete Institute |
| ACPA | American Concrete Pipe Association |
| ADF | Maximum total average daily flow |
| ADWF | Average Dry Weather Flow |
| ASCE | American Society of Civil Engineers |
| ASTM | American Society for Testing and Materials |
| AWWA | American Water Works Association |
| CCFRPM | Centrifugally cast fiberglass reinforced polymer mortar |
| cfs | Cubic Feet per Second |
| CSD | Community Services District |
| Elev | Elevation |
| fps | Feet per Second |
| ft | Foot |
| HART | Highway Addressable Remote Transducer |
| HEC | Hydraulic Engineering Circular |
| HGL | Hydraulic Grade Line |
| HSG | Hydrologic Soil Group |
| hr | Hour |
| in | Inch |
| IDF | Intensity-Duration-Frequency |
| mgd | Million gallons per day |
| mDWF | Minimum Dry Weather Flow |
| NEMA | National Electrical Manufacturers Association |
| NRCS | Natural Resources Conservation Service |
| OH&P | Overhead and Profit |
| PDWF | Peak Dry Weather Flow |
| PWWF | Peak Wet Weather Flow |
| RCP | Reinforced Concrete Pipe |
| RWQCP | Riverside Regional Water Quality Control Plant |
| SCS | Soil Conservation Service |
| SS | Stainless Steel |
| SSA | Autodesk Storm and Sanitary Analysis 2012 software |
| Tc | Time of Concentration |
| URS | URS Corporation |
| V | Volt |
| VCP | Vitrified Clay Pipe |
| WSEL | Water Surface Elevation |
| yr | Year |

# 1.0 INTRODUCTION

URS Corporation (URS) has been contracted by the City of Riverside Public Works Department (City) to update the Influent Metering Study prepared by Carollo Engineers, Inc. (May 2005), prepare construction documents, and provide engineering support during construction for three new influent flow metering facilities for the Riverside Regional Water Quality Control Plant (RWQCP or Plant). The purpose of this project is to upgrade the flow meters on three existing influent sewage trunk lines entering the RWQCP to obtain more accurate and reliable flow measurements.

## 1.1 EXISTING INFLUENT METERING

Owned and operated by the City, RWQCP is located at 5950 Acorn Street in Riverside, California. The Plant receives domestic, commercial, and industrial wastewater from a 87.4-square mile service area that includes the City of Riverside, the community services districts (CSDs) of Edgemont, Jurupa, Rubidoux, and, starting in 2015, development in the Highgrove Community. The Plant is capable of generating tertiary effluent compliant with nitrified Title 22 standards for groundwater recharge, stream augmentation of the Santa Ana River, and landscape irrigation. The current design capacity of the Plant is 40 million gallons per day (mgd) on an average annual basis. The maximum total average daily flow (ADF) was 34.0 mgd in the year 2005 based on annual averages for the time period 2000 to 2012. At the completion of a Phase 1 expansion project, the Plant will be able to treat up to 46 mgd, which is higher than the projected flow rate for the year 2035. Future expansion may increase Plant capacity to 52 mgd.

Influent is currently conveyed to the RWQCP via five influent trunk sewers, which are metered as follows:

- Rubidoux trunk sewer: 14-inch ABB MagMaster electromagnetic flow meter.
- Jurupa trunk sewer: Marsh-McBirney Flo-Dar™ (Flo-Dar) radar velocity and ultrasonic depth flow meter.
- Acorn and Arlanza trunk sewers: Combined at a junction structure within the Plant and their combined flow is metered with a Flo-Dar unit.
- Santa Ana River (Riverside/Hillside) trunk sewers: Combined at a junction structure upstream of the Plant and their combined flow is metered with a Flo-Dar unit.

The ABB MagMaster electromagnetic flow meter on the Rubidoux trunk sewer measures full pipe flow with a rated accuracy of approximately ±0.3% for velocities above 1.64 feet per second (fps) and is checked for calibration on an annual basis. In contrast, the radar/ultrasonic flow meters on the other influent trunk sewers measure open channel flow with a manufacturer-stated accuracy of ±5% in uniform flow conditions without surcharge, and require more frequent (e.g., quarterly and after major storm events). Note that unsteady flow conditions or submergence of the meter during high flows can cause inaccurate flow reporting.

# 2.0 SCOPE OF WORK

The scope of this project is to replace the two existing Flo-Dar meters for the Acorn/Arlanza combined trunk sewers and Santa Ana River (Riverside/Hillside) combined trunk sewers with three electromagnetic flow meters (i.e., by adding separate meters for the Acorn and Arlanza trunk sewers) to allow the City to obtain more accurate influent flow data. This influent flow data is critical for RWQCP operations staff to monitor and control the Plant's treatment processes.

In addition, a separate project under consideration is to re-route the Jurupa trunk sewer into a sewer force main, which would relocate it from its current location and connect it to the 51-inch diameter Arlanza line within the Plant's property. The design of the new Arlanza metering facility is to accommodate a stub-out for the future addition of the Jurupa sewer force main.

This preliminary design report is the deliverable for "Task 4 – Preliminary Design Report Update" within "Phase I: Preliminary Design" of the project scope of work. The assessment of the existing flow metering facilities is based on URS' observations during site visits, review of existing documentation provided by the City, and an open channel flow study performed by URS. The scope of this preliminary design report includes:

- Calculations and analysis of the hydraulic performance of the existing metering facilities:
- Preliminary design criteria for the proposed metering facilities;
- A proposed design for the metering facilities based on the updated baseline and future flow rates for each of the trunk lines developed by URS in Technical Memorandum #1;
- A preliminary construction cost estimate;
- Preliminary design plans; and
- A proposed site investigation program.

# 3.0 HYDRAULIC DESIGN

## 3.1 DESIGN FLOW RATES

Using the data provided by the City and the URS' flow study, revised present and future design flow rates were developed in URS' Technical Memorandum #1 ( November 25, 2013). Despite steady population growth in the Plant's service area over the time period from 2005 to 2013, influent flows have been on a relatively flat or declining trend due to water conservation practices and recent economic conditions. In part, the conservation has contributed to the reduction in the revised present design flows.

The City has developed flow projections based on projected population growth to the year 2035. In addition, the City plans to increase capacity of RWQCP from 40 mgd to 52 mgd. The ultimate flow rates for the individual lines were determined by using the contracted maximum capacities for Edgemont CSD, Jurupa CSD, Rubidoux CSD, and the Highgrove Community area. The remaining flows for the Santa Ana and Acorn/Arlanza trunk sewers were increased by preserving the same flow percentage for each influent line as in the revised present flows, and scaling up the result to the total combined flow of 52 mgd. Flow projections for each of the influent trunk sewers are summarized in Table 1. These revised present and future design flows were used to check the current capacity of the influent trunk lines, and also to design the proposed flow meters. Figure G-1 in Appendix A – Plans provides a site overview plan identifying the proposed metering locations.

**Table 1: Revised Present and Future Design Flows for Plant Influent Trunk Sewers**

| RWQCP Influent Trunk Sewers | Revised Present Design Flows (mgd) | | | | | Ultimate Future Design Flows (mgd) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | AADF | % of total flow | mDWF | PDWF | PWWF | AADF | % of total flow | mDWF | PDWF | PWWF |
| Rubidoux | 2.0 | 6% | 0.1 | 3.0 | 4.5 | 3.1 | 6% | 0.1 | 4.6 | 6.7 |
| Jurupa | 3.2 | 10% | 0.0 | 4.7 | 6.9 | 4.0 | 8% | 0.0 | 6.0 | 8.8 |
| Santa Ana | 11.6 | 36% | 5.8 | 17.4 | 25.5 | 21.9 | 42% | 11.0 | 32.9 | 48.2 |
| *Riverside/Hillside* | *11.0* | *34%* | *5.5* | *16.6* | *24.3* | *16.6* | *32%* | *8.3* | *25.0* | *36.6* |
| *Edgemont* | *0.6* | *2%* | *0.3* | *0.9* | *1.3* | *0.9* | *2%* | *0.4* | *1.3* | *2.0* |
| *Highgrove* | *0.0* | *0%* | *0.0* | *0.0* | *0.0* | *4.4* | *8%* | *2.2* | *6.6* | *9.7* |
| Acorn/Arlanza | 15.9 | 49% | 7.9 | 23.8 | 34.9 | 23.0 | 44% | 11.5 | 34.5 | 50.6 |
| *Arlanza* | *14.5* | *45%* | *7.3* | *21.8* | *32.0* | *21.1* | *41%* | *10.6* | *31.7* | *46.4* |
| *Acorn* | *1.3* | *4%* | *0.7* | *2.0* | *2.9* | *1.9* | *4%* | *1.0* | *2.9* | *4.2* |
| TOTALS | 32.7 | 100% | 13.8 | 49.0 | 71.8 | 52.0 | 100% | 26.0 | 78.0 | 114.4 |

## 3.2 HYDRAULIC PROFILE MODELING

The Acorn, Arlanza, and Santa Ana influent trunk lines were hydraulically modeled to determine if each line has sufficient capacity for the present design flows and ultimate future design flows shown in Table 1 above. The model predicts hydraulic surcharges for each flow scenario. Also, the model determines if the addition of the proposed metering facilities will significantly impact the hydraulic profiles and, if so, can be used to identify locations of surcharge mitigation strategies.

Using as-built plans of the existing influent trunk lines provided by the City and the design flow rates, URS developed a tabular calculation to generate hydraulic grade line profiles of each influent trunk line. The downstream boundary condition for the hydraulic profiles was set at the Plant headworks and the upstream boundary condition was set at the manhole upstream of the proposed metering facilities (with the exception of the Santa Ana line, where the upstream boundary condition was set at the seventh manhole upstream of the proposed facilities).

Where data was lacking from the as-built plans provided by the City, reasonable assumptions were made for the purposes of this assessment. These assumed values are planned to be replaced with data obtained in the proposed field investigations (see Section 6.0 – Proposed Site Investigation).

## 3.3 HYDRAULIC CALCULATIONS METHODOLOGY

URS performed tabular calculations of the invert elevation changes, minor flow losses, and pipe friction losses to determine the hydraulic grade line (HGL) for each influent trunk line.

The Jurupa, Santa Ana (Riverside/Hillside), Acorn, and Arlanza influent trunk lines were evaluated from where they each connect to the headworks of the RWQCP. All influent trunk line invert elevations, lengths, diameters, flow rates, velocities, pipeline material, associated bends, manholes, and junction boxes were determined. From the headworks, each influent trunk line's minor headloss for all pipe bends, and all inlets and outlets from junction boxes and manholes were determined using the product of its associated minor loss coefficient and velocity head. The friction losses for influent trunk line lengths were determined using the Hazen-Williams equation. Using the water surface elevation within the headworks, the minor headloss and friction losses were added to the invert elevations throughout each influent trunk line to determine its HGL. The HGL was compared to the ground surface elevation of each manhole to determine if overflow could potentially occur. The calculations for the hydraulic profiles are provided in Appendix B – Calculations.

The tabular calculations were first performed for existing conditions to determine if the existing influent trunk lines had sufficient capacity at present and ultimate future flows. An additional set of tabular calculations was performed for the proposed metering facilities.

## 3.4 HYDRAULIC PROFILE RESULTS AND CONCLUSION

The hydraulic calculations show that surcharge or near surcharge conditions may occur under high flow scenarios for all of the Plant's influent lines in their existing condition. Adding the proposed metering facilities will increase hydraulic losses on the lines, and therefore will increase the likelihood of surcharge conditions. These results are summarized in Table 2. However, it should be noted that under the proposed condition, a bypass line will be added to prevent flooding at the metering facility. This bypass line will add additional capacity for large flow events, which should mitigate the impact of the increased head losses resulting from the addition of the proposed metering facilities.

**Table 2: Hydraulic Analysis Results for Existing and Proposed Facilities**

| Scenario | Flow Criteria | Hydraulic Surcharge (Yes / No) | | | |
|---|---|---|---|---|---|
| | | Minimum Dry Weather Flow (mDWF) | Annual Average Daily Flow (AADF) | Peak Dry Weather Flow (PDWF) | Peak Wet Weather Flow (PWWF) |
| Existing | Revised Present Flows | No | No | No | <1' Freeboard on Santa Ana and Arlanza |
| | Ultimate Future Flows | No | <1' Freeboard on Santa Ana | Yes, Santa Ana (0.47' surcharge); <1' Freeboard on Arlanza | Yes, Santa Ana (4.08' surcharge), Arlanza (6.62' surcharge), and Acorn (0.60' surcharge) |
| Proposed | Revised Present Flows | No | No | <1' Freeboard on Santa Ana | Yes, Santa Ana and Arlanza |
| | Ultimate Future Flows | No | Yes, Santa Ana (0.02' surcharge) | Yes, Santa Ana (2.69' surcharge) and Arlanza (0.73' surcharge) | Yes, Santa Ana (8.78' surcharge), Arlanza (9.92' surcharge), and Acorn (1.30' surcharge) |

# 4.0 DESIGN CRITERIA AND METER SIZING

The design objective for the proposed flow meters is to achieve self-cleansing velocity at least once a day (i.e., greater than 2 feet per second [fps] minimum) while not exceeding pipe scouring velocity at the flow meter. For increased accuracy, design self-cleansing velocities were selected according to relative pipe diameter, based on recommended values for tractive force for sanitary sewers (Schafer, 1994). In addition, the proposed flow meters are designed to minimize scouring potential due to extremely high flow velocities. Industry-standard practice is to use 10 fps for maximum scouring velocity in pipes (e.g., in ASCE Manual 60), and 8 fps is commonly used for force mains.

As a secondary consideration, electromagnetic flow meters are typically more accurate at velocities greater than 1 to 2 fps. The design criteria for the proposed flow meters are summarized in Table 3:

**Table 3: Flow Meter Design Criteria for Pipe Flow Velocities**

| Pipe Diameter | Self-Cleansing Velocity (min. velocity at present PDWF) | Pipe Scouring Velocity (max. velocity at future PDWF) |
|---|---|---|
| ≤18” | 3.6 fps desirable; 2.0 fps min. | 7.0 fps |
| 24” | 3.8 fps | 7.0 fps |
| 36” | 4.0 fps | 7.5 fps |
| 48” | 4.3 fps | 8.0 fps |

The bypass pipe design will be such that, during wet weather events, the weir gates can be operated to divert flows from the metered run and avoid exceeding the maximum scouring velocities.

## 4.1 RECOMMENDED METER SIZES

The hydraulic calculations were used to predict the flow velocity within each proposed flow meter. The sizing of the flow meters and associated pipes were selected to meet the design criteria as close as feasible. Table 4 provides the recommended flow meter sizes for each influent trunk sewer. Note that the metered run of pipe is designed to always have full flow, and thus the diameter of this section of pipe is reduced in size compared to the existing trunk line to achieve higher flow velocities.

**Table 4: Recommended Flow Meter Sizes for Plant Influent Trunk Sewers**

| Influent Trunk Line and Existing Diameter | Metered Run Pipe Diameter and Flow Meter Diameter | Self-Cleansing Velocity (min. velocity at present PDWF) | Pipe Scouring Velocity (max. velocity at future PDWF) |
|---|---|---|---|
| Acorn, 27” | 16” | 2.2 fps | 3.2 fps |
| Arlanza, 51” | 36” | 4.8 fps | 6.9 fps |
| Santa Ana, 48” | 36” | 3.8 fps | 7.2 fps |

# 5.0 PROPOSED DESIGN

## 5.1 ELECTROMAGNETIC FLOW METERS

Based on available electromagnetic flow meters that are considered suitable for the proposed metering facilities, two different meter types were initially considered:

1. Full flow meters: Although widely used, these meters require that the pipe run be depressed and/or use a smaller diameter pipe to achieve full flow.

2. Partial flow meters: Using a newer technology, these meters can accurately measure flow as long as the pipe is running approximately 10% full or more.

After further evaluation, partial flow meters were eliminated from consideration because of their significantly higher cost. Therefore, the preliminary design includes full flow type meters that meet the following criteria:

- Standard water temperatures
- Hard rubber flow tube lining
- Stainless steel (SS) flanges rated AWWA Class D (150 psi)
- NEMA 4X enclosures
- 316 SS electrodes
- Grounding rings
- Remote mounted transmitter with 5 meters of cable
- RS232/485 and HART protocol

Commercial vendors for Toshiba, ABB, Siemens, and Sparling provided budgetary quotes for suitable electromagnetic flow meters, as shown in Table 5.

**Table 5: Quotations from Vendors for Recommended Electromagnetic Flow Meters**

| Manufacturer / Product line | Accuracy | Pipe flow Configuration | Straight Pipe Requirements | Budgetary Cost (24" Ø) | Budgetary Cost (48" Ø) |
|---|---|---|---|---|---|
| Toshiba LF664 | ±0.5% of flow rate | Full flow | 3Ø upstream 0Ø downstream | $11,848 | $40,285 |
| ABB FEW | ±0.4% of flow rate | Full flow | 5Ø upstream 2Ø downstream | $20,600 | $63,500 |
| Siemens Mag 5100W | ±2.0% of flow rate | Full flow | 5Ø upstream 3Ø downstream | $11,591 | $29,501 |
| Sparling FM656 | ±1.0% of flow rate | Full flow | 3Ø upstream 3Ø downstream | $11,560 | $26,216 |

*Notes:*

1. *Upstream pipe length "L" of 5Ø for 90 bend, tee, diffuser, or fully opened sluice valve; L=10Ø for other valves (not fully opened); optional mount anywhere with L=3Ø*
2. *Budgetary costs are for meters only and do not include shipping, handling, installation, and contractor overhead and profit (OH&P).*

## 5.2 PROPOSED METERING FACILITIES

The proposed metering facility layout for each trunk line has been designed to meet the unique constraints at each specific location, to minimize cost, and to simplify construction. The configuration of the proposed metering facilities for the Acorn, Arlanza, and Santa Ana lines are similar in many respects. All three configurations include an influent diversion structure, a meter vault that protects the meter, an outlet diversion structure, and a bypass pipeline, as shown in Appendix A - Plans. Wastewater is directed to the flow meter or through the bypass line using sluice gates that are located within the influent diversion structure and outlet diversion structure. All flow meters will be located a full pipe diameter below the invert of the outlet structure to provide the full flow conditions required to obtain accurate readings. In the event the flow meter or bypass requires servicing, the sluice gates could be closed in such a manner that the flow meter or bypass line can be serviced.

Note that it is assumed that bypass flow events will be rare and that they are not required to be metered.

## 5.3 DIVERSION STRUCTURE SLUICE GATES

The Acorn, Arlanza, and Santa Ana flow meters will require sluice gates to divert wastewater through either the flow pipeline or the bypass line. The sluice gates will be selected based on the following design criteria:

- Mechanical actuation for ease of operation including:
    - Electric motor-driven floor stand. Electrical power requirement assumed to be 3-phase 480-volt (to be confirmed in subsequent design stages).
    - Hydraulic cylinder operator, which is optimal when there are multiple gates that can be operated from a single system.
    - Rising stem.
- Fabricated 316 SS preferred over cast iron to increase service life in the corrosive environment found at wastewater treatment plants.
- Seated and unseated design head will be determined during final design.
- Operating floor elevation and invert elevation will be determined during final design.

Recommended manufacturers include:

- **Rodney Hunt:** Manufacturer of sluice gates that are in full compliance with AWWA Standard C-560 and provide low leakage, complete drainage systems. A wide variety of sizes are available, ranging from 8-inch x 8-inch to 120-inch x 120-inch.
- **Waterman Industries**: Manufacturer of heavy duty sluice gates with different shapes and dimensions that are expected to meet the size requirements for the proposed influent metering system, from round to square gates ranging from 6-inch to 144-inch dimensions. These gates meet the leakage requirements of AWWA Standard C-561.

## 5.4 VAULTS

The influent diversion structure, the outlet diversion structure, and the meter vault structure selections were based on the following design criteria:

- Cast-in-place vaults
- Penetrations for pipes with seals
- Sump pump for interior space of the meter vault
- Vehicle loading H-20 rated hatches
- T-lock lining

Recommended vault manufacturers include:

- **OldCastle Precast:** Leading manufacturer of precast concrete, polymer concrete, and plastic products in the United States. They manufacture a wide range of utility vaults, reinforced concrete pipe, catch basins, retaining walls, and concrete barriers that meet all appropriate ACI, ASTM, and ACPA standards for precast concrete vaults.
- **Jensen Precast:** For over 40 years, Jensen Precast has designed and manufactured standard and custom precast concrete products to meet large-scale infrastructure demands for highway construction, retaining walls, electric and gas utility, telecommunications, water, on-site wastewater, and sewage applications.

## 5.5 Pipes and Valves

For the proposed piping, we recommend using centrifugally cast fiberglass reinforced polymer mortar (CCFRPM) pipe and welded steel pipe. In addition because of its low pipe friction losses, CCFRPM pipe is also proposed to be used for the bypass pipe and the section of meter pipe downstream of the meter vault. A welded steel pipe is required for the metered pipe run from the influent vault, connecting via bolted flanges to the electromagnetic meter, and transitioning to CCFRPM beyond the meter vault. Rubber seals are proposed for all pipe penetrations through proposed vaults and manholes to provide water-tight seals.

Recommended pipe manufacturers include:

- **Northwest Pipe:** Northwest pipe has manufactured welded steel pipe since 1966. Their rolled and welded steel pipes are applicable to water transmission pipelines and water and wastewater treatment plants. Their pipes are designed to meet and exceed ASTM A53 for durability.
- **Hobas Pipe:** Hobas Pipe USA is a producer and supplier of CCFRPM pipe systems. Hobas Pipe meets ASTM D3262, ASTM D3754, AWWA C950, and AWWA M45 for non-pressure sanitary sewers, sewer force mains and industrial effluents, pressure water systems, and fiberglass design, respectively.

# 6.0 PROPOSED SITE INVESTIGATION

## 6.1 GEOTECHNICAL STUDY AND INVESTIGATION

URS proposes to advance one test borehole at each proposed metering facility using a standard truck-mounted drill rig and hollow-stem augers. The boreholes will be located as near the proposed meter vaults as feasible given site constraints (ground slope, existing aboveground and belowground facilities, etc.). The boreholes will be advanced to a depth of approximately 30 feet below ground surface or to hollow-stem auger refusal. Samples of the materials encountered will be taken at approximately 5-foot depth intervals using Standard Penetration Test (SPT) or California samplers.

Prior to drilling, URS will mark the proposed borehole locations at the site and will contact Underground Service Alert of Southern California (DigAlert) so that member agencies can review the proposed borehole locations for potential interference with existing underground facilities. The property owner will also be asked to review the drilling locations for potential damage to existing facilities.

URS will perform laboratory testing to classify subsurface materials. We anticipate that the soils encountered will be mainly granular, so the testing will primarily consist of particle-size distribution and water content tests.

URS will perform additional geotechnical analyses to support the recommendations for selection of the underground vaults. The analysis will include evaluation of soil bearing, lateral earth pressures acting on the proposed vaults, and resistance to lateral loading due to passive pressure. Engineering properties needed for the analyses will be based on the identified soil types and blow counts.

A single geotechnical report will be prepared for the three proposed metering sites. The report will include borehole logs and laboratory test results, a brief description of subsurface conditions, as well as recommendations for the facilities. The report will be stamped by the California-registered engineer in responsible charge of the work.

## 6.2 SITE SURVEY

URS proposes to utilize our surveyor subconsultant to locate surficial utility features (e.g., manholes, valve cans, etc.) and determine existing topography to develop a current base map. URS has collected as-built drawings from the City and others as necessary to create a utility base map. We will also perform a web search of USA Digalert's online database to locate all known utilities in the project vicinity as necessary and then contact each utility owner to request a copy of the available as-built drawings. In addition, the surveyor will locate missing invert elevations for manholes as required to finalize the hydraulic model.

## 6.3 POTHOLING

URS proposes to utilize our potholing subconsultant to perform utility potholing to determine the location and depth of the existing utilities. The potholing subconsultant will obtain accurate subsurface utility information using a non-destructive, minimally invasive, vacuum process, which exposes the utility for visual verification, identification, inspection, measurements, and vertical and horizontal location of the utility. URS will obtain, if required, encroachment permits from the City for the potholing fieldwork. URS anticipates that 20 utility potholes would be required.

# 7.0 CONSTRUCTION

Construction of the influent flow metering facilities is estimated to require approximately 10 months, which accounts for procurement lead time and sequencing of construction activities to maintain Plant operations. Construction of the metering systems will require the following:

- Mobilization
- Site demolition and clearing
- Open excavation
- Bypass pumping installation
- Existing pipe demolition
- Construction of inlet, outlet and meter vaults
- Installation of sluice gates, valves, hatches, and fittings
- Installation of bypass pipe, meter pipe, flow meter
- Backfill
- Demobilization

Excavations are likely to be vertically-sided and shored because of site constraints. Some existing utilities and pipelines may need to be temporarily relocated and supported.

## 7.1 PRELIMINARY CONSTRUCTION COSTS

An Engineer's Opinion of Probable Construction Costs for the project is included in Appendix C. This cost estimate is based on preliminary design data and was developed using commercial vendor quotes, recent cost data from similar projects, and the 2012 Caltrans Contract Cost Data Book. The estimate includes a 30% contingency, a 15% general contractor overhead and profit, a 3% escalation to construction midpoint, and a 10% owner's reserve for change orders. The accuracy of the cost estimate follows the guidelines established by the American Association of Cost Engineers for a preliminary (or concept level) design, with a +50% to -30% accuracy.

## 7.2 REGULATORY AUTHORITIES AND PERMITTING

Table 6 outlines the regulatory reviews and permits anticipated for this project.

**Table 6: Regulatory Reviews and Permits for Proposed Metering Facilities**

| Agency | Approval/Permit Description |
|---|---|
| City of Riverside<br>Public Works Department | Demolition Permit<br>Building Permit<br>Encroachment Permit<br>Construction Permit |

# 8.0 ENGINEERING CERTIFICATION

This report has been prepared under my direction as the professional engineer in direct responsible charge of the work, in accordance with the provisions of the Professional Engineer's Act of the State of California.

Bryan C. Paine, PE

Registration No.: C64334

Date: October 6, 2014

## 8.1 STATEMENT OF LIMITATIONS

This report has been prepared by URS for the sole use of the City in the evaluation of the Project described above and is not to be distributed to third parties outside the City's organization. The scope of services performed during this preliminary design stage may not be appropriate to satisfy the needs of other users, and any use or re-use of this document or of the findings, conclusions, or recommendations presented herein is at the sole risk of said user.

Background information, design bases, and other data have been furnished to URS by the City and/or third parties, which URS has used in preparing this report. URS has relied on this information as furnished, and is neither responsible for nor has confirmed the accuracy of this information. This report is based on data, site conditions and other information that is generally applicable as of October 2014, and the conclusions and recommendations herein are therefore applicable only to that time frame.

This report has been prepared based on certain key assumptions made by URS which substantially affect the conclusions and recommendations of this report. These assumptions, although thought to be reasonable and appropriate, may not prove to be true in the future. The conclusions and recommendations of URS are conditioned upon these assumptions.

# 9.0 REFERENCES

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22. *City of Riverside Capital Improvement Program and Rate Development Study*. MWH Americas, Inc. and Carollo Engineers, Inc., 2014.

# APPENDIX A – PLANS

SANTA ANA RIVER

RWQCP

PROPOSED SANTA ANA METERING FACILITIES (SEE FIGURES C-3 AND M-3)

PROPOSED ACORN METERING FACILITIES (SEE FIGURES C-1 AND M-1)

PROPOSED ARLANZA METERING FACILITIES (SEE FIGURES C-2 AND M-2)

VAN BUREN BLVD

JURUPA AVE

ACORN ST

N

0 150 300

SCALE: 1" = 300

# PLAN

NOT FOR CONSTRUCTION

URS

CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT
WATER QUALITY CONTROL PLANT ENGINEERING SECTION
5950 ACORN STREET
RIVERSIDE, CA 92504

SITE OVERVIEW PLAN

| | |
|---|---|
| URS Proj. No.: 29880194 | Date: OCTOBER 2014 |
| Project: INFLUENT FLOW METERING PROJECT 30% PRELIMINARY DESIGN | Figure: G-1 |

T:\071\City Of Riverside\Influent Flow Metering (29880194)\6 Engineering\CADD\020 Civil\C-PDR Vault Layouts.dwg , File date: 9/29/2014 6:47 PM , Print date: 10/1/2014 4:35 PM by: Chase, Nathan

EXIST. ADMIN/LAB BUILDING

EXIST. 3" STL. NATURAL GAS

EXIST. 12" A.C. POTABLE WATER

EXIST. 27" RCP ACORN INFLUENT

EXIST. 36" RCP STORM DRAIN

ACORN STREET

PROPOSED MANHOLE AC-1

REMOVE EXIST. PIPE

EXIST. 36" RCP ACORN INFLUENT

TO EXIST. ACORN / ARLANZA LINE JUNCTION STRUCTURE

EXIST. CATCH BASIN, TYP.

EXIST. SIDEWALK

PROPOSED MANHOLE AC-3

EXIST. LIGHT STANDARD, TYP.

PROPOSED MANHOLE AC-2

EXIST. ROCK LANDSCAPE AREA

PROPOSED METER VAULT

PROPOSED OUTLET VAULT

RELOCATE EXIST. TREE AS NEEDED, TYP.

PROPOSED INFLUENT VAULT

EXIST. PARKING LOT

NOT FOR CONSTRUCTION

N

# PLAN

0 10 20

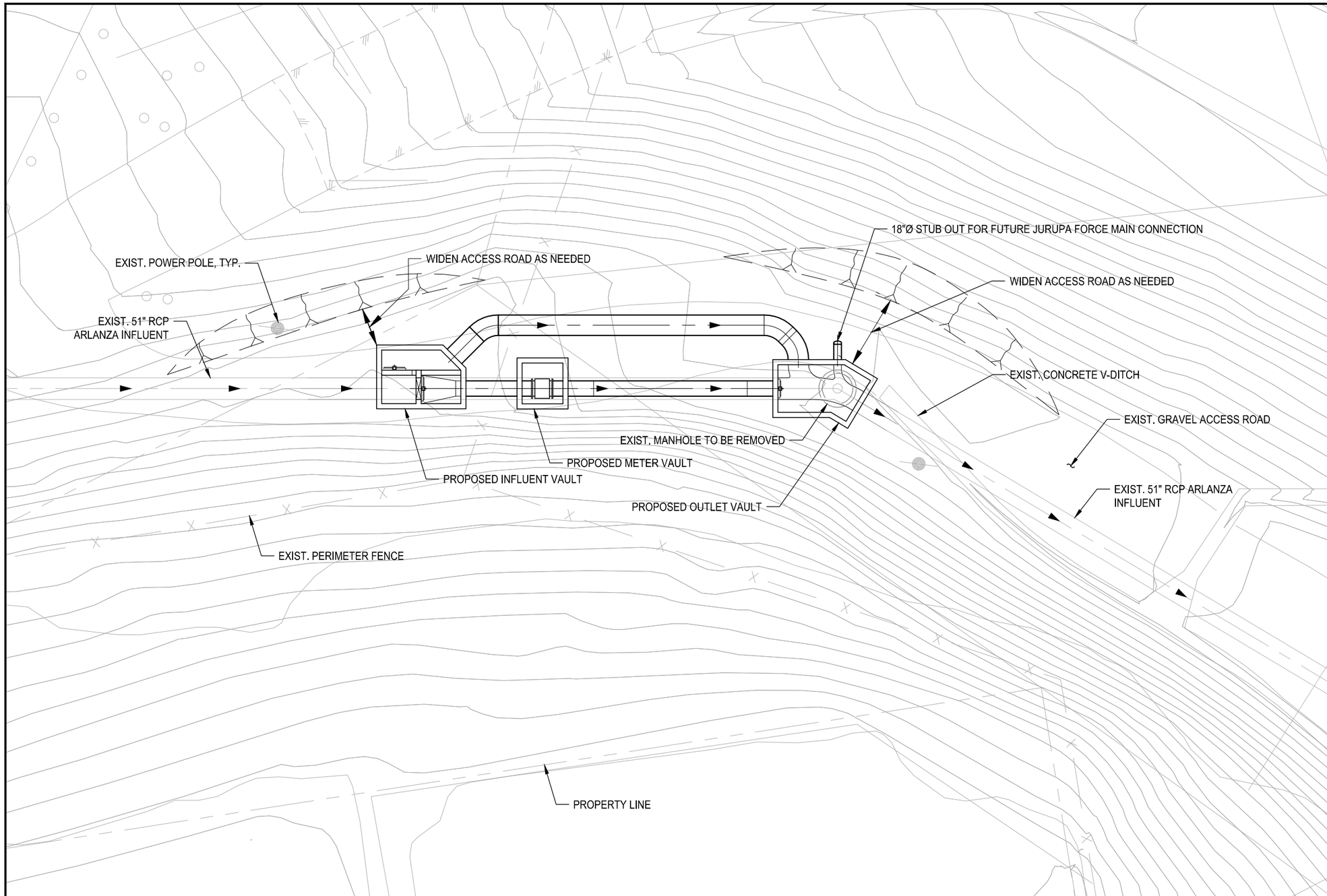
SCALE: 1" = 20

URS

CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT
WATER QUALITY CONTROL PLANT ENGINEERING SECTION
5950 ACORN STREET
RIVERSIDE, CA 92504

ACORN PLAN

| | |
|---|---|
| URS Proj. No.: 29880194 | Date: OCTOBER 2014 |
| Project: INFLUENT FLOW METERING PROJECT 30% PRELIMINARY DESIGN | Figure: C-1 |




# PLAN

NOT FOR CONSTRUCTION

URS

CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT
WATER QUALITY CONTROL PLANT ENGINEERING SECTION
5950 ACORN STREET
RIVERSIDE, CA 92504

ARLANZA PLAN

| | |
|---|---|
| URS Proj. No.: 29880194 | Date: OCTOBER 2014 |
| Project: INFLUENT FLOW METERING PROJECT<br>30% PRELIMINARY DESIGN | Figure: C-2 |

EXIST. CHAIN LINK FENCE

REGRADE 3:1 SLOPE AS NEEDED

EXIST. DUAL 12" STL. JURUPA SEWER FORCEMAINS

PROPOSED METER VAULT

WIDEN ACCESS ROAD AS NEEDED

EXIST. BOLLARDS, TYP.

REMOVE INTERFERING PORTIONS OF EXIST. 45" PIPE (ABANDONED)

EXIST. MANHOLE, TYP.

PROPOSED INFLUENT VAULT

PROPOSED OUTLET VAULT

EXIST. 41" CCFRPM SEWER

EXIST. JURUPA / RUBIDOUX INFLUENT MONITORING STATION

EXIST. 21" CCFRPM SEWER

EXIST. 48" SANTA ANA INFLUENT

EXIST. 14" RUBIDOUX BYPASS STEEL SEWER FORCEMAIN

EXIST. 18" SEWER

EXIST. 18" RUBIDOUX SEWER FORCEMAIN

EXIST. GRAVEL ACCESS ROAD

EXIST. SANTA ANA INFLUENT MONITORING STATION

EXIST. DE ANZA GATE

EXIST. RUBIDOUX FLOW METERING VAULT

PROPERTY LINE

EXIST. 42" RCP TO HEADWORKS (TYP. OF 2)

NOT FOR CONSTRUCTION

# PLAN

N

0 10 20

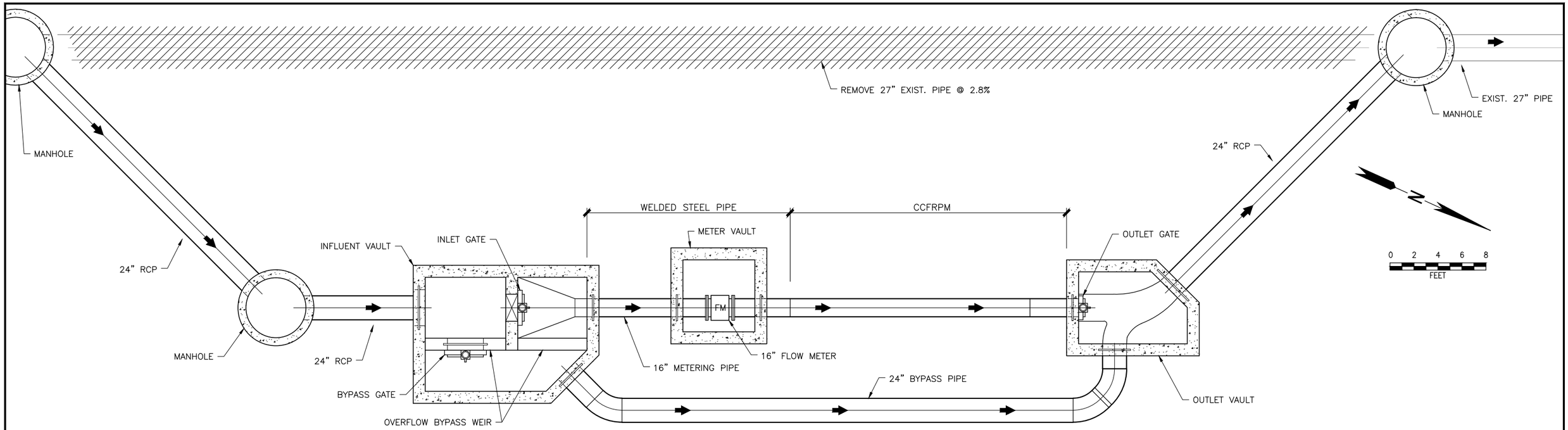
SCALE: 1" = 20

URS

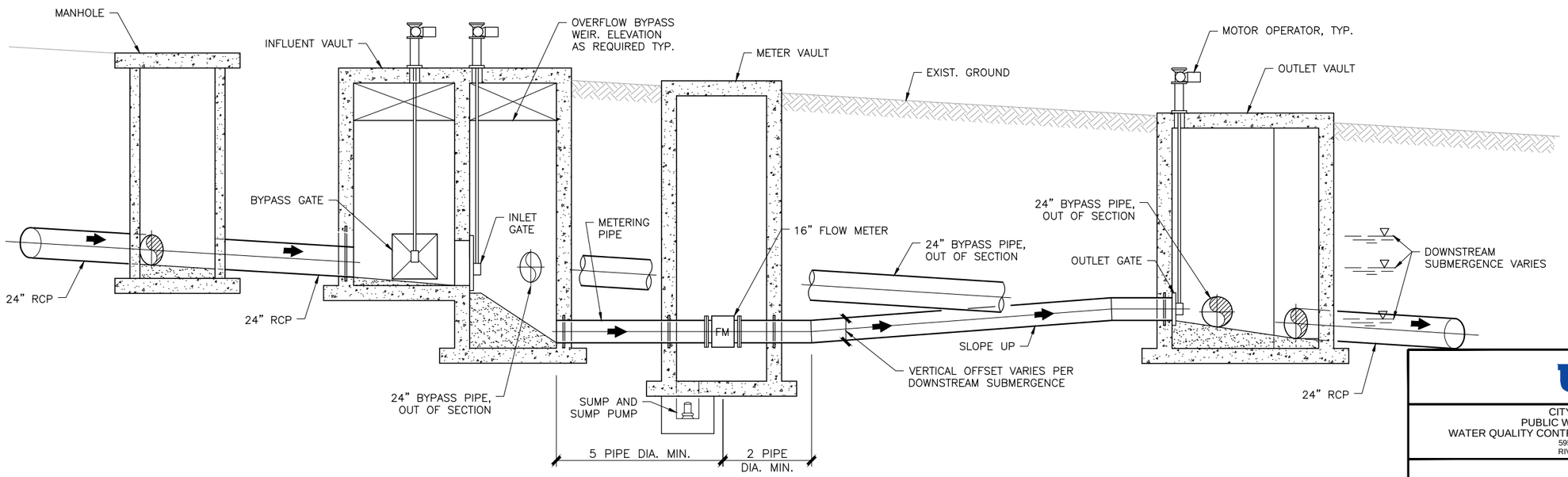
CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT
WATER QUALITY CONTROL PLANT ENGINEERING SECTION
5950 ACORN STREET
RIVERSIDE, CA 92504

SANTA ANA PLAN

| | |
|---|---|
| URS Proj. No.: 29880194 | Date: OCTOBER 2014 |
| Project: INFLUENT FLOW METERING PROJECT 30% PRELIMINARY DESIGN | Figure: C-3 |




ACORN PLAN




SECTION

NOT FOR CONSTRUCTION

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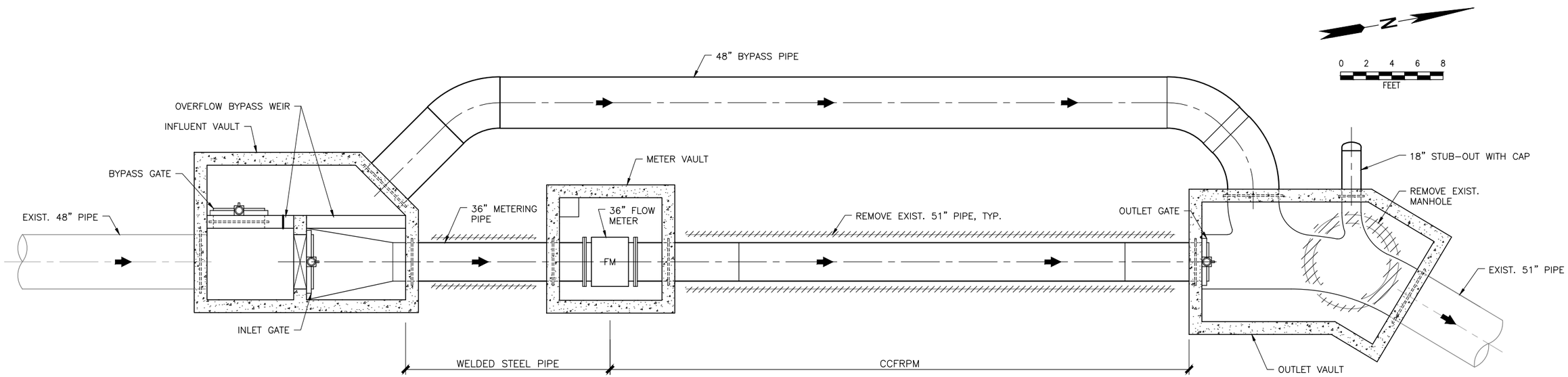
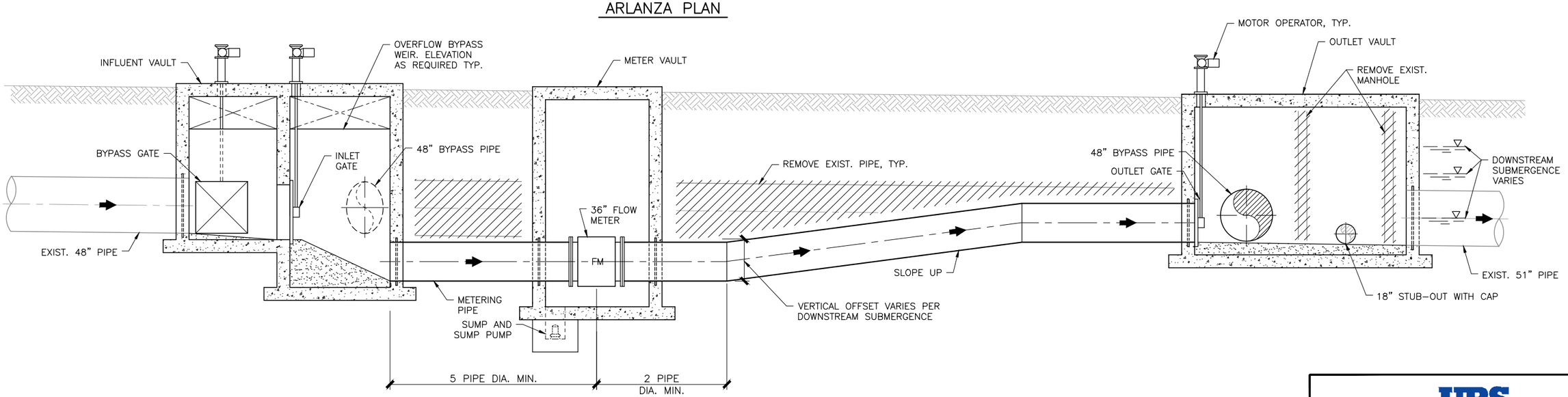
PUBLIC WORKS DEPARTMENT
WATER QUALITY CONTROL PLANT ENGINEERING SECTION
5950 ACORN STREET
RIVERSIDE, CA 92504

ACORN PLAN AND SECTION

| | |
|---|---|
| URS Proj. No.: 29880194 | Date: OCTOBER 2014 |
| Project: INFLUENT FLOW METERING PROJECT 30% PRELIMINARY DESIGN | Figure: M-1 |




ARLANZA PLAN

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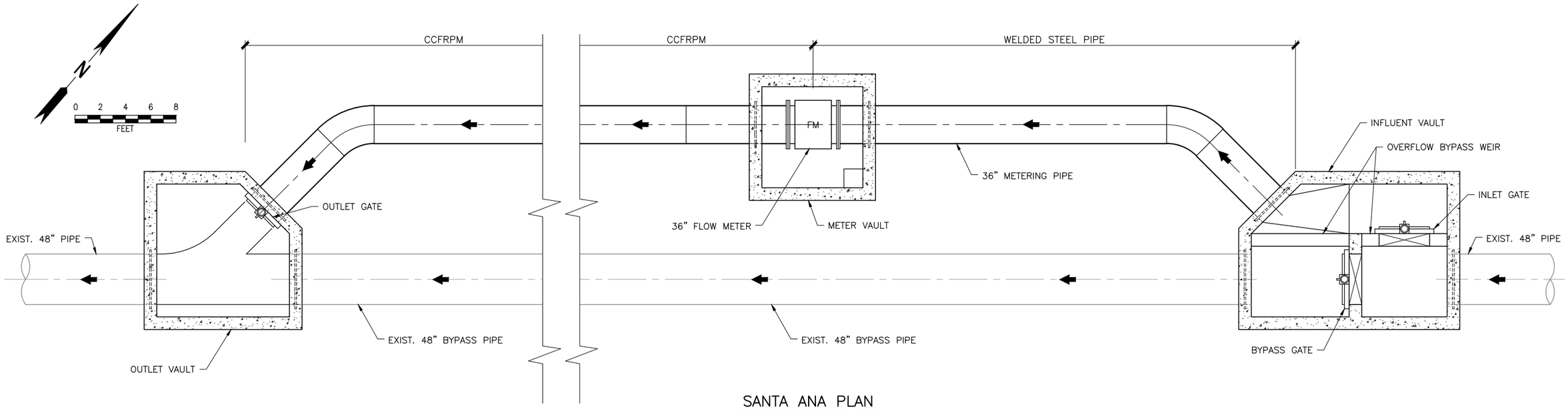
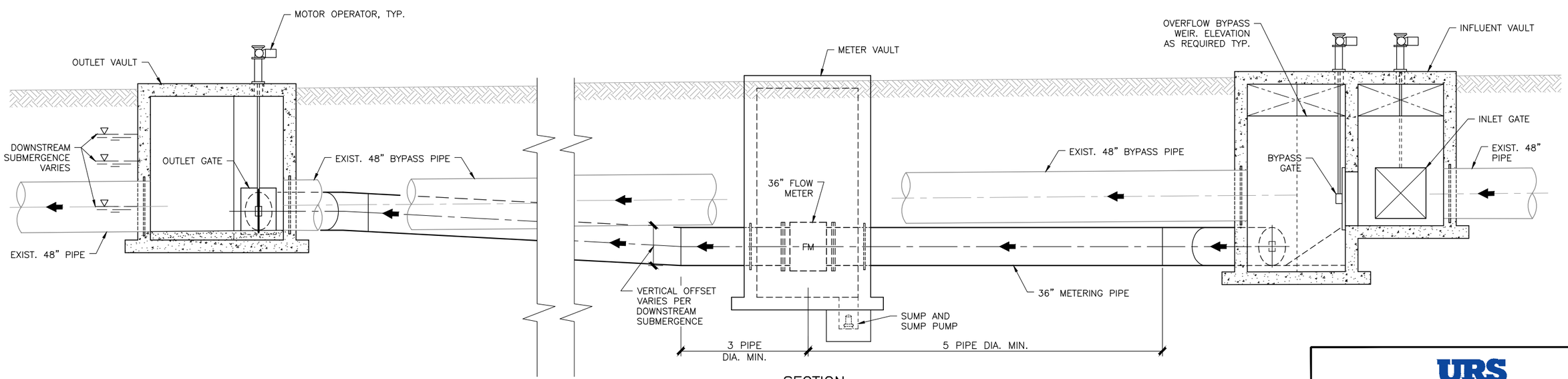
CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT
WATER QUALITY CONTROL PLANT ENGINEERING SECTION
5950 ACORN STREET
RIVERSIDE, CA 92504

ARLANZA PLAN AND SECTION

| | |
|---|---|
| URS Proj. No.: 29880194 | Date: OCTOBER 2014 |
| Project: INFLUENT FLOW METERING PROJECT 30% PRELIMINARY DESIGN | Figure: M-2 |




SANTA ANA PLAN




SECTION

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URS

CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT
WATER QUALITY CONTROL PLANT ENGINEERING SECTION
5950 ACORN STREET
RIVERSIDE, CA 92504

SANTA ANA PLAN AND SECTION

| | |
|---|---|
| URS Proj. No.: 29880194 | Date: OCTOBER 2014 |
| Project: INFLUENT FLOW METERING PROJECT 30% PRELIMINARY DESIGN | Figure: M-3 |

# APPENDIX B – CALCULATIONS

# EQUATIONS USED

*HGL = Hydraulic Grade Line*
*HGL= Downstream HGL + Head Losses*
*Head Losses = Minor Head Losses + Pipe Friction Losses*
*Δh= hm + hf*
*where,*
*Δh= Head Losses (ft)*
*hm= Minor Loss (ft)*
*hf= Friction Loss (ft)*

*Minor Loss Equation*

$$hm = \frac{Kv^2}{2g}$$

*where,*
*hm= Minor Loss (ft)*
*K= Minor Loss Coefficient*
*v= Velocity (ft/sec)*
*g= Gravitational Constant (32.2 ft/s²)*

*Pipe Flow Velocity Equation*

$$V = \frac{Q}{\pi D^2/4}$$

*where,*
*Q= Flow (cfs)*
*D= Pipe Diameter (ft)*
*v= Velocity (ft/sec)*

*Hazen-Williams Pipe Friction Loss Equation*

$$hf = \frac{4.727\ LQ^{1.85}}{D^{4.87}C^{1.85}}$$

*where,*
*L= Length of Pipe (ft)*
*Q= Flow Rate (cfs)*
*D= Diameter of Pipe (ft)*
*C= Hazen-Williams Coefficient (unitless)*

URS

## EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Rubidoux Flow Rate =** 3.0 MGD
**Checked by:** BCP Date: 9/29/2014

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Acorn Flow Rate =** 2.0 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined at Revised Present Design Flow - PDWF* | | *36.8* | | | | | | | | | | | | |
| | *Acorn at Revised Present Design Flow - PDWF* | | *3.1* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 2 | 18.4 | 42 | 1.9 | 0.06 | 0.5 | | | 0.03 | 705.70 | | **710.48** | **713.70** | **3.22** |
| 4 | RCP (Headworks) | 1 | 36.8 | 48 | | | | 110 | 21 | 0.02 | 705.75 | 0.05 | 710.54 | | |
| 5 | 90 Degree Bend | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.9 | | | 0.12 | 705.75 | 0.00 | 710.66 | | |
| 6 | RCP (Headworks) | 1 | 36.8 | 48 | | | | 110 | 79 | 0.06 | 705.94 | 0.19 | 710.91 | | |
| 7 | Meter Box Outlet | 1 | 36.8 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 705.94 | 0.00 | 711.04 | | |
| 8 | **Meter Box Inlet** | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.5 | | | 0.07 | 705.96 | 0.02 | **711.13** | **715.90** | **4.77** |
| 9 | RCP (Headworks) | 1 | 36.8 | 48 | | | | 110 | 491 | 0.36 | 707.10 | 1.14 | 712.63 | | |
| 10 | 35 Degree Bend | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.4 | | | 0.05 | 707.10 | 0.00 | 712.68 | | |
| 11 | RCP (Headworks) | 1 | 36.8 | 48 | | | | 110 | 129 | 0.09 | 707.40 | 0.30 | 713.08 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 36.8 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 707.40 | 0.00 | 713.21 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 3.1 | 48 | 0.2 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **713.21** | **716.10** | **2.89** |
| 14 | RCP (Acorn) | 1 | 3.1 | 36 | | | | 110 | 148 | 0.00 | 711.31 | 3.91 | 717.12 | | |
| 15 | RCP (Acorn) | 1 | 3.1 | 27 | | | | 110 | 180 | 0.02 | 716.30 | 8.90 | 722.13 | | |
| 16 | Acorn Manhole #1 Exit | 1 | 3.1 | 27 | 0.8 | 0.01 | 1 | | | 0.01 | 716.30 | 0.00 | 722.14 | | |
| 17 | **Acorn Manhole #1 Inlet** | 1 | 3.1 | 27 | 0.8 | 0.01 | 0.5 | | | 0.00 | 716.30 | 0.00 | **722.15** | **728.30** | **6.15** |
| 18 | RCP (Acorn) | 1 | 3.1 | 27 | | | | 110 | 339 | 0.04 | 735.49 | 19.19 | 741.38 | | |
| 19 | Acorn Manhole #2 Exit | 1 | 3.1 | 27 | 0.8 | 0.01 | 1 | | | 0.01 | 735.49 | 0.00 | 741.39 | | |
| 20 | **Acorn Manhole #2 Inlet** | 1 | 3.1 | 27 | 0.8 | 0.01 | 0.5 | | | 0.00 | 735.49 | 0.00 | **741.39** | **747.50** | **6.11** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 21.8 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined at Revised Present Design* | | *36.8* | | | | | | | | | | | | |
| | *Arlanza at Revised Present Design Flow - PDWF* | | *33.7* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 18.4 | 42 | 1.9 | 0.06 | 0.5 | | | 0.03 | 705.70 | | **710.48** | **713.70** | **3.22** |
| 4 | RCP (Headworks) | 1 | 36.8 | 48 | | | | 110 | 21 | 0.02 | 705.75 | 0.05 | 709.71 | | |
| 5 | 90 Degree Bend | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.9 | | | 0.12 | 705.75 | 0.00 | 709.83 | | |
| 6 | RCP (Headworks) | 1 | 36.8 | 48 | | | | 110 | 79 | 0.06 | 705.94 | 0.19 | 710.07 | | |
| 7 | Meter Box Outlet | 1 | 36.8 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 705.94 | 0.00 | 710.21 | | |
| 8 | **Meter Box Inlet** | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.5 | | | 0.07 | 705.96 | 0.02 | **710.30** | **716.00** | **5.70** |
| 9 | RCP (Headworks) | 1 | 36.8 | 48 | | | | 110 | 491 | 0.36 | 707.10 | 1.14 | 711.79 | | |
| 10 | 35 Degree Bend | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.4 | | | 0.05 | 707.10 | 0.00 | 711.84 | | |
| 11 | RCP (Headworks) | 1 | 36.8 | 48 | | | | 110 | 129 | 0.09 | 707.40 | 0.30 | 712.24 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 36.8 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 707.40 | 0.00 | 712.37 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 18.4 | 48 | 1.5 | 0.03 | 0.5 | | | 0.02 | 707.40 | 0.00 | **712.39** | **716.10** | **3.71** |
| 14 | RCP (Arlanza) | 2 | 16.9 | 48 | | | | 110 | 293 | 0.05 | 708.00 | 0.60 | 713.04 | | |
| 15 | Arlanza Junction Box Exit | 2 | 16.9 | 48 | 1.3 | 0.03 | 1 | | | 0.03 | 708.00 | 0.00 | 713.07 | | |
| 16 | **Inlet - Arlanza Junction Box #1** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 708.00 | 0.00 | **713.11** | **717.50** | **4.39** |
| 17 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 713.13 | | |
| 18 | 30 Degree Bend | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.4 | | | 0.04 | 708.02 | 0.00 | 713.17 | | |
| 19 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 17 | 0.01 | 708.08 | 0.06 | 713.23 | | |
| 20 | 20 Degree Bend | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.25 | | | 0.02 | 708.08 | 0.00 | 713.26 | | |
| 21 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 86 | 0.04 | 708.39 | 0.31 | 713.61 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 708.39 | 0.00 | 713.69 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 708.39 | 0.00 | **713.74** | **724.80** | **11.06** |
| 24 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 161 | 0.07 | 708.97 | 0.58 | 714.39 | | |
| 25 | 50 Degree Bend | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.7 | | | 0.06 | 708.97 | 0.00 | 714.45 | | |
| 26 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 24 | 0.01 | 709.05 | 0.09 | 714.55 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 709.05 | 0.00 | 714.64 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 709.05 | 0.00 | **714.68** | **727.50** | **12.82** |
| 29 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 150 | 0.07 | 709.59 | 0.54 | 715.29 | | |
| 30 | Exit - Junction Box | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 709.59 | 0.00 | 715.38 | | |
| 31 | **Inlet - Arlanza Junction Box #2** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 709.59 | 0.00 | **715.42** | **725.00** | **9.58** |
| 32 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 303 | 0.14 | 710.68 | 1.09 | 716.65 | | |
| 33 | 20 Degree Bend | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.25 | | | 0.02 | 710.68 | 0.00 | 716.67 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 21.8 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 495 | 0.23 | 712.46 | 1.78 | 718.68 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 712.46 | 0.00 | 718.77 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 712.46 | 0.00 | **718.81** | **724.00** | **5.19** |
| 37 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 428 | 0.20 | 713.32 | 0.86 | 719.87 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 713.32 | 0.00 | 719.95 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 713.32 | 0.00 | **720.00** | **726.30** | **6.30** |
| 40 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 428 | 0.20 | 714.06 | 0.74 | 720.94 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 714.06 | 0.00 | 721.03 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 714.06 | 0.00 | **721.07** | **725.60** | **4.53** |
| 40 | RCP (Arlanza) | 1 | 33.7 | 51 | 2.4 | 0.09 | | 110 | 246 | 0.11 | 718.46 | 4.40 | 725.59 | | |
| 41 | Arlanza Manhole #6 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 718.46 | 0.00 | 725.67 | | |
| 42 | **Arlanza Manhole #6 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 718.46 | 0.00 | **725.72** | **730.00** | **4.28** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box #1 invert elevation and manhole elevation were found on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 17.4 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) at Revised Present Design Flow - PDWF* | | *36.6* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) at Revised Present Design Flow - PDWF* | | *26.9* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 18.3 | 42 | 1.9 | 0.06 | 0.5 | | | 0.03 | 705.70 | | **710.48** | **713.70** | **3.22** |
| 4 | RCP (Headworks) | 2 | 18.3 | 42 | | | | 125 | 28 | 0.01 | 705.87 | 0.17 | 710.66 | | |
| 5 | 35 Degree Bend | 2 | 18.3 | 42 | 1.9 | 0.06 | 0.4 | | | 0.02 | 705.87 | 0.00 | 710.68 | | |
| 6 | RCP (Headworks) | 2 | 18.3 | 42 | | | | 125 | 200 | 0.06 | 1285.53 | 579.66 | 1290.40 | | |
| 7 | 35 Degree Bend | 2 | 18.3 | 42 | 1.9 | 0.06 | 0.4 | | | 0.02 | 1285.53 | 0.00 | 1290.42 | | |
| 8 | RCP (Headworks) | 2 | 18.3 | 42 | | | | 125 | 16 | 0.00 | 707.18 | -578.35 | 712.08 | | |
| 9 | Exit from Junction | 2 | 18.3 | 42 | 1.9 | 0.06 | 1 | | | 0.06 | 707.18 | 0.00 | 712.13 | | |
| 10 | **Inlet to Junction** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.18 | 0.00 | **712.17** | **717.00** | **4.83** |
| 11 | RCP (Headworks) | 1 | 26.9 | 48 | | | | 125 | 10 | 0.00 | 707.30 | 0.12 | 712.29 | | |
| 12 | CCFRPM Pipe (Riverside/Hillside) | 1 | 26.9 | 48 | | | | 155 | 35 | 0.01 | 707.70 | 0.40 | 712.70 | | |
| 13 | Manhole #1 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 707.70 | 0.00 | 712.77 | | |
| 14 | **Manhole #1 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.70 | 0.00 | **712.81** | **716.20** | **3.39** |
| 15 | CCFRPM Pipe (Riverside/Hillside) | 1 | 26.9 | 48 | | | | 155 | 174 | 0.04 | 708.06 | 0.36 | 713.20 | | |
| 16 | Manhole #2 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 708.06 | 0.00 | 713.28 | | |
| 17 | **Manhole #2 Inlet** | 1 | 26.9 | 41 | 2.9 | 0.13 | 0.5 | | | 0.07 | 708.06 | 0.00 | **713.34** | **714.60** | **1.26** |
| 18 | CCFRPM Pipe (Riverside/Hillside) | 1 | 23.5 | 41 | | | | 155 | 13 | 0.00 | 708.10 | 0.04 | 713.39 | | |
| 19 | Manhole #3 Exit | 1 | 23.5 | 41 | 2.6 | 0.10 | 1 | | | 0.10 | 708.10 | 0.00 | 713.49 | | |
| 20 | **Manhole #3 Inlet** | 1 | 23.5 | 41 | 2.6 | 0.10 | 0.5 | | | 0.05 | 708.10 | 0.00 | **713.54** | **714.90** | **1.36** |
| 21 | CCFRPM Pipe (Riverside/Hillside) | 1 | 23.5 | 41 | | | | 155 | 95 | 0.03 | 708.26 | 0.16 | 713.73 | | |
| 22 | Manhole #4 Exit | 1 | 23.5 | 41 | 2.6 | 0.10 | 1 | | | 0.10 | 708.26 | 0.00 | 713.84 | | |
| 23 | **Manhole #4 Inlet** | 1 | 23.5 | 41 | 2.6 | 0.10 | 0.5 | | | 0.05 | 708.26 | 0.00 | **713.89** | **715.50** | **1.61** |
| 24 | CCFRPM Pipe (Riverside/Hillside) | 1 | 23.5 | 41 | | | | 155 | 13 | 0.00 | 708.29 | 0.03 | 713.92 | | |
| 25 | Manhole #5 Exit | 1 | 23.5 | 41 | 2.6 | 0.10 | 1 | | | 0.10 | 708.29 | 0.00 | 714.02 | | |
| 26 | **Manhole #5 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 708.29 | 0.00 | **714.06** | **716.30** | **2.24** |
| 27 | CCFRPM Pipe (Riverside/Hillside) | 1 | 26.9 | 48 | | | | 155 | 78 | 0.02 | 708.43 | 0.14 | 714.22 | | |
| 28 | Manhole #6 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 708.43 | 0.00 | 714.29 | | |
| 29 | **Manhole #6 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 708.43 | 0.00 | **714.32** | **719.40** | **5.08** |
| 30 | CCFRPM Pipe (Riverside/Hillside) | 1 | 26.9 | 48 | | | | 155 | 55 | 0.01 | 708.53 | 0.10 | 714.44 | | |
| 31 | Manhole #7 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 708.53 | 0.00 | 714.51 | | |
| 32 | **Manhole #7 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 708.53 | 0.00 | **714.54** | **719.40** | **4.86** |
| 33 | CCFRPM Pipe (Riverside/Hillside) | 1 | 26.9 | 48 | | | | 155 | 215.5 | 0.05 | 708.92 | 0.39 | 714.98 | | |
| 34 | Manhole #8 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 708.92 | 0.00 | 715.05 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 17.4 MGD
**Checked by:** BCP Date: 9/29/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 35 | **Manhole #8 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 708.92 | 0.00 | **715.09** | **722.20** | **7.11** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Rubidoux Flow Rate =** 4.5 MGD
**Checked by:** BCP Date: 9/29/2014

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Acorn Flow Rate =** 2.9 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined at Revised Present Design Flow - PWWF* | | *54.0* | | | | | | | | | | | | |
| | *Acorn at Revised Present Design Flow - PWWF* | | *4.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 2 | 27.0 | 42 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.70 | | **710.51** | **713.70** | **3.19** |
| 4 | RCP (Headworks) | 1 | 54.0 | 48 | | | | 110 | 21 | 0.03 | 705.75 | 0.05 | 710.59 | | |
| 5 | 90 Degree Bend | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.9 | | | 0.26 | 705.75 | 0.00 | 710.85 | | |
| 6 | RCP (Headworks) | 1 | 54.0 | 48 | | | | 110 | 79 | 0.12 | 705.94 | 0.19 | 711.16 | | |
| 7 | Meter Box Outlet | 1 | 54.0 | 48 | 4.3 | 0.29 | 1 | | | 0.29 | 705.94 | 0.00 | 711.44 | | |
| 8 | **Meter Box Inlet** | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.5 | | | 0.14 | 705.96 | 0.02 | **711.61** | **715.90** | **4.29** |
| 9 | RCP (Headworks) | 1 | 54.0 | 48 | | | | 110 | 491 | 0.73 | 707.10 | 1.14 | 713.48 | | |
| 10 | 35 Degree Bend | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.4 | | | 0.11 | 707.10 | 0.00 | 713.59 | | |
| 11 | RCP (Headworks) | 1 | 54.0 | 48 | | | | 110 | 129 | 0.19 | 707.40 | 0.30 | 714.08 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 54.0 | 48 | 4.3 | 0.29 | 1 | | | 0.29 | 707.40 | 0.00 | 714.37 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 4.5 | 48 | 0.4 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **714.37** | **716.10** | **1.73** |
| 14 | RCP (Acorn) | 1 | 4.5 | 36 | | | | 110 | 148 | 0.01 | 711.31 | 3.91 | 718.29 | | |
| 15 | RCP (Acorn) | 1 | 4.5 | 27 | | | | 110 | 180 | 0.04 | 716.30 | 8.90 | 723.31 | | |
| 16 | Acorn Manhole #1 Exit | 1 | 4.5 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 716.30 | 0.00 | 723.33 | | |
| 17 | **Acorn Manhole #1 Inlet** | 1 | 4.5 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 716.30 | 0.00 | **723.34** | **728.30** | **4.96** |
| 18 | RCP (Acorn) | 1 | 4.5 | 27 | | | | 110 | 339 | 0.08 | 735.49 | 19.19 | 742.61 | | |
| 19 | Acorn Manhole #2 Exit | 1 | 4.5 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 735.49 | 0.00 | 742.63 | | |
| 20 | **Acorn Manhole #2 Inlet** | 1 | 4.5 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 735.49 | 0.00 | **742.64** | **747.50** | **4.86** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 32.0 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined at Revised Present Design* | | *54.0* | | | | | | | | | | | | |
| | *Arlanza at Revised Present Design Flow - PWWF* | | *49.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 27.0 | 42 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.70 | | **710.51** | **713.70** | **3.19** |
| 4 | RCP (Headworks) | 1 | 54.0 | 48 | | | | 110 | 21 | 0.03 | 705.75 | 0.05 | 709.72 | | |
| 5 | 90 Degree Bend | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.9 | | | 0.26 | 705.75 | 0.00 | 709.98 | | |
| 6 | RCP (Headworks) | 1 | 54.0 | 48 | | | | 110 | 79 | 0.12 | 705.94 | 0.19 | 710.29 | | |
| 7 | Meter Box Outlet | 1 | 54.0 | 48 | 4.3 | 0.29 | 1 | | | 0.29 | 705.94 | 0.00 | 710.57 | | |
| 8 | **Meter Box Inlet** | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.5 | | | 0.14 | 705.96 | 0.02 | **710.74** | **716.00** | **5.26** |
| 9 | RCP (Headworks) | 1 | 54.0 | 48 | | | | 110 | 491 | 0.73 | 707.10 | 1.14 | 712.60 | | |
| 10 | 35 Degree Bend | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.4 | | | 0.11 | 707.10 | 0.00 | 712.72 | | |
| 11 | RCP (Headworks) | 1 | 54.0 | 48 | | | | 110 | 129 | 0.19 | 707.40 | 0.30 | 713.21 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 54.0 | 48 | 4.3 | 0.29 | 1 | | | 0.29 | 707.40 | 0.00 | 713.50 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 27.0 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.40 | 0.00 | **713.53** | **716.10** | **2.57** |
| 14 | RCP (Arlanza) | 2 | 24.7 | 48 | | | | 110 | 293 | 0.10 | 708.00 | 0.60 | 714.23 | | |
| 15 | Arlanza Junction Box Exit | 2 | 24.7 | 48 | 2.0 | 0.06 | 1 | | | 0.06 | 708.00 | 0.00 | 714.29 | | |
| 16 | **Inlet - Arlanza Junction Box #1** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 708.00 | 0.00 | **714.39** | **717.50** | **3.11** |
| 17 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 714.41 | | |
| 18 | 30 Degree Bend | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.4 | | | 0.08 | 708.02 | 0.00 | 714.49 | | |
| 19 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 17 | 0.02 | 708.08 | 0.06 | 714.56 | | |
| 20 | 20 Degree Bend | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.25 | | | 0.05 | 708.08 | 0.00 | 714.61 | | |
| 21 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 86 | 0.08 | 708.39 | 0.31 | 715.00 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 708.39 | 0.00 | 715.19 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 708.39 | 0.00 | **715.28** | **724.80** | **9.52** |
| 24 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 161 | 0.15 | 708.97 | 0.58 | 716.01 | | |
| 25 | 50 Degree Bend | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.7 | | | 0.13 | 708.97 | 0.00 | 716.15 | | |
| 26 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 24 | 0.02 | 709.05 | 0.09 | 716.26 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 709.05 | 0.00 | 716.44 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 709.05 | 0.00 | **716.54** | **727.50** | **10.96** |
| 29 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 150 | 0.14 | 709.59 | 0.54 | 717.22 | | |
| 30 | Exit - Junction Box | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 709.59 | 0.00 | 717.41 | | |
| 31 | **Inlet - Arlanza Junction Box #2** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 709.59 | 0.00 | **717.50** | **725.00** | **7.50** |
| 32 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 303 | 0.28 | 710.68 | 1.09 | 718.88 | | |
| 33 | 20 Degree Bend | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.25 | | | 0.05 | 710.68 | 0.00 | 718.92 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 32.0 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 495 | 0.46 | 712.46 | 1.78 | 721.17 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 712.46 | 0.00 | 721.36 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 712.46 | 0.00 | **721.45** | **724.00** | **2.55** |
| 37 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 428 | 0.40 | 713.32 | 0.86 | 722.71 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 713.32 | 0.00 | 722.90 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 713.32 | 0.00 | **722.99** | **726.30** | **3.31** |
| 40 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 428 | 0.40 | 714.06 | 0.74 | 724.14 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 714.06 | 0.00 | 724.33 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 714.06 | 0.00 | **724.42** | **725.60** | **1.18** |
| 40 | RCP (Arlanza) | 1 | 49.5 | 51 | 3.5 | 0.19 | | 110 | 246 | 0.23 | 718.46 | 4.40 | 729.05 | | |
| 41 | Arlanza Manhole #6 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 718.46 | 0.00 | 729.24 | | |
| 42 | **Arlanza Manhole #6 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 718.46 | 0.00 | **729.34** | **730.00** | **0.66** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box #1 invert elevation and manhole elevation were found on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 25.5 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) at Revised Present Design Flow - PWWF* | | *51.4* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) at Revised Present Design Flow - PWWF* | | *39.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 25.7 | 42 | 2.7 | 0.11 | 0.5 | | | 0.06 | 705.70 | | **710.51** | **713.70** | **3.19** |
| 4 | RCP (Headworks) | 2 | 25.7 | 42 | | | | 125 | 28 | 0.02 | 705.87 | 0.17 | 710.69 | | |
| 5 | 35 Degree Bend | 2 | 25.7 | 42 | 2.7 | 0.11 | 0.4 | | | 0.04 | 705.87 | 0.00 | 710.74 | | |
| 6 | RCP (Headworks) | 2 | 25.7 | 42 | | | | 125 | 200 | 0.11 | 1285.53 | 579.66 | 1290.50 | | |
| 7 | 35 Degree Bend | 2 | 25.7 | 42 | 2.7 | 0.11 | 0.4 | | | 0.04 | 1285.53 | 0.00 | 1290.55 | | |
| 8 | RCP (Headworks) | 2 | 25.7 | 42 | | | | 125 | 16 | 0.01 | 707.18 | -578.35 | 712.21 | | |
| 9 | Exit from Junction | 2 | 25.7 | 42 | 2.7 | 0.11 | 1 | | | 0.11 | 707.18 | 0.00 | 712.32 | | |
| 10 | **Inlet to Junction** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 707.18 | 0.00 | **712.40** | **717.00** | **4.60** |
| 11 | RCP (Headworks) | 1 | 39.5 | 48 | | | | 125 | 10 | 0.01 | 707.30 | 0.12 | 712.52 | | |
| 12 | CCFRPM Pipe (Riverside/Hillside) | 1 | 39.5 | 48 | | | | 155 | 35 | 0.02 | 707.70 | 0.40 | 712.94 | | |
| 13 | Manhole #1 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 707.70 | 0.00 | 713.10 | | |
| 14 | **Manhole #1 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 707.70 | 0.00 | **713.17** | **716.20** | **3.03** |
| 15 | CCFRPM Pipe (Riverside/Hillside) | 1 | 39.5 | 48 | | | | 155 | 174 | 0.08 | 708.06 | 0.36 | 713.61 | | |
| 16 | Manhole #2 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 708.06 | 0.00 | 713.76 | | |
| 17 | **Manhole #2 Inlet** | 1 | 39.5 | 41 | 4.3 | 0.29 | 0.5 | | | 0.14 | 708.06 | 0.00 | **713.91** | **714.60** | **0.69** |
| 18 | CCFRPM Pipe (Riverside/Hillside) | 1 | 34.4 | 41 | | | | 155 | 13 | 0.01 | 708.10 | 0.04 | 713.96 | | |
| 19 | Manhole #3 Exit | 1 | 34.4 | 41 | 3.8 | 0.22 | 1 | | | 0.22 | 708.10 | 0.00 | 714.18 | | |
| 20 | **Manhole #3 Inlet** | 1 | 34.4 | 41 | 3.8 | 0.22 | 0.5 | | | 0.11 | 708.10 | 0.00 | **714.28** | **714.90** | **0.62** |
| 21 | CCFRPM Pipe (Riverside/Hillside) | 1 | 34.4 | 41 | | | | 155 | 95 | 0.07 | 708.26 | 0.16 | 714.51 | | |
| 22 | Manhole #4 Exit | 1 | 34.4 | 41 | 3.8 | 0.22 | 1 | | | 0.22 | 708.26 | 0.00 | 714.73 | | |
| 23 | **Manhole #4 Inlet** | 1 | 34.4 | 41 | 3.8 | 0.22 | 0.5 | | | 0.11 | 708.26 | 0.00 | **714.84** | **715.50** | **0.66** |
| 24 | CCFRPM Pipe (Riverside/Hillside) | 1 | 34.4 | 41 | | | | 155 | 13 | 0.01 | 708.29 | 0.03 | 714.88 | | |
| 25 | Manhole #5 Exit | 1 | 34.4 | 41 | 3.8 | 0.22 | 1 | | | 0.22 | 708.29 | 0.00 | 715.10 | | |
| 26 | **Manhole #5 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 708.29 | 0.00 | **715.18** | **716.30** | **1.12** |
| 27 | CCFRPM Pipe (Riverside/Hillside) | 1 | 39.5 | 48 | | | | 155 | 78 | 0.03 | 708.43 | 0.14 | 715.35 | | |
| 28 | Manhole #6 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 708.43 | 0.00 | 715.51 | | |
| 29 | **Manhole #6 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 708.43 | 0.00 | **715.58** | **719.40** | **3.82** |
| 30 | CCFRPM Pipe (Riverside/Hillside) | 1 | 39.5 | 48 | | | | 155 | 55 | 0.02 | 708.53 | 0.10 | 715.71 | | |
| 31 | Manhole #7 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 708.53 | 0.00 | 715.86 | | |
| 32 | **Manhole #7 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 708.53 | 0.00 | **715.94** | **719.40** | **3.46** |
| 33 | CCFRPM Pipe (Riverside/Hillside) | 1 | 39.5 | 48 | | | | 155 | 215.5 | 0.10 | 708.92 | 0.39 | 716.42 | | |
| 34 | Manhole #8 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 708.92 | 0.00 | 716.58 | | |

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 25.5 MGD
**Checked by:** BCP Date: 9/29/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 35 | **Manhole #8 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 708.92 | 0.00 | **716.65** | **722.20** | **5.55** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Ultimate Future Design Flow - AADF

**Calculated by:** NTC Date: 9/26/2014 **Rubidoux Flow Rate =** 3.1 MGD
**Checked by:** BCP Date: 9/29/2014

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/26/2014 **Acorn Flow Rate =** 1.9 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined at Ultimate Future Design Flow - AADF* | | *35.6* | | | | | | | | | | | | |
| | *Acorn at Ultimate Future Design Flow - AADF* | | *3.0* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 2 | 17.8 | 42 | 1.9 | 0.05 | 0.5 | | | 0.03 | 705.70 | | **710.48** | **713.70** | **3.22** |
| 4 | RCP (Headworks) | 1 | 35.6 | 48 | | | | 110 | 21 | 0.01 | 705.75 | 0.05 | 710.54 | | |
| 5 | 90 Degree Bend | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.9 | | | 0.11 | 705.75 | 0.00 | 710.65 | | |
| 6 | RCP (Headworks) | 1 | 35.6 | 48 | | | | 110 | 79 | 0.05 | 705.94 | 0.19 | 710.90 | | |
| 7 | Meter Box Outlet | 1 | 35.6 | 48 | 2.8 | 0.12 | 1 | | | 0.12 | 705.94 | 0.00 | 711.02 | | |
| 8 | **Meter Box Inlet** | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.96 | 0.02 | **711.11** | **715.90** | **4.79** |
| 9 | RCP (Headworks) | 1 | 35.6 | 48 | | | | 110 | 491 | 0.34 | 707.10 | 1.14 | 712.58 | | |
| 10 | 35 Degree Bend | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.4 | | | 0.05 | 707.10 | 0.00 | 712.63 | | |
| 11 | RCP (Headworks) | 1 | 35.6 | 48 | | | | 110 | 129 | 0.09 | 707.40 | 0.30 | 713.02 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 35.6 | 48 | 2.8 | 0.12 | 1 | | | 0.12 | 707.40 | 0.00 | 713.14 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 3.0 | 48 | 0.2 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **713.14** | **716.10** | **2.96** |
| 14 | RCP (Acorn) | 1 | 3.0 | 36 | | | | 110 | 148 | 0.00 | 711.31 | 3.91 | 717.06 | | |
| 15 | RCP (Acorn) | 1 | 3.0 | 27 | | | | 110 | 180 | 0.02 | 716.30 | 8.90 | 722.06 | | |
| 16 | Acorn Manhole #1 Exit | 1 | 3.0 | 27 | 0.7 | 0.01 | 1 | | | 0.01 | 716.30 | 0.00 | 722.07 | | |
| 17 | **Acorn Manhole #1 Inlet** | 1 | 3.0 | 27 | 0.7 | 0.01 | 0.5 | | | 0.00 | 716.30 | 0.00 | **722.08** | **728.30** | **6.22** |
| 18 | RCP (Acorn) | 1 | 3.0 | 27 | | | | 110 | 339 | 0.04 | 735.49 | 19.19 | 741.31 | | |
| 19 | Acorn Manhole #2 Exit | 1 | 3.0 | 27 | 0.7 | 0.01 | 1 | | | 0.01 | 735.49 | 0.00 | 741.31 | | |
| 20 | **Acorn Manhole #2 Inlet** | 1 | 3.0 | 27 | 0.7 | 0.01 | 0.5 | | | 0.00 | 735.49 | 0.00 | **741.32** | **747.50** | **6.18** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 21.1 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined at Ultimate Future Design* | | *35.6* | | | | | | | | | | | | |
| | *Arlanza at Ultimate Future Design Flow - AADF* | | *32.6* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 17.8 | 42 | 1.9 | 0.05 | 0.5 | | | 0.03 | 705.70 | | **710.48** | **713.70** | **3.22** |
| 4 | RCP (Headworks) | 1 | 35.6 | 48 | | | | 110 | 21 | 0.01 | 705.75 | 0.05 | 709.70 | | |
| 5 | 90 Degree Bend | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.9 | | | 0.11 | 705.75 | 0.00 | 709.82 | | |
| 6 | RCP (Headworks) | 1 | 35.6 | 48 | | | | 110 | 79 | 0.05 | 705.94 | 0.19 | 710.06 | | |
| 7 | Meter Box Outlet | 1 | 35.6 | 48 | 2.8 | 0.12 | 1 | | | 0.12 | 705.94 | 0.00 | 710.19 | | |
| 8 | **Meter Box Inlet** | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.96 | 0.02 | **710.27** | **716.00** | **5.73** |
| 9 | RCP (Headworks) | 1 | 35.6 | 48 | | | | 110 | 491 | 0.34 | 707.10 | 1.14 | 711.75 | | |
| 10 | 35 Degree Bend | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.4 | | | 0.05 | 707.10 | 0.00 | 711.80 | | |
| 11 | RCP (Headworks) | 1 | 35.6 | 48 | | | | 110 | 129 | 0.09 | 707.40 | 0.30 | 712.18 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 35.6 | 48 | 2.8 | 0.12 | 1 | | | 0.12 | 707.40 | 0.00 | 712.31 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 17.8 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 707.40 | 0.00 | **712.32** | **716.10** | **3.78** |
| 14 | RCP (Arlanza) | 2 | 16.3 | 48 | | | | 110 | 293 | 0.05 | 708.00 | 0.60 | 712.97 | | |
| 15 | Arlanza Junction Box Exit | 2 | 16.3 | 48 | 1.3 | 0.03 | 1 | | | 0.03 | 708.00 | 0.00 | 713.00 | | |
| 16 | **Inlet - Arlanza Junction Box #1** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 708.00 | 0.00 | **713.04** | **717.50** | **4.46** |
| 17 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 713.06 | | |
| 18 | 30 Degree Bend | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.4 | | | 0.03 | 708.02 | 0.00 | 713.09 | | |
| 19 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 17 | 0.01 | 708.08 | 0.06 | 713.16 | | |
| 20 | 20 Degree Bend | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.25 | | | 0.02 | 708.08 | 0.00 | 713.18 | | |
| 21 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 86 | 0.04 | 708.39 | 0.31 | 713.53 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 708.39 | 0.00 | 713.61 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 708.39 | 0.00 | **713.65** | **724.80** | **11.15** |
| 24 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 161 | 0.07 | 708.97 | 0.58 | 714.30 | | |
| 25 | 50 Degree Bend | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.7 | | | 0.06 | 708.97 | 0.00 | 714.36 | | |
| 26 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 24 | 0.01 | 709.05 | 0.09 | 714.45 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 709.05 | 0.00 | 714.53 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 709.05 | 0.00 | **714.58** | **727.50** | **12.92** |
| 29 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 150 | 0.07 | 709.59 | 0.54 | 715.18 | | |
| 30 | Exit - Junction Box | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 709.59 | 0.00 | 715.26 | | |
| 31 | **Inlet - Arlanza Junction Box #2** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 709.59 | 0.00 | **715.30** | **725.00** | **9.70** |
| 32 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 303 | 0.13 | 710.68 | 1.09 | 716.52 | | |
| 33 | 20 Degree Bend | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.25 | | | 0.02 | 710.68 | 0.00 | 716.54 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 21.1 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 495 | 0.22 | 712.46 | 1.78 | 718.54 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 712.46 | 0.00 | 718.62 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 712.46 | 0.00 | **718.66** | **724.00** | **5.34** |
| 37 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 428 | 0.19 | 713.32 | 0.86 | 719.70 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 713.32 | 0.00 | 719.79 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 713.32 | 0.00 | **719.83** | **726.30** | **6.47** |
| 40 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 428 | 0.19 | 714.06 | 0.74 | 720.76 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 714.06 | 0.00 | 720.84 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 714.06 | 0.00 | **720.88** | **725.60** | **4.72** |
| 40 | RCP (Arlanza) | 1 | 32.6 | 51 | 2.3 | 0.08 | | 110 | 246 | 0.11 | 718.46 | 4.40 | 725.39 | | |
| 41 | Arlanza Manhole #6 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 718.46 | 0.00 | 725.47 | | |
| 42 | **Arlanza Manhole #6 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 718.46 | 0.00 | **725.51** | **730.00** | **4.49** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box #1 invert elevation and manhole elevation were found on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 21.9 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) at Ultimate Future Design Flow - AADF* | | *43.6* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) at Ultimate Future Design Flow - AADF* | | *33.9* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 21.8 | 42 | 2.3 | 0.08 | 0.5 | | | 0.04 | 705.70 | | **710.49** | **713.70** | **3.21** |
| 4 | RCP (Headworks) | 2 | 21.8 | 42 | | | | 125 | 28 | 0.01 | 705.87 | 0.17 | 710.67 | | |
| 5 | 35 Degree Bend | 2 | 21.8 | 42 | 2.3 | 0.08 | 0.4 | | | 0.03 | 705.87 | 0.00 | 710.70 | | |
| 6 | RCP (Headworks) | 2 | 21.8 | 42 | | | | 125 | 200 | 0.08 | 1285.53 | 579.66 | 1290.44 | | |
| 7 | 35 Degree Bend | 2 | 21.8 | 42 | 2.3 | 0.08 | 0.4 | | | 0.03 | 1285.53 | 0.00 | 1290.47 | | |
| 8 | RCP (Headworks) | 2 | 21.8 | 42 | | | | 125 | 16 | 0.01 | 707.18 | -578.35 | 712.14 | | |
| 9 | Exit from Junction | 2 | 21.8 | 42 | 2.3 | 0.08 | 1 | | | 0.08 | 707.18 | 0.00 | 712.22 | | |
| 10 | **Inlet to Junction** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 707.18 | 0.00 | **712.27** | **717.00** | **4.73** |
| 11 | RCP (Headworks) | 1 | 33.9 | 48 | | | | 125 | 10 | 0.00 | 707.30 | 0.12 | 712.39 | | |
| 12 | CCFRPM Pipe (Riverside/Hillside) | 1 | 33.9 | 48 | | | | 155 | 35 | 0.01 | 707.70 | 0.40 | 712.81 | | |
| 13 | Manhole #1 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 707.70 | 0.00 | 712.92 | | |
| 14 | **Manhole #1 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 707.70 | 0.00 | **712.98** | **716.20** | **3.22** |
| 15 | CCFRPM Pipe (Riverside/Hillside) | 1 | 33.9 | 48 | | | | 155 | 174 | 0.06 | 708.06 | 0.36 | 713.40 | | |
| 16 | Manhole #2 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 708.06 | 0.00 | 713.51 | | |
| 17 | **Manhole #2 Inlet** | 1 | 33.9 | 41 | 3.7 | 0.21 | 0.5 | | | 0.11 | 708.06 | 0.00 | **713.62** | **714.60** | **0.98** |
| 18 | CCFRPM Pipe (Riverside/Hillside) | 1 | 29.6 | 41 | | | | 155 | 13 | 0.01 | 708.10 | 0.04 | 713.66 | | |
| 19 | Manhole #3 Exit | 1 | 29.6 | 41 | 3.2 | 0.16 | 1 | | | 0.16 | 708.10 | 0.00 | 713.82 | | |
| 20 | **Manhole #3 Inlet** | 1 | 29.6 | 41 | 3.2 | 0.16 | 0.5 | | | 0.08 | 708.10 | 0.00 | **713.91** | **714.90** | **0.99** |
| 21 | CCFRPM Pipe (Riverside/Hillside) | 1 | 29.6 | 41 | | | | 155 | 95 | 0.05 | 708.26 | 0.16 | 714.12 | | |
| 22 | Manhole #4 Exit | 1 | 29.6 | 41 | 3.2 | 0.16 | 1 | | | 0.16 | 708.26 | 0.00 | 714.28 | | |
| 23 | **Manhole #4 Inlet** | 1 | 29.6 | 41 | 3.2 | 0.16 | 0.5 | | | 0.08 | 708.26 | 0.00 | **714.36** | **715.50** | **1.14** |
| 24 | CCFRPM Pipe (Riverside/Hillside) | 1 | 29.6 | 41 | | | | 155 | 13 | 0.01 | 708.29 | 0.03 | 714.40 | | |
| 25 | Manhole #5 Exit | 1 | 29.6 | 41 | 3.2 | 0.16 | 1 | | | 0.16 | 708.29 | 0.00 | 714.56 | | |
| 26 | **Manhole #5 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 708.29 | 0.00 | **714.62** | **716.30** | **1.68** |
| 27 | CCFRPM Pipe (Riverside/Hillside) | 1 | 33.9 | 48 | | | | 155 | 78 | 0.03 | 708.43 | 0.14 | 714.78 | | |
| 28 | Manhole #6 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 708.43 | 0.00 | 714.89 | | |
| 29 | **Manhole #6 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 708.43 | 0.00 | **714.95** | **719.40** | **4.45** |
| 30 | CCFRPM Pipe (Riverside/Hillside) | 1 | 33.9 | 48 | | | | 155 | 55 | 0.02 | 708.53 | 0.10 | 715.07 | | |
| 31 | Manhole #7 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 708.53 | 0.00 | 715.18 | | |
| 32 | **Manhole #7 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 708.53 | 0.00 | **715.24** | **719.40** | **4.16** |
| 33 | CCFRPM Pipe (Riverside/Hillside) | 1 | 33.9 | 48 | | | | 155 | 215.5 | 0.07 | 708.92 | 0.39 | 715.70 | | |
| 34 | Manhole #8 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 708.92 | 0.00 | 715.81 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 21.9 MGD
**Checked by:** BCP Date: 9/29/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 35 | **Manhole #8 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 708.92 | 0.00 | **715.87** | **722.20** | **6.33** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Ultimate Future Design Flow - PDWF

**Calculated by:** NTC Date: 9/26/2014 **Rubidoux Flow Rate =** 4.6 MGD
**Checked by:** BCP Date: 9/29/2014

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Acorn Flow Rate =** 2.9 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined at Ultimate Future Design Flow - PDWF* | | *53.4* | | | | | | | | | | | | |
| | *Acorn at Ultimate Future Design Flow - PDWF* | | *4.4* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 2 | 26.7 | 42 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.70 | | **710.51** | **713.70** | **3.19** |
| 4 | RCP (Headworks) | 1 | 53.4 | 48 | | | | 110 | 21 | 0.03 | 705.75 | 0.05 | 710.59 | | |
| 5 | 90 Degree Bend | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.9 | | | 0.25 | 705.75 | 0.00 | 710.84 | | |
| 6 | RCP (Headworks) | 1 | 53.4 | 48 | | | | 110 | 79 | 0.11 | 705.94 | 0.19 | 711.15 | | |
| 7 | Meter Box Outlet | 1 | 53.4 | 48 | 4.2 | 0.28 | 1 | | | 0.28 | 705.94 | 0.00 | 711.43 | | |
| 8 | **Meter Box Inlet** | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.5 | | | 0.14 | 705.96 | 0.02 | **711.59** | **715.90** | **4.31** |
| 9 | RCP (Headworks) | 1 | 53.4 | 48 | | | | 110 | 491 | 0.71 | 707.10 | 1.14 | 713.44 | | |
| 10 | 35 Degree Bend | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.4 | | | 0.11 | 707.10 | 0.00 | 713.55 | | |
| 11 | RCP (Headworks) | 1 | 53.4 | 48 | | | | 110 | 129 | 0.19 | 707.40 | 0.30 | 714.04 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 53.4 | 48 | 4.2 | 0.28 | 1 | | | 0.28 | 707.40 | 0.00 | 714.32 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 4.4 | 48 | 0.4 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **714.32** | **716.10** | **1.78** |
| 14 | RCP (Acorn) | 1 | 4.4 | 36 | | | | 110 | 148 | 0.01 | 711.31 | 3.91 | 718.24 | | |
| 15 | RCP (Acorn) | 1 | 4.4 | 27 | | | | 110 | 180 | 0.04 | 716.30 | 8.90 | 723.26 | | |
| 16 | Acorn Manhole #1 Exit | 1 | 4.4 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 716.30 | 0.00 | 723.28 | | |
| 17 | **Acorn Manhole #1 Inlet** | 1 | 4.4 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 716.30 | 0.00 | **723.29** | **728.30** | **5.01** |
| 18 | RCP (Acorn) | 1 | 4.4 | 27 | | | | 110 | 339 | 0.08 | 735.49 | 19.19 | 742.56 | | |
| 19 | Acorn Manhole #2 Exit | 1 | 4.4 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 735.49 | 0.00 | 742.58 | | |
| 20 | **Acorn Manhole #2 Inlet** | 1 | 4.4 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 735.49 | 0.00 | **742.59** | **747.50** | **4.91** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 31.7 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined at Ultimate Future Design* | | *53.4* | | | | | | | | | | | | |
| | *Arlanza at Ultimate Future Design Flow - PDWF* | | *49.0* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 26.7 | 42 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.70 | | **710.51** | **713.70** | **3.19** |
| 4 | RCP (Headworks) | 1 | 53.4 | 48 | | | | 110 | 21 | 0.03 | 705.75 | 0.05 | 709.72 | | |
| 5 | 90 Degree Bend | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.9 | | | 0.25 | 705.75 | 0.00 | 709.97 | | |
| 6 | RCP (Headworks) | 1 | 53.4 | 48 | | | | 110 | 79 | 0.11 | 705.94 | 0.19 | 710.28 | | |
| 7 | Meter Box Outlet | 1 | 53.4 | 48 | 4.2 | 0.28 | 1 | | | 0.28 | 705.94 | 0.00 | 710.56 | | |
| 8 | **Meter Box Inlet** | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.5 | | | 0.14 | 705.96 | 0.02 | **710.72** | **716.00** | **5.28** |
| 9 | RCP (Headworks) | 1 | 53.4 | 48 | | | | 110 | 491 | 0.71 | 707.10 | 1.14 | 712.57 | | |
| 10 | 35 Degree Bend | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.4 | | | 0.11 | 707.10 | 0.00 | 712.68 | | |
| 11 | RCP (Headworks) | 1 | 53.4 | 48 | | | | 110 | 129 | 0.19 | 707.40 | 0.30 | 713.17 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 53.4 | 48 | 4.2 | 0.28 | 1 | | | 0.28 | 707.40 | 0.00 | 713.45 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 26.7 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.40 | 0.00 | **713.49** | **716.10** | **2.61** |
| 14 | RCP (Arlanza) | 2 | 24.5 | 48 | | | | 110 | 293 | 0.10 | 708.00 | 0.60 | 714.19 | | |
| 15 | Arlanza Junction Box Exit | 2 | 24.5 | 48 | 1.9 | 0.06 | 1 | | | 0.06 | 708.00 | 0.00 | 714.25 | | |
| 16 | **Inlet - Arlanza Junction Box #1** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 708.00 | 0.00 | **714.34** | **717.50** | **3.16** |
| 17 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 714.36 | | |
| 18 | 30 Degree Bend | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.4 | | | 0.07 | 708.02 | 0.00 | 714.43 | | |
| 19 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 17 | 0.02 | 708.08 | 0.06 | 714.51 | | |
| 20 | 20 Degree Bend | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.25 | | | 0.05 | 708.08 | 0.00 | 714.56 | | |
| 21 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 86 | 0.08 | 708.39 | 0.31 | 714.95 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 708.39 | 0.00 | 715.13 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 708.39 | 0.00 | **715.22** | **724.80** | **9.58** |
| 24 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 161 | 0.15 | 708.97 | 0.58 | 715.95 | | |
| 25 | 50 Degree Bend | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.7 | | | 0.13 | 708.97 | 0.00 | 716.08 | | |
| 26 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 24 | 0.02 | 709.05 | 0.09 | 716.19 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 709.05 | 0.00 | 716.37 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 709.05 | 0.00 | **716.47** | **727.50** | **11.03** |
| 29 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 150 | 0.14 | 709.59 | 0.54 | 717.14 | | |
| 30 | Exit - Junction Box | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 709.59 | 0.00 | 717.33 | | |
| 31 | **Inlet - Arlanza Junction Box #2** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 709.59 | 0.00 | **717.42** | **725.00** | **7.58** |
| 32 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 303 | 0.28 | 710.68 | 1.09 | 718.79 | | |
| 33 | 20 Degree Bend | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.25 | | | 0.05 | 710.68 | 0.00 | 718.83 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 31.7 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 495 | 0.46 | 712.46 | 1.78 | 721.07 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 712.46 | 0.00 | 721.25 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 712.46 | 0.00 | **721.35** | **724.00** | **2.65** |
| 37 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 428 | 0.39 | 713.32 | 0.86 | 722.60 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 713.32 | 0.00 | 722.78 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 713.32 | 0.00 | **722.87** | **726.30** | **3.43** |
| 40 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 428 | 0.39 | 714.06 | 0.74 | 724.01 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 714.06 | 0.00 | 724.20 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 714.06 | 0.00 | **724.29** | **725.60** | **1.31** |
| 40 | RCP (Arlanza) | 1 | 49.0 | 51 | 3.5 | 0.18 | | 110 | 246 | 0.23 | 718.46 | 4.40 | 728.92 | | |
| 41 | Arlanza Manhole #6 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 718.46 | 0.00 | 729.10 | | |
| 42 | **Arlanza Manhole #6 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 718.46 | 0.00 | **729.19** | **730.00** | **0.81** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box #1 invert elevation and manhole elevation were found on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 32.9 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) at Ultimate Future Design Flow - PDWF* | | *62.9* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) at Ultimate Future Design Flow - PDWF* | | *50.9* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 31.5 | 42 | 3.3 | 0.17 | 0.5 | | | 0.08 | 705.70 | | **710.53** | **713.70** | **3.17** |
| 4 | RCP (Headworks) | 2 | 31.5 | 42 | | | | 125 | 28 | 0.02 | 705.87 | 0.17 | 710.73 | | |
| 5 | 35 Degree Bend | 2 | 31.5 | 42 | 3.3 | 0.17 | 0.4 | | | 0.07 | 705.87 | 0.00 | 710.79 | | |
| 6 | RCP (Headworks) | 2 | 31.5 | 42 | | | | 125 | 200 | 0.17 | 1285.53 | 579.66 | 1290.61 | | |
| 7 | 35 Degree Bend | 2 | 31.5 | 42 | 3.3 | 0.17 | 0.4 | | | 0.07 | 1285.53 | 0.00 | 1290.68 | | |
| 8 | RCP (Headworks) | 2 | 31.5 | 42 | | | | 125 | 16 | 0.01 | 707.18 | -578.35 | 712.35 | | |
| 9 | Exit from Junction | 2 | 31.5 | 42 | 3.3 | 0.17 | 1 | | | 0.17 | 707.18 | 0.00 | 712.51 | | |
| 10 | **Inlet to Junction** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 707.18 | 0.00 | **712.64** | **717.00** | **4.36** |
| 11 | RCP (Headworks) | 1 | 50.9 | 48 | | | | 125 | 10 | 0.01 | 707.30 | 0.12 | 712.77 | | |
| 12 | CCFRPM Pipe (Riverside/Hillside) | 1 | 50.9 | 48 | | | | 155 | 35 | 0.02 | 707.70 | 0.40 | 713.20 | | |
| 13 | Manhole #1 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 707.70 | 0.00 | 713.45 | | |
| 14 | **Manhole #1 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 707.70 | 0.00 | **713.58** | **716.20** | **2.62** |
| 15 | CCFRPM Pipe (Riverside/Hillside) | 1 | 50.9 | 48 | | | | 155 | 174 | 0.12 | 708.06 | 0.36 | 714.06 | | |
| 16 | Manhole #2 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 708.06 | 0.00 | 714.31 | | |
| 17 | **Manhole #2 Inlet** | 1 | 50.9 | 41 | 5.6 | 0.48 | 0.5 | | | 0.24 | 708.06 | 0.00 | **714.55** | **714.60** | **0.05** |
| 18 | CCFRPM Pipe (Riverside/Hillside) | 1 | 44.3 | 41 | | | | 155 | 13 | 0.02 | 708.10 | 0.04 | 714.61 | | |
| 19 | Manhole #3 Exit | 1 | 44.3 | 41 | 4.8 | 0.36 | 1 | | | 0.36 | 708.10 | 0.00 | 714.97 | | |
| 20 | **Manhole #3 Inlet** | 1 | 44.3 | 41 | 4.8 | 0.36 | 0.5 | | | 0.18 | 708.10 | 0.00 | **715.15** | **714.90** | **-0.25** |
| 21 | CCFRPM Pipe (Riverside/Hillside) | 1 | 44.3 | 41 | | | | 155 | 95 | 0.11 | 708.26 | 0.16 | 715.43 | | |
| 22 | Manhole #4 Exit | 1 | 44.3 | 41 | 4.8 | 0.36 | 1 | | | 0.36 | 708.26 | 0.00 | 715.79 | | |
| 23 | **Manhole #4 Inlet** | 1 | 44.3 | 41 | 4.8 | 0.36 | 0.5 | | | 0.18 | 708.26 | 0.00 | **715.97** | **715.50** | **-0.47** |
| 24 | CCFRPM Pipe (Riverside/Hillside) | 1 | 44.3 | 41 | | | | 155 | 13 | 0.02 | 708.29 | 0.03 | 716.02 | | |
| 25 | Manhole #5 Exit | 1 | 44.3 | 41 | 4.8 | 0.36 | 1 | | | 0.36 | 708.29 | 0.00 | 716.38 | | |
| 26 | **Manhole #5 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 708.29 | 0.00 | **716.51** | **716.30** | **-0.21** |
| 27 | CCFRPM Pipe (Riverside/Hillside) | 1 | 50.9 | 48 | | | | 155 | 78 | 0.05 | 708.43 | 0.14 | 716.70 | | |
| 28 | Manhole #6 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 708.43 | 0.00 | 716.96 | | |
| 29 | **Manhole #6 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 708.43 | 0.00 | **717.08** | **719.40** | **2.32** |
| 30 | CCFRPM Pipe (Riverside/Hillside) | 1 | 50.9 | 48 | | | | 155 | 55 | 0.04 | 708.53 | 0.10 | 717.22 | | |
| 31 | Manhole #7 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 708.53 | 0.00 | 717.48 | | |
| 32 | **Manhole #7 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 708.53 | 0.00 | **717.60** | **719.40** | **1.80** |
| 33 | CCFRPM Pipe (Riverside/Hillside) | 1 | 50.9 | 48 | | | | 155 | 215.5 | 0.15 | 708.92 | 0.39 | 718.15 | | |
| 34 | Manhole #8 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 708.92 | 0.00 | 718.40 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 32.9 MGD
**Checked by:** BCP Date: 9/29/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 35 | **Manhole #8 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 708.92 | 0.00 | **718.53** | **722.20** | **3.67** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Ultimate Future Design Flow - PWWF

**Calculated by:** NTC Date: 9/26/2014 **Rubidoux Flow Rate =** 6.7 MGD
**Checked by:** BCP Date: 9/29/2014

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Acorn Flow Rate =** 4.2 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined at Ultimate Future Design Flow - PWWF* | | *78.3* | | | | | | | | | | | | |
| | *Acorn at Ultimate Future Design Flow - PWWF* | | *6.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 2 | 39.2 | 42 | 4.1 | 0.26 | 0.5 | | | 0.13 | 705.70 | | **710.58** | **713.70** | **3.12** |
| 4 | RCP (Headworks) | 1 | 78.3 | 48 | | | | 110 | 21 | 0.06 | 705.75 | 0.05 | 710.69 | | |
| 5 | 90 Degree Bend | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.9 | | | 0.54 | 705.75 | 0.00 | 711.23 | | |
| 6 | RCP (Headworks) | 1 | 78.3 | 48 | | | | 110 | 79 | 0.23 | 705.94 | 0.19 | 711.66 | | |
| 7 | Meter Box Outlet | 1 | 78.3 | 48 | 6.2 | 0.60 | 1 | | | 0.60 | 705.94 | 0.00 | 712.26 | | |
| 8 | **Meter Box Inlet** | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.5 | | | 0.30 | 705.96 | 0.02 | **712.58** | **715.90** | **3.32** |
| 9 | RCP (Headworks) | 1 | 78.3 | 48 | | | | 110 | 491 | 1.45 | 707.10 | 1.14 | 715.17 | | |
| 10 | 35 Degree Bend | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.4 | | | 0.24 | 707.10 | 0.00 | 715.41 | | |
| 11 | RCP (Headworks) | 1 | 78.3 | 48 | | | | 110 | 129 | 0.38 | 707.40 | 0.30 | 716.09 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 78.3 | 48 | 6.2 | 0.60 | 1 | | | 0.60 | 707.40 | 0.00 | 716.69 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 6.5 | 48 | 0.5 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **716.70** | **716.10** | **-0.60** |
| 14 | RCP (Acorn) | 1 | 6.5 | 36 | | | | 110 | 148 | 0.02 | 711.31 | 3.91 | 720.62 | | |
| 15 | RCP (Acorn) | 1 | 6.5 | 27 | | | | 110 | 180 | 0.09 | 716.30 | 8.90 | 725.68 | | |
| 16 | Acorn Manhole #1 Exit | 1 | 6.5 | 27 | 1.6 | 0.04 | 1 | | | 0.04 | 716.30 | 0.00 | 725.72 | | |
| 17 | **Acorn Manhole #1 Inlet** | 1 | 6.5 | 27 | 1.6 | 0.04 | 0.5 | | | 0.02 | 716.30 | 0.00 | **725.75** | **728.30** | **2.55** |
| 18 | RCP (Acorn) | 1 | 6.5 | 27 | | | | 110 | 339 | 0.16 | 735.49 | 19.19 | 745.10 | | |
| 19 | Acorn Manhole #2 Exit | 1 | 6.5 | 27 | 1.6 | 0.04 | 1 | | | 0.04 | 735.49 | 0.00 | 745.14 | | |
| 20 | **Acorn Manhole #2 Inlet** | 1 | 6.5 | 27 | 1.6 | 0.04 | 0.5 | | | 0.02 | 735.49 | 0.00 | **745.16** | **747.50** | **2.34** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 46.4 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined at Ultimate Future Design* | | *78.3* | | | | | | | | | | | | |
| | *Arlanza at Ultimate Future Design Flow - PWWF* | | *71.8* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 39.2 | 42 | 4.1 | 0.26 | 0.5 | | | 0.13 | 705.70 | | **710.58** | **713.70** | **3.12** |
| 4 | RCP (Headworks) | 1 | 78.3 | 48 | | | | 110 | 21 | 0.06 | 705.75 | 0.05 | 709.75 | | |
| 5 | 90 Degree Bend | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.9 | | | 0.54 | 705.75 | 0.00 | 710.30 | | |
| 6 | RCP (Headworks) | 1 | 78.3 | 48 | | | | 110 | 79 | 0.23 | 705.94 | 0.19 | 710.72 | | |
| 7 | Meter Box Outlet | 1 | 78.3 | 48 | 6.2 | 0.60 | 1 | | | 0.60 | 705.94 | 0.00 | 711.32 | | |
| 8 | **Meter Box Inlet** | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.5 | | | 0.30 | 705.96 | 0.02 | **711.65** | **716.00** | **4.35** |
| 9 | RCP (Headworks) | 1 | 78.3 | 48 | | | | 110 | 491 | 1.45 | 707.10 | 1.14 | 714.23 | | |
| 10 | 35 Degree Bend | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.4 | | | 0.24 | 707.10 | 0.00 | 714.47 | | |
| 11 | RCP (Headworks) | 1 | 78.3 | 48 | | | | 110 | 129 | 0.38 | 707.40 | 0.30 | 715.15 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 78.3 | 48 | 6.2 | 0.60 | 1 | | | 0.60 | 707.40 | 0.00 | 715.75 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 39.2 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 707.40 | 0.00 | **715.83** | **716.10** | **0.27** |
| 14 | RCP (Arlanza) | 2 | 35.9 | 48 | | | | 110 | 293 | 0.20 | 708.00 | 0.60 | 716.63 | | |
| 15 | Arlanza Junction Box Exit | 2 | 35.9 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 708.00 | 0.00 | 716.76 | | |
| 16 | **Inlet - Arlanza Junction Box #1** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 708.00 | 0.00 | **716.96** | **717.50** | **0.54** |
| 17 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 5 | 0.01 | 708.02 | 0.02 | 716.99 | | |
| 18 | 30 Degree Bend | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.4 | | | 0.16 | 708.02 | 0.00 | 717.15 | | |
| 19 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 17 | 0.03 | 708.08 | 0.06 | 717.24 | | |
| 20 | 20 Degree Bend | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.25 | | | 0.10 | 708.08 | 0.00 | 717.34 | | |
| 21 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 86 | 0.16 | 708.39 | 0.31 | 717.81 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 708.39 | 0.00 | 718.21 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 708.39 | 0.00 | **718.41** | **724.80** | **6.39** |
| 24 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 161 | 0.30 | 708.97 | 0.58 | 719.29 | | |
| 25 | 50 Degree Bend | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.7 | | | 0.28 | 708.97 | 0.00 | 719.56 | | |
| 26 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 24 | 0.04 | 709.05 | 0.09 | 719.70 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 709.05 | 0.00 | 720.09 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 709.05 | 0.00 | **720.29** | **727.50** | **7.21** |
| 29 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 150 | 0.28 | 709.59 | 0.54 | 721.11 | | |
| 30 | Exit - Junction Box | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 709.59 | 0.00 | 721.51 | | |
| 31 | **Inlet - Arlanza Junction Box #2** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 709.59 | 0.00 | **721.71** | **725.00** | **3.29** |
| 32 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 303 | 0.57 | 710.68 | 1.09 | 723.36 | | |
| 33 | 20 Degree Bend | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.25 | | | 0.10 | 710.68 | 0.00 | 723.46 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Arlanza Flow Rate =** 46.4 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 495 | 0.93 | 712.46 | 1.78 | 726.17 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 712.46 | 0.00 | 726.57 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 712.46 | 0.00 | **726.76** | **724.00** | **-2.76** |
| 37 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 428 | 0.80 | 713.32 | 0.86 | 728.42 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 713.32 | 0.00 | 728.82 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 713.32 | 0.00 | **729.02** | **726.30** | **-2.72** |
| 40 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 428 | 0.80 | 714.06 | 0.74 | 730.56 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 714.06 | 0.00 | 730.96 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 714.06 | 0.00 | **731.16** | **725.60** | **-5.56** |
| 40 | RCP (Arlanza) | 1 | 71.8 | 51 | 5.1 | 0.40 | | 110 | 246 | 0.46 | 718.46 | 4.40 | 736.02 | | |
| 41 | Arlanza Manhole #6 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 718.46 | 0.00 | 736.42 | | |
| 42 | **Arlanza Manhole #6 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 718.46 | 0.00 | **736.62** | **730.00** | **-6.62** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box #1 invert elevation and manhole elevation were found on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 48.2 MGD
**Checked by:** BCP Date: 9/29/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) at Ultimate Future Design Flow - PWWF* | | *90.0* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) at Ultimate Future Design Flow - PWWF* | | *74.6* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 45.0 | 42 | 4.7 | 0.34 | 0.5 | | | 0.17 | 705.70 | | **710.62** | **713.70** | **3.08** |
| 4 | RCP (Headworks) | 2 | 45.0 | 42 | | | | 125 | 28 | 0.04 | 705.87 | 0.17 | 710.83 | | |
| 5 | 35 Degree Bend | 2 | 45.0 | 42 | 4.7 | 0.34 | 0.4 | | | 0.14 | 705.87 | 0.00 | 710.97 | | |
| 6 | RCP (Headworks) | 2 | 45.0 | 42 | | | | 125 | 200 | 0.32 | 1285.53 | 579.66 | 1290.95 | | |
| 7 | 35 Degree Bend | 2 | 45.0 | 42 | 4.7 | 0.34 | 0.4 | | | 0.14 | 1285.53 | 0.00 | 1291.08 | | |
| 8 | RCP (Headworks) | 2 | 45.0 | 42 | | | | 125 | 16 | 0.03 | 707.18 | -578.35 | 712.76 | | |
| 9 | Exit from Junction | 2 | 45.0 | 42 | 4.7 | 0.34 | 1 | | | 0.34 | 707.18 | 0.00 | 713.10 | | |
| 10 | **Inlet to Junction** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 707.18 | 0.00 | **713.38** | **717.00** | **3.62** |
| 11 | RCP (Headworks) | 1 | 74.6 | 48 | | | | 125 | 10 | 0.02 | 707.30 | 0.12 | 713.51 | | |
| 12 | CCFRPM Pipe (Riverside/Hillside) | 1 | 74.6 | 48 | | | | 155 | 35 | 0.05 | 707.70 | 0.40 | 713.97 | | |
| 13 | Manhole #1 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 707.70 | 0.00 | 714.51 | | |
| 14 | **Manhole #1 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 707.70 | 0.00 | **714.79** | **716.20** | **1.41** |
| 15 | CCFRPM Pipe (Riverside/Hillside) | 1 | 74.6 | 48 | | | | 155 | 174 | 0.25 | 708.06 | 0.36 | 715.40 | | |
| 16 | Manhole #2 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 708.06 | 0.00 | 715.94 | | |
| 17 | **Manhole #2 Inlet** | 1 | 74.6 | 41 | 8.1 | 1.03 | 0.5 | | | 0.51 | 708.06 | 0.00 | **716.46** | **714.60** | **-1.86** |
| 18 | CCFRPM Pipe (Riverside/Hillside) | 1 | 65.0 | 41 | | | | 155 | 13 | 0.03 | 708.10 | 0.04 | 716.53 | | |
| 19 | Manhole #3 Exit | 1 | 65.0 | 41 | 7.1 | 0.78 | 1 | | | 0.78 | 708.10 | 0.00 | 717.31 | | |
| 20 | **Manhole #3 Inlet** | 1 | 65.0 | 41 | 7.1 | 0.78 | 0.5 | | | 0.39 | 708.10 | 0.00 | **717.70** | **714.90** | **-2.80** |
| 21 | CCFRPM Pipe (Riverside/Hillside) | 1 | 65.0 | 41 | | | | 155 | 95 | 0.23 | 708.26 | 0.16 | 718.09 | | |
| 22 | Manhole #4 Exit | 1 | 65.0 | 41 | 7.1 | 0.78 | 1 | | | 0.78 | 708.26 | 0.00 | 718.87 | | |
| 23 | **Manhole #4 Inlet** | 1 | 65.0 | 41 | 7.1 | 0.78 | 0.5 | | | 0.39 | 708.26 | 0.00 | **719.26** | **715.50** | **-3.76** |
| 24 | CCFRPM Pipe (Riverside/Hillside) | 1 | 65.0 | 41 | | | | 155 | 13 | 0.03 | 708.29 | 0.03 | 719.32 | | |
| 25 | Manhole #5 Exit | 1 | 65.0 | 41 | 7.1 | 0.78 | 1 | | | 0.78 | 708.29 | 0.00 | 720.10 | | |
| 26 | **Manhole #5 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 708.29 | 0.00 | **720.38** | **716.30** | **-4.08** |
| 27 | CCFRPM Pipe (Riverside/Hillside) | 1 | 74.6 | 48 | | | | 155 | 78 | 0.11 | 708.43 | 0.14 | 720.63 | | |
| 28 | Manhole #6 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 708.43 | 0.00 | 721.18 | | |
| 29 | **Manhole #6 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 708.43 | 0.00 | **721.45** | **719.40** | **-2.05** |
| 30 | CCFRPM Pipe (Riverside/Hillside) | 1 | 74.6 | 48 | | | | 155 | 55 | 0.08 | 708.53 | 0.10 | 721.63 | | |
| 31 | Manhole #7 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 708.53 | 0.00 | 722.18 | | |
| 32 | **Manhole #7 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 708.53 | 0.00 | **722.45** | **719.40** | **-3.05** |
| 33 | CCFRPM Pipe (Riverside/Hillside) | 1 | 74.6 | 48 | | | | 155 | 215.5 | 0.31 | 708.92 | 0.39 | 723.15 | | |
| 34 | Manhole #8 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 708.92 | 0.00 | 723.70 | | |

URS

# EXISTING CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Flow Projection:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/26/2014 **Santa Ana Flow Rate =** 48.2 MGD
**Checked by:** BCP Date: 9/29/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 35 | **Manhole #8 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 708.92 | 0.00 | **723.97** | **722.20** | **-1.77** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Revised Present Design Flow - AADF

**Calculated by:** NTC Date: 9/29/2014 **Rubidoux Flow Rate =** 2.0 MGD
**Checked by:** BCP Date: 9/30/2014

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Revised Present Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 1.3 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined Flow* | | *24.5* | | | | | | | | | | | | |
| | *Acorn Flow* | | *2.0* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 1 | 24.5 | 48 | 2.0 | 0.06 | 0.5 | | | 0.03 | 705.70 | | **710.48** | **713.70** | **3.22** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 24.5 | 48 | | | | 110 | 21 | 0.01 | 705.75 | 0.05 | 710.54 | | |
| 5 | 90 Degree Bend | 1 | 24.5 | 48 | 2.0 | 0.06 | 0.9 | | | 0.05 | 705.75 | 0.00 | 710.59 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 24.5 | 48 | | | | 110 | 79 | 0.03 | 705.94 | 0.19 | 710.81 | | |
| 7 | Meter Box Outlet | 1 | 24.5 | 48 | 2.0 | 0.06 | 1 | | | 0.06 | 705.94 | 0.00 | 710.87 | | |
| 8 | **Meter Box Inlet** | 1 | 24.5 | 48 | 2.0 | 0.06 | 0.5 | | | 0.03 | 705.96 | 0.02 | **710.92** | **715.90** | **4.98** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 24.5 | 48 | | | | 110 | 491 | 0.17 | 707.10 | 1.14 | 712.23 | | |
| 10 | 35 Degree Bend | 1 | 24.5 | 48 | 2.0 | 0.06 | 0.4 | | | 0.02 | 707.10 | 0.00 | 712.25 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 24.5 | 48 | | | | 110 | 129 | 0.04 | 707.40 | 0.30 | 712.59 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 24.5 | 48 | 2.0 | 0.06 | 1 | | | 0.06 | 707.40 | 0.00 | 712.65 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 2.0 | 48 | 0.2 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **712.65** | **716.10** | **3.45** |
| 14 | RCP (Acorn) | 1 | 2.0 | 36 | | | | 110 | 109 | 0.00 | 711.29 | 3.89 | 716.55 | | |
| 15 | RCP (Acorn) | 1 | 2.0 | 27 | | | | 110 | 39 | 0.00 | 712.36 | 1.07 | 717.62 | | |
| 16 | New MH AC-3 Outlet | 1 | 2.0 | 27 | 0.5 | 0.00 | 1 | | | 0.00 | 712.36 | 0.00 | 717.63 | | |
| 17 | 45 Degree Bend | 1 | 2.0 | 24 | 0.6 | 0.01 | 0.42 | | | 0.00 | 712.36 | 0.00 | 717.63 | | |
| 18 | **New MH AC-3 Inlet** | 1 | 2.0 | 24 | 0.6 | 0.01 | 0.5 | | | 0.00 | 712.36 | 0.00 | **717.63** | **720.70** | **3.07** |
| 19 | New RCP (Acorn) | 1 | 2.0 | 24 | | | | 140 | 28 | 0.00 | 712.93 | 0.57 | 718.20 | | |
| 20 | 45 Degree Bend | 1 | 2.0 | 24 | 0.6 | 0.01 | 0.42 | | | 0.00 | 714.24 | 1.31 | 719.51 | | |
| 21 | Tee from Bypass Run | 1 | 2.0 | 24 | 0.6 | 0.01 | 0.6 | | | 0.00 | 714.24 | 0.00 | 719.52 | | |
| 22 | Outlet Gate | 1 | 2.0 | 16 | 1.5 | 0.03 | 0.19 | | | 0.01 | 714.24 | 0.00 | 719.52 | | |
| 23 | Pipe Bend | 1 | 2.0 | 16 | 1.5 | 0.03 | 0.2 | | | 0.01 | 712.74 | -1.50 | 718.03 | | |
| 24 | New CCFRPM (Acorn Meter Run) | 1 | 2.0 | 16 | | | | 155 | 23 | 0.01 | 712.74 | 0.00 | 718.04 | | |
| 25 | New WSP (Acorn Meter Run) | 1 | 2.0 | 16 | | | | 140 | 17 | 0.01 | 712.74 | 0.00 | 718.04 | | |
| 26 | Inlet Structure Outlet | 1 | 2.0 | 16 | 1.5 | 0.03 | 1 | | | 0.03 | 712.74 | 0.00 | 718.08 | | |
| 27 | Inlet Structure Losses | 1 | 2.0 | 16 | 1.5 | 0.03 | 0.5 | | | 0.02 | 714.24 | 1.50 | 719.59 | | |
| 28 | Inlet Gate | 1 | 2.0 | 16 | 1.5 | 0.03 | 0.19 | | | 0.01 | 714.24 | 0.00 | 719.60 | | |
| 29 | **Inlet Structure Inlet (Acorn)** | 1 | 2.0 | 24 | 0.6 | 0.01 | 0.5 | | | 0.00 | 714.74 | 0.50 | **720.10** | **724.00** | **3.90** |
| 30 | New RCP (Acorn) | 1 | 2.0 | 24 | | | | 140 | 12 | 0.00 | 714.98 | 0.24 | 720.34 | | |
| 31 | New MH AC-2 Outlet | 1 | 2.0 | 24 | 0.6 | 0.01 | 1 | | | 0.01 | 714.98 | 0.00 | 720.35 | | |
| 32 | 45 Degree Bend | 1 | 2.0 | 24 | 0.6 | 0.01 | 0.42 | | | 0.00 | 714.98 | 0.00 | 720.35 | | |
| 33 | **New MH AC-2 Inlet** | 1 | 2.0 | 24 | 0.6 | 0.01 | 0.5 | | | 0.00 | 714.98 | 0.00 | **720.36** | **725.90** | **5.54** |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Revised Present Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 1.3 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 34 | New RCP (Acorn) | 1 | 2.0 | 24 | | | | 140 | 31 | 0.00 | 715.60 | 0.62 | 720.98 | | |
| 35 | New MH AC-1 Outlet | 1 | 2.0 | 24 | 0.6 | 0.01 | 1 | | | 0.01 | 715.60 | 0.00 | 720.99 | | |
| 36 | 45 Degree Bend | 1 | 2.0 | 24 | 0.6 | 0.01 | 0.42 | | | 0.00 | 715.60 | 0.00 | 720.99 | | |
| 37 | **New MH AC-1 Inlet** | 1 | 2.0 | 27 | 0.5 | 0.00 | 0.5 | | | 0.00 | 715.60 | 0.00 | **720.99** | **726.60** | **5.61** |
| 38 | RCP (Acorn) | 1 | 2.0 | 27 | | | | 110 | 25 | 0.00 | 716.30 | 0.70 | 721.69 | | |
| 39 | Acorn Manhole #1 Outlet | 1 | 2.0 | 27 | 0.5 | 0.00 | 1 | | | 0.00 | 716.30 | 0.00 | 721.70 | | |
| 40 | **Acorn Manhole #1 Inlet** | 1 | 2.0 | 27 | 0.5 | 0.00 | 0.5 | | | 0.00 | 716.30 | 0.00 | **721.70** | **728.30** | **6.60** |
| 41 | RCP (Acorn) | 1 | 2.0 | 27 | | | | 110 | 339 | 0.02 | 735.49 | 19.19 | 740.91 | | |
| 42 | Acorn Manhole #2 Exit | 1 | 2.0 | 27 | 0.5 | 0.00 | 1 | | | 0.00 | 735.49 | 0.00 | 740.91 | | |
| 43 | **Acorn Manhole #2 Inlet** | 1 | 2.0 | 27 | 0.5 | 0.00 | 0.5 | | | 0.00 | 735.49 | 0.00 | **740.91** | **747.50** | **6.59** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

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# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 14.5 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined Flow* | | *24.5* | | | | | | | | | | | | |
| | *Arlanza Flow* | | *22.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 1 | 24.5 | 48 | 2.0 | 0.06 | 0.5 | | | 0.03 | 705.70 | | **710.48** | **713.70** | **3.22** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 24.5 | 48 | | | | 110 | 21 | 0.01 | 705.75 | 0.05 | 709.70 | | |
| 5 | 90 Degree Bend | 1 | 24.5 | 48 | 2.0 | 0.06 | 0.9 | | | 0.05 | 705.75 | 0.00 | 709.75 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 24.5 | 48 | | | | 110 | 79 | 0.03 | 705.94 | 0.19 | 709.97 | | |
| 7 | Meter Box Outlet | 1 | 24.5 | 48 | 2.0 | 0.06 | 1 | | | 0.06 | 705.94 | 0.00 | 710.03 | | |
| 8 | **Meter Box Inlet** | 1 | 24.5 | 48 | 2.0 | 0.06 | 0.5 | | | 0.03 | 705.96 | 0.02 | **710.08** | **716.00** | **5.92** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 24.5 | 48 | | | | 110 | 491 | 0.17 | 707.10 | 1.14 | 711.39 | | |
| 10 | 35 Degree Bend | 1 | 24.5 | 48 | 2.0 | 0.06 | 0.4 | | | 0.02 | 707.10 | 0.00 | 711.41 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 24.5 | 48 | | | | 110 | 129 | 0.04 | 707.40 | 0.30 | 711.75 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 24.5 | 48 | 2.0 | 0.06 | 1 | | | 0.06 | 707.40 | 0.00 | 711.81 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 12.3 | 48 | 1.0 | 0.01 | 0.5 | | | 0.01 | 707.40 | 0.00 | **711.82** | **716.10** | **4.28** |
| 14 | Reinforced Concrete Pipe (RCP, Arlanza) | 2 | 11.2 | 48 | | | | 110 | 293 | 0.02 | 708.00 | 0.60 | 712.44 | | |
| 15 | Arlanza Junction Box Exit | 2 | 11.2 | 48 | 0.9 | 0.01 | 1 | | | 0.01 | 708.00 | 0.00 | 712.46 | | |
| 16 | **Arlanza Junction Box Inlet** | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.5 | | | 0.02 | 708.00 | 0.00 | **712.48** | **717.50** | **5.02** |
| 17 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 712.50 | | |
| 18 | 30 Degree Bend | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.4 | | | 0.02 | 708.02 | 0.00 | 712.51 | | |
| 19 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 17 | 0.00 | 708.08 | 0.06 | 712.58 | | |
| 20 | 20 Degree Bend | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.25 | | | 0.01 | 708.08 | 0.00 | 712.59 | | |
| 21 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 86 | 0.02 | 708.39 | 0.31 | 712.91 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 22.5 | 51 | 1.6 | 0.04 | 1 | | | 0.04 | 708.39 | 0.00 | 712.95 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.5 | | | 0.02 | 708.39 | 0.00 | **712.97** | **724.80** | **11.83** |
| 24 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 161 | 0.04 | 708.97 | 0.58 | 713.59 | | |
| 25 | 50 Degree Bend | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.7 | | | 0.03 | 708.97 | 0.00 | 713.61 | | |
| 26 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 24 | 0.01 | 709.05 | 0.09 | 713.70 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 22.5 | 51 | 1.6 | 0.04 | 1 | | | 0.04 | 709.05 | 0.00 | 713.74 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.5 | | | 0.02 | 709.05 | 0.00 | **713.76** | **727.50** | **13.74** |
| 29 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 150 | 0.03 | 709.59 | 0.54 | 714.33 | | |
| 30 | Exit - Junction Box | 1 | 22.5 | 51 | 1.6 | 0.04 | 1 | | | 0.04 | 709.59 | 0.00 | 714.37 | | |
| 31 | **Inlet - Junction Box** | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.5 | | | 0.02 | 709.59 | 0.00 | **714.39** | **725.00** | **10.61** |
| 32 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 303 | 0.07 | 710.68 | 1.09 | 715.55 | | |
| 33 | 20 Degree Bend | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.25 | | | 0.01 | 710.68 | 0.00 | 715.56 | | |

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# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 14.5 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 495 | 0.11 | 712.46 | 1.78 | 717.45 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 22.5 | 51 | 1.6 | 0.04 | 1 | | | 0.04 | 712.46 | 0.00 | 717.48 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.5 | | | 0.02 | 712.46 | 0.00 | **717.50** | **724.00** | **6.50** |
| 37 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 428 | 0.09 | 713.32 | 0.86 | 718.45 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 22.5 | 51 | 1.6 | 0.04 | 1 | | | 0.04 | 713.32 | 0.00 | 718.49 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.5 | | | 0.02 | 713.32 | 0.00 | **718.51** | **726.30** | **7.79** |
| 40 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 22.5 | 51 | | | | 110 | 428 | 0.09 | 714.06 | 0.74 | 719.35 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 22.5 | 51 | 1.6 | 0.04 | 1 | | | 0.04 | 714.06 | 0.00 | 719.39 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.5 | | | 0.02 | 714.06 | 0.00 | **719.41** | **725.60** | **6.19** |
| 43 | RCP (Arlanza) | 1 | 22.5 | 51 | | | | 110 | 21 | 0.00 | 715.67 | 1.61 | 721.02 | | |
| 44 | Tee from future Jurupa Run | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.6 | | | 0.02 | 715.67 | 0.00 | 721.04 | | |
| 45 | Tee from Bypass Run | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.6 | | | 0.02 | 715.67 | 0.00 | 721.07 | | |
| 46 | Outlet Gate | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.19 | | | 0.01 | 715.67 | 0.00 | 721.07 | | |
| 47 | Pipe Bend | 1 | 22.5 | 36 | 3.2 | 0.16 | 0.2 | | | 0.03 | 712.67 | -3.00 | 718.11 | | |
| 48 | New CCFRPM (Arlanza Meter Run) | 1 | 22.5 | 36 | | | | 155 | 45 | 0.03 | 712.67 | 0.00 | 718.13 | | |
| 49 | New WSP (Arlanza Meter Run) | 1 | 22.5 | 36 | | | | 140 | 16 | 0.01 | 712.67 | 0.00 | 718.15 | | |
| 50 | Inlet Structure Outlet | 1 | 22.5 | 36 | 3.2 | 0.16 | 1 | | | 0.16 | 712.67 | 0.00 | 718.30 | | |
| 51 | Inlet Structure Losses | 1 | 22.5 | 48 | 1.8 | 0.05 | 0.5 | | | 0.02 | 715.67 | 3.00 | 721.33 | | |
| 52 | Inlet Gate | 1 | 22.5 | 48 | 1.8 | 0.05 | 0.19 | | | 0.01 | 715.67 | 0.00 | 721.34 | | |
| 53 | **Inlet Structure Inlet (Arlanza)** | 1 | 22.5 | 48 | 1.8 | 0.05 | 0.5 | | | 0.02 | 715.67 | 0.00 | **721.36** | **731.00** | **9.64** |
| 54 | RCP (Arlanza) | 1 | 22.5 | 51 | | | | 110 | 156 | 0.03 | 718.46 | 2.79 | 724.19 | | |
| 55 | Arlanza Manhole #6 Outlet | 1 | 22.5 | 51 | 1.6 | 0.04 | 1 | | | 0.04 | 718.46 | 0.00 | 724.23 | | |
| 56 | **Arlanza Manhole #6 Inlet** | 1 | 22.5 | 51 | 1.6 | 0.04 | 0.5 | | | 0.02 | 718.46 | 0.00 | **724.25** | **730.00** | **5.75** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box invert elevation and manhole elevation were determined on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

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# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Revised Present Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 11.6 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) Combined Flow* | | *28.4* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) Flow* | | *18.0* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 14.2 | 42 | 1.5 | 0.03 | 0.5 | | | 0.02 | 705.70 | | **710.47** | **713.70** | **3.23** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 14.2 | 42 | | | | 125 | 28 | 0.01 | 705.87 | 0.17 | 710.64 | | |
| 5 | 35 Degree Bend | 2 | 14.2 | 42 | 1.5 | 0.03 | 0.4 | | | 0.01 | 705.87 | 0.00 | 710.66 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 14.2 | 42 | | | | 125 | 200 | 0.04 | 707.08 | 1.21 | 711.91 | | |
| 7 | 35 Degree Bend | 2 | 14.2 | 42 | 1.5 | 0.03 | 0.4 | | | 0.01 | 707.08 | 0.00 | 711.92 | | |
| 8 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 14.2 | 42 | | | | 125 | 16 | 0.00 | 707.18 | 0.10 | 712.02 | | |
| 9 | Exit from Junction | 2 | 14.2 | 42 | 1.5 | 0.03 | 1 | | | 0.03 | 707.18 | 0.00 | 712.05 | | |
| 10 | **Inlet to Junction** | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 707.18 | 0.00 | **712.07** | **717.00** | **4.93** |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 18.0 | 48 | | | | 125 | 10 | 0.00 | 707.30 | 0.12 | 712.19 | | |
| 12 | CCFRPM Pipe (Santa Ana) | 1 | 18.0 | 48 | | | | 155 | 35 | 0.00 | 707.70 | 0.40 | 712.60 | | |
| 13 | Manhole #1 Exit | 1 | 18.0 | 48 | 1.4 | 0.03 | 1 | | | 0.03 | 707.70 | 0.00 | 712.63 | | |
| 14 | **Manhole #1 Inlet** | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 707.70 | 0.00 | **712.64** | **716.20** | **3.56** |
| 15 | New CCFRPM (Santa Ana) | 1 | 18.0 | 36 | | | | 155 | 12 | 0.00 | 707.99 | 0.29 | 712.94 | | |
| 16 | Tee from Bypass Run | 1 | 18.0 | 36 | 2.5 | 0.10 | 0.6 | | | 0.06 | 707.99 | 0.00 | 713.00 | | |
| 17 | Outlet Gate | 1 | 18.0 | 36 | 2.5 | 0.10 | 0.19 | | | 0.02 | 707.99 | 0.00 | 713.02 | | |
| 18 | Pipe Bend, vertical | 1 | 18.0 | 36 | 2.5 | 0.10 | 0.2 | | | 0.02 | 704.99 | -3.00 | 710.04 | | |
| 19 | New CCFRPM (Santa Ana Meter Run) | 1 | 18.0 | 36 | | | | 155 | 71 | 0.03 | 704.99 | 0.00 | 710.07 | | |
| 20 | New WSP (Santa Ana Meter Run) | 1 | 18.0 | 36 | | | | 140 | 38 | 0.02 | 704.99 | 0.00 | 710.09 | | |
| 21 | Inlet Structure Outlet | 1 | 18.0 | 36 | 2.5 | 0.10 | 1 | | | 0.10 | 704.99 | 0.00 | 710.19 | | |
| 22 | Inlet Structure Losses | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 707.99 | 3.00 | 713.21 | | |
| 23 | Inlet Gate | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.19 | | | 0.01 | 707.99 | 0.00 | 713.21 | | |
| 24 | **Inlet Structure Inlet (Santa Ana)** | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 707.99 | 0.00 | **713.23** | **717.80** | **4.57** |
| 25 | CCFRPM Pipe (Santa Ana) | 1 | 18.0 | 48 | 1.4 | 0.03 | | 155 | 31 | 0.00 | 708.06 | 0.07 | 713.30 | | |
| 26 | Manhole #2 Exit | 1 | 18.0 | 48 | 1.4 | 0.03 | 1 | | | 0.03 | 708.06 | 0.00 | 713.33 | | |
| 27 | **Manhole #2 Inlet** | 1 | 18.0 | 41 | 2.0 | 0.06 | 0.5 | | | 0.03 | 708.06 | 0.00 | **713.36** | **714.60** | **1.24** |
| 28 | CCFRPM Pipe (Santa Ana) | 1 | 15.7 | 41 | | | | 155 | 13 | 0.00 | 708.10 | 0.04 | 713.40 | | |
| 29 | Manhole #3 Exit | 1 | 15.7 | 41 | 1.7 | 0.05 | 1 | | | 0.05 | 708.10 | 0.00 | 713.45 | | |
| 30 | **Manhole #3 Inlet** | 1 | 15.7 | 41 | 1.7 | 0.05 | 0.5 | | | 0.02 | 708.10 | 0.00 | **713.47** | **714.90** | **1.43** |
| 31 | CCFRPM Pipe (Santa Ana) | 1 | 15.7 | 41 | | | | 155 | 95 | 0.02 | 708.26 | 0.16 | 713.64 | | |
| 32 | Manhole #4 Exit | 1 | 15.7 | 41 | 1.7 | 0.05 | 1 | | | 0.05 | 708.26 | 0.00 | 713.69 | | |
| 33 | **Manhole #4 Inlet** | 1 | 15.7 | 41 | 1.7 | 0.05 | 0.5 | | | 0.02 | 708.26 | 0.00 | **713.71** | **715.50** | **1.79** |
| 34 | CCFRPM Pipe (Santa Ana) | 1 | 15.7 | 41 | | | | 155 | 13 | 0.00 | 708.29 | 0.03 | 713.74 | | |
| 35 | Manhole #5 Exit | 1 | 15.7 | 41 | 1.7 | 0.05 | 1 | | | 0.05 | 708.29 | 0.00 | 713.79 | | |

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# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Revised Present Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 11.6 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36 | **Manhole #5 Inlet** | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 708.29 | 0.00 | **713.81** | **716.30** | **2.49** |
| 37 | CCFRPM Pipe (Santa Ana) | 1 | 18.0 | 48 | | | | 155 | 78 | 0.01 | 708.43 | 0.14 | 713.95 | | |
| 38 | Manhole #6 Exit | 1 | 18.0 | 48 | 1.4 | 0.03 | 1 | | | 0.03 | 708.43 | 0.00 | 713.99 | | |
| 39 | **Manhole #6 Inlet** | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 708.43 | 0.00 | **714.00** | **719.40** | **5.40** |
| 40 | CCFRPM Pipe (Santa Ana) | 1 | 18.0 | 48 | | | | 155 | 55 | 0.01 | 708.53 | 0.10 | 714.11 | | |
| 41 | Manhole #7 Exit | 1 | 18.0 | 48 | 1.4 | 0.03 | 1 | | | 0.03 | 708.53 | 0.00 | 714.14 | | |
| 42 | **Manhole #7 Inlet** | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 708.53 | 0.00 | **714.15** | **719.40** | **5.25** |
| 43 | CCFRPM Pipe (Santa Ana) | 1 | 18.0 | 48 | | | | 155 | 216 | 0.02 | 708.92 | 0.39 | 714.57 | | |
| 44 | Manhole #8 Exit | 1 | 18.0 | 48 | 1.4 | 0.03 | 1 | | | 0.03 | 708.92 | 0.00 | 714.60 | | |
| 45 | **Manhole #8 Inlet** | 1 | 18.0 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 708.92 | 0.00 | **714.61** | **722.20** | **7.59** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Rubidoux Flow Rate =** 3.0 MGD
**Checked by:** BCP Date: 9/30/2014

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 2.0 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined Flow* | | *36.8* | | | | | | | | | | | | |
| | *Acorn Flow* | | *3.1* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.5 | | | 0.07 | 705.70 | | **710.52** | **713.70** | **3.18** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 36.8 | 48 | | | | 110 | 21 | 0.02 | 705.75 | 0.05 | 710.58 | | |
| 5 | 90 Degree Bend | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.9 | | | 0.12 | 705.75 | 0.00 | 710.70 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 36.8 | 48 | | | | 110 | 79 | 0.06 | 705.94 | 0.19 | 710.95 | | |
| 7 | Meter Box Outlet | 1 | 36.8 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 705.94 | 0.00 | 711.08 | | |
| 8 | **Meter Box Inlet** | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.5 | | | 0.07 | 705.96 | 0.02 | **711.17** | **715.90** | **4.73** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 36.8 | 48 | | | | 110 | 491 | 0.36 | 707.10 | 1.14 | 712.67 | | |
| 10 | 35 Degree Bend | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.4 | | | 0.05 | 707.10 | 0.00 | 712.72 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 36.8 | 48 | | | | 110 | 129 | 0.09 | 707.40 | 0.30 | 713.11 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 36.8 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 707.40 | 0.00 | 713.25 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 3.1 | 48 | 0.2 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **713.25** | **716.10** | **2.85** |
| 14 | RCP (Acorn) | 1 | 3.1 | 36 | | | | 110 | 109 | 0.00 | 711.29 | 3.89 | 717.15 | | |
| 15 | RCP (Acorn) | 1 | 3.1 | 27 | | | | 110 | 39 | 0.00 | 712.36 | 1.07 | 718.22 | | |
| 16 | New MH AC-3 Outlet | 1 | 3.1 | 27 | 0.8 | 0.01 | 1 | | | 0.01 | 712.36 | 0.00 | 718.23 | | |
| 17 | 45 Degree Bend | 1 | 3.1 | 24 | 1.0 | 0.01 | 0.42 | | | 0.01 | 712.36 | 0.00 | 718.24 | | |
| 18 | **New MH AC-3 Inlet** | 1 | 3.1 | 24 | 1.0 | 0.01 | 0.5 | | | 0.01 | 712.36 | 0.00 | **718.24** | **720.70** | **2.46** |
| 19 | New RCP (Acorn) | 1 | 3.1 | 24 | | | | 140 | 28 | 0.00 | 712.93 | 0.57 | 718.81 | | |
| 20 | 45 Degree Bend | 1 | 3.1 | 24 | 1.0 | 0.01 | 0.42 | | | 0.01 | 714.24 | 1.31 | 720.13 | | |
| 21 | Tee from Bypass Run | 1 | 3.1 | 24 | 1.0 | 0.01 | 0.6 | | | 0.01 | 714.24 | 0.00 | 720.14 | | |
| 22 | Outlet Gate | 1 | 3.1 | 16 | 2.2 | 0.07 | 0.19 | | | 0.01 | 714.24 | 0.00 | 720.15 | | |
| 23 | Pipe Bend | 1 | 3.1 | 16 | 2.2 | 0.07 | 0.2 | | | 0.01 | 712.74 | -1.50 | 718.67 | | |
| 24 | New CCFRPM (Acorn Meter Run) | 1 | 3.1 | 16 | | | | 155 | 23 | 0.02 | 712.74 | 0.00 | 718.69 | | |
| 25 | New WSP (Acorn Meter Run) | 1 | 3.1 | 16 | | | | 140 | 17 | 0.02 | 712.74 | 0.00 | 718.70 | | |
| 26 | Inlet Structure Outlet | 1 | 3.1 | 16 | 2.2 | 0.07 | 1 | | | 0.07 | 712.74 | 0.00 | 718.78 | | |
| 27 | Inlet Structure Losses | 1 | 3.1 | 16 | 2.2 | 0.07 | 0.5 | | | 0.04 | 714.24 | 1.50 | 720.31 | | |
| 28 | Inlet Gate | 1 | 3.1 | 16 | 2.2 | 0.07 | 0.19 | | | 0.01 | 714.24 | 0.00 | 720.33 | | |
| 29 | **Inlet Structure Inlet (Acorn)** | 1 | 3.1 | 24 | 1.0 | 0.01 | 0.5 | | | 0.01 | 714.74 | 0.50 | **720.84** | **724.00** | **3.16** |
| 30 | New RCP (Acorn) | 1 | 3.1 | 24 | | | | 140 | 12 | 0.00 | 714.98 | 0.24 | 721.08 | | |
| 31 | New MH AC-2 Outlet | 1 | 3.1 | 24 | 1.0 | 0.01 | 1 | | | 0.01 | 714.98 | 0.00 | 721.09 | | |
| 32 | 45 Degree Bend | 1 | 3.1 | 24 | 1.0 | 0.01 | 0.42 | | | 0.01 | 714.98 | 0.00 | 721.10 | | |
| 33 | **New MH AC-2 Inlet** | 1 | 3.1 | 24 | 1.0 | 0.01 | 0.5 | | | 0.01 | 714.98 | 0.00 | **721.10** | **725.90** | **4.80** |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 2.0 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 34 | New RCP (Acorn) | 1 | 3.1 | 24 | | | | 140 | 31 | 0.00 | 715.60 | 0.62 | 721.73 | | |
| 35 | New MH AC-1 Outlet | 1 | 3.1 | 24 | 1.0 | 0.01 | 1 | | | 0.01 | 715.60 | 0.00 | 721.74 | | |
| 36 | 45 Degree Bend | 1 | 3.1 | 24 | 1.0 | 0.01 | 0.42 | | | 0.01 | 715.60 | 0.00 | 721.75 | | |
| 37 | **New MH AC-1 Inlet** | 1 | 3.1 | 27 | 0.8 | 0.01 | 0.5 | | | 0.00 | 715.60 | 0.00 | **721.75** | **726.60** | **4.85** |
| 38 | RCP (Acorn) | 1 | 3.1 | 27 | | | | 110 | 25 | 0.00 | 716.30 | 0.70 | 722.46 | | |
| 39 | Acorn Manhole #1 Outlet | 1 | 3.1 | 27 | 0.8 | 0.01 | 1 | | | 0.01 | 716.30 | 0.00 | 722.47 | | |
| 40 | **Acorn Manhole #1 Inlet** | 1 | 3.1 | 27 | 0.8 | 0.01 | 0.5 | | | 0.00 | 716.30 | 0.00 | **722.47** | **728.30** | **5.83** |
| 41 | RCP (Acorn) | 1 | 3.1 | 27 | | | | 110 | 339 | 0.04 | 735.49 | 19.19 | 741.70 | | |
| 42 | Acorn Manhole #2 Exit | 1 | 3.1 | 27 | 0.8 | 0.01 | 1 | | | 0.01 | 735.49 | 0.00 | 741.71 | | |
| 43 | **Acorn Manhole #2 Inlet** | 1 | 3.1 | 27 | 0.8 | 0.01 | 0.5 | | | 0.00 | 735.49 | 0.00 | **741.72** | **747.50** | **5.78** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 21.8 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| | *Acorn/Arlanza Combined Flow* | | *36.8* | | | | | | | | | | | | |
| | *Arlanza Flow* | | *33.7* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.5 | | | 0.07 | 705.70 | | **710.52** | **713.70** | **3.18** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 36.8 | 48 | | | | 110 | 21 | 0.02 | 705.75 | 0.05 | 709.71 | | |
| 5 | 90 Degree Bend | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.9 | | | 0.12 | 705.75 | 0.00 | 709.83 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 36.8 | 48 | | | | 110 | 79 | 0.06 | 705.94 | 0.19 | 710.07 | | |
| 7 | Meter Box Outlet | 1 | 36.8 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 705.94 | 0.00 | 710.21 | | |
| 8 | **Meter Box Inlet** | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.5 | | | 0.07 | 705.96 | 0.02 | **710.30** | **716.00** | **5.70** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 36.8 | 48 | | | | 110 | 491 | 0.36 | 707.10 | 1.14 | 711.79 | | |
| 10 | 35 Degree Bend | 1 | 36.8 | 48 | 2.9 | 0.13 | 0.4 | | | 0.05 | 707.10 | 0.00 | 711.84 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 36.8 | 48 | | | | 110 | 129 | 0.09 | 707.40 | 0.30 | 712.24 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 36.8 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 707.40 | 0.00 | 712.37 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 18.4 | 48 | 1.5 | 0.03 | 0.5 | | | 0.02 | 707.40 | 0.00 | **712.39** | **716.10** | **3.71** |
| 14 | Reinforced Concrete Pipe (RCP, Arlanza) | 2 | 16.9 | 48 | | | | 110 | 293 | 0.05 | 708.00 | 0.60 | 713.04 | | |
| 15 | Arlanza Junction Box Exit | 2 | 16.9 | 48 | 1.3 | 0.03 | 1 | | | 0.03 | 708.00 | 0.00 | 713.07 | | |
| 16 | **Arlanza Junction Box Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 708.00 | 0.00 | **713.11** | **717.50** | **4.39** |
| 17 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 713.13 | | |
| 18 | 30 Degree Bend | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.4 | | | 0.04 | 708.02 | 0.00 | 713.17 | | |
| 19 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 17 | 0.01 | 708.08 | 0.06 | 713.23 | | |
| 20 | 20 Degree Bend | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.25 | | | 0.02 | 708.08 | 0.00 | 713.26 | | |
| 21 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 86 | 0.04 | 708.39 | 0.31 | 713.61 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 708.39 | 0.00 | 713.69 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 708.39 | 0.00 | **713.74** | **724.80** | **11.06** |
| 24 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 161 | 0.07 | 708.97 | 0.58 | 714.39 | | |
| 25 | 50 Degree Bend | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.7 | | | 0.06 | 708.97 | 0.00 | 714.45 | | |
| 26 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 24 | 0.01 | 709.05 | 0.09 | 714.55 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 709.05 | 0.00 | 714.64 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 709.05 | 0.00 | **714.68** | **727.50** | **12.82** |
| 29 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 150 | 0.07 | 709.59 | 0.54 | 715.29 | | |
| 30 | Exit - Junction Box | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 709.59 | 0.00 | 715.38 | | |
| 31 | **Inlet - Junction Box** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 709.59 | 0.00 | **715.42** | **725.00** | **9.58** |
| 32 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 303 | 0.14 | 710.68 | 1.09 | 716.65 | | |
| 33 | 20 Degree Bend | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.25 | | | 0.02 | 710.68 | 0.00 | 716.67 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 21.8 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 495 | 0.23 | 712.46 | 1.78 | 718.68 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 712.46 | 0.00 | 718.77 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 712.46 | 0.00 | **718.81** | **724.00** | **5.19** |
| 37 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 428 | 0.20 | 713.32 | 0.86 | 719.87 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 713.32 | 0.00 | 719.95 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 713.32 | 0.00 | **720.00** | **726.30** | **6.30** |
| 40 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 33.7 | 51 | | | | 110 | 428 | 0.20 | 714.06 | 0.74 | 720.94 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 714.06 | 0.00 | 721.03 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 714.06 | 0.00 | **721.07** | **725.60** | **4.53** |
| 43 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 21 | 0.01 | 715.67 | 1.61 | 722.69 | | |
| 44 | Tee from future Jurupa Run | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.6 | | | 0.05 | 715.67 | 0.00 | 722.74 | | |
| 45 | Tee from Bypass Run | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.6 | | | 0.05 | 715.67 | 0.00 | 722.79 | | |
| 46 | Outlet Gate | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.19 | | | 0.02 | 715.67 | 0.00 | 722.81 | | |
| 47 | Pipe Bend | 1 | 33.7 | 36 | 4.8 | 0.35 | 0.2 | | | 0.07 | 712.67 | -3.00 | 719.88 | | |
| 48 | New CCFRPM (Arlanza Meter Run) | 1 | 33.7 | 36 | | | | 155 | 45 | 0.06 | 712.67 | 0.00 | 719.94 | | |
| 49 | New WSP (Arlanza Meter Run) | 1 | 33.7 | 36 | | | | 140 | 16 | 0.03 | 712.67 | 0.00 | 719.97 | | |
| 50 | Inlet Structure Outlet | 1 | 33.7 | 36 | 4.8 | 0.35 | 1 | | | 0.35 | 712.67 | 0.00 | 720.32 | | |
| 51 | Inlet Structure Losses | 1 | 33.7 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 715.67 | 3.00 | 723.38 | | |
| 52 | Inlet Gate | 1 | 33.7 | 48 | 2.7 | 0.11 | 0.19 | | | 0.02 | 715.67 | 0.00 | 723.40 | | |
| 53 | **Inlet Structure Inlet (Arlanza)** | 1 | 33.7 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 715.67 | 0.00 | **723.46** | **731.00** | **7.54** |
| 54 | RCP (Arlanza) | 1 | 33.7 | 51 | | | | 110 | 156 | 0.07 | 718.46 | 2.79 | 726.32 | | |
| 55 | Arlanza Manhole #6 Outlet | 1 | 33.7 | 51 | 2.4 | 0.09 | 1 | | | 0.09 | 718.46 | 0.00 | 726.41 | | |
| 56 | **Arlanza Manhole #6 Inlet** | 1 | 33.7 | 51 | 2.4 | 0.09 | 0.5 | | | 0.04 | 718.46 | 0.00 | **726.45** | **730.00** | **3.55** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box invert elevation and manhole elevation were determined on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 17.4 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) Combined Flow* | | *39.0* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) Flow* | | *26.9* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 19.5 | 42 | 2.0 | 0.06 | 0.5 | | | 0.03 | 705.70 | | **710.48** | **713.70** | **3.22** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 19.5 | 42 | | | | 125 | 28 | 0.01 | 705.87 | 0.17 | 710.66 | | |
| 5 | 35 Degree Bend | 2 | 19.5 | 42 | 2.0 | 0.06 | 0.4 | | | 0.03 | 705.87 | 0.00 | 710.69 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 19.5 | 42 | | | | 125 | 200 | 0.07 | 707.08 | 1.21 | 711.97 | | |
| 7 | 35 Degree Bend | 2 | 19.5 | 42 | 2.0 | 0.06 | 0.4 | | | 0.03 | 707.08 | 0.00 | 711.99 | | |
| 8 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 19.5 | 42 | | | | 125 | 16 | 0.01 | 707.18 | 0.10 | 712.10 | | |
| 9 | Exit from Junction | 2 | 19.5 | 42 | 2.0 | 0.06 | 1 | | | 0.06 | 707.18 | 0.00 | 712.16 | | |
| 10 | **Inlet to Junction** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.18 | 0.00 | **712.20** | **717.00** | **4.80** |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 26.9 | 48 | | | | 125 | 10 | 0.00 | 707.30 | 0.12 | 712.31 | | |
| 12 | CCFRPM Pipe (Santa Ana) | 1 | 26.9 | 48 | | | | 155 | 35 | 0.01 | 707.70 | 0.40 | 712.73 | | |
| 13 | Manhole #1 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 707.70 | 0.00 | 712.80 | | |
| 14 | **Manhole #1 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.70 | 0.00 | **712.83** | **716.20** | **3.37** |
| 15 | New CCFRPM (Santa Ana) | 1 | 26.9 | 36 | | | | 155 | 12 | 0.01 | 707.99 | 0.29 | 713.14 | | |
| 16 | Tee from Bypass Run | 1 | 26.9 | 36 | 3.8 | 0.23 | 0.6 | | | 0.14 | 707.99 | 0.00 | 713.27 | | |
| 17 | Outlet Gate | 1 | 26.9 | 36 | 3.8 | 0.23 | 0.19 | | | 0.04 | 707.99 | 0.00 | 713.32 | | |
| 18 | Pipe Bend, vertical | 1 | 26.9 | 36 | 3.8 | 0.23 | 0.2 | | | 0.05 | 704.99 | -3.00 | 710.36 | | |
| 19 | New CCFRPM (Santa Ana Meter Run) | 1 | 26.9 | 36 | | | | 155 | 71 | 0.06 | 704.99 | 0.00 | 710.42 | | |
| 20 | New WSP (Santa Ana Meter Run) | 1 | 26.9 | 36 | | | | 140 | 38 | 0.04 | 704.99 | 0.00 | 710.47 | | |
| 21 | Inlet Structure Outlet | 1 | 26.9 | 36 | 3.8 | 0.23 | 1 | | | 0.23 | 704.99 | 0.00 | 710.69 | | |
| 22 | Inlet Structure Losses | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.99 | 3.00 | 713.73 | | |
| 23 | Inlet Gate | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.19 | | | 0.01 | 707.99 | 0.00 | 713.74 | | |
| 24 | **Inlet Structure Inlet (Santa Ana)** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.99 | 0.00 | **713.78** | **717.80** | **4.02** |
| 25 | CCFRPM Pipe (Santa Ana) | 1 | 26.9 | 48 | 2.1 | 0.07 | | 155 | 31 | 0.01 | 708.06 | 0.07 | 713.85 | | |
| 26 | Manhole #2 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 708.06 | 0.00 | 713.92 | | |
| 27 | **Manhole #2 Inlet** | 1 | 26.9 | 41 | 2.9 | 0.13 | 0.5 | | | 0.07 | 708.06 | 0.00 | **713.99** | **714.60** | **0.61** |
| 28 | CCFRPM Pipe (Santa Ana) | 1 | 23.5 | 41 | | | | 155 | 13 | 0.00 | 708.10 | 0.04 | 714.03 | | |
| 29 | Manhole #3 Exit | 1 | 23.5 | 41 | 2.6 | 0.10 | 1 | | | 0.10 | 708.10 | 0.00 | 714.13 | | |
| 30 | **Manhole #3 Inlet** | 1 | 23.5 | 41 | 2.6 | 0.10 | 0.5 | | | 0.05 | 708.10 | 0.00 | **714.18** | **714.90** | **0.72** |
| 31 | CCFRPM Pipe (Santa Ana) | 1 | 23.5 | 41 | | | | 155 | 95 | 0.03 | 708.26 | 0.16 | 714.38 | | |
| 32 | Manhole #4 Exit | 1 | 23.5 | 41 | 2.6 | 0.10 | 1 | | | 0.10 | 708.26 | 0.00 | 714.48 | | |
| 33 | **Manhole #4 Inlet** | 1 | 23.5 | 41 | 2.6 | 0.10 | 0.5 | | | 0.05 | 708.26 | 0.00 | **714.53** | **715.50** | **0.97** |
| 34 | CCFRPM Pipe (Santa Ana) | 1 | 23.5 | 41 | | | | 155 | 13 | 0.00 | 708.29 | 0.03 | 714.57 | | |
| 35 | Manhole #5 Exit | 1 | 23.5 | 41 | 2.6 | 0.10 | 1 | | | 0.10 | 708.29 | 0.00 | 714.67 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Revised Present Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 17.4 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36 | **Manhole #5 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 708.29 | 0.00 | **714.70** | **716.30** | **1.60** |
| 37 | CCFRPM Pipe (Santa Ana) | 1 | 26.9 | 48 | | | | 155 | 78 | 0.02 | 708.43 | 0.14 | 714.86 | | |
| 38 | Manhole #6 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 708.43 | 0.00 | 714.93 | | |
| 39 | **Manhole #6 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 708.43 | 0.00 | **714.97** | **719.40** | **4.43** |
| 40 | CCFRPM Pipe (Santa Ana) | 1 | 26.9 | 48 | | | | 155 | 55 | 0.01 | 708.53 | 0.10 | 715.08 | | |
| 41 | Manhole #7 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 708.53 | 0.00 | 715.15 | | |
| 42 | **Manhole #7 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 708.53 | 0.00 | **715.19** | **719.40** | **4.21** |
| 43 | CCFRPM Pipe (Santa Ana) | 1 | 26.9 | 48 | | | | 155 | 216 | 0.05 | 708.92 | 0.39 | 715.62 | | |
| 44 | Manhole #8 Exit | 1 | 26.9 | 48 | 2.1 | 0.07 | 1 | | | 0.07 | 708.92 | 0.00 | 715.69 | | |
| 45 | **Manhole #8 Inlet** | 1 | 26.9 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 708.92 | 0.00 | **715.73** | **722.20** | **6.47** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Rubidoux Flow Rate =** 4.5 MGD
**Checked by:** BCP Date: 9/30/2014

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 2.9 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined Flow* | | *54.0* | | | | | | | | | | | | |
| | *Acorn Flow* | | *4.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.5 | | | 0.14 | 705.70 | | **710.59** | **713.70** | **3.11** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 54.0 | 48 | | | | 110 | 21 | 0.03 | 705.75 | 0.05 | 710.67 | | |
| 5 | 90 Degree Bend | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.9 | | | 0.26 | 705.75 | 0.00 | 710.93 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 54.0 | 48 | | | | 110 | 79 | 0.12 | 705.94 | 0.19 | 711.24 | | |
| 7 | Meter Box Outlet | 1 | 54.0 | 48 | 4.3 | 0.29 | 1 | | | 0.29 | 705.94 | 0.00 | 711.53 | | |
| 8 | **Meter Box Inlet** | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.5 | | | 0.14 | 705.96 | 0.02 | **711.69** | **715.90** | **4.21** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 54.0 | 48 | | | | 110 | 491 | 0.73 | 707.10 | 1.14 | 713.56 | | |
| 10 | 35 Degree Bend | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.4 | | | 0.11 | 707.10 | 0.00 | 713.67 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 54.0 | 48 | | | | 110 | 129 | 0.19 | 707.40 | 0.30 | 714.16 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 54.0 | 48 | 4.3 | 0.29 | 1 | | | 0.29 | 707.40 | 0.00 | 714.45 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 4.5 | 48 | 0.4 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **714.45** | **716.10** | **1.65** |
| 14 | RCP (Acorn) | 1 | 4.5 | 36 | | | | 110 | 109 | 0.01 | 711.29 | 3.89 | 718.35 | | |
| 15 | RCP (Acorn) | 1 | 4.5 | 27 | | | | 110 | 39 | 0.01 | 712.36 | 1.07 | 719.43 | | |
| 16 | New MH AC-3 Outlet | 1 | 4.5 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 712.36 | 0.00 | 719.45 | | |
| 17 | 45 Degree Bend | 1 | 4.5 | 24 | 1.4 | 0.03 | 0.42 | | | 0.01 | 712.36 | 0.00 | 719.46 | | |
| 18 | **New MH AC-3 Inlet** | 1 | 4.5 | 24 | 1.4 | 0.03 | 0.5 | | | 0.02 | 712.36 | 0.00 | **719.48** | **720.70** | **1.22** |
| 19 | New RCP (Acorn) | 1 | 4.5 | 24 | | | | 140 | 28 | 0.01 | 712.93 | 0.57 | 720.05 | | |
| 20 | 45 Degree Bend | 1 | 4.5 | 24 | 1.4 | 0.03 | 0.42 | | | 0.01 | 714.24 | 1.31 | 721.38 | | |
| 21 | Tee from Bypass Run | 1 | 4.5 | 24 | 1.4 | 0.03 | 0.6 | | | 0.02 | 714.24 | 0.00 | 721.39 | | |
| 22 | Outlet Gate | 1 | 4.5 | 16 | 3.2 | 0.16 | 0.19 | | | 0.03 | 714.24 | 0.00 | 721.42 | | |
| 23 | Pipe Bend | 1 | 4.5 | 16 | 3.2 | 0.16 | 0.2 | | | 0.03 | 712.74 | -1.50 | 719.96 | | |
| 24 | New CCFRPM (Acorn Meter Run) | 1 | 4.5 | 16 | | | | 155 | 23 | 0.04 | 712.74 | 0.00 | 719.99 | | |
| 25 | New WSP (Acorn Meter Run) | 1 | 4.5 | 16 | | | | 140 | 17 | 0.03 | 712.74 | 0.00 | 720.03 | | |
| 26 | Inlet Structure Outlet | 1 | 4.5 | 16 | 3.2 | 0.16 | 1 | | | 0.16 | 712.74 | 0.00 | 720.19 | | |
| 27 | Inlet Structure Losses | 1 | 4.5 | 16 | 3.2 | 0.16 | 0.5 | | | 0.08 | 714.24 | 1.50 | 721.77 | | |
| 28 | Inlet Gate | 1 | 4.5 | 16 | 3.2 | 0.16 | 0.19 | | | 0.03 | 714.24 | 0.00 | 721.80 | | |
| 29 | **Inlet Structure Inlet (Acorn)** | 1 | 4.5 | 24 | 1.4 | 0.03 | 0.5 | | | 0.02 | 714.74 | 0.50 | **722.31** | **724.00** | **1.69** |
| 30 | New RCP (Acorn) | 1 | 4.5 | 24 | | | | 140 | 12 | 0.00 | 714.98 | 0.24 | 722.56 | | |
| 31 | New MH AC-2 Outlet | 1 | 4.5 | 24 | 1.4 | 0.03 | 1 | | | 0.03 | 714.98 | 0.00 | 722.59 | | |
| 32 | 45 Degree Bend | 1 | 4.5 | 24 | 1.4 | 0.03 | 0.42 | | | 0.01 | 714.98 | 0.00 | 722.60 | | |
| 33 | **New MH AC-2 Inlet** | 1 | 4.5 | 24 | 1.4 | 0.03 | 0.5 | | | 0.02 | 714.98 | 0.00 | **722.62** | **725.90** | **3.28** |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 2.9 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 34 | New RCP (Acorn) | 1 | 4.5 | 24 | | | | 140 | 31 | 0.01 | 715.60 | 0.62 | 723.25 | | |
| 35 | New MH AC-1 Outlet | 1 | 4.5 | 24 | 1.4 | 0.03 | 1 | | | 0.03 | 715.60 | 0.00 | 723.28 | | |
| 36 | 45 Degree Bend | 1 | 4.5 | 24 | 1.4 | 0.03 | 0.42 | | | 0.01 | 715.60 | 0.00 | 723.29 | | |
| 37 | **New MH AC-1 Inlet** | 1 | 4.5 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 715.60 | 0.00 | **723.30** | **726.60** | **3.30** |
| 38 | RCP (Acorn) | 1 | 4.5 | 27 | | | | 110 | 25 | 0.01 | 716.30 | 0.70 | 724.01 | | |
| 39 | Acorn Manhole #1 Outlet | 1 | 4.5 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 716.30 | 0.00 | 724.03 | | |
| 40 | **Acorn Manhole #1 Inlet** | 1 | 4.5 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 716.30 | 0.00 | **724.04** | **728.30** | **4.26** |
| 41 | RCP (Acorn) | 1 | 4.5 | 27 | | | | 110 | 339 | 0.08 | 735.49 | 19.19 | 743.31 | | |
| 42 | Acorn Manhole #2 Exit | 1 | 4.5 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 735.49 | 0.00 | 743.33 | | |
| 43 | **Acorn Manhole #2 Inlet** | 1 | 4.5 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 735.49 | 0.00 | **743.34** | **747.50** | **4.16** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 32.0 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined Flow* | | *54.0* | | | | | | | | | | | | |
| | *Arlanza Flow* | | *49.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.5 | | | 0.14 | 705.70 | | **710.59** | **713.70** | **3.11** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 54.0 | 48 | | | | 110 | 21 | 0.03 | 705.75 | 0.05 | 709.72 | | |
| 5 | 90 Degree Bend | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.9 | | | 0.26 | 705.75 | 0.00 | 709.98 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 54.0 | 48 | | | | 110 | 79 | 0.12 | 705.94 | 0.19 | 710.29 | | |
| 7 | Meter Box Outlet | 1 | 54.0 | 48 | 4.3 | 0.29 | 1 | | | 0.29 | 705.94 | 0.00 | 710.57 | | |
| 8 | **Meter Box Inlet** | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.5 | | | 0.14 | 705.96 | 0.02 | **710.74** | **716.00** | **5.26** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 54.0 | 48 | | | | 110 | 491 | 0.73 | 707.10 | 1.14 | 712.60 | | |
| 10 | 35 Degree Bend | 1 | 54.0 | 48 | 4.3 | 0.29 | 0.4 | | | 0.11 | 707.10 | 0.00 | 712.72 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 54.0 | 48 | | | | 110 | 129 | 0.19 | 707.40 | 0.30 | 713.21 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 54.0 | 48 | 4.3 | 0.29 | 1 | | | 0.29 | 707.40 | 0.00 | 713.50 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 27.0 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.40 | 0.00 | **713.53** | **716.10** | **2.57** |
| 14 | Reinforced Concrete Pipe (RCP, Arlanza) | 2 | 24.7 | 48 | | | | 110 | 293 | 0.10 | 708.00 | 0.60 | 714.23 | | |
| 15 | Arlanza Junction Box Exit | 2 | 24.7 | 48 | 2.0 | 0.06 | 1 | | | 0.06 | 708.00 | 0.00 | 714.29 | | |
| 16 | **Arlanza Junction Box Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 708.00 | 0.00 | **714.39** | **717.50** | **3.11** |
| 17 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 714.41 | | |
| 18 | 30 Degree Bend | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.4 | | | 0.08 | 708.02 | 0.00 | 714.49 | | |
| 19 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 17 | 0.02 | 708.08 | 0.06 | 714.56 | | |
| 20 | 20 Degree Bend | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.25 | | | 0.05 | 708.08 | 0.00 | 714.61 | | |
| 21 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 86 | 0.08 | 708.39 | 0.31 | 715.00 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 708.39 | 0.00 | 715.19 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 708.39 | 0.00 | **715.28** | **724.80** | **9.52** |
| 24 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 161 | 0.15 | 708.97 | 0.58 | 716.01 | | |
| 25 | 50 Degree Bend | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.7 | | | 0.13 | 708.97 | 0.00 | 716.15 | | |
| 26 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 24 | 0.02 | 709.05 | 0.09 | 716.26 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 709.05 | 0.00 | 716.44 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 709.05 | 0.00 | **716.54** | **727.50** | **10.96** |
| 29 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 150 | 0.14 | 709.59 | 0.54 | 717.22 | | |
| 30 | Exit - Junction Box | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 709.59 | 0.00 | 717.41 | | |
| 31 | **Inlet - Junction Box** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 709.59 | 0.00 | **717.50** | **725.00** | **7.50** |
| 32 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 303 | 0.28 | 710.68 | 1.09 | 718.88 | | |
| 33 | 20 Degree Bend | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.25 | | | 0.05 | 710.68 | 0.00 | 718.92 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 32.0 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 495 | 0.46 | 712.46 | 1.78 | 721.17 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 712.46 | 0.00 | 721.36 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 712.46 | 0.00 | **721.45** | **724.00** | **2.55** |
| 37 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 428 | 0.40 | 713.32 | 0.86 | 722.71 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 713.32 | 0.00 | 722.90 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 713.32 | 0.00 | **722.99** | **726.30** | **3.31** |
| 40 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.5 | 51 | | | | 110 | 428 | 0.40 | 714.06 | 0.74 | 724.14 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 714.06 | 0.00 | 724.33 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 714.06 | 0.00 | **724.42** | **725.60** | **1.18** |
| 43 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 21 | 0.02 | 715.67 | 1.61 | 726.05 | | |
| 44 | Tee from future Jurupa Run | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.6 | | | 0.11 | 715.67 | 0.00 | 726.16 | | |
| 45 | Tee from Bypass Run | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.6 | | | 0.11 | 715.67 | 0.00 | 726.28 | | |
| 46 | Outlet Gate | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.19 | | | 0.04 | 715.67 | 0.00 | 726.31 | | |
| 47 | Pipe Bend | 1 | 49.5 | 36 | 7.0 | 0.76 | 0.2 | | | 0.15 | 712.67 | -3.00 | 723.46 | | |
| 48 | New CCFRPM (Arlanza Meter Run) | 1 | 49.5 | 36 | | | | 155 | 45 | 0.12 | 712.67 | 0.00 | 723.59 | | |
| 49 | New WSP (Arlanza Meter Run) | 1 | 49.5 | 36 | | | | 140 | 16 | 0.05 | 712.67 | 0.00 | 723.64 | | |
| 50 | Inlet Structure Outlet | 1 | 49.5 | 36 | 7.0 | 0.76 | 1 | | | 0.76 | 712.67 | 0.00 | 724.40 | | |
| 51 | Inlet Structure Losses | 1 | 49.5 | 48 | 3.9 | 0.24 | 0.5 | | | 0.12 | 715.67 | 3.00 | 727.52 | | |
| 52 | Inlet Gate | 1 | 49.5 | 48 | 3.9 | 0.24 | 0.19 | | | 0.05 | 715.67 | 0.00 | 727.57 | | |
| 53 | **Inlet Structure Inlet (Arlanza)** | 1 | 49.5 | 48 | 3.9 | 0.24 | 0.5 | | | 0.12 | 715.67 | 0.00 | **727.69** | **731.00** | **3.31** |
| 54 | RCP (Arlanza) | 1 | 49.5 | 51 | | | | 110 | 156 | 0.15 | 718.46 | 2.79 | 730.63 | | |
| 55 | Arlanza Manhole #6 Outlet | 1 | 49.5 | 51 | 3.5 | 0.19 | 1 | | | 0.19 | 718.46 | 0.00 | 730.81 | | |
| 56 | **Arlanza Manhole #6 Inlet** | 1 | 49.5 | 51 | 3.5 | 0.19 | 0.5 | | | 0.09 | 718.46 | 0.00 | **730.91** | **730.00** | **-0.91** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box invert elevation and manhole elevation were determined on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 25.5 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) Combined Flow* | | *53.7* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) Flow* | | *39.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 26.9 | 42 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.70 | | **710.51** | **713.70** | **3.19** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 26.9 | 42 | | | | 125 | 28 | 0.02 | 705.87 | 0.17 | 710.70 | | |
| 5 | 35 Degree Bend | 2 | 26.9 | 42 | 2.8 | 0.12 | 0.4 | | | 0.05 | 705.87 | 0.00 | 710.75 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 26.9 | 42 | | | | 125 | 200 | 0.12 | 707.08 | 1.21 | 712.08 | | |
| 7 | 35 Degree Bend | 2 | 26.9 | 42 | 2.8 | 0.12 | 0.4 | | | 0.05 | 707.08 | 0.00 | 712.13 | | |
| 8 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 26.9 | 42 | | | | 125 | 16 | 0.01 | 707.18 | 0.10 | 712.24 | | |
| 9 | Exit from Junction | 2 | 26.9 | 42 | 2.8 | 0.12 | 1 | | | 0.12 | 707.18 | 0.00 | 712.36 | | |
| 10 | **Inlet to Junction** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 707.18 | 0.00 | **712.44** | **717.00** | **4.56** |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 39.5 | 48 | | | | 125 | 10 | 0.01 | 707.30 | 0.12 | 712.56 | | |
| 12 | CCFRPM Pipe (Santa Ana) | 1 | 39.5 | 48 | | | | 155 | 35 | 0.02 | 707.70 | 0.40 | 712.98 | | |
| 13 | Manhole #1 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 707.70 | 0.00 | 713.13 | | |
| 14 | **Manhole #1 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 707.70 | 0.00 | **713.21** | **716.20** | **2.99** |
| 15 | New CCFRPM (Santa Ana) | 1 | 39.5 | 36 | | | | 155 | 12 | 0.02 | 707.99 | 0.29 | 713.52 | | |
| 16 | Tee from Bypass Run | 1 | 39.5 | 36 | 5.6 | 0.49 | 0.6 | | | 0.29 | 707.99 | 0.00 | 713.82 | | |
| 17 | Outlet Gate | 1 | 39.5 | 36 | 5.6 | 0.49 | 0.19 | | | 0.09 | 707.99 | 0.00 | 713.91 | | |
| 18 | Pipe Bend, vertical | 1 | 39.5 | 36 | 5.6 | 0.49 | 0.2 | | | 0.10 | 704.99 | -3.00 | 711.00 | | |
| 19 | New CCFRPM (Santa Ana Meter Run) | 1 | 39.5 | 36 | | | | 155 | 71 | 0.13 | 704.99 | 0.00 | 711.13 | | |
| 20 | New WSP (Santa Ana Meter Run) | 1 | 39.5 | 36 | | | | 140 | 38 | 0.08 | 704.99 | 0.00 | 711.21 | | |
| 21 | Inlet Structure Outlet | 1 | 39.5 | 36 | 5.6 | 0.49 | 1 | | | 0.49 | 704.99 | 0.00 | 711.70 | | |
| 22 | Inlet Structure Losses | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 707.99 | 3.00 | 714.78 | | |
| 23 | Inlet Gate | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.19 | | | 0.03 | 707.99 | 0.00 | 714.81 | | |
| 24 | **Inlet Structure Inlet (Santa Ana)** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 707.99 | 0.00 | **714.88** | **717.80** | **2.92** |
| 25 | CCFRPM Pipe (Santa Ana) | 1 | 39.5 | 48 | 3.1 | 0.15 | | 155 | 31 | 0.01 | 708.06 | 0.07 | 714.96 | | |
| 26 | Manhole #2 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 708.06 | 0.00 | 715.11 | | |
| 27 | **Manhole #2 Inlet** | 1 | 39.5 | 41 | 4.3 | 0.29 | 0.5 | | | 0.14 | 708.06 | 0.00 | **715.26** | **714.60** | **-0.66** |
| 28 | CCFRPM Pipe (Santa Ana) | 1 | 34.4 | 41 | | | | 155 | 13 | 0.01 | 708.10 | 0.04 | 715.31 | | |
| 29 | Manhole #3 Exit | 1 | 34.4 | 41 | 3.8 | 0.22 | 1 | | | 0.22 | 708.10 | 0.00 | 715.53 | | |
| 30 | **Manhole #3 Inlet** | 1 | 34.4 | 41 | 3.8 | 0.22 | 0.5 | | | 0.11 | 708.10 | 0.00 | **715.64** | **714.90** | **-0.74** |
| 31 | CCFRPM Pipe (Santa Ana) | 1 | 34.4 | 41 | | | | 155 | 95 | 0.07 | 708.26 | 0.16 | 715.87 | | |
| 32 | Manhole #4 Exit | 1 | 34.4 | 41 | 3.8 | 0.22 | 1 | | | 0.22 | 708.26 | 0.00 | 716.09 | | |
| 33 | **Manhole #4 Inlet** | 1 | 34.4 | 41 | 3.8 | 0.22 | 0.5 | | | 0.11 | 708.26 | 0.00 | **716.20** | **715.50** | **-0.70** |
| 34 | CCFRPM Pipe (Santa Ana) | 1 | 34.4 | 41 | | | | 155 | 13 | 0.01 | 708.29 | 0.03 | 716.24 | | |
| 35 | Manhole #5 Exit | 1 | 34.4 | 41 | 3.8 | 0.22 | 1 | | | 0.22 | 708.29 | 0.00 | 716.45 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Revised Present Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 25.5 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36 | **Manhole #5 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 708.29 | 0.00 | **716.53** | **716.30** | **-0.23** |
| 37 | CCFRPM Pipe (Santa Ana) | 1 | 39.5 | 48 | | | | 155 | 78 | 0.03 | 708.43 | 0.14 | 716.71 | | |
| 38 | Manhole #6 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 708.43 | 0.00 | 716.86 | | |
| 39 | **Manhole #6 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 708.43 | 0.00 | **716.94** | **719.40** | **2.46** |
| 40 | CCFRPM Pipe (Santa Ana) | 1 | 39.5 | 48 | | | | 155 | 55 | 0.02 | 708.53 | 0.10 | 717.06 | | |
| 41 | Manhole #7 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 708.53 | 0.00 | 717.21 | | |
| 42 | **Manhole #7 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 708.53 | 0.00 | **717.29** | **719.40** | **2.11** |
| 43 | CCFRPM Pipe (Santa Ana) | 1 | 39.5 | 48 | | | | 155 | 216 | 0.10 | 708.92 | 0.39 | 717.78 | | |
| 44 | Manhole #8 Exit | 1 | 39.5 | 48 | 3.1 | 0.15 | 1 | | | 0.15 | 708.92 | 0.00 | 717.93 | | |
| 45 | **Manhole #8 Inlet** | 1 | 39.5 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 708.92 | 0.00 | **718.01** | **722.20** | **4.19** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Rubidoux Flow Rate =** 3.1 MGD
**Checked by:** BCP Date: 9/30/2014

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 1.9 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined Flow* | | *35.6* | | | | | | | | | | | | |
| | *Acorn Flow* | | *3.0* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.70 | | **710.51** | **713.70** | **3.19** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 35.6 | 48 | | | | 110 | 21 | 0.01 | 705.75 | 0.05 | 710.58 | | |
| 5 | 90 Degree Bend | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.9 | | | 0.11 | 705.75 | 0.00 | 710.69 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 35.6 | 48 | | | | 110 | 79 | 0.05 | 705.94 | 0.19 | 710.93 | | |
| 7 | Meter Box Outlet | 1 | 35.6 | 48 | 2.8 | 0.12 | 1 | | | 0.12 | 705.94 | 0.00 | 711.06 | | |
| 8 | **Meter Box Inlet** | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.96 | 0.02 | **711.14** | **715.90** | **4.76** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 35.6 | 48 | | | | 110 | 491 | 0.34 | 707.10 | 1.14 | 712.62 | | |
| 10 | 35 Degree Bend | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.4 | | | 0.05 | 707.10 | 0.00 | 712.67 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 35.6 | 48 | | | | 110 | 129 | 0.09 | 707.40 | 0.30 | 713.05 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 35.6 | 48 | 2.8 | 0.12 | 1 | | | 0.12 | 707.40 | 0.00 | 713.18 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 3.0 | 48 | 0.2 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **713.18** | **716.10** | **2.92** |
| 14 | RCP (Acorn) | 1 | 3.0 | 36 | | | | 110 | 109 | 0.00 | 711.29 | 3.89 | 717.08 | | |
| 15 | RCP (Acorn) | 1 | 3.0 | 27 | | | | 110 | 39 | 0.00 | 712.36 | 1.07 | 718.15 | | |
| 16 | New MH AC-3 Outlet | 1 | 3.0 | 27 | 0.7 | 0.01 | 1 | | | 0.01 | 712.36 | 0.00 | 718.16 | | |
| 17 | 45 Degree Bend | 1 | 3.0 | 24 | 0.9 | 0.01 | 0.42 | | | 0.01 | 712.36 | 0.00 | 718.17 | | |
| 18 | **New MH AC-3 Inlet** | 1 | 3.0 | 24 | 0.9 | 0.01 | 0.5 | | | 0.01 | 712.36 | 0.00 | **718.17** | **720.70** | **2.53** |
| 19 | New RCP (Acorn) | 1 | 3.0 | 24 | | | | 140 | 28 | 0.00 | 712.93 | 0.57 | 718.74 | | |
| 20 | 45 Degree Bend | 1 | 3.0 | 24 | 0.9 | 0.01 | 0.42 | | | 0.01 | 714.24 | 1.31 | 720.06 | | |
| 21 | Tee from Bypass Run | 1 | 3.0 | 24 | 0.9 | 0.01 | 0.6 | | | 0.01 | 714.24 | 0.00 | 720.07 | | |
| 22 | Outlet Gate | 1 | 3.0 | 16 | 2.1 | 0.07 | 0.19 | | | 0.01 | 714.24 | 0.00 | 720.08 | | |
| 23 | Pipe Bend | 1 | 3.0 | 16 | 2.1 | 0.07 | 0.2 | | | 0.01 | 712.74 | -1.50 | 718.59 | | |
| 24 | New CCFRPM (Acorn Meter Run) | 1 | 3.0 | 16 | | | | 155 | 23 | 0.02 | 712.74 | 0.00 | 718.61 | | |
| 25 | New WSP (Acorn Meter Run) | 1 | 3.0 | 16 | | | | 140 | 17 | 0.02 | 712.74 | 0.00 | 718.63 | | |
| 26 | Inlet Structure Outlet | 1 | 3.0 | 16 | 2.1 | 0.07 | 1 | | | 0.07 | 712.74 | 0.00 | 718.70 | | |
| 27 | Inlet Structure Losses | 1 | 3.0 | 16 | 2.1 | 0.07 | 0.5 | | | 0.03 | 714.24 | 1.50 | 720.23 | | |
| 28 | Inlet Gate | 1 | 3.0 | 16 | 2.1 | 0.07 | 0.19 | | | 0.01 | 714.24 | 0.00 | 720.24 | | |
| 29 | **Inlet Structure Inlet (Acorn)** | 1 | 3.0 | 24 | 0.9 | 0.01 | 0.5 | | | 0.01 | 714.74 | 0.50 | **720.75** | **724.00** | **3.25** |
| 30 | New RCP (Acorn) | 1 | 3.0 | 24 | | | | 140 | 12 | 0.00 | 714.98 | 0.24 | 720.99 | | |
| 31 | New MH AC-2 Outlet | 1 | 3.0 | 24 | 0.9 | 0.01 | 1 | | | 0.01 | 714.98 | 0.00 | 721.01 | | |
| 32 | 45 Degree Bend | 1 | 3.0 | 24 | 0.9 | 0.01 | 0.42 | | | 0.01 | 714.98 | 0.00 | 721.01 | | |
| 33 | **New MH AC-2 Inlet** | 1 | 3.0 | 24 | 0.9 | 0.01 | 0.5 | | | 0.01 | 714.98 | 0.00 | **721.02** | **725.90** | **4.88** |

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# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 1.9 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 34 | New RCP (Acorn) | 1 | 3.0 | 24 | | | | 140 | 31 | 0.00 | 715.60 | 0.62 | 721.64 | | |
| 35 | New MH AC-1 Outlet | 1 | 3.0 | 24 | 0.9 | 0.01 | 1 | | | 0.01 | 715.60 | 0.00 | 721.66 | | |
| 36 | 45 Degree Bend | 1 | 3.0 | 24 | 0.9 | 0.01 | 0.42 | | | 0.01 | 715.60 | 0.00 | 721.66 | | |
| 37 | **New MH AC-1 Inlet** | 1 | 3.0 | 27 | 0.7 | 0.01 | 0.5 | | | 0.00 | 715.60 | 0.00 | **721.67** | **726.60** | **4.93** |
| 38 | RCP (Acorn) | 1 | 3.0 | 27 | | | | 110 | 25 | 0.00 | 716.30 | 0.70 | 722.37 | | |
| 39 | Acorn Manhole #1 Outlet | 1 | 3.0 | 27 | 0.7 | 0.01 | 1 | | | 0.01 | 716.30 | 0.00 | 722.38 | | |
| 40 | **Acorn Manhole #1 Inlet** | 1 | 3.0 | 27 | 0.7 | 0.01 | 0.5 | | | 0.00 | 716.30 | 0.00 | **722.38** | **728.30** | **5.92** |
| 41 | RCP (Acorn) | 1 | 3.0 | 27 | | | | 110 | 339 | 0.04 | 735.49 | 19.19 | 741.61 | | |
| 42 | Acorn Manhole #2 Exit | 1 | 3.0 | 27 | 0.7 | 0.01 | 1 | | | 0.01 | 735.49 | 0.00 | 741.62 | | |
| 43 | **Acorn Manhole #2 Inlet** | 1 | 3.0 | 27 | 0.7 | 0.01 | 0.5 | | | 0.00 | 735.49 | 0.00 | **741.62** | **747.50** | **5.88** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

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# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 21.1 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined Flow* | | *35.6* | | | | | | | | | | | | |
| | *Arlanza Flow* | | *32.6* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.70 | | **710.51** | **713.70** | **3.19** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 35.6 | 48 | | | | 110 | 21 | 0.01 | 705.75 | 0.05 | 709.70 | | |
| 5 | 90 Degree Bend | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.9 | | | 0.11 | 705.75 | 0.00 | 709.82 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 35.6 | 48 | | | | 110 | 79 | 0.05 | 705.94 | 0.19 | 710.06 | | |
| 7 | Meter Box Outlet | 1 | 35.6 | 48 | 2.8 | 0.12 | 1 | | | 0.12 | 705.94 | 0.00 | 710.19 | | |
| 8 | **Meter Box Inlet** | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.5 | | | 0.06 | 705.96 | 0.02 | **710.27** | **716.00** | **5.73** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 35.6 | 48 | | | | 110 | 491 | 0.34 | 707.10 | 1.14 | 711.75 | | |
| 10 | 35 Degree Bend | 1 | 35.6 | 48 | 2.8 | 0.12 | 0.4 | | | 0.05 | 707.10 | 0.00 | 711.80 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 35.6 | 48 | | | | 110 | 129 | 0.09 | 707.40 | 0.30 | 712.18 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 35.6 | 48 | 2.8 | 0.12 | 1 | | | 0.12 | 707.40 | 0.00 | 712.31 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 17.8 | 48 | 1.4 | 0.03 | 0.5 | | | 0.02 | 707.40 | 0.00 | **712.32** | **716.10** | **3.78** |
| 14 | Reinforced Concrete Pipe (RCP, Arlanza) | 2 | 16.3 | 48 | | | | 110 | 293 | 0.05 | 708.00 | 0.60 | 712.97 | | |
| 15 | Arlanza Junction Box Exit | 2 | 16.3 | 48 | 1.3 | 0.03 | 1 | | | 0.03 | 708.00 | 0.00 | 713.00 | | |
| 16 | **Arlanza Junction Box Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 708.00 | 0.00 | **713.04** | **717.50** | **4.46** |
| 17 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 713.06 | | |
| 18 | 30 Degree Bend | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.4 | | | 0.03 | 708.02 | 0.00 | 713.09 | | |
| 19 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 17 | 0.01 | 708.08 | 0.06 | 713.16 | | |
| 20 | 20 Degree Bend | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.25 | | | 0.02 | 708.08 | 0.00 | 713.18 | | |
| 21 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 86 | 0.04 | 708.39 | 0.31 | 713.53 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 708.39 | 0.00 | 713.61 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 708.39 | 0.00 | **713.65** | **724.80** | **11.15** |
| 24 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 161 | 0.07 | 708.97 | 0.58 | 714.30 | | |
| 25 | 50 Degree Bend | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.7 | | | 0.06 | 708.97 | 0.00 | 714.36 | | |
| 26 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 24 | 0.01 | 709.05 | 0.09 | 714.45 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 709.05 | 0.00 | 714.53 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 709.05 | 0.00 | **714.58** | **727.50** | **12.92** |
| 29 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 150 | 0.07 | 709.59 | 0.54 | 715.18 | | |
| 30 | Exit - Junction Box | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 709.59 | 0.00 | 715.26 | | |
| 31 | **Inlet - Junction Box** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 709.59 | 0.00 | **715.30** | **725.00** | **9.70** |
| 32 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 303 | 0.13 | 710.68 | 1.09 | 716.52 | | |
| 33 | 20 Degree Bend | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.25 | | | 0.02 | 710.68 | 0.00 | 716.54 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 21.1 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| 34 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 495 | 0.22 | 712.46 | 1.78 | 718.54 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 712.46 | 0.00 | 718.62 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 712.46 | 0.00 | **718.66** | **724.00** | **5.34** |
| 37 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 428 | 0.19 | 713.32 | 0.86 | 719.70 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 713.32 | 0.00 | 719.79 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 713.32 | 0.00 | **719.83** | **726.30** | **6.47** |
| 40 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 32.6 | 51 | | | | 110 | 428 | 0.19 | 714.06 | 0.74 | 720.76 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 714.06 | 0.00 | 720.84 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 714.06 | 0.00 | **720.88** | **725.60** | **4.72** |
| 43 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 21 | 0.01 | 715.67 | 1.61 | 722.50 | | |
| 44 | Tee from future Jurupa Run | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.6 | | | 0.05 | 715.67 | 0.00 | 722.55 | | |
| 45 | Tee from Bypass Run | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.6 | | | 0.05 | 715.67 | 0.00 | 722.60 | | |
| 46 | Outlet Gate | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.19 | | | 0.02 | 715.67 | 0.00 | 722.61 | | |
| 47 | Pipe Bend | 1 | 32.6 | 36 | 4.6 | 0.33 | 0.2 | | | 0.07 | 712.67 | -3.00 | 719.68 | | |
| 48 | New CCFRPM (Arlanza Meter Run) | 1 | 32.6 | 36 | | | | 155 | 45 | 0.06 | 712.67 | 0.00 | 719.73 | | |
| 49 | New WSP (Arlanza Meter Run) | 1 | 32.6 | 36 | | | | 140 | 16 | 0.02 | 712.67 | 0.00 | 719.76 | | |
| 50 | Inlet Structure Outlet | 1 | 32.6 | 36 | 4.6 | 0.33 | 1 | | | 0.33 | 712.67 | 0.00 | 720.09 | | |
| 51 | Inlet Structure Losses | 1 | 32.6 | 48 | 2.6 | 0.10 | 0.5 | | | 0.05 | 715.67 | 3.00 | 723.14 | | |
| 52 | Inlet Gate | 1 | 32.6 | 48 | 2.6 | 0.10 | 0.19 | | | 0.02 | 715.67 | 0.00 | 723.16 | | |
| 53 | **Inlet Structure Inlet (Arlanza)** | 1 | 32.6 | 48 | 2.6 | 0.10 | 0.5 | | | 0.05 | 715.67 | 0.00 | **723.22** | **731.00** | **7.78** |
| 54 | RCP (Arlanza) | 1 | 32.6 | 51 | | | | 110 | 156 | 0.07 | 718.46 | 2.79 | 726.08 | | |
| 55 | Arlanza Manhole #6 Outlet | 1 | 32.6 | 51 | 2.3 | 0.08 | 1 | | | 0.08 | 718.46 | 0.00 | 726.16 | | |
| 56 | **Arlanza Manhole #6 Inlet** | 1 | 32.6 | 51 | 2.3 | 0.08 | 0.5 | | | 0.04 | 718.46 | 0.00 | **726.20** | **730.00** | **3.80** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box invert elevation and manhole elevation were determined on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 21.9 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) Combined Flow* | | *46.0* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) Flow* | | *33.9* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 23.0 | 42 | 2.4 | 0.09 | 0.5 | | | 0.04 | 705.70 | | **710.49** | **713.70** | **3.21** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 23.0 | 42 | | | | 125 | 28 | 0.01 | 705.87 | 0.17 | 710.68 | | |
| 5 | 35 Degree Bend | 2 | 23.0 | 42 | 2.4 | 0.09 | 0.4 | | | 0.04 | 705.87 | 0.00 | 710.71 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 23.0 | 42 | | | | 125 | 200 | 0.09 | 707.08 | 1.21 | 712.02 | | |
| 7 | 35 Degree Bend | 2 | 23.0 | 42 | 2.4 | 0.09 | 0.4 | | | 0.04 | 707.08 | 0.00 | 712.05 | | |
| 8 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 23.0 | 42 | | | | 125 | 16 | 0.01 | 707.18 | 0.10 | 712.16 | | |
| 9 | Exit from Junction | 2 | 23.0 | 42 | 2.4 | 0.09 | 1 | | | 0.09 | 707.18 | 0.00 | 712.25 | | |
| 10 | **Inlet to Junction** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 707.18 | 0.00 | **712.30** | **717.00** | **4.70** |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 33.9 | 48 | | | | 125 | 10 | 0.00 | 707.30 | 0.12 | 712.42 | | |
| 12 | CCFRPM Pipe (Santa Ana) | 1 | 33.9 | 48 | | | | 155 | 35 | 0.01 | 707.70 | 0.40 | 712.84 | | |
| 13 | Manhole #1 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 707.70 | 0.00 | 712.95 | | |
| 14 | **Manhole #1 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 707.70 | 0.00 | **713.01** | **716.20** | **3.19** |
| 15 | New CCFRPM (Santa Ana) | 1 | 33.9 | 36 | | | | 155 | 12 | 0.02 | 707.99 | 0.29 | 713.32 | | |
| 16 | Tee from Bypass Run | 1 | 33.9 | 36 | 4.8 | 0.36 | 0.6 | | | 0.21 | 707.99 | 0.00 | 713.53 | | |
| 17 | Outlet Gate | 1 | 33.9 | 36 | 4.8 | 0.36 | 0.19 | | | 0.07 | 707.99 | 0.00 | 713.60 | | |
| 18 | Pipe Bend, vertical | 1 | 33.9 | 36 | 4.8 | 0.36 | 0.2 | | | 0.07 | 704.99 | -3.00 | 710.67 | | |
| 19 | New CCFRPM (Santa Ana Meter Run) | 1 | 33.9 | 36 | | | | 155 | 71 | 0.10 | 704.99 | 0.00 | 710.77 | | |
| 20 | New WSP (Santa Ana Meter Run) | 1 | 33.9 | 36 | | | | 140 | 38 | 0.06 | 704.99 | 0.00 | 710.83 | | |
| 21 | Inlet Structure Outlet | 1 | 33.9 | 36 | 4.8 | 0.36 | 1 | | | 0.36 | 704.99 | 0.00 | 711.19 | | |
| 22 | Inlet Structure Losses | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 707.99 | 3.00 | 714.25 | | |
| 23 | Inlet Gate | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.19 | | | 0.02 | 707.99 | 0.00 | 714.27 | | |
| 24 | **Inlet Structure Inlet (Santa Ana)** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 707.99 | 0.00 | **714.33** | **717.80** | **3.47** |
| 25 | CCFRPM Pipe (Santa Ana) | 1 | 33.9 | 48 | 2.7 | 0.11 | | 155 | 31 | 0.01 | 708.06 | 0.07 | 714.40 | | |
| 26 | Manhole #2 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 708.06 | 0.00 | 714.51 | | |
| 27 | **Manhole #2 Inlet** | 1 | 33.9 | 41 | 3.7 | 0.21 | 0.5 | | | 0.11 | 708.06 | 0.00 | **714.62** | **714.60** | **-0.02** |
| 28 | CCFRPM Pipe (Santa Ana) | 1 | 29.6 | 41 | | | | 155 | 13 | 0.01 | 708.10 | 0.04 | 714.67 | | |
| 29 | Manhole #3 Exit | 1 | 29.6 | 41 | 3.2 | 0.16 | 1 | | | 0.16 | 708.10 | 0.00 | 714.83 | | |
| 30 | **Manhole #3 Inlet** | 1 | 29.6 | 41 | 3.2 | 0.16 | 0.5 | | | 0.08 | 708.10 | 0.00 | **714.91** | **714.90** | **-0.01** |
| 31 | CCFRPM Pipe (Santa Ana) | 1 | 29.6 | 41 | | | | 155 | 95 | 0.05 | 708.26 | 0.16 | 715.12 | | |
| 32 | Manhole #4 Exit | 1 | 29.6 | 41 | 3.2 | 0.16 | 1 | | | 0.16 | 708.26 | 0.00 | 715.28 | | |
| 33 | **Manhole #4 Inlet** | 1 | 29.6 | 41 | 3.2 | 0.16 | 0.5 | | | 0.08 | 708.26 | 0.00 | **715.36** | **715.50** | **0.14** |
| 34 | CCFRPM Pipe (Santa Ana) | 1 | 29.6 | 41 | | | | 155 | 13 | 0.01 | 708.29 | 0.03 | 715.40 | | |
| 35 | Manhole #5 Exit | 1 | 29.6 | 41 | 3.2 | 0.16 | 1 | | | 0.16 | 708.29 | 0.00 | 715.56 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Ultimate Future Design Flow - AADF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 21.9 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36 | **Manhole #5 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 708.29 | 0.00 | **715.62** | **716.30** | **0.68** |
| 37 | CCFRPM Pipe (Santa Ana) | 1 | 33.9 | 48 | | | | 155 | 78 | 0.03 | 708.43 | 0.14 | 715.79 | | |
| 38 | Manhole #6 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 708.43 | 0.00 | 715.90 | | |
| 39 | **Manhole #6 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 708.43 | 0.00 | **715.96** | **719.40** | **3.44** |
| 40 | CCFRPM Pipe (Santa Ana) | 1 | 33.9 | 48 | | | | 155 | 55 | 0.02 | 708.53 | 0.10 | 716.07 | | |
| 41 | Manhole #7 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 708.53 | 0.00 | 716.19 | | |
| 42 | **Manhole #7 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 708.53 | 0.00 | **716.24** | **719.40** | **3.16** |
| 43 | CCFRPM Pipe (Santa Ana) | 1 | 33.9 | 48 | | | | 155 | 216 | 0.07 | 708.92 | 0.39 | 716.71 | | |
| 44 | Manhole #8 Exit | 1 | 33.9 | 48 | 2.7 | 0.11 | 1 | | | 0.11 | 708.92 | 0.00 | 716.82 | | |
| 45 | **Manhole #8 Inlet** | 1 | 33.9 | 48 | 2.7 | 0.11 | 0.5 | | | 0.06 | 708.92 | 0.00 | **716.87** | **722.20** | **5.33** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Rubidoux Flow Rate =** 4.6 MGD
**Checked by:** BCP Date: 9/30/2014

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 2.9 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined Flow* | | *53.4* | | | | | | | | | | | | |
| | *Acorn Flow* | | *4.4* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.5 | | | 0.14 | 705.70 | | **710.59** | **713.70** | **3.11** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 53.4 | 48 | | | | 110 | 21 | 0.03 | 705.75 | 0.05 | 710.67 | | |
| 5 | 90 Degree Bend | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.9 | | | 0.25 | 705.75 | 0.00 | 710.92 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 53.4 | 48 | | | | 110 | 79 | 0.11 | 705.94 | 0.19 | 711.23 | | |
| 7 | Meter Box Outlet | 1 | 53.4 | 48 | 4.2 | 0.28 | 1 | | | 0.28 | 705.94 | 0.00 | 711.51 | | |
| 8 | **Meter Box Inlet** | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.5 | | | 0.14 | 705.96 | 0.02 | **711.67** | **715.90** | **4.23** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 53.4 | 48 | | | | 110 | 491 | 0.71 | 707.10 | 1.14 | 713.52 | | |
| 10 | 35 Degree Bend | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.4 | | | 0.11 | 707.10 | 0.00 | 713.63 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 53.4 | 48 | | | | 110 | 129 | 0.19 | 707.40 | 0.30 | 714.12 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 53.4 | 48 | 4.2 | 0.28 | 1 | | | 0.28 | 707.40 | 0.00 | 714.40 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 4.4 | 48 | 0.4 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **714.40** | **716.10** | **1.70** |
| 14 | RCP (Acorn) | 1 | 4.4 | 36 | | | | 110 | 109 | 0.01 | 711.29 | 3.89 | 718.30 | | |
| 15 | RCP (Acorn) | 1 | 4.4 | 27 | | | | 110 | 39 | 0.01 | 712.36 | 1.07 | 719.38 | | |
| 16 | New MH AC-3 Outlet | 1 | 4.4 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 712.36 | 0.00 | 719.40 | | |
| 17 | 45 Degree Bend | 1 | 4.4 | 24 | 1.4 | 0.03 | 0.42 | | | 0.01 | 712.36 | 0.00 | 719.41 | | |
| 18 | **New MH AC-3 Inlet** | 1 | 4.4 | 24 | 1.4 | 0.03 | 0.5 | | | 0.02 | 712.36 | 0.00 | **719.43** | **720.70** | **1.27** |
| 19 | New RCP (Acorn) | 1 | 4.4 | 24 | | | | 140 | 28 | 0.01 | 712.93 | 0.57 | 720.00 | | |
| 20 | 45 Degree Bend | 1 | 4.4 | 24 | 1.4 | 0.03 | 0.42 | | | 0.01 | 714.24 | 1.31 | 721.33 | | |
| 21 | Tee from Bypass Run | 1 | 4.4 | 24 | 1.4 | 0.03 | 0.6 | | | 0.02 | 714.24 | 0.00 | 721.34 | | |
| 22 | Outlet Gate | 1 | 4.4 | 16 | 3.2 | 0.16 | 0.19 | | | 0.03 | 714.24 | 0.00 | 721.37 | | |
| 23 | Pipe Bend | 1 | 4.4 | 16 | 3.2 | 0.16 | 0.2 | | | 0.03 | 712.74 | -1.50 | 719.91 | | |
| 24 | New CCFRPM (Acorn Meter Run) | 1 | 4.4 | 16 | | | | 155 | 23 | 0.04 | 712.74 | 0.00 | 719.94 | | |
| 25 | New WSP (Acorn Meter Run) | 1 | 4.4 | 16 | | | | 140 | 17 | 0.03 | 712.74 | 0.00 | 719.98 | | |
| 26 | Inlet Structure Outlet | 1 | 4.4 | 16 | 3.2 | 0.16 | 1 | | | 0.16 | 712.74 | 0.00 | 720.13 | | |
| 27 | Inlet Structure Losses | 1 | 4.4 | 16 | 3.2 | 0.16 | 0.5 | | | 0.08 | 714.24 | 1.50 | 721.71 | | |
| 28 | Inlet Gate | 1 | 4.4 | 16 | 3.2 | 0.16 | 0.19 | | | 0.03 | 714.24 | 0.00 | 721.74 | | |
| 29 | **Inlet Structure Inlet (Acorn)** | 1 | 4.4 | 24 | 1.4 | 0.03 | 0.5 | | | 0.02 | 714.74 | 0.50 | **722.26** | **724.00** | **1.74** |
| 30 | New RCP (Acorn) | 1 | 4.4 | 24 | | | | 140 | 12 | 0.00 | 714.98 | 0.24 | 722.50 | | |
| 31 | New MH AC-2 Outlet | 1 | 4.4 | 24 | 1.4 | 0.03 | 1 | | | 0.03 | 714.98 | 0.00 | 722.53 | | |
| 32 | 45 Degree Bend | 1 | 4.4 | 24 | 1.4 | 0.03 | 0.42 | | | 0.01 | 714.98 | 0.00 | 722.54 | | |
| 33 | **New MH AC-2 Inlet** | 1 | 4.4 | 24 | 1.4 | 0.03 | 0.5 | | | 0.02 | 714.98 | 0.00 | **722.56** | **725.90** | **3.34** |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 2.9 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 34 | New RCP (Acorn) | 1 | 4.4 | 24 | | | | 140 | 31 | 0.01 | 715.60 | 0.62 | 723.19 | | |
| 35 | New MH AC-1 Outlet | 1 | 4.4 | 24 | 1.4 | 0.03 | 1 | | | 0.03 | 715.60 | 0.00 | 723.22 | | |
| 36 | 45 Degree Bend | 1 | 4.4 | 24 | 1.4 | 0.03 | 0.42 | | | 0.01 | 715.60 | 0.00 | 723.23 | | |
| 37 | **New MH AC-1 Inlet** | 1 | 4.4 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 715.60 | 0.00 | **723.24** | **726.60** | **3.36** |
| 38 | RCP (Acorn) | 1 | 4.4 | 27 | | | | 110 | 25 | 0.01 | 716.30 | 0.70 | 723.95 | | |
| 39 | Acorn Manhole #1 Outlet | 1 | 4.4 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 716.30 | 0.00 | 723.97 | | |
| 40 | **Acorn Manhole #1 Inlet** | 1 | 4.4 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 716.30 | 0.00 | **723.98** | **728.30** | **4.32** |
| 41 | RCP (Acorn) | 1 | 4.4 | 27 | | | | 110 | 339 | 0.08 | 735.49 | 19.19 | 743.25 | | |
| 42 | Acorn Manhole #2 Exit | 1 | 4.4 | 27 | 1.1 | 0.02 | 1 | | | 0.02 | 735.49 | 0.00 | 743.27 | | |
| 43 | **Acorn Manhole #2 Inlet** | 1 | 4.4 | 27 | 1.1 | 0.02 | 0.5 | | | 0.01 | 735.49 | 0.00 | **743.28** | **747.50** | **4.22** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 31.7 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | (ft) | | | |
| | *Acorn/Arlanza Combined Flow* | | *53.4* | | | | | | | | | | | | |
| | *Arlanza Flow* | | *49.0* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.5 | | | 0.14 | 705.70 | | **710.59** | **713.70** | **3.11** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 53.4 | 48 | | | | 110 | 21 | 0.03 | 705.75 | 0.05 | 709.72 | | |
| 5 | 90 Degree Bend | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.9 | | | 0.25 | 705.75 | 0.00 | 709.97 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 53.4 | 48 | | | | 110 | 79 | 0.11 | 705.94 | 0.19 | 710.28 | | |
| 7 | Meter Box Outlet | 1 | 53.4 | 48 | 4.2 | 0.28 | 1 | | | 0.28 | 705.94 | 0.00 | 710.56 | | |
| 8 | **Meter Box Inlet** | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.5 | | | 0.14 | 705.96 | 0.02 | **710.72** | **716.00** | **5.28** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 53.4 | 48 | | | | 110 | 491 | 0.71 | 707.10 | 1.14 | 712.57 | | |
| 10 | 35 Degree Bend | 1 | 53.4 | 48 | 4.2 | 0.28 | 0.4 | | | 0.11 | 707.10 | 0.00 | 712.68 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 53.4 | 48 | | | | 110 | 129 | 0.19 | 707.40 | 0.30 | 713.17 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 53.4 | 48 | 4.2 | 0.28 | 1 | | | 0.28 | 707.40 | 0.00 | 713.45 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 26.7 | 48 | 2.1 | 0.07 | 0.5 | | | 0.04 | 707.40 | 0.00 | **713.49** | **716.10** | **2.61** |
| 14 | Reinforced Concrete Pipe (RCP, Arlanza) | 2 | 24.5 | 48 | | | | 110 | 293 | 0.10 | 708.00 | 0.60 | 714.19 | | |
| 15 | Arlanza Junction Box Exit | 2 | 24.5 | 48 | 1.9 | 0.06 | 1 | | | 0.06 | 708.00 | 0.00 | 714.25 | | |
| 16 | **Arlanza Junction Box Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 708.00 | 0.00 | **714.34** | **717.50** | **3.16** |
| 17 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 5 | 0.00 | 708.02 | 0.02 | 714.36 | | |
| 18 | 30 Degree Bend | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.4 | | | 0.07 | 708.02 | 0.00 | 714.43 | | |
| 19 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 17 | 0.02 | 708.08 | 0.06 | 714.51 | | |
| 20 | 20 Degree Bend | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.25 | | | 0.05 | 708.08 | 0.00 | 714.56 | | |
| 21 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 86 | 0.08 | 708.39 | 0.31 | 714.95 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 708.39 | 0.00 | 715.13 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 708.39 | 0.00 | **715.22** | **724.80** | **9.58** |
| 24 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 161 | 0.15 | 708.97 | 0.58 | 715.95 | | |
| 25 | 50 Degree Bend | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.7 | | | 0.13 | 708.97 | 0.00 | 716.08 | | |
| 26 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 24 | 0.02 | 709.05 | 0.09 | 716.19 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 709.05 | 0.00 | 716.37 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 709.05 | 0.00 | **716.47** | **727.50** | **11.03** |
| 29 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 150 | 0.14 | 709.59 | 0.54 | 717.14 | | |
| 30 | Exit - Junction Box | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 709.59 | 0.00 | 717.33 | | |
| 31 | **Inlet - Junction Box** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 709.59 | 0.00 | **717.42** | **725.00** | **7.58** |
| 32 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 303 | 0.28 | 710.68 | 1.09 | 718.79 | | |
| 33 | 20 Degree Bend | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.25 | | | 0.05 | 710.68 | 0.00 | 718.83 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 31.7 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 495 | 0.46 | 712.46 | 1.78 | 721.07 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 712.46 | 0.00 | 721.25 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 712.46 | 0.00 | **721.35** | **724.00** | **2.65** |
| 37 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 428 | 0.39 | 713.32 | 0.86 | 722.60 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 713.32 | 0.00 | 722.78 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 713.32 | 0.00 | **722.87** | **726.30** | **3.43** |
| 40 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 49.0 | 51 | | | | 110 | 428 | 0.39 | 714.06 | 0.74 | 724.01 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 714.06 | 0.00 | 724.20 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 714.06 | 0.00 | **724.29** | **725.60** | **1.31** |
| 43 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 21 | 0.02 | 715.67 | 1.61 | 725.92 | | |
| 44 | Tee from future Jurupa Run | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.6 | | | 0.11 | 715.67 | 0.00 | 726.03 | | |
| 45 | Tee from Bypass Run | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.6 | | | 0.11 | 715.67 | 0.00 | 726.14 | | |
| 46 | Outlet Gate | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.19 | | | 0.04 | 715.67 | 0.00 | 726.17 | | |
| 47 | Pipe Bend | 1 | 49.0 | 36 | 6.9 | 0.75 | 0.2 | | | 0.15 | 712.67 | -3.00 | 723.32 | | |
| 48 | New CCFRPM (Arlanza Meter Run) | 1 | 49.0 | 36 | | | | 155 | 45 | 0.12 | 712.67 | 0.00 | 723.44 | | |
| 49 | New WSP (Arlanza Meter Run) | 1 | 49.0 | 36 | | | | 140 | 16 | 0.05 | 712.67 | 0.00 | 723.49 | | |
| 50 | Inlet Structure Outlet | 1 | 49.0 | 36 | 6.9 | 0.75 | 1 | | | 0.75 | 712.67 | 0.00 | 724.24 | | |
| 51 | Inlet Structure Losses | 1 | 49.0 | 48 | 3.9 | 0.24 | 0.5 | | | 0.12 | 715.67 | 3.00 | 727.36 | | |
| 52 | Inlet Gate | 1 | 49.0 | 48 | 3.9 | 0.24 | 0.19 | | | 0.04 | 715.67 | 0.00 | 727.40 | | |
| 53 | **Inlet Structure Inlet (Arlanza)** | 1 | 49.0 | 48 | 3.9 | 0.24 | 0.5 | | | 0.12 | 715.67 | 0.00 | **727.52** | **731.00** | **3.48** |
| 54 | RCP (Arlanza) | 1 | 49.0 | 51 | | | | 110 | 156 | 0.14 | 718.46 | 2.79 | 730.46 | | |
| 55 | Arlanza Manhole #6 Outlet | 1 | 49.0 | 51 | 3.5 | 0.18 | 1 | | | 0.18 | 718.46 | 0.00 | 730.64 | | |
| 56 | **Arlanza Manhole #6 Inlet** | 1 | 49.0 | 51 | 3.5 | 0.18 | 0.5 | | | 0.09 | 718.46 | 0.00 | **730.73** | **730.00** | **-0.73** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box invert elevation and manhole elevation were determined on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 32.9 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) Combined Flow* | | *65.3* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) Flow* | | *50.9* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 32.6 | 42 | 3.4 | 0.18 | 0.5 | | | 0.09 | 705.70 | | **710.54** | **713.70** | **3.16** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 32.6 | 42 | | | | 125 | 28 | 0.02 | 705.87 | 0.17 | 710.73 | | |
| 5 | 35 Degree Bend | 2 | 32.6 | 42 | 3.4 | 0.18 | 0.4 | | | 0.07 | 705.87 | 0.00 | 710.81 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 32.6 | 42 | | | | 125 | 200 | 0.18 | 707.08 | 1.21 | 712.20 | | |
| 7 | 35 Degree Bend | 2 | 32.6 | 42 | 3.4 | 0.18 | 0.4 | | | 0.07 | 707.08 | 0.00 | 712.27 | | |
| 8 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 32.6 | 42 | | | | 125 | 16 | 0.01 | 707.18 | 0.10 | 712.38 | | |
| 9 | Exit from Junction | 2 | 32.6 | 42 | 3.4 | 0.18 | 1 | | | 0.18 | 707.18 | 0.00 | 712.56 | | |
| 10 | **Inlet to Junction** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 707.18 | 0.00 | **712.68** | **717.00** | **4.32** |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 50.9 | 48 | | | | 125 | 10 | 0.01 | 707.30 | 0.12 | 712.81 | | |
| 12 | CCFRPM Pipe (Santa Ana) | 1 | 50.9 | 48 | | | | 155 | 35 | 0.02 | 707.70 | 0.40 | 713.24 | | |
| 13 | Manhole #1 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 707.70 | 0.00 | 713.49 | | |
| 14 | **Manhole #1 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 707.70 | 0.00 | **713.62** | **716.20** | **2.58** |
| 15 | New CCFRPM (Santa Ana) | 1 | 50.9 | 36 | | | | 155 | 12 | 0.03 | 707.99 | 0.29 | 713.95 | | |
| 16 | Tee from Bypass Run | 1 | 50.9 | 36 | 7.2 | 0.80 | 0.6 | | | 0.48 | 707.99 | 0.00 | 714.43 | | |
| 17 | Outlet Gate | 1 | 50.9 | 36 | 7.2 | 0.80 | 0.19 | | | 0.15 | 707.99 | 0.00 | 714.59 | | |
| 18 | Pipe Bend, vertical | 1 | 50.9 | 36 | 7.2 | 0.80 | 0.2 | | | 0.16 | 704.99 | -3.00 | 711.75 | | |
| 19 | New CCFRPM (Santa Ana Meter Run) | 1 | 50.9 | 36 | | | | 155 | 71 | 0.20 | 704.99 | 0.00 | 711.95 | | |
| 20 | New WSP (Santa Ana Meter Run) | 1 | 50.9 | 36 | | | | 140 | 38 | 0.13 | 704.99 | 0.00 | 712.08 | | |
| 21 | Inlet Structure Outlet | 1 | 50.9 | 36 | 7.2 | 0.80 | 1 | | | 0.80 | 704.99 | 0.00 | 712.89 | | |
| 22 | Inlet Structure Losses | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 707.99 | 3.00 | 716.01 | | |
| 23 | Inlet Gate | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.19 | | | 0.05 | 707.99 | 0.00 | 716.06 | | |
| 24 | **Inlet Structure Inlet (Santa Ana)** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 707.99 | 0.00 | **716.19** | **717.80** | **1.61** |
| 25 | CCFRPM Pipe (Santa Ana) | 1 | 50.9 | 48 | 4.0 | 0.25 | | 155 | 31 | 0.02 | 708.06 | 0.07 | 716.28 | | |
| 26 | Manhole #2 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 708.06 | 0.00 | 716.53 | | |
| 27 | **Manhole #2 Inlet** | 1 | 50.9 | 41 | 5.6 | 0.48 | 0.5 | | | 0.24 | 708.06 | 0.00 | **716.77** | **714.60** | **-2.17** |
| 28 | CCFRPM Pipe (Santa Ana) | 1 | 44.3 | 41 | | | | 155 | 13 | 0.02 | 708.10 | 0.04 | 716.83 | | |
| 29 | Manhole #3 Exit | 1 | 44.3 | 41 | 4.8 | 0.36 | 1 | | | 0.36 | 708.10 | 0.00 | 717.19 | | |
| 30 | **Manhole #3 Inlet** | 1 | 44.3 | 41 | 4.8 | 0.36 | 0.5 | | | 0.18 | 708.10 | 0.00 | **717.37** | **714.90** | **-2.47** |
| 31 | CCFRPM Pipe (Santa Ana) | 1 | 44.3 | 41 | | | | 155 | 95 | 0.11 | 708.26 | 0.16 | 717.64 | | |
| 32 | Manhole #4 Exit | 1 | 44.3 | 41 | 4.8 | 0.36 | 1 | | | 0.36 | 708.26 | 0.00 | 718.01 | | |
| 33 | **Manhole #4 Inlet** | 1 | 44.3 | 41 | 4.8 | 0.36 | 0.5 | | | 0.18 | 708.26 | 0.00 | **718.19** | **715.50** | **-2.69** |
| 34 | CCFRPM Pipe (Santa Ana) | 1 | 44.3 | 41 | | | | 155 | 13 | 0.02 | 708.29 | 0.03 | 718.23 | | |
| 35 | Manhole #5 Exit | 1 | 44.3 | 41 | 4.8 | 0.36 | 1 | | | 0.36 | 708.29 | 0.00 | 718.60 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Ultimate Future Design Flow - PDWF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 32.9 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36 | **Manhole #5 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 708.29 | 0.00 | **718.72** | **716.30** | **-2.42** |
| 37 | CCFRPM Pipe (Santa Ana) | 1 | 50.9 | 48 | | | | 155 | 78 | 0.05 | 708.43 | 0.14 | 718.92 | | |
| 38 | Manhole #6 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 708.43 | 0.00 | 719.17 | | |
| 39 | **Manhole #6 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 708.43 | 0.00 | **719.30** | **719.40** | **0.10** |
| 40 | CCFRPM Pipe (Santa Ana) | 1 | 50.9 | 48 | | | | 155 | 55 | 0.04 | 708.53 | 0.10 | 719.44 | | |
| 41 | Manhole #7 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 708.53 | 0.00 | 719.69 | | |
| 42 | **Manhole #7 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 708.53 | 0.00 | **719.82** | **719.40** | **-0.42** |
| 43 | CCFRPM Pipe (Santa Ana) | 1 | 50.9 | 48 | | | | 155 | 216 | 0.15 | 708.92 | 0.39 | 720.36 | | |
| 44 | Manhole #8 Exit | 1 | 50.9 | 48 | 4.0 | 0.25 | 1 | | | 0.25 | 708.92 | 0.00 | 720.62 | | |
| 45 | **Manhole #8 Inlet** | 1 | 50.9 | 48 | 4.0 | 0.25 | 0.5 | | | 0.13 | 708.92 | 0.00 | **720.74** | **722.20** | **1.46** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

URS

## PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Flow for Rubidoux Interceptor (for use in Santa Ana hydraulic profile calculation)
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Rubidoux Flow Rate =** 6.7 MGD
**Checked by:** BCP Date: 9/30/2014

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 4.2 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Acorn/Arlanza Combined Flow* | | *78.3* | | | | | | | | | | | | |
| | *Acorn Flow* | | *6.5* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction Box | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction Box** | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.5 | | | 0.30 | 705.70 | | **710.75** | **713.70** | **2.95** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 78.3 | 48 | | | | 110 | 21 | 0.06 | 705.75 | 0.05 | 710.86 | | |
| 5 | 90 Degree Bend | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.9 | | | 0.54 | 705.75 | 0.00 | 711.41 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 78.3 | 48 | | | | 110 | 79 | 0.23 | 705.94 | 0.19 | 711.83 | | |
| 7 | Meter Box Outlet | 1 | 78.3 | 48 | 6.2 | 0.60 | 1 | | | 0.60 | 705.94 | 0.00 | 712.43 | | |
| 8 | **Meter Box Inlet** | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.5 | | | 0.30 | 705.96 | 0.02 | **712.76** | **715.90** | **3.14** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 78.3 | 48 | | | | 110 | 491 | 1.45 | 707.10 | 1.14 | 715.34 | | |
| 10 | 35 Degree Bend | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.4 | | | 0.24 | 707.10 | 0.00 | 715.58 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 78.3 | 48 | | | | 110 | 129 | 0.38 | 707.40 | 0.30 | 716.26 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 78.3 | 48 | 6.2 | 0.60 | 1 | | | 0.60 | 707.40 | 0.00 | 716.87 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 1 | 6.5 | 48 | 0.5 | 0.00 | 0.5 | | | 0.00 | 707.40 | 0.00 | **716.87** | **716.10** | **-0.77** |
| 14 | RCP (Acorn) | 1 | 6.5 | 36 | | | | 110 | 109 | 0.01 | 711.29 | 3.89 | 720.78 | | |
| 15 | RCP (Acorn) | 1 | 6.5 | 27 | | | | 110 | 39 | 0.02 | 712.36 | 1.07 | 721.87 | | |
| 16 | New MH AC-3 Outlet | 1 | 6.5 | 27 | 1.6 | 0.04 | 1 | | | 0.04 | 712.36 | 0.00 | 721.91 | | |
| 17 | 45 Degree Bend | 1 | 6.5 | 24 | 2.1 | 0.07 | 0.42 | | | 0.03 | 712.36 | 0.00 | 721.93 | | |
| 18 | **New MH AC-3 Inlet** | 1 | 6.5 | 24 | 2.1 | 0.07 | 0.5 | | | 0.03 | 712.36 | 0.00 | **721.97** | **720.70** | **-1.27** |
| 19 | New RCP (Acorn) | 1 | 6.5 | 24 | | | | 140 | 28 | 0.02 | 712.93 | 0.57 | 722.55 | | |
| 20 | 45 Degree Bend | 1 | 6.5 | 24 | 2.1 | 0.07 | 0.42 | | | 0.03 | 714.24 | 1.31 | 723.89 | | |
| 21 | Tee from Bypass Run | 1 | 6.5 | 24 | 2.1 | 0.07 | 0.6 | | | 0.04 | 714.24 | 0.00 | 723.93 | | |
| 22 | Outlet Gate | 1 | 6.5 | 16 | 4.7 | 0.34 | 0.19 | | | 0.06 | 714.24 | 0.00 | 723.99 | | |
| 23 | Pipe Bend | 1 | 6.5 | 16 | 4.7 | 0.34 | 0.2 | | | 0.07 | 712.74 | -1.50 | 722.56 | | |
| 24 | New CCFRPM (Acorn Meter Run) | 1 | 6.5 | 16 | | | | 155 | 23 | 0.08 | 712.74 | 0.00 | 722.63 | | |
| 25 | New WSP (Acorn Meter Run) | 1 | 6.5 | 16 | | | | 140 | 17 | 0.07 | 712.74 | 0.00 | 722.70 | | |
| 26 | Inlet Structure Outlet | 1 | 6.5 | 16 | 4.7 | 0.34 | 1 | | | 0.34 | 712.74 | 0.00 | 723.04 | | |
| 27 | Inlet Structure Losses | 1 | 6.5 | 16 | 4.7 | 0.34 | 0.5 | | | 0.17 | 714.24 | 1.50 | 724.71 | | |
| 28 | Inlet Gate | 1 | 6.5 | 16 | 4.7 | 0.34 | 0.19 | | | 0.06 | 714.24 | 0.00 | 724.77 | | |
| 29 | **Inlet Structure Inlet (Acorn)** | 1 | 6.5 | 24 | 2.1 | 0.07 | 0.5 | | | 0.03 | 714.74 | 0.50 | **725.30** | **724.00** | **-1.30** |
| 30 | New RCP (Acorn) | 1 | 6.5 | 24 | | | | 140 | 12 | 0.01 | 714.98 | 0.24 | 725.55 | | |
| 31 | New MH AC-2 Outlet | 1 | 6.5 | 24 | 2.1 | 0.07 | 1 | | | 0.07 | 714.98 | 0.00 | 725.62 | | |
| 32 | 45 Degree Bend | 1 | 6.5 | 24 | 2.1 | 0.07 | 0.42 | | | 0.03 | 714.98 | 0.00 | 725.64 | | |
| 33 | **New MH AC-2 Inlet** | 1 | 6.5 | 24 | 2.1 | 0.07 | 0.5 | | | 0.03 | 714.98 | 0.00 | **725.68** | **725.90** | **0.22** |

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# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Acorn Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Acorn Flow Rate =** 4.2 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 34 | New RCP (Acorn) | 1 | 6.5 | 24 | | | | 140 | 31 | 0.02 | 715.60 | 0.62 | 726.31 | | |
| 35 | New MH AC-1 Outlet | 1 | 6.5 | 24 | 2.1 | 0.07 | 1 | | | 0.07 | 715.60 | 0.00 | 726.38 | | |
| 36 | 45 Degree Bend | 1 | 6.5 | 24 | 2.1 | 0.07 | 0.42 | | | 0.03 | 715.60 | 0.00 | 726.41 | | |
| 37 | **New MH AC-1 Inlet** | 1 | 6.5 | 27 | 1.6 | 0.04 | 0.5 | | | 0.02 | 715.60 | 0.00 | **726.43** | **726.60** | **0.17** |
| 38 | RCP (Acorn) | 1 | 6.5 | 27 | | | | 110 | 25 | 0.01 | 716.30 | 0.70 | 727.14 | | |
| 39 | Acorn Manhole #1 Outlet | 1 | 6.5 | 27 | 1.6 | 0.04 | 1 | | | 0.04 | 716.30 | 0.00 | 727.18 | | |
| 40 | **Acorn Manhole #1 Inlet** | 1 | 6.5 | 27 | 1.6 | 0.04 | 0.5 | | | 0.02 | 716.30 | 0.00 | **727.20** | **728.30** | **1.10** |
| 41 | RCP (Acorn) | 1 | 6.5 | 27 | | | | 110 | 339 | 0.16 | 735.49 | 19.19 | 746.56 | | |
| 42 | Acorn Manhole #2 Exit | 1 | 6.5 | 27 | 1.6 | 0.04 | 1 | | | 0.04 | 735.49 | 0.00 | 746.60 | | |
| 43 | **Acorn Manhole #2 Inlet** | 1 | 6.5 | 27 | 1.6 | 0.04 | 0.5 | | | 0.02 | 735.49 | 0.00 | **746.62** | **747.50** | **0.88** |

**Data Source(s):**
John A. Carollo Engineers. (1960). City of Riverside, California Sewage Treatment Plant Additions T5. Phoenix, Arizona.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Acorn manhole invert was estimated at 12 ft bgs based on a photograph of the inside of the manhole.
Acorn manhole elevation was determined using 2012 topographic survey.

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# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 46.4 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses: Flow, Q (cfs) | Minor Losses: Diameter, D (in) | Minor Losses: Velocity, V (ft/s) | Minor Losses: $V^2/2g$ (ft) | Minor Losses: K | Pipe Friction: C | Pipe Friction: L (ft) | Head Loss: Delta h (ft) | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | *Acorn/Arlanza Combined Flow* | | *78.3* | | | | | | | | | | | | |
| | *Arlanza Flow* | | *71.8* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.5 | | | 0.30 | 705.70 | | **710.75** | **713.70** | **2.95** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 78.3 | 48 | | | | 110 | 21 | 0.06 | 705.75 | 0.05 | 709.75 | | |
| 5 | 90 Degree Bend | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.9 | | | 0.54 | 705.75 | 0.00 | 710.30 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 78.3 | 48 | | | | 110 | 79 | 0.23 | 705.94 | 0.19 | 710.72 | | |
| 7 | Meter Box Outlet | 1 | 78.3 | 48 | 6.2 | 0.60 | 1 | | | 0.60 | 705.94 | 0.00 | 711.32 | | |
| 8 | **Meter Box Inlet** | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.5 | | | 0.30 | 705.96 | 0.02 | **711.65** | **716.00** | **4.35** |
| 9 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 78.3 | 48 | | | | 110 | 491 | 1.45 | 707.10 | 1.14 | 714.23 | | |
| 10 | 35 Degree Bend | 1 | 78.3 | 48 | 6.2 | 0.60 | 0.4 | | | 0.24 | 707.10 | 0.00 | 714.47 | | |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 78.3 | 48 | | | | 110 | 129 | 0.38 | 707.40 | 0.30 | 715.15 | | |
| 12 | Exit - Acorn/Arlanza Junction Box | 1 | 78.3 | 48 | 6.2 | 0.60 | 1 | | | 0.60 | 707.40 | 0.00 | 715.75 | | |
| 13 | **Inlet - Acorn/Arlanza Junction Box** | 2 | 39.2 | 48 | 3.1 | 0.15 | 0.5 | | | 0.08 | 707.40 | 0.00 | **715.83** | **716.10** | **0.27** |
| 14 | Reinforced Concrete Pipe (RCP, Arlanza) | 2 | 35.9 | 48 | | | | 110 | 293 | 0.20 | 708.00 | 0.60 | 716.63 | | |
| 15 | Arlanza Junction Box Exit | 2 | 35.9 | 48 | 2.9 | 0.13 | 1 | | | 0.13 | 708.00 | 0.00 | 716.76 | | |
| 16 | **Arlanza Junction Box Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 708.00 | 0.00 | **716.96** | **717.50** | **0.54** |
| 17 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 5 | 0.01 | 708.02 | 0.02 | 716.99 | | |
| 18 | 30 Degree Bend | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.4 | | | 0.16 | 708.02 | 0.00 | 717.15 | | |
| 19 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 17 | 0.03 | 708.08 | 0.06 | 717.24 | | |
| 20 | 20 Degree Bend | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.25 | | | 0.10 | 708.08 | 0.00 | 717.34 | | |
| 21 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 86 | 0.16 | 708.39 | 0.31 | 717.81 | | |
| 22 | Arlanza Manhole #1 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 708.39 | 0.00 | 718.21 | | |
| 23 | **Arlanza Manhole #1 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 708.39 | 0.00 | **718.41** | **724.80** | **6.39** |
| 24 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 161 | 0.30 | 708.97 | 0.58 | 719.29 | | |
| 25 | 50 Degree Bend | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.7 | | | 0.28 | 708.97 | 0.00 | 719.56 | | |
| 26 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 24 | 0.04 | 709.05 | 0.09 | 719.70 | | |
| 27 | Arlanza Manhole #2 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 709.05 | 0.00 | 720.09 | | |
| 28 | **Arlanza Manhole #2 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 709.05 | 0.00 | **720.29** | **727.50** | **7.21** |
| 29 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 150 | 0.28 | 709.59 | 0.54 | 721.11 | | |
| 30 | Exit - Junction Box | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 709.59 | 0.00 | 721.51 | | |
| 31 | **Inlet - Junction Box** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 709.59 | 0.00 | **721.71** | **725.00** | **3.29** |
| 32 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 303 | 0.57 | 710.68 | 1.09 | 723.36 | | |
| 33 | 20 Degree Bend | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.25 | | | 0.10 | 710.68 | 0.00 | 723.46 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Arlanza Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Arlanza Flow Rate =** 46.4 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Delta h (ft) | | | | | |
| 34 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 495 | 0.93 | 712.46 | 1.78 | 726.17 | | |
| 35 | Arlanza Manhole #3 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 712.46 | 0.00 | 726.57 | | |
| 36 | **Arlanza Manhole #3 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 712.46 | 0.00 | **726.76** | **724.00** | **-2.76** |
| 37 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 428 | 0.80 | 713.32 | 0.86 | 728.42 | | |
| 38 | Arlanza Manhole #4 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 713.32 | 0.00 | 728.82 | | |
| 39 | **Arlanza Manhole #4 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 713.32 | 0.00 | **729.02** | **726.30** | **-2.72** |
| 40 | Reinforced Concrete Pipe (RCP, Arlanza) | 1 | 71.8 | 51 | | | | 110 | 428 | 0.80 | 714.06 | 0.74 | 730.56 | | |
| 41 | Arlanza Manhole #5 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 714.06 | 0.00 | 730.96 | | |
| 42 | **Arlanza Manhole #5 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 714.06 | 0.00 | **731.16** | **725.60** | **-5.56** |
| 43 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 21 | 0.04 | 715.67 | 1.61 | 732.81 | | |
| 44 | Tee from future Jurupa Run | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.6 | | | 0.24 | 715.67 | 0.00 | 733.04 | | |
| 45 | Tee from Bypass Run | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.6 | | | 0.24 | 715.67 | 0.00 | 733.28 | | |
| 46 | Outlet Gate | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.19 | | | 0.08 | 715.67 | 0.00 | 733.36 | | |
| 47 | Pipe Bend | 1 | 71.8 | 36 | 10.2 | 1.60 | 0.2 | | | 0.32 | 712.67 | -3.00 | 730.68 | | |
| 48 | New CCFRPM (Arlanza Meter Run) | 1 | 71.8 | 36 | | | | 155 | 45 | 0.24 | 712.67 | 0.00 | 730.92 | | |
| 49 | New WSP (Arlanza Meter Run) | 1 | 71.8 | 36 | | | | 140 | 16 | 0.10 | 712.67 | 0.00 | 731.03 | | |
| 50 | Inlet Structure Outlet | 1 | 71.8 | 36 | 10.2 | 1.60 | 1 | | | 1.60 | 712.67 | 0.00 | 732.63 | | |
| 51 | Inlet Structure Losses | 1 | 71.8 | 48 | 5.7 | 0.51 | 0.5 | | | 0.25 | 715.67 | 3.00 | 735.88 | | |
| 52 | Inlet Gate | 1 | 71.8 | 48 | 5.7 | 0.51 | 0.19 | | | 0.10 | 715.67 | 0.00 | 735.98 | | |
| 53 | **Inlet Structure Inlet (Arlanza)** | 1 | 71.8 | 48 | 5.7 | 0.51 | 0.5 | | | 0.25 | 715.67 | 0.00 | **736.23** | **731.00** | **-5.23** |
| 54 | RCP (Arlanza) | 1 | 71.8 | 51 | | | | 110 | 156 | 0.29 | 718.46 | 2.79 | 739.32 | | |
| 55 | Arlanza Manhole #6 Outlet | 1 | 71.8 | 51 | 5.1 | 0.40 | 1 | | | 0.40 | 718.46 | 0.00 | 739.72 | | |
| 56 | **Arlanza Manhole #6 Inlet** | 1 | 71.8 | 51 | 5.1 | 0.40 | 0.5 | | | 0.20 | 718.46 | 0.00 | **739.92** | **730.00** | **-9.92** |

**Data Source(s):**
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.
Arlanza Pipeline: John Carollo Engineers. (1990). City of Riverside, CA Department of Public Works Water Quality Control Plant 1990 Tertiary Expansion. Phoenix, AZ.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
Acorn/Arlanza Junction Box invert elevation and manhole elevation were found on C14 (Brown and Caldwell 1996).
Arlanza Junction Box invert elevation and manhole elevation were determined on C15 (Brown and Caldwell 1996)
Arlanza Manhole #3 invert elevation was found on drawing Y-6 (John Carollo Engineers, 1990).
The average slope from the Arlanza junction box (708.0 ft invert elevation) and the Arlanza Manhole #3 (717.25 invert elevation) was determined to be 0.0074 ft/ft, using the length between the Arlanza junction box and the Arlanza Manhole #3.
Arlanza Manholes #1- #3 invert elevations were determined using a slc 0.00359 ft/ft and pipe lengths between each manhole.
The slope between Arlanza Manhole #3 and Arlanza Manhole #5 was determined 0.002 ft/ft
Arlanza Manhole #4 was determined using the slope between Arlanza Manhole #3 and Arlanza Manhole #5 and the pipe length between Arlanza Manhole #3 and Arlanza Manhole #4.

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 48.2 MGD
**Checked by:** BCP Date: 9/30/2014

| Point No. | Item | Qty | Minor Losses | | | | | Pipe Friction | | Head Loss | Invert Elevation (ft) | ΔElevation (ft) | Hydraulic Grade Line (ft) | Rim Elevation (ft) | Freeboard (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Flow, Q (cfs) | Diameter, D (in) | Velocity, V (ft/s) | $V^2/2g$ (ft) | K | C | L (ft) | Δh (ft) | | | | | |
| | *Rubidoux, Jurupa, Santa Ana (Riverside/Hillside) Combined Flow* | | *92.4* | | | | | | | | | | | | |
| | *Santa Ana (Riverside/Hillside) Flow* | | *74.6* | | | | | | | | | | | | |
| 1 | **Headworks Water Surface Elevation** | | | | | | | | | | | | **709.64** | **713.70** | **4.06** |
| 2 | Exit from Headworks Junction | | | | | | | | | | | | 710.45 | | |
| 3 | **Inlet to Headworks Junction** | 2 | 46.2 | 42 | 4.8 | 0.36 | 0.5 | | | 0.18 | 705.70 | | **710.63** | **713.70** | **3.07** |
| 4 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 46.2 | 42 | | | | 125 | 28 | 0.05 | 705.87 | 0.17 | 710.85 | | |
| 5 | 35 Degree Bend | 2 | 46.2 | 42 | 4.8 | 0.36 | 0.4 | | | 0.14 | 705.87 | 0.00 | 710.99 | | |
| 6 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 46.2 | 42 | | | | 125 | 200 | 0.34 | 707.08 | 1.21 | 712.54 | | |
| 7 | 35 Degree Bend | 2 | 46.2 | 42 | 4.8 | 0.36 | 0.4 | | | 0.14 | 707.08 | 0.00 | 712.68 | | |
| 8 | Reinforced Concrete Pipe (RCP, Headworks) | 2 | 46.2 | 42 | | | | 125 | 16 | 0.03 | 707.18 | 0.10 | 712.80 | | |
| 9 | Exit from Junction | 2 | 46.2 | 42 | 4.8 | 0.36 | 1 | | | 0.36 | 707.18 | 0.00 | 713.16 | | |
| 10 | **Inlet to Junction** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 707.18 | 0.00 | **713.44** | **717.00** | **3.56** |
| 11 | Reinforced Concrete Pipe (RCP, Headworks) | 1 | 74.6 | 48 | | | | 125 | 10 | 0.02 | 707.30 | 0.12 | 713.57 | | |
| 12 | CCFRPM Pipe (Santa Ana) | 1 | 74.6 | 48 | | | | 155 | 35 | 0.05 | 707.70 | 0.40 | 714.03 | | |
| 13 | Manhole #1 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 707.70 | 0.00 | 714.58 | | |
| 14 | **Manhole #1 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 707.70 | 0.00 | **714.85** | **716.20** | **1.35** |
| 15 | New CCFRPM (Santa Ana) | 1 | 74.6 | 36 | | | | 155 | 12 | 0.07 | 707.99 | 0.29 | 715.21 | | |
| 16 | Tee from Bypass Run | 1 | 74.6 | 36 | 10.6 | 1.73 | 0.6 | | | 1.04 | 707.99 | 0.00 | 716.25 | | |
| 17 | Outlet Gate | 1 | 74.6 | 36 | 10.6 | 1.73 | 0.19 | | | 0.33 | 707.99 | 0.00 | 716.58 | | |
| 18 | Pipe Bend, vertical | 1 | 74.6 | 36 | 10.6 | 1.73 | 0.2 | | | 0.35 | 704.99 | -3.00 | 713.93 | | |
| 19 | New CCFRPM (Santa Ana Meter Run) | 1 | 74.6 | 36 | | | | 155 | 71 | 0.41 | 704.99 | 0.00 | 714.34 | | |
| 20 | New WSP (Santa Ana Meter Run) | 1 | 74.6 | 36 | | | | 140 | 38 | 0.27 | 704.99 | 0.00 | 714.61 | | |
| 21 | Inlet Structure Outlet | 1 | 74.6 | 36 | 10.6 | 1.73 | 1 | | | 1.73 | 704.99 | 0.00 | 716.34 | | |
| 22 | Inlet Structure Losses | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 707.99 | 3.00 | 719.61 | | |
| 23 | Inlet Gate | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.19 | | | 0.10 | 707.99 | 0.00 | 719.72 | | |
| 24 | **Inlet Structure Inlet (Santa Ana)** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 707.99 | 0.00 | **719.99** | **717.80** | **-2.19** |
| 25 | CCFRPM Pipe (Santa Ana) | 1 | 74.6 | 48 | 5.9 | 0.55 | | 155 | 31 | 0.05 | 708.06 | 0.07 | 720.10 | | |
| 26 | Manhole #2 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 708.06 | 0.00 | 720.65 | | |
| 27 | **Manhole #2 Inlet** | 1 | 74.6 | 41 | 8.1 | 1.03 | 0.5 | | | 0.51 | 708.06 | 0.00 | **721.16** | **714.60** | **-6.56** |
| 28 | CCFRPM Pipe (Santa Ana) | 1 | 65.0 | 41 | | | | 155 | 13 | 0.03 | 708.10 | 0.04 | 721.23 | | |
| 29 | Manhole #3 Exit | 1 | 65.0 | 41 | 7.1 | 0.78 | 1 | | | 0.78 | 708.10 | 0.00 | 722.02 | | |
| 30 | **Manhole #3 Inlet** | 1 | 65.0 | 41 | 7.1 | 0.78 | 0.5 | | | 0.39 | 708.10 | 0.00 | **722.41** | **714.90** | **-7.51** |
| 31 | CCFRPM Pipe (Santa Ana) | 1 | 65.0 | 41 | | | | 155 | 95 | 0.23 | 708.26 | 0.16 | 722.79 | | |
| 32 | Manhole #4 Exit | 1 | 65.0 | 41 | 7.1 | 0.78 | 1 | | | 0.78 | 708.26 | 0.00 | 723.57 | | |
| 33 | **Manhole #4 Inlet** | 1 | 65.0 | 41 | 7.1 | 0.78 | 0.5 | | | 0.39 | 708.26 | 0.00 | **723.97** | **715.50** | **-8.47** |
| 34 | CCFRPM Pipe (Santa Ana) | 1 | 65.0 | 41 | | | | 155 | 13 | 0.03 | 708.29 | 0.03 | 724.03 | | |
| 35 | Manhole #5 Exit | 1 | 65.0 | 41 | 7.1 | 0.78 | 1 | | | 0.78 | 708.29 | 0.00 | 724.81 | | |

URS

# PROPOSED CONDITIONS

**Project:** City of Riverside Regional Water Quality Control Plant, Influent Flow Metering
**Calculation:** Hydraulic Profile for Santa Ana (Riverside/Hillside) Interceptor
**Design Case:** Ultimate Future Design Flow - PWWF
**Calculated by:** NTC Date: 9/29/2014 **Santa Ana Flow Rate =** 48.2 MGD
**Checked by:** BCP Date: 9/30/2014

| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36 | **Manhole #5 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 708.29 | 0.00 | **725.08** | **716.30** | **-8.78** |
| 37 | CCFRPM Pipe (Santa Ana) | 1 | 74.6 | 48 | | | | 155 | 78 | 0.11 | 708.43 | 0.14 | 725.33 | | |
| 38 | Manhole #6 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 708.43 | 0.00 | 725.88 | | |
| 39 | **Manhole #6 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 708.43 | 0.00 | **726.15** | **719.40** | **-6.75** |
| 40 | CCFRPM Pipe (Santa Ana) | 1 | 74.6 | 48 | | | | 155 | 55 | 0.08 | 708.53 | 0.10 | 726.33 | | |
| 41 | Manhole #7 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 708.53 | 0.00 | 726.88 | | |
| 42 | **Manhole #7 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 708.53 | 0.00 | **727.15** | **719.40** | **-7.75** |
| 43 | CCFRPM Pipe (Santa Ana) | 1 | 74.6 | 48 | | | | 155 | 216 | 0.31 | 708.92 | 0.39 | 727.85 | | |
| 44 | Manhole #8 Exit | 1 | 74.6 | 48 | 5.9 | 0.55 | 1 | | | 0.55 | 708.92 | 0.00 | 728.40 | | |
| 45 | **Manhole #8 Inlet** | 1 | 74.6 | 48 | 5.9 | 0.55 | 0.5 | | | 0.27 | 708.92 | 0.00 | **728.67** | **722.20** | **-6.47** |

**Data Source(s):**
Riverside/Hillside Pipeline: Brown and Caldwell. (2011). Riverside California Public Works Department Santa Ana River Trunk Sewer Replacement Project - Phase 1. San Diego, CA.
Headworks Pipeline: Brown and Caldwell. (1996). City of Riverside Regional Water Quality Control Plant Headworks Project. Irvine, CA.

**Note(s):**
Headworks WSEL based on 70 MGD of flow according to plant design hydraulic profile by Brown & Caldwell, 1996.
There is no junction elevation rise between 42" and 48" pipe based on S-2054 Sheet 6 of 33 (Brown & Caldwell 2011).
The length of pipes between the manholes were determined using S-2054 Sheet 6 of 33 (Brown and Caldwell 2011).
Invert elevations were determined using (Brown & Caldwell 1996, 2011).
The Riverside/Hillside CCFRPM pipeline was constructed in 2012; based on the pipe's age, a Hazen Williams coefficient of 155 was chosen.
The dual 42" RCPs to headworks were constructed in 1996; based on the pipe's age, a Hazen Williams coefficient of 125 was chosen.

# APPENDIX C – COST ESTIMATE

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**ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COSTS**
**30% PRELIMINARY DESIGN LEVEL**

Project: Riverside Regional Water Quality Control Plant Influent Flow Metering Project
Client : City of Riverside, Public Works Department, Water Quality Control Plant Engineering Section
Engineer : URS Corporation
Rev : October 6, 2014

| Item | Description | Estimated Quantity | Unit | Unit Price | Total Price | Source |
|---|---|---|---|---|---|---|
| **1** | **General Contractor General Conditions** | | | **Subtotal:** | **$300,000** | |
| | Mobilization, Demobilization, Cleanup | 1 | LS | $100,000 | $100,000 | [8] |
| | Other General Conditions | 1 | LS | $200,000 | $200,000 | [8] |
| **2** | **Santa Ana (Riverside-Hillside) Metering System** | | | **Subtotal:** | **$806,100** | |
| | Bypass Pumping (30 days) | 1 | LS | $180,000 | $180,000 | [7] |
| | Demo and clearing including disposal and pipe | 1 | LS | $20,000 | $20,000 | [8] |
| | Excavation including hauling and shoring | 1,900 | CY | $30 | $57,000 | [8] |
| | Backfill | 1,300 | CY | $50 | $65,000 | [8] |
| | 6-inch AC Pavement/class 2 base | 500 | SF | $8 | $4,000 | [8] |
| | 48" CCFRPM Pipe and Fitting (Bypass Pipe) | 110 | LF | $400 | $44,000 | [1] |
| | 36" CCFRPM Pipe and Fittings (Meter Pipe) | 71 | LF | $300 | $21,300 | [1] |
| | 36" Steel Pipe and Fittings (Meter Pipe) | 38 | LF | $600 | $22,800 | [5] |
| | Inlet Vault and hatches | 1 | LS | $60,000 | $60,000 | [8] |
| | Meter vault and hatches | 1 | LS | $25,000 | $25,000 | [8] |
| | Outlet vault and hatches | 1 | LS | $20,000 | $20,000 | [8] |
| | T-lock lining for wet vaults | 1 | LS | $40,000 | $40,000 | [8] |
| | Fabricated 316SS sluice gates (48"x48") | 1 | LS | $30,000 | $30,000 | [4] |
| | Fabricated 316SS sluice gates (36"x36") | 2 | LS | $27,000 | $54,000 | [4] |
| | Misc. Metals | 1 | LS | $10,000 | $10,000 | [8] |
| | Painting and Coatings | 1 | LS | $15,000 | $15,000 | [8] |
| | Sump Pump (0.5 hp) and piping | 1 | LS | $5,000 | $5,000 | [6] |
| | 18" x 18" sump catch basin and grate | 1 | LS | $1,000 | $1,000 | [8] |
| | Electrical | 1 | LS | $40,000 | $40,000 | [8] |
| | Ventilation | 1 | LS | $7,000 | $7,000 | [8] |
| | Instrumentation Including 36" Meter and SCADA | 1 | LS | $60,000 | $60,000 | [3] & [7] |
| | Miscellaneous appurtenances | 1 | LS | $25,000 | $25,000 | [8] |
| **3** | **Acorn Metering System** | | | **Subtotal:** | **$557,000** | |
| | Bypass Pumping (24 days) | 1 | LS | $45,000 | $45,000 | [7] |
| | Demo and clearing including disposal and pipe | 1 | LS | $15,000 | $15,000 | [8] |
| | Excavation including hauling and shoring | 1,700 | CY | $30 | $51,000 | [8] |
| | Backfill | 1,200 | CY | $50 | $60,000 | [8] |
| | 6-inch AC Pavement/class 2 base | 1,000 | SF | $8 | $8,000 | [8] |
| | 24" CCFRPM Pipe and Fitting (Bypass Pipe) | 40 | LF | $200 | $8,000 | [1] |
| | 16" CCFRPM Pipe and Fittings (Meter Pipe) | 23 | LF | $140 | $3,220 | [1] |
| | 16" Steel Pipe and Fittings (Meter Pipe) | 17 | LF | $300 | $5,100 | [2] |
| | 24" RCP (New Pipe Alignment) | 71 | LF | $200 | $14,200 | [2] |
| | Sewer Manholes | 3 | LS | $7,000 | $21,000 | [8] |
| | Inlet Vault and hatches | 1 | LS | $50,000 | $50,000 | [8] |
| | Meter vault and hatches | 1 | LS | $30,000 | $30,000 | [8] |
| | Outlet vault and hatches | 1 | LS | $15,000 | $15,000 | [8] |
| | T-lock lining for wet vaults and sewer manholes | 1 | LS | $55,000 | $55,000 | [8] |
| | Fabricated 316SS sluice gates (36" x36") | 1 | LS | $27,000 | $27,000 | [4] |
| | Fabricated 316SS sluice gates (18" x18") | 2 | LS | $22,000 | $44,000 | [4] |
| | Misc. Metals | 1 | LS | $10,000 | $10,000 | [8] |
| | Painting and Coatings | 1 | LS | $13,000 | $13,000 | [8] |
| | Sump Pump (0.5 hp) | 1 | LS | $1,500 | $1,500 | [6] |
| | 18" x 18" sump catch basin and grate | 1 | LS | $1,000 | $1,000 | [8] |
| | Electrical | 1 | LS | $30,000 | $30,000 | [8] |
| | Ventilation | 1 | LS | $5,000 | $5,000 | [8] |
| | Instrumentation Including 16" Meter and SCADA | 1 | LS | $30,000 | $30,000 | [3] & [8] |
| | Miscellaneous appurtenances | 1 | LS | $15,000 | $15,000 | [8] |

URS

| Item | Description | Estimated Quantity | Unit | Unit Price | Total Price | Source |
|---|---|---|---|---|---|---|
| **4** | **Arlanza Metering System** | | | **Subtotal:** | **$805,600** | |
| | Bypass Pumping (30 days) | 1 | LS | $180,000 | $180,000 | [7] |
| | Demo and clearing including disposal and pipe | 1 | LS | $20,000 | $20,000 | [8] |
| | Excavation including hauling and shoring | 1,900 | CY | $30 | $57,000 | [8] |
| | Backfill | 1,400 | CY | $50 | $70,000 | [8] |
| | 6-inch AC Pavement/class 2 base | 2,000 | SF | $8 | $16,000 | [8] |
| | 48" CCFRPM Pipe and Fitting (Bypass Pipe) | 65 | LF | $400 | $26,000 | [1] |
| | 36" CCFRPM Pipe and Fittings (Meter Pipe) | 45 | LF | $300 | $13,500 | [1] |
| | 36" Steel Pipe and Fittings (Meter Pipe) | 16 | LF | $600 | $9,600 | [5] |
| | Inlet Vault and hatches | 1 | LS | $60,000 | $60,000 | [8] |
| | Meter vault and hatches | 1 | LS | $25,000 | $25,000 | [8] |
| | Outlet vault and hatches | 1 | LS | $40,000 | $40,000 | [8] |
| | T-lock lining for wet vaults | 1 | LS | $40,000 | $40,000 | [8] |
| | Fabricated 316SS sluice gates (48"x48") | 1 | LS | $30,000 | $30,000 | [4] |
| | Fabricated 316SS sluice gates (36"x36") | 2 | LS | $27,000 | $54,000 | [4] |
| | Misc. Metals | 1 | LS | $10,000 | $10,000 | [8] |
| | Painting and Coatings | 1 | LS | $15,000 | $15,000 | [8] |
| | Sump Pump (0.5 hp) | 1 | LS | $1,500 | $1,500 | [6] |
| | 18" x 18" sump catch basin and grate | 1 | LS | $1,000 | $1,000 | [8] |
| | Electrical | 1 | LS | $40,000 | $40,000 | [8] |
| | Ventilation | 1 | LS | $7,000 | $7,000 | [8] |
| | Instrumentation Including 36" Meter and SCADA | 1 | LS | $60,000 | $60,000 | [8] |
| | Miscellaneous appurtenances | 1 | LS | $30,000 | $30,000 | [8] |
| **5** | **Start-up Testing and Training** | | | **Subtotal:** | **$20,000** | |
| | | 1 | LS | $20,000 | $20,000 | [8] |
| **6** | **Prepare Operation and Maintenance Manuals** | | | **Subtotal:** | **$10,000** | |
| | | 1 | LS | $10,000 | $10,000 | [8] |
| **7** | **Prepare and Maintain Record Drawings** | | | **Subtotal:** | **$10,000** | |
| | | 1 | LS | $10,000 | $10,000 | [8] |
| | | | | **SUBTOTAL - DIRECT COSTS ONLY** | **$2,508,700** | |
| **8** | **Bonding and All Risk Insurance** | | | 2% | $50,200 | |
| | | | | **Subtotal:** | **$2,558,900** | |
| **9** | **Contingency** | | | 30% | $752,700 | |
| | | | | **Subtotal:** | **$3,261,400** | |
| **10** | **General Contractor Overhead and Profit** | | | 15% | $489,300 | |
| | | | | **Subtotal:** | **$3,750,700** | |
| **11** | **Escalation to Construction Midpoint** | | | 3% | $112,600 | |
| | | | | **Subtotal:** | **$3,863,300** | |
| **12** | **Owner's Reserve for Change Orders** | | | 10% | $386,400 | |
| | | | | **TOTAL ESTIMATED CONSTRUCTION COST** | **$4,249,700** | |

**Notes:**
1. Minor discrepancies in subtotals are possible due to rounding.
2. This estimate is based on the Project Preliminary Design Report prepared by URS dated 10/06/2014.

**Sources:**
[1] Vendor quote, Hobas Pipe - CCFRPM (2013).
[2] Caltrans Contract Cost Data Book (2012).
[3] Vendor quote, Siemens (2013).
[4] Vendor quote, CS-AMSCO (2013).
[5] Vendor quote, Northwest Pipe (2013).
[6] Vendor quote, Pump Man/Pump Source USA (2013).
[7] Vendor quote, Rain for Rent (2013).
[8] Experience from previous projects.

Attachment B

# DETAILED COST ESTIMATE

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**CITY OF RIVERSIDE PUBLIC WORKS**

**UPDATED INTEGRATED MASTER PLAN FOR WASTEWATER COLLECTION AND TREATMENT FACILITIES PROJECT**

| | | | |
|---|---|---|---|
| **TASK :** | 7 - CIP and Financial Plan and User Rates and Fees | **ESTIMATE PREPARATION DATE :** | 3/1/2016 |
| **JOB # :** | 10495A.00 | **PREPARED BY :** | TetraTech |
| **LOCATION :** | Riverside, CA | **REVIEWED BY :** | |
| **TITLE :** | Levee Rehabilitation Phase II | | |

| ITEM NO. | DESCRIPTION | QTY | UNIT | UNIT COST | SUBTOTAL | TOTAL |
|---|---|---|---|---|---|---|
| | | | | | | |
| 1 | **Mobilization** | | | | | |
| | Mobilization | 1.0 | LS | $85,654 | $85,700 | |
| | Total | | | | | $85,700 |
| | | | | | | |
| 2 | **Site Preparation** | | | | | |
| | Site Preparation | 1 | LS | $41,783 | $41,800 | |
| | Total | | | | | $41,800 |
| | | | | | | |
| 3 | **Earthwork** | | | | | |
| | Structural Excavation | 81925 | CY | $30 | $2,457,800 | |
| | Grouted Riprap | 2102 | CY | $200 | $420,400 | |
| | Total | | | | | $2,878,200 |
| | | | | | | |
| 4 | **Permitting** | | | | | |
| | Permitting | 1.0 | LS | $1,000,000 | $1,000,000 | |
| | Total | | | | | $1,000,000 |
| | | | | | | |
| | | | | | | |
| | **ITEM NO. 1,2,3,and 4 SUBTOTAL** | | | | | **$4,119,121** |
| | | | | | | |
| 5 | **Allowances** | | | | | |
| | | | | | | |
| | Construction Difficulty | 5 | % | | $205,956 | |
| | Total | | | | | $206,000 |
| | | | | | | |
| | **SUBTOTAL** | | | | | **$4,325,121** |
| | | | | | | |
| | Estimating Contingency | 30 | % | | | $1,298,000 |
| | **SUBTOTAL** | | | | | **$5,623,121** |
| | | | | | | |
| | General Conditions | 10 | % | | | $562,312 |
| | **SUBTOTAL** | | | | | **$6,185,434** |
| | | | | | | |
| | General Contractor Overhead & Profit | 15 | % | | | $646,659 |
| | **SUBTOTAL** | | | | | **$6,269,780** |
| | | | | | | |
| | Escalation to June 2018 | 3 | % | | | $188,093 |
| | **SUBTOTAL** | | | | | **$6,457,874** |
| | | | | | | |
| | Sales Tax on 50% of Subtotal Above | 8.75 | % | | | $549,000 |
| | **CONSTRUCTION COST SUBTOTAL** | | | | | **$6,818,780** |
| | | | | | | |
| | Engineering, Management, and Legal | 30 | % | | | $2,046,000 |
| | Phave II Preliminary Design Report Allowance | | | | | $250,000 |
| | | | | | | |
| | **PROJECT COST** | | | | | **$9,114,800** |

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 5

## Wastewater Treatment System

## Primary Treatment

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City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 5: PRIMARY TREATMENT

FINAL | June 2019

City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 5: PRIMARY TREATMENT

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:19:20-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 5

# PRIMARY TREATMENT

## 5.1 Purpose

The purpose of this Chapter is to describe upgrades to the existing primary sludge and scum pump stations associated with the ACT treatment train (Plant 2) and to estimate the overall capital cost of improvements through 2037. This Chapter includes recommendations based on initial investigations. However, final decisions about specific equipment types or layouts should be determined during the preliminary and final design stages of each associated project.

## 5.2 Conclusions and Recommendations

- For estimated sludge production through 2037, 300 gpm of reliable pumping capacity is required.
- Existing pump stations are highly congested and revised layouts should be considered to improve operation and maintenance conditions within the stations.
- A scum recirculation/mixing system is recommended for the primary sludge/scum pump stations to mitigate clogging.
- Submersible chopper pumps are recommended for scum services. The recommended pump configuration for the ACT treatment train (Plant 2) scum pumps is one duty pump at each station and one uninstalled standby.
- Two 150-gpm rotary lobe pumps (one duty and one standby) are recommended for each ACT treatment train (Plant 2) primary sludge pump station.

## 5.3 Background

There are two existing sludge and scum pump stations. The first was installed in 1967 and serves ACT treatment train (Plant 2) primaries 1 and 2. The second was installed in 1982 and serves ACT treatment train (Plant 2) primaries 3 and 4. Each station contains two sludge and one scum pump. Existing pumps are all progressive cavity and the layouts are highly confined. Primary Sludge Pump Station 2 (1982) is more congested than Primary Sludge Pump Station 1 (1967). Table 5.1 presents the design criteria for existing primary facilities on the ACT treatment train (Plant 2) side.

Table 5.1 Existing Primary Facilities

| Description | Value |
|---|---|
| Primary Clarifiers | |
| Number | 4 |
| Diameter, ft. | 95 |
| Overflow Rate, gal/day/sq. ft. | 705 |
| Detention Time, hrs | 2.54 |
| Side Water Depth, ft. | 9 |
| Sludge Pumps | |
| Number | 4 |
| Type | Progressive Cavity |
| Size, gpm/each | 150 |

Table 5.1 Existing Primary Facilities (continued)

| Description | Value |
|---|---|
| Scum Pumps | |
| Number | 2 |
| Type | Progressive Cavity |
| Size, gpm/each | 150 |

## 5.4 Primary Sludge Pumps

There are four existing ACT treatment train (Plant 2) primaries; two are serviced by Primary Sludge Pump Station 1 (1967), and two are serviced by Primary Sludge Pump Station 2 (1982). Each pump station has two sludge pumps and one scum pump installed. The sludge pumps are capable of pumping 150 gpm of sludge each and operate in a cyclical fashion pulling sludge from each clarifier for a specified amount of time.

Based on process modeling, approximately 44,600 ppd of sludge can be generated from the entire ACT treatment train (Plant 2) primary clarifier system assuming the ACT treatment train (Plant 2) side receives 20 mgd of flow on an average annual basis. For this evaluation, two separate thickening percentages were used to determine preliminary sludge pump capacity requirements. Assuming one sludge pump from each pump station is operating at a time and the entire 44,600 ppd of sludge is evenly distributed between all four primary clarifiers, it would take 30 min/hr to pump the required sludge volume at 2.5-percent thickened sludge, or 16 min/hr to pump the required sludge volume at 4.5-percent thickened sludge. This means that the existing pump system with four sludge pumps, two in each station, one duty and one standby per station operating at 150 gpm each (300 gpm total in service), is sufficient to meet the sludge production requirements throughout the planning period. Table 5.2 shows preliminary design criteria for each sludge thickening scenario.

Table 5.2 Preliminary Sludge Pump Design Criteria

| Parameter | 2.5% Thickened Sludge | 4.5% Thickened Sludge |
|---|---|---|
| Sludge Production, lbs/d | 44,600 | |
| Sludge Production, gpd | 214,000 | 119,000 |
| Run Time at 300 gpm | 30 mins/hr | 16 mins/hr |
| Design per Station (duty, standby) | 2 pumps (1+1), 150 gpm each | |
| Total Installed Pump Capacity | 600 gpm | |

The existing pumps are progressive cavity pumps, which are effective at pumping sludge; their dimensions require more space than other pump types. As a result, the pump and piping layout in the existing pump stations is congested. As congestion is the primary concern in these two stations, the main focus of this evaluation is space-saving methodologies. For this update to the Master Plan, rotary lobe sludge pumps were considered. During the preliminary design phase, additional alternative technologies may also be considered.

In comparison to progressive cavity pumps for sludge pumping, single-lobe rotary pumps offer the same capabilities but in a more compact footprint. Figure 5.1 shows the existing pumping layout with two progressive cavity sludge pumps and one progressive cavity scum pump in each station. Figure 5.2 shows an alternative pumping layout with only two rotary lobe sludge pumps (scum pumps are evaluated separately below). By comparison, rotary lobe pumps result in a more

spacious layout. Furthermore, maintenance of the lobe and stator is not limited with the new rotary lobe designs because the front of the pump, which is unhindered in the proposed layout, can be removed for easy access to the lobes. Grinders are recommended upstream for pump protection.

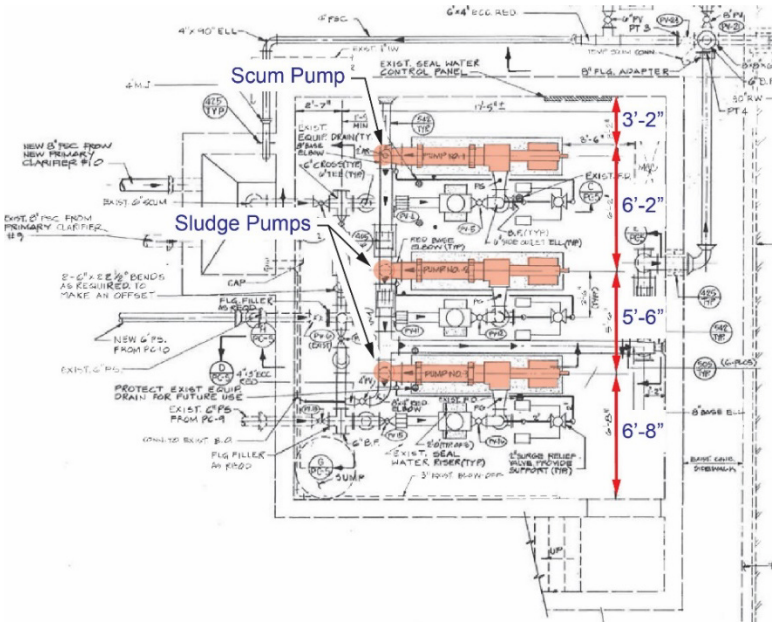


Figure 5.1 Existing Primary Sludge/Scum Pump Station

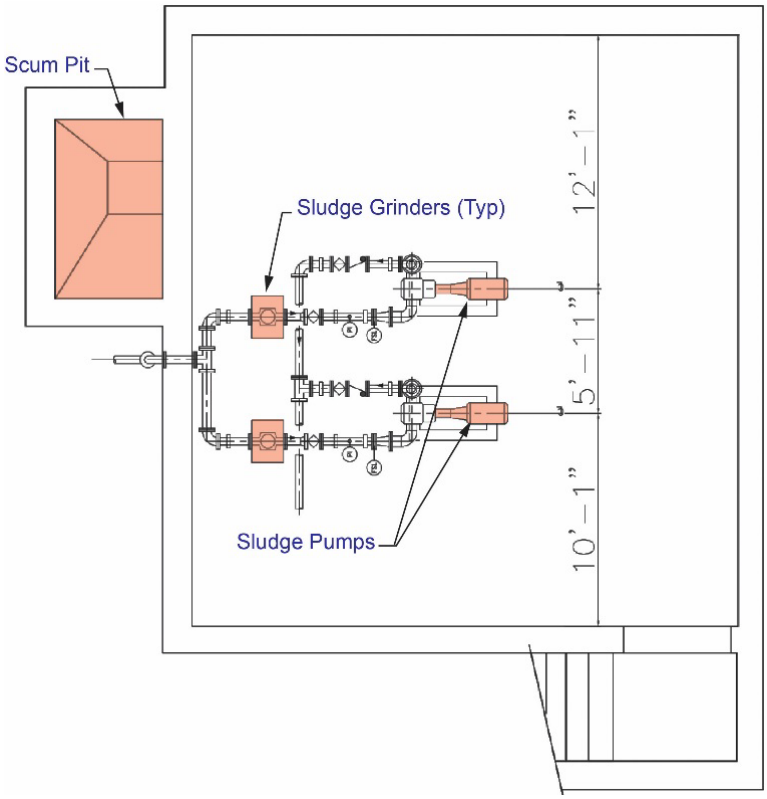


Figure 5.2 Proposed Typical Sludge Pump Layout

At the June 28, 2017, workshop, the City decided that rotary lobes pumps were the recommended alternative. If necessary, this technology can be evaluated further during preliminary design.

### 5.4.1 Design Considerations

For this update to the Master Plan, a Vogelsang model single-lobe rotary pump was considered as an example for design consideration purposes. Table 5.3 presents design criteria for the pumps. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $0.6 million.

Table 5.3 Primary Sludge Pump Design Criteria

| Description | Value |
|---|---|
| Type | Rotary Lobe |
| Manufacturer/Model | Vogelsang VS136-140Q |
| Design Flow, gpm | 150 |
| Design Discharge Pressure, psi | 60 |
| Motor Size, hp | 10 |
| Maximum Pump Speed, rpm | 300 |

## 5.5 Primary Scum Pumps

Although each station has one progressive cavity scum pump, the capacity of each pump is unclear. It is assumed to be the same as the sludge pumps (150 gpm), but this should be verified during preliminary design. For comparison, the MBR treatment train (Plant 1) has a total of four horizontal chopper scum pumps (two duty and two standby) with in-line grinders at 230-gpm maximum flow each. The process model does not estimate scum production; further consideration should be given during preliminary design for scum pump capacity for future conditions. For this update to the Master Plan, alternative technologies were considered including horizontal chopper, submersible chopper, and rotary lobe pumps.

### 5.5.1 Horizontal Chopper

A typical layout of the horizontal chopper pump for the ACT treatment train (Plant 2) scum pumps is shown on Figure 5.3. These pumps are smaller than the progressive cavity pumps currently installed, but do take up space in the Primary Sludge/Scum Pump Stations. Table 5.4 shows the advantages and disadvantages of this type of pump. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $0.6 million.

Table 5.4 Summary of Horizontal Chopper Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Similar design to those installed in the MBR treatment train (Plant 1) | Requires space in the Primary Sludge/Scum pump stations |
| Dry-pit installation improves maintenance accessibility | Suction pipe is prone to clogging |
| Layout can tie-in to existing piping with limited pump station structural modification | |

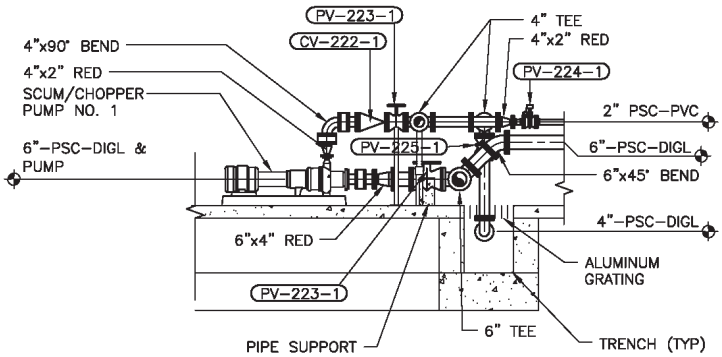


Figure 5.3 Typical Horizontal Chopper Installation

### 5.5.2 Submersible Choppers

Submersible chopper pumps would be installed within the scum pit itself offering the obvious advantage of freeing space in the Primary Sludge/Scum pump stations. Due to limited scum pit area, an uninstalled standby is recommended for this alternative. The submersible chopper impeller breaks up scum against the impeller casing providing the chopping action. Discharge piping can be routed to connect to existing piping within the Primary Sludge/Scum pump stations, but this requires modification to the pump station structure (i.e., likely a roof penetration). Figure 5.4 shows a conceptual layout of a scum pump installation. Table 5.5 shows the advantages and disadvantages of this type of pump. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $0.8 million.

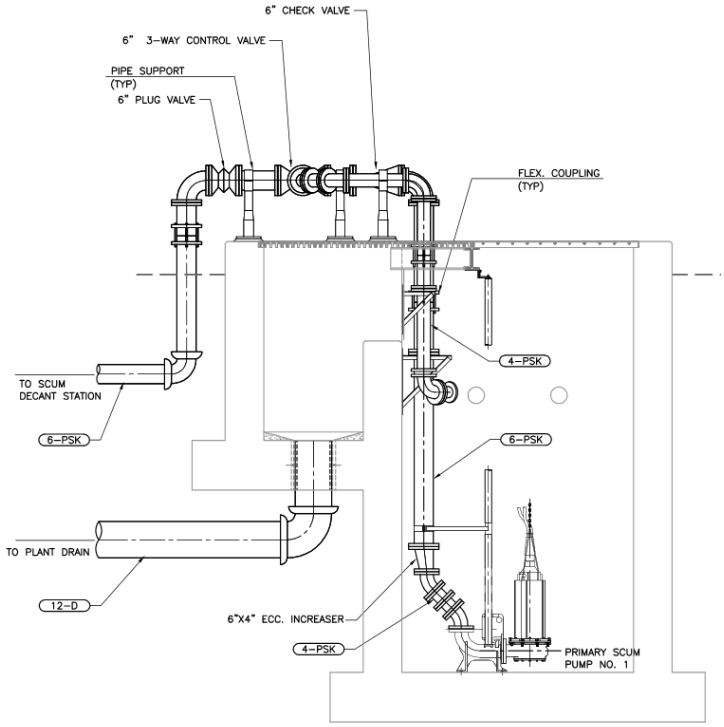


Figure 5.4 Typical Submersible Chopper Installation

Table 5.5 Summary of Submersible Chopper Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Successful in similar applications | Requires removal from the scum pit for maintenance |
| Frees space in the Primary Sludge/Scum pump stations | Different design with unfamiliar technology |
| No suction piping to clog | Tie-in to existing piping inside pump stations requires some structural modification |

### 5.5.3 Rotary Lobe

This alternative uses the same pumps as the primary sludge pumps described above. Instead of two pumps, a three pump configuration can be set up within each pump station. Although rotary lobe pumps are smaller than progressive cavity pumps, this alternative essentially maintains the existing piping layout with a different pump type. Rotary lobe pumps do offer an increased maintenance accessibility in comparison to progressive cavity pumps. Figure 5.5 shows a proposed layout with three rotary lobe pumps (one for scum, two for sludge). Table 5.6 shows the advantages and disadvantages of this type of pump for scum application. Planning level cost estimates indicate this alternative would have an associated project cost of approximately $0.7 million.

Table 5.6 Summary of Rotary Lobe Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Successful in similar applications | Requires space in Primary Sludge/Scum pump stations |
| Common standby with sludge pumps | Suction pipe is prone to clogging |
| Dry-pit installation improves maintenance accessibility | |
| Layout can tie-in to existing piping with limited pump station structural modification | |

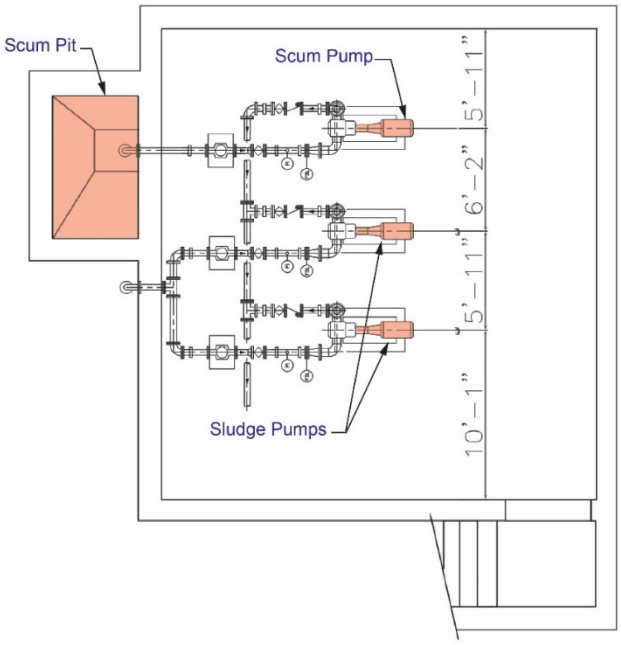


Figure 5.5 Proposed Rotary Lobe Layout

### 5.5.4 Scum Pit Recirculation/Mixing

Regardless of the scum pumping technology selected, if the scum is not completely mixed, all scum pumps will have a problem with clogging when the scum blanket (top layer of buoyant scum) hits the pump suction. For this reason, it is recommended to reconfigure existing scum discharge piping to provide recirculation within the scum pit as shown in Figure 5.6. After the scum pump begins pumping, the pump discharge is pumped back to the scum pit through a fluidizing jet stream to mix the settled scum blanket back into suspension. After a set time period, the recirculation valving turns off allowing normal discharge to the solids blending facility. With this recirculation methodology in place, the scum remains fluidized during normal pump operation preventing clogging.

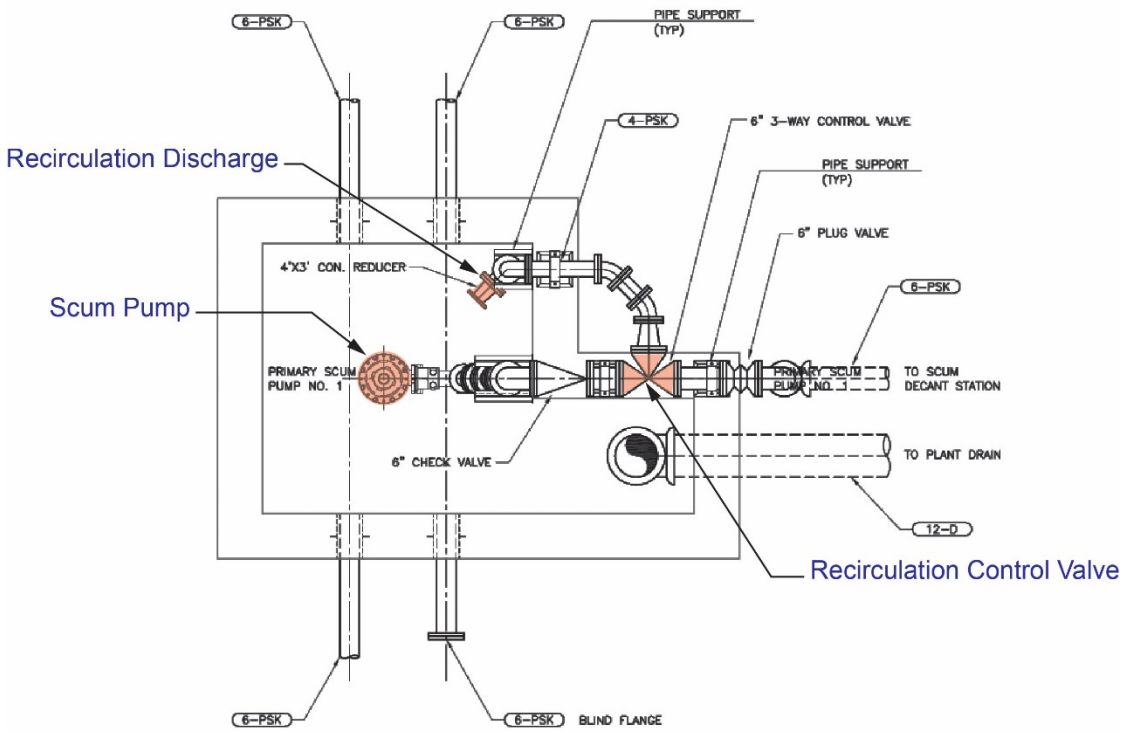


Figure 5.6 Typical Scum Pit Recirculation/Mixing Plan

### 5.5.5 Alternatives Comparison and Recommendation

Table 5.7 shows a comparison of the evaluated alternatives. For this update to the Master Plan, submersible chopper pumps are recommended for the scum application freeing up the most space in the Primary Sludge/Scum pump stations. It is recommended that an uninstalled standby pump be available. In addition, it is recommended that provisions be included for scum pit recirculation/mixing to provide a consistent fluid blend for the pumps.

Table 5.7 Comparison of Conveyance Alternatives

| Parameter | Horizontal Chopper | Submersible Chopper | Rotary Lobe |
|---|---|---|---|
| Pump Station Space Impact | Requires Space in the Primary Sludge/Scum Pump Station | Frees Space in the Primary Sludge/Scum Pump Station | Requires Space in the Primary Sludge/Scum Pump Station |
| Recirculation/Mixing Impact | Requires Piping Modification to Achieve Recirculation and Mixing | Recirculation and Mixing can be Built Into Pump Installation Design | Requires Piping Modification to Achieve Recirculation and Mixing |
| Structural Modification for Tie-In | Pump Station Structure Does Not Need to be Modified | A New Pipe Penetration is Required | Pump Station Structure Does Not Need to be Modified |
| Standby Requirements | Uninstalled Standby | Uninstalled Standby | Joint Standby with Sludge Alternatives |
| RWQCP Experience | Installed at MBR treatment train (Plant 1) | New Technology for the City | New Technology for the City |
| Project Cost | $0.6 million | $0.8 million | $0.7 million |

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 6

## Wastewater Treatment System

## Secondary Treatment

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City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 6: SECONDARY TREATMENT

FINAL | June 2019

City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 6: SECONDARY TREATMENT

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:20:42-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 6

# SECONDARY TREATMENT

## 6.1 Purpose

The purpose of this Chapter is to describe upgrades to the existing secondary treatment facility at the City RWQCP and estimate the overall capital cost of improvements through 2037. The improvements include:

1. Increasing the capacity of the MBR treatment train (Plant 1) to 32-mgd AADF.
2. Enhanced nutrient removal upgrades to meet future nutrient limits.

## 6.2 Conclusions

- The Phase II expansion includes the construction of a sixth aeration basin and will increase the MBR treatment train (Plant 1) capacity by 6 mgd. The cost of the Phase II modifications is estimated at $20,517,500.
- The replacement of existing membranes with Fibreplate™ membranes by Anaergia Inc., will increase the MBR treatment train (Plant 1) membrane tank's buildout capacity to 81 mgd. However, a complete retrofit of the membrane tanks will be necessary to install the Fibreplate™ membranes. This would require the demolition of the existing equipment which still have a considerable amount of useful life left. Additionally, to increase the MBR treatment train (Plant 1) capacity above 32 mgd would require new piping, utilization of the ACT treatment train (Plant 2) aeration basins, and several other minor modifications.
- To remove nitrogen and phosphorous in the MBR treatment train (Plant 1), the chosen method is to 1) modify the existing aeration basin to provide an anaerobic zone, 2) construct a seventh aeration basin, and 3) add supplemental carbon to aid in denitrification. The cost of the enhanced nutrient removal modifications is estimated at $24,207,600, with an annual supplemental carbon expenditure of $617,300.
- To remove nitrogen and phosphorous at the ACT treatment train (Plant 2), the chosen method is to 1) modify the existing aeration basin to provide an anaerobic zone, 2) retrofitting the existing tertiary filter to become a denitrifying filter, 3) add a coagulant to precipitate phosphorous, and 4) add supplemental carbon to aid in denitrification. The cost of the enhanced nutrient removal modifications is estimated at $38,423,200, with an annual supplemental carbon expenditure of $376,400.

## 6.3 Background

As presented in Volume 2, Chapter 3, Population, Loading and Flow Projections, the anticipated influent flow to RWQCP in year 2037 is 39-mgd ADF. The existing secondary treatment capacity of the RWQCP is 39.7 mgd, as described in Volume 4, Chapter 3, Process Design and Reliability Criteria. This capacity assumes one MBR treatment train (Plant 1) tank offline and all secondary clarifiers in the ACT treatment train (Plant 2) online. Based on these studies, the existing secondary treatment facilities have adequate capacity to handle influent flows up to the year 2037.

In the summer of 2017, the City finished the first part of the expansion of the secondary treatment process referred to as the Phase I expansion. The expansion converted Plant 1 from conventional

activated sludge to an MBR treatment train (Plant 1) process. This increased capacity by 6 mgd for MBR treatment train (Plant 1) from 20 mgd to 26 mgd ADF, with provisions for increasing capacity to 32 mgd. The current trend of water conservation resulted in lower influent flows to the RWQCP than was previously anticipated. The lower flows compounded with the desire to reduce O&M costs, has led the City to consider expanding the MBR treatment train (Plant 1) to 32 mgd. The expansion will include an additional (sixth) aeration basin, additional MBR treatment train (Plant 1) cassettes, some additional blower capacity, and an additional (third) RAS pump.

Furthermore, the EPA and other states have enacted more restrictive nutrient standards to protect aquatic organisms, particularly benthic macroinvertebrates. California is in the process of a similar effort to develop a Nutrient Policy, as discussed in Volume 2, Chapter 2, Regulatory and Climate Change Considerations. Because of the anticipated Nutrient Policy a consensus was reached with the City Staff that future permits may limit effluent quality to 3 mg/L TN and 1 mg/L TP. In addition, an estimated enforcement date of 2035 for these nutrient limits was agreed upon. This would require planning of additions or modifications to the treatment system to meet these nutrient limits to begin in 2029 (six years prior to the enforcement date).

## 6.4 Existing Treatment

### 6.4.1 Description

Figure 6.1 shows the layout of existing facilities. Figure 6.2 shows the BioWin flow schematic of existing facilities. The influent wastewater stream is divided to the MBR treatment train (Plant 1) and the ACT treatment train (Plant 2), as indicated by the blue and orange boxes, respectively.

In both the MBR treatment train (Plant 1) and ACT treatment train (Plant 2), wastewater is settled in primary clarifiers before biological treatment. The MBR treatment train (Plant 1) has five rectangular aeration basins and eight MBR treatment train (Plant 1) membrane tanks. The ACT treatment train (Plant 2) has six rectangular aeration basins and four circular secondary clarifiers. For more detail on the existing facilities reference Volume 4, Chapter 1, Existing Facilities.

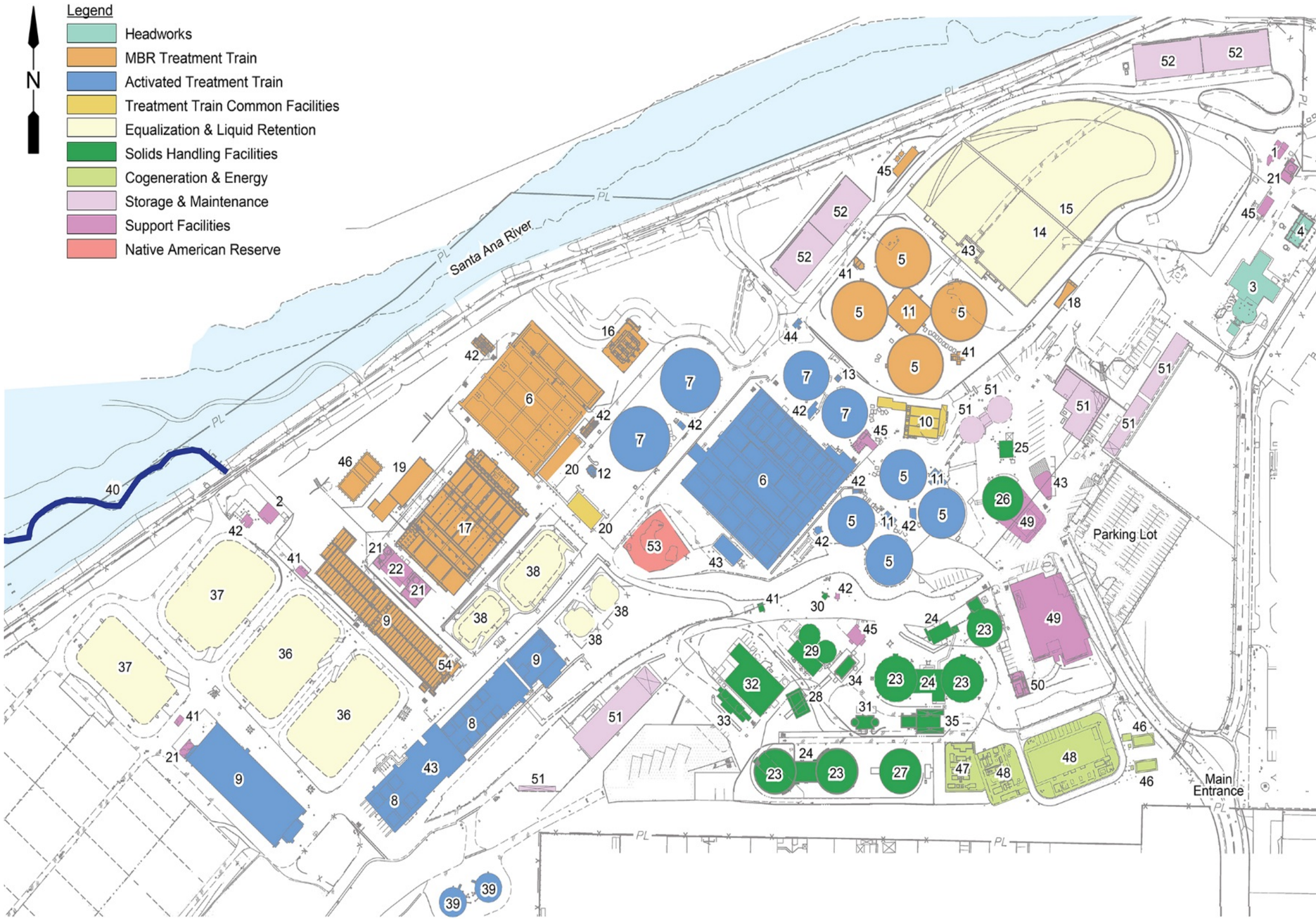
### 6.4.2 Design Criteria

Table 6.1 is a summary of the design criteria of the existing facilities. For a detailed description of the design criteria, refer to Volume 2, Chapter 4, Population, Loading, and Flow Projections.

Table 6.1 Summary of the Design Criteria of the RWQCP

| Element | Unit | Value |
|---|---|---|
| **Design Flow** | | |
| ADWF | mgd | 39 |
| PWWF | mgd | 86(1) |
| Approximate Recycle Flow to Headworks | mgd | 1.7 |
| **Average Influent Wastewater Characteristics(2)** | | |
| BOD | mg/L | 370 |
| TSS | mg/L | 340 |
| TKN | mg N/L | 48 |
| **Effluent Quality Requirements (2017)** | | |
| BOD | mg/L | <10 |
| TSS | mg/L | <10 |
| TIN | mg N/L | <10 |

Notes:
(1) Based on a wet weather peak hour flow factor of 2.2.
(2) Does not include impact of recycle streams from dewatering, thickening, and tertiary filter back wash.




| KEY | |
|---|---|
| 1. | Influent Flow Monitoring |
| 2. | Final Effluent Metering Flow Control Outflow |
| 3. | Headworks Facility |
| 4. | Headworks Odor Control Facility |
| 5. | Primary Clarifier |
| 6. | Aeration Basin |
| 7. | Secondary Clarifier |
| 8. | Tertiary Filter |
| 9. | Chlorine Contact Basin |
| 10. | Primary Clarifiers Odor Control Facility |
| 11. | Primary Sludge and Scum Pump Station |
| 12. | RAS/WAS Pump Station |
| 13. | RAS Pump Station |
| 14. | Diurnal Flow Equalization Basin |
| 15. | Wet Weather Equalization Basin |
| 16. | MBR Fine Screen Facility |
| 17. | MBR Filtration Units |
| 18. | MBR Train Influent Flow Meter |
| 19. | MBR Air Scour/Blower Building |
| 20. | Blower Building |
| 21. | Chemical Storage and Feed Area |
| 22. | Dechlorination Building |
| 23. | Digester |
| 24. | Digester Control Building |
| 25. | Digested Sludge Pump Station |
| 26. | Digested Sludge Storage Tank |
| 27. | Digester Gas Storage Facility |
| 28. | Digester Flares |
| 29. | Dissolved Air Flotation Thickeners |
| 30. | Solids Metering Facility |
| 31. | Bio-Solids Blending Facility |
| 32. | Bio-Solids Dewatering Facility |
| 33. | Bio-Solids Silos/Loading Facility |
| 34. | TWAS Disintegration Facility |
| 35. | FOG Receiving Station |
| 36. | Secondary Effluent Equalization Pond |
| 37. | Stormwater Retention Pond |
| 38. | Waste Backwash Lagoon |
| 39. | Backwash Storage Tank |
| 40. | RWQCP Plant Effluent Outfall |
| 41. | Flow Monitoring or Flow Control Structure |
| 42. | Splitter Box or Diversion Structure |
| 43. | Pump Station/Switchgear Room |
| 44. | Sampling Station |
| 45. | Electrical or MCC Structure |
| 46. | Power Generation or Standby Generators |
| 47. | Fuel Cell |
| 48. | Cogeneration |
| 49. | Administration Building |
| 50. | Cooling Tower |
| 51. | Maintenance, Storage, or Misc. |
| 52. | Material Storage Bed |
| 53. | Native American Rock Reserve Area |
| 54. | Recycled Water Pump Station |

Figure 6.1 Existing Facilities Layout

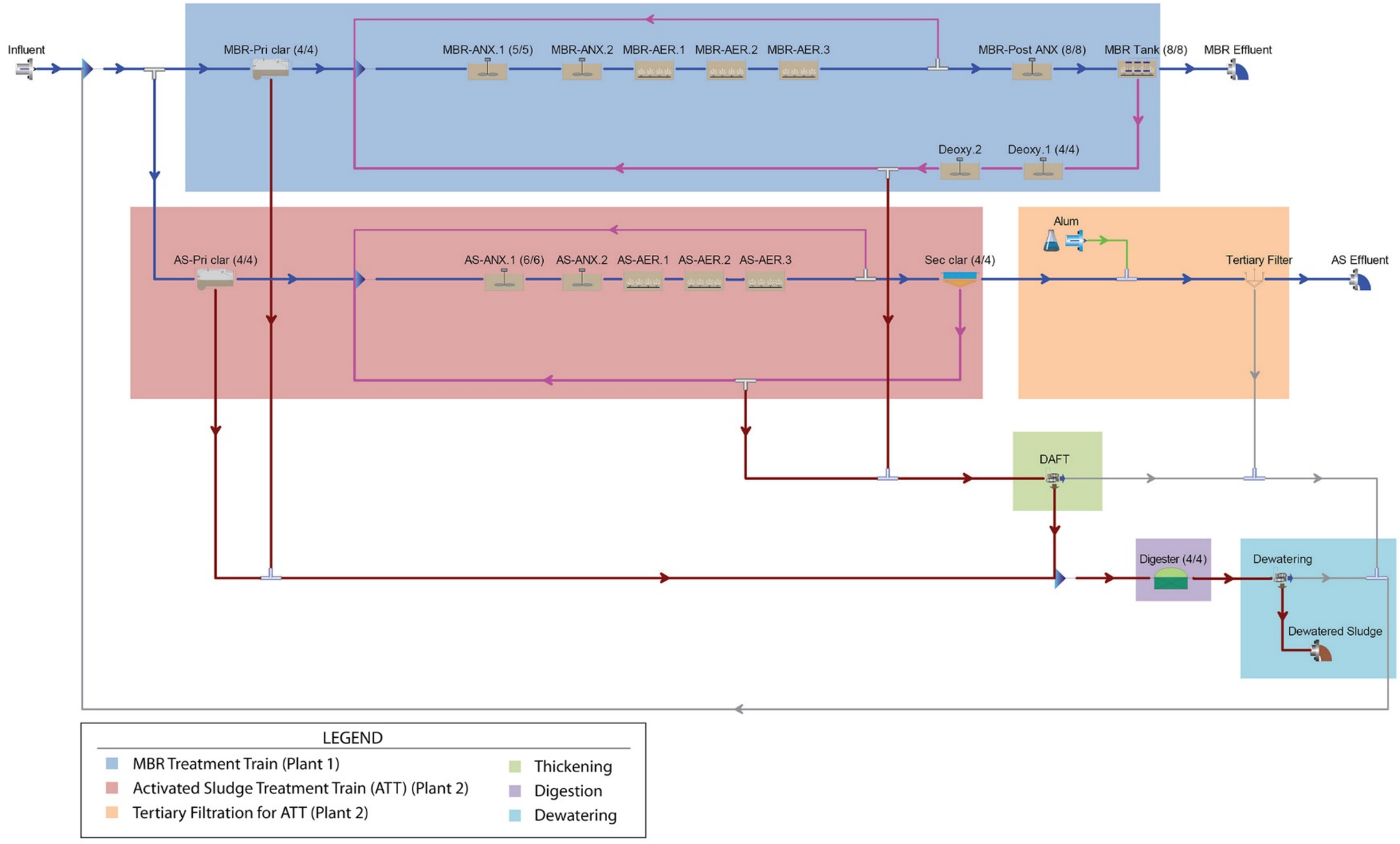



Figure 6.2 BioWin Schematic of Existing Facilities

## 6.5 Phase II Expansion of MBR Treatment Train (Plant 1)

In this section the improvements that are necessary to increase the MBR treatment train (Plant 1) treatment capacity from 26 mgd to 32 mgd are described. The Phase II expansion of the MBR treatment train (Plant 1) capacity will include the following project elements:

- Addition of one new fine screen in a space provided for in the fine screen facility.
- Addition of one new (sixth) aeration basin in a space provided for in the Phase I design.
- Additional of a new aeration blower in the space provided for in the Phase I design.
- Addition of membrane cassettes in slots provided for in the Phase I design.
- Addition of one scour aeration blower in a space provided for in the Phase I design.
- Addition of one RAS recycle pump in a space provided for in the Phase I design.

Figure 6.3 shows a BioWin schematic of the proposed changes to the MBR treatment train (Plant 1). Figure 6.4 shows the proposed location for the sixth aeration basin. The total cost for this project is $20,517,500. A detailed cost estimate is included in Appendix 6A.

However, there is an alternative to the expansion of the existing membrane tanks. Instead of adding more of the same membranes in the provided expansion slots, the existing membranes can be replaced with different membranes. There have been advancements in membrane technology that have intensified the process by increasing capacity of membrane cassettes without increasing their footprint. Currently, there are membranes manufactured by Anaergia Inc. called Fibreplate™ membranes. These membranes are housed in cassettes similar to the existing GE cartridges, and preliminary estimates show that each membrane tank can house the same number of cassettes (20 cassettes) as the existing GE membrane cassettes. However, Fibreplate™ membranes cassettes have approximately 3.5 times the surface area than the existing membrane cassettes. Estimates show that replacement of existing membrane cassettes with Fibreplate™ membranes cassettes will provide a membrane tank capacity of 81 mgd.

However, there are a number of modifications in addition to the MBR treatment train (Plant 1) Phase II Expansion modifications that will be necessary to take advantage of the significant increase in the membrane tank capacity. In Volume 4, Chapter 3, Process Design and Reliability Criteria, MBR treatment train (Plant 1) capacity is limited by the following:

1. The primary clarifiers (36 mgd ADWF).
2. Chlorine contact time (30.7 mgd ADWF, 46 mgd peak flow).
3. Aeration air (26 mgd ADWF).

To circumvent these additional capacity bottlenecks, piping will have to be installed to route flows through ACT treatment train (Plant 2) and utilize the ACT treatment train (Plant 2) primary clarifiers, and aeration basin. Furthermore, the Fibreplate™ membranes cassettes are not identical to the existing membrane cassettes, this would mean that a retrofit of the MBR tanks would be necessary before the new membrane cassettes can be installed. This would require the demolition of the existing membrane tank equipment which still has a significant amount of useful life still left.

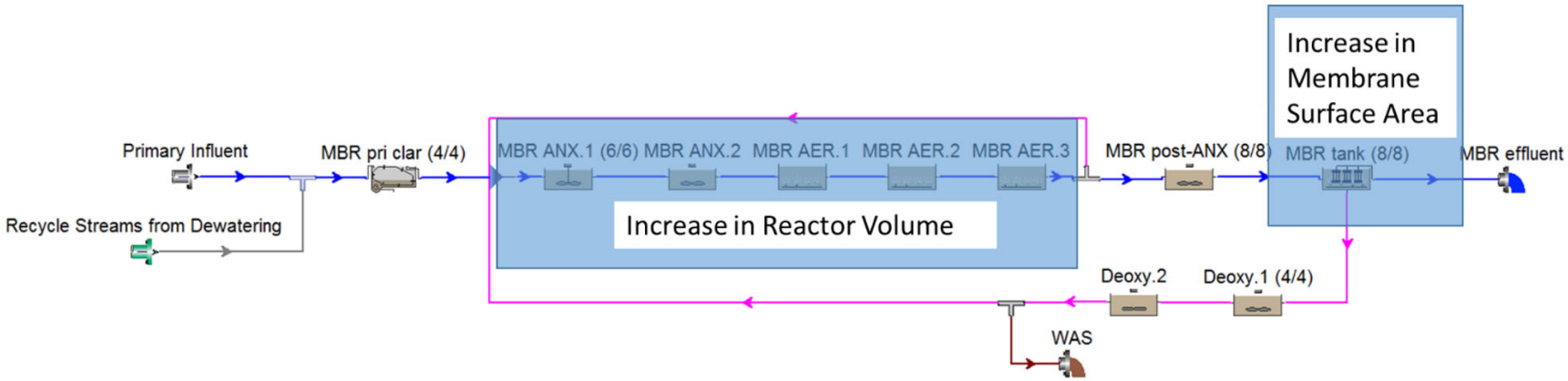



Figure 6.3 BioWin Schematic of Phase II Expansion on MBR Treatment Train (Plant 1)

Figure 6.4 Location of the Sixth Aeration Basin

## 6.6 Enhanced Nutrient Removal for Future Nutrient Limits

As mentioned previously, future permits may limit effluent quality to 3 mg/L TN and 1 mg/L TP, for nitrogen and phosphorous, respectively.

In this Section, additions and modifications that are needed for the MBR treatment train (Plant 1) and ACT treatment train (Plant 2) to reach these nutrient limits are described.

### 6.6.1 Enhanced Nutrient Removal for the MBR Treatment Train (Plant 1)

BNR was selected as the best method to reach the assumed nutrient limits for the future MBR treatment train (Plant 1). There are several ways to achieve BNR, however, the common feature in all BNR processes is that they are divided into zones containing different biochemical environments that allow the BNR process to remove phosphorus and/or nitrogen. For this system both phosphorous and nitrogen are to be removed.

The removal of phosphorous requires the use of baffles to create a first zone within the aeration basin that is anaerobic. This anaerobic environment encourages the growth of specialized phosphorous-storing bacteria.

The removal of nitrogen requires separate sectors within the aeration basin to promote nitrification and denitrification. The process converts ammonia to nitrate and nitrate to nitrogen gas, respectively. This process requires baffles to create a second and a third zone, which are anoxic and aerobic zones, respectively. Biodegradable organic material in the influent wastewater

is used as the carbon source for denitrification by the incorporation of an internal mixed liquor recycle, which supplies nitrate from the end of the aerobic zone (Zone 3) to the anoxic zone (Zone 2). Figure 6.5 shows the process flow diagram of the proposed secondary treatment configuration for the MBR treatment train (Plant 1). A supplemental carbon source, such as methanol or Micro-C, is required, because the influent wastewater does not contain sufficient biodegradable organic carbon relative to the influent nitrate for the 3.0 mg/L TN value to be achieved.

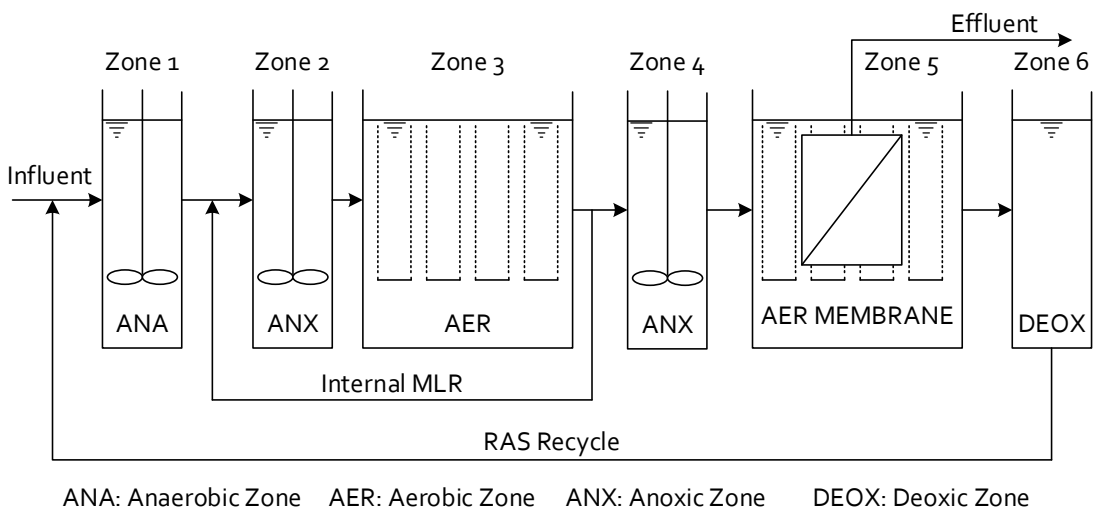



Figure 6.5 Process Flow Diagram of the MBR Treatment Train (Plant 1) Enhanced Nutrient Removal Upgrade

The current configuration has anoxic and aerobic zones to remove nitrogen to meet the current effluent TIN limit of 10 mg/L. The addition of a third zone within the existing aeration basins plus the one future aeration basin is necessary to meet the lower 3.0 mg/L TN value. However, adding this zone will reduce the treatment capacity of the MBR treatment train (Plant 1). A seventh aeration basin is necessary to mitigate the loss of capacity. For planning purposes, the seventh aeration basin will be located where the existing MBR treatment train (Plant 1) Screening Facility resides. In this scenario, relocation of the MBR treatment train (Plant 1) Screening facility may need to occur in order to provide an open location for the seventh aeration basin. Therefore, a new MBR treatment train (Plant 1) Screening Facility may have to be built further east of the MBR treatment train (Plant 1) aeration basins and the existing MBR treatment train (Plant 1) Screening Facility may have to be demolished. Because this project is not needed until 2035, the actual layout can be reconfigured during preliminary design, which may provide an alternative to relocating the fine screen facility.

Figure 6.6 shows the location of the seventh aeration basin. The major elements of the enhanced nutrient removal project for the MBR treatment train (Plant 1) will include:

- Demolish the existing MBR treatment train (Plant 1) Screening Facility.
- Construct a new MBR treatment train (Plant 1) Screening Facility.
- Retrofit the existing aeration basins with methanol addition.
- Reconfigure the existing anoxic zones as staged anaerobic zones.
- Reroute the internal mixed liquor recycle to anoxic Zone 1.
- Construct a seventh aeration basin.
- Construct a methanol storage facility.

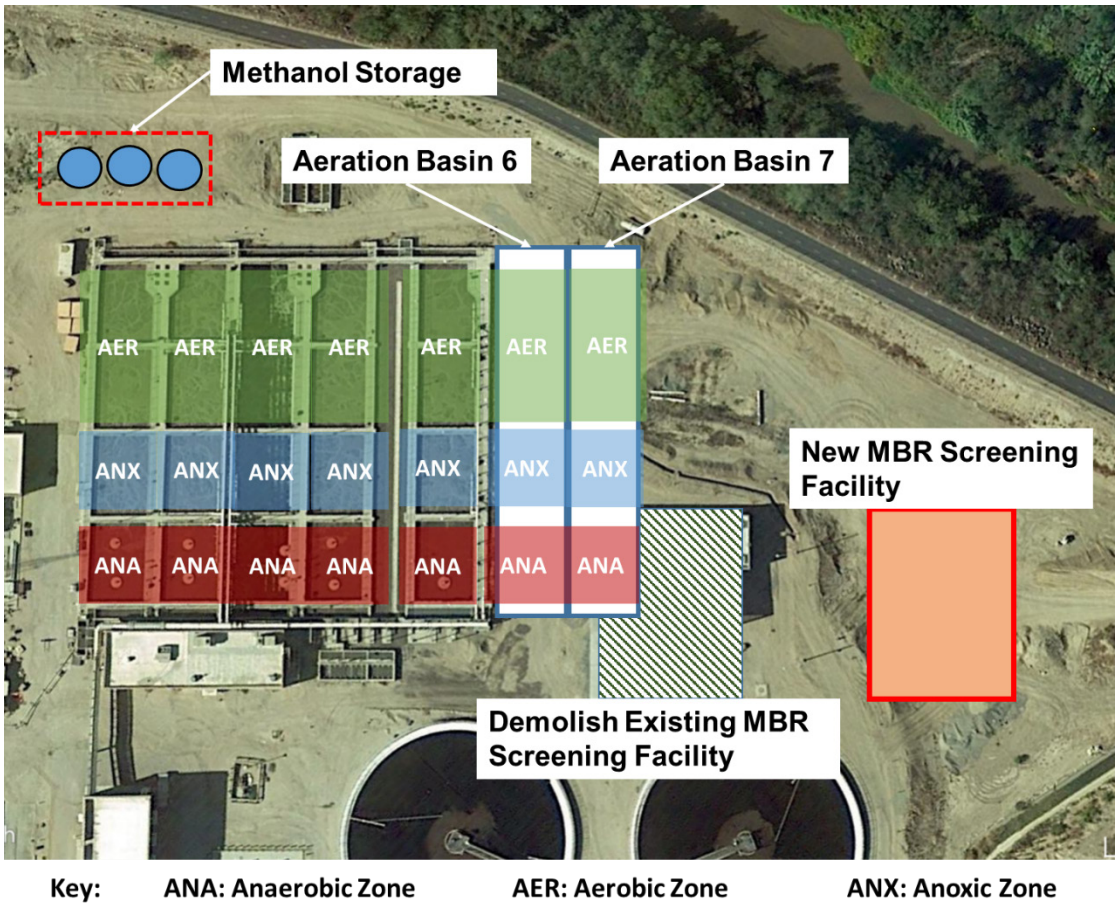



Figure 6.6 Location of the Modifications and Additions for Enhanced Nutrient Removal at MBR Treatment Train (Plant 1)

Figure 6.7 shows the changes to the model. The total cost for this project is $24,207,600. The BioWin model predicted an estimated annual methanol expenditure of $617,300. A detailed cost estimate is included in Appendix 6A.

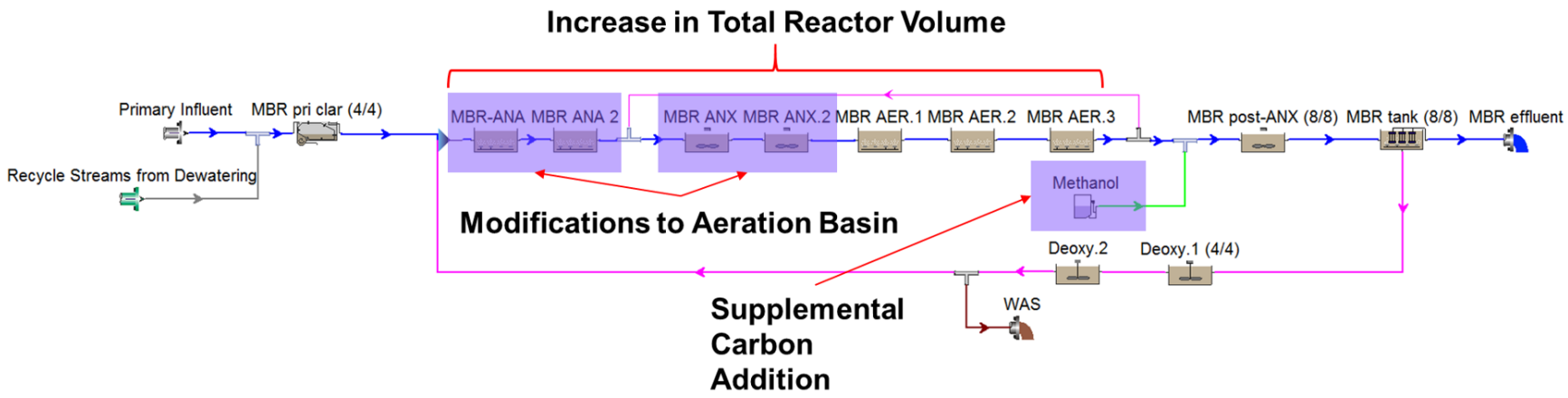



Figure 6.7 BioWin Schematic of Enhanced Nutrient Removal on MBR Treatment Train (Plant 1)

### 6.6.2 Enhanced Nutrient Removal for the ACT Treatment Train (Plant 2)

BNR was also selected as the method to remove nitrogen and phosphorous for the ACT treatment train (Plant 2). In addition, the existing ACT treatment train (Plant 2) aeration basins are to be modified in the same way as the MBR treatment train (Plant 1) to provide 3 zones, anaerobic (Zone 1), anoxic (Zone 2), and aerobic (Zone 3) (Figure 6.8). However, just as in the case of the MBR treatment train (Plant 1), the creation of an anaerobic zone reduces the treatment capacity of the ACT treatment train (Plant 2). Unfortunately in this case, the addition of another basin is not feasible for the ACT treatment train (Plant 2) as there is no room for expansion (see Figure 6.9). Therefore, post-nutrient processes were explored. It was decided that the existing granular filters (tertiary filters), which are scheduled to be replaced by cloth disk filter filters (see Volume 4, Chapter 7, Tertiary Treatment) can be repurposed as denitrifying filters.

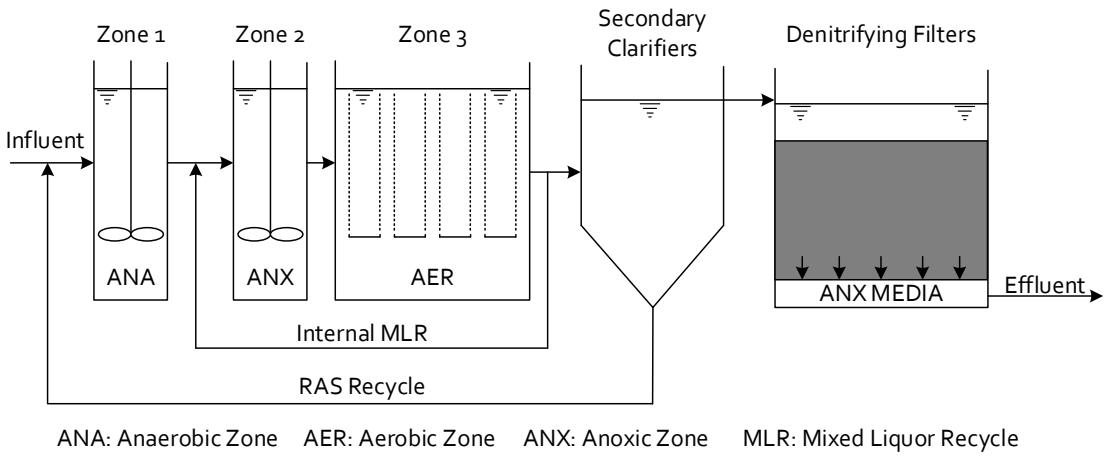


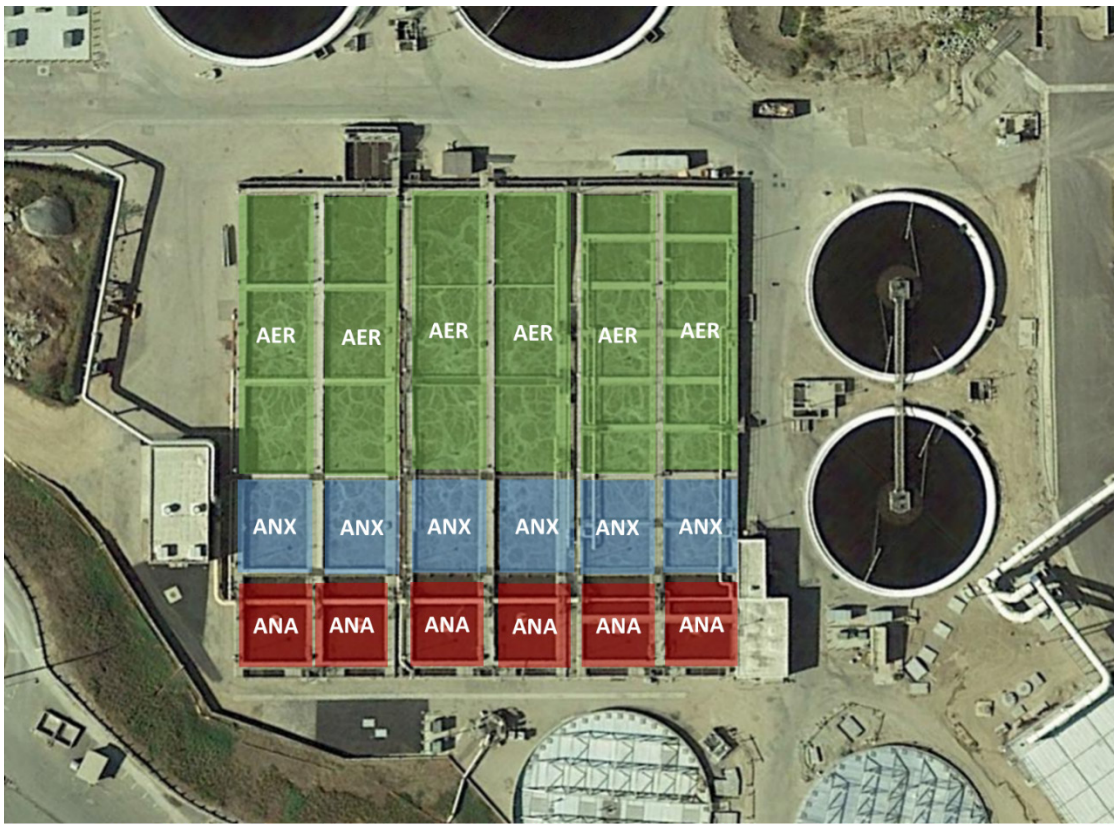

Figure 6.8 Process Flow Diagram of the ACT Treatment Train (Plant 2) Enhanced Nutrient Removal Upgrade

Denitrifying filters consist of a reactor vessel, support media for biofilm growth, an influent distribution system, an effluent withdrawal system, and an air and water backwash system. Wastewater is applied to the top and passes downward through the media to an effluent collection system. The addition of wastewater to the submerged media propagates the growth of attached microorganisms, thereby accomplishing biochemical conversion. Relatively small diameter media and modest superficial velocities are used to prevent biomass from being detached and transported through the media bed. Suspended solids contained in the wastewater will accumulate in the filter, therefore, backwashing is required periodically. Backwashing also removes microorganisms that grow within the media, thereby controlling the biomass inventory. Figure 6.8 shows the process flow diagram of the proposed secondary treatment configuration for the ACT treatment train (Plant 2). The denitrifying filters provide both biological treatment and filtration, which is necessary for the removal of both phosphorous and nitrogen, as further explained below.

The existing ACT treatment train (Plant 2) includes a facility to introduce coagulants prior to the tertiary filters. This in conjunction with the filter properties of the denitrification filters can be used to chemically remove phosphorous. Coagulants introduced to the wastewater prior to the

denitrification filters will cause the precipitation of phosphorous, which in turn will be filtered out and removed from the wastewater.

Since the nitrification has occurred in the ACT treatment train (Plant 2) aeration basins, the denitrifying filters need to facilitate denitrification. For denitrification to occur, both biodegradable organic matter and nitrate must be present in the influent wastewater. The BioWin model indicates that there is insufficient organic matter in the nitrified secondary effluent supplied to the denitrifying filters. Therefore, supplemental carbon (i.e., methanol) must be added. Nitrogen gas will accumulate in a denitrifying filter, therefore, this gas must be purged periodically from the denitrifying filters or it will displace fluid from the interstitial spaces and restrict liquid flow. Purging is often accomplished by temporarily stopping the influent flow and using an air wash cycle.




**Key:** **ANA: Anaerobic Zone** **AER: Aerobic Zone** **ANX: Anoxic Zone**

Figure 6.9 Location of the Aeration Basin Modifications for Enhanced Nutrient Removal at ACT Treatment Train (Plant 2)

Figures 6.10 and 6.11 show the intended modifications to the aeration basins and tertiary filters, respectively. The major elements of the enhanced nutrient removal project for the ACT treatment train (Plant 2) will include:

- Reconfigure the existing anoxic zones as staged anaerobic zones.
- Add a new anoxic zone downstream of the anaerobic zone.
- Reroute the internal mixed liquor recycle to the new anoxic zone.
- Retrofit the granular media denitrifying filters for methanol addition.
- Construct a methanol storage facility.

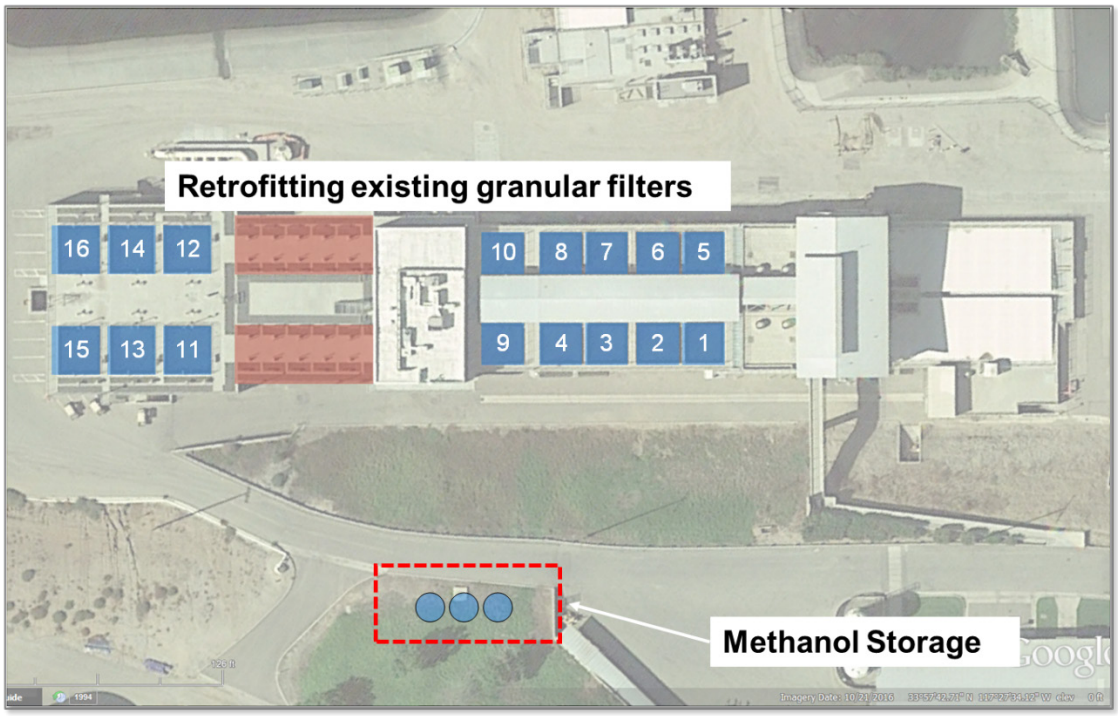



Figure 6.10 Location of the Tertiary Filter Modifications and Additions for Enhanced Nutrient Removal at ACT Treatment Train (Plant 2)

Figure 6.11 shows the changes to the model. The total cost for this project is $38,423,200. The model predicted an estimated annual menthol expenditure of $376,400. A detailed cost estimate is included in Appendix 6A.

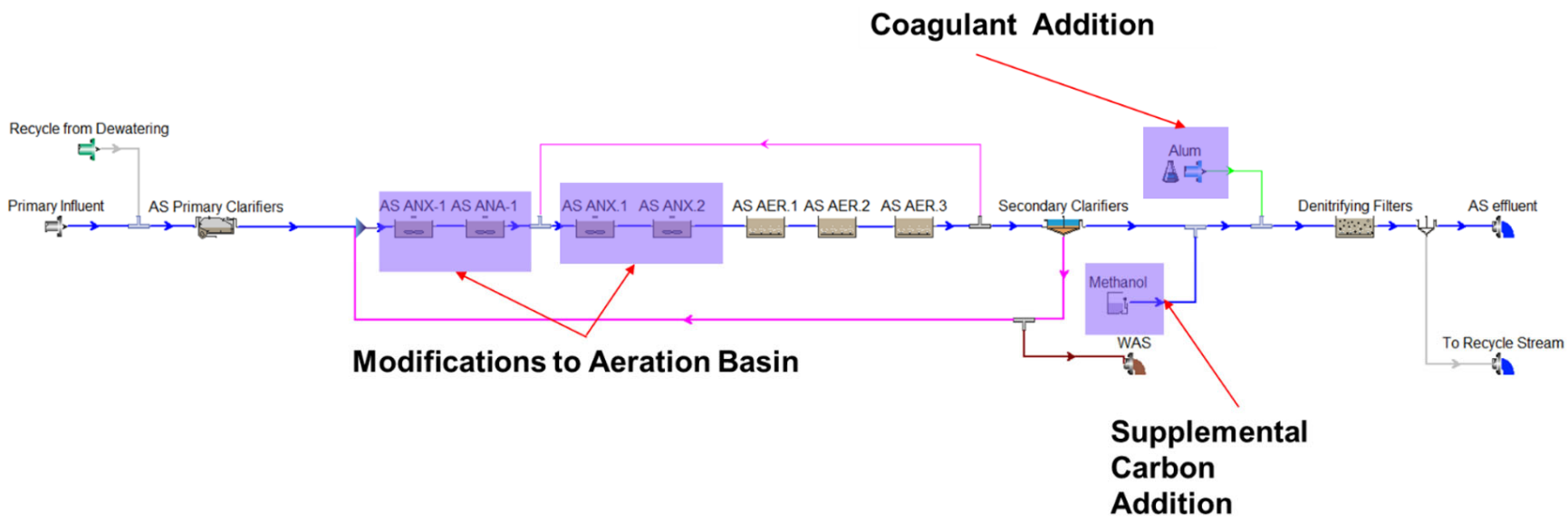



Figure 6.11 BioWin Schematic of Enhanced Nutrient Removal on MBR Treatment Train (Plant 1)

Appendix 6A

# DETAILED COST ESTIMATE

| Methanol Consumption | | | | | | |
|---|---|---|---|---|---|---|
| | Q, mgd | NO3-N, mgN/L | NO3-N, lbN/d | DO, mg/L | DO, lb/d | NO3-N + DO, lb/d |
| MBR | 26.0 | 6.2 | 1,344 | 2.0 | 434 | |
| Activated sludge | 13.0 | 8.0 | 867 | 2.0 | 217 | |
| | **39.0** | | **2,212** | | **651** | **2,862** |
| | | | | | | **4.7** |
| | | | | | | **13,453** |
| | | | | | | **$ 993,700** |
| | | | | | MeOH strength | 100% |
| | | | | | MeOH s.g. | 0.79 |
| | | | | | MeOH, lb/gal | 6.59 |
| | | | | | COD, lb/lb | 1.50 |
| | | | | | COD, lb/gal | 9.88 |
| | | | | | MeOH cost, $/gal | $ 2.00 |
| | | | | | MeOH cost $/lb COD | $ 0.20 |

Figure A-1 Methanol Demand Calculations for Secondary Treatment

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## CITY OF RIVERSIDE PUBLIC WORKS

### UPDATED INTEGRATED MASTER PLAN FOR WASTEWATER COLLECTION AND TREATMENT FACILITIES PROJECT

**TASK :** 7 - CIP and Financial Plan and User Rates and Fees
**JOB # :** 10495A.00
**LOCATION :** Riverside, CA
**TITLE :** Phase II Expansion

**ESTIMATE PREPARATION DATE :** 12/15/2017
**PREPARED BY :** WME
**REVIEWED BY :**

| ITEM NO. | DESCRIPTION | QTY | UNIT | UNIT COST | SUBTOTAL | TOTAL |
|---|---|---|---|---|---|---|
| | | | | | | |
| 1 | **Aeration Basin 6** | | | | | |
| | | | | | | |
| | Fine Pore Diffused Aeration Equipment | 1 | LS | $134,075 | $134,075 | |
| | Construction | 1 | LS | $3,933,631 | $3,933,631 | |
| | Total | | | | | $4,067,707 |
| | | | | | | |
| 2 | **Blower Building 2** | | | | | |
| | | | | | | |
| | Blower | 1 | EA | $267,470 | $267,470 | |
| | Instalation | 1 | LS | $1,478 | $1,478 | |
| | Total | | | | | $268,949 |
| | | | | | | |
| 3 | **MBR Tank** | | | | | |
| | | | | | | |
| | Membrane Cassettes | 1 | LS | $2,225,440 | $2,225,440 | |
| | Instalation | 1 | LS | $70,970 | $70,970 | |
| | Total | | | | | $2,296,410 |
| | | | | | | |
| 4 | **MBR Scour Building** | | | | | |
| | | | | | | |
| | Blower | 1 | LS | $345,284 | $345,284 | |
| | Instalation | 1 | LS | $49,850 | $49,850 | |
| | Total | | | | | $395,134 |
| | | | | | | |
| 6 | **MBR RAS Pump** | | | | | |
| | | | | | | |
| | Vertical propeller pumps | 1 | EA | $98,892 | $98,892 | |
| | Instalation | 1 | LS | $24,141 | $24,141 | |
| | Total | | | | | $123,033 |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | **ITEM NO. 1, 2, 3, 4, 5, & 6 SUBTOTAL** | | | | | **$7,151,232** |
| | | | | | | |
| 5 | **Allowances** | | | | | |
| | | | | | | |
| | Yard Piping & Site Civil Allowance | 10 | % | | $715,123 | |
| | Electrical and Instrumentation Allowance | 20 | % | | $1,430,246 | |
| | Construction Difficulty | 10 | % | | $715,123 | |
| | Total | | | | | $2,860,500 |
| | | | | | | |
| | **SUBTOTAL** | | | | | **$10,011,732** |
| | | | | | | |
| | Estimating Contingency | 30 | % | | | $3,004,000 |
| | **SUBTOTAL** | | | | | **$13,015,732** |
| | | | | | | |
| | General Conditions | 10 | % | | | $1,301,573 |
| | **SUBTOTAL** | | | | | **$14,317,305** |
| | | | | | | |
| | General Contractor Overhead & Profit | 15 | % | | | $1,496,809 |
| | **SUBTOTAL** | | | | | **$14,512,541** |
| | | | | | | |
| | Escalation to June 2018 | 3 | % | | | $435,376 |
| | **SUBTOTAL** | | | | | **$14,947,918** |
| | | | | | | |
| | Sales Tax on 50% of Subtotal Above | 8.75 | % | | | $1,270,000 |
| | **CONSTRUCTION COST SUBTOTAL** | | | | | **$15,782,541** |
| | | | | | | |
| | Engineering, Management, and Legal | 30 | % | | | $4,735,000 |
| | **PROJECT COST** | | | | | **$20,517,500** |

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## CITY OF RIVERSIDE PUBLIC WORKS

### UPDATED INTEGRATED MASTER PLAN FOR WASTEWATER COLLECTION AND TREATMENT FACILITIES PROJECT

**TASK :** 7 - CIP and Financial Plan and User Rates and Fees
**JOB # :** 10495A.00
**LOCATION :** Riverside, CA
**TITLE :** MBR (Plant 1) ENR Upgrade

**ESTIMATE PREPARATION DATE :** 12/15/2017
**PREPARED BY :** WME
**REVIEWED BY :**

| ITEM NO. | DESCRIPTION | QTY | UNIT | UNIT COST | SUBTOTAL | TOTAL |
|---|---|---|---|---|---|---|
| | | | | | | |
| 1 | **Aeration Basin 1-6** | | | | | |
| | | | | | | |
| | Mechanical mixers | 24 | EA | $25,000 | $600,000 | |
| | Installation of Mixers | 1 | LS | $180,000 | $180,000 | |
| | Existing Baffle wall demo | 6 | LS | $25,000 | $150,000 | |
| | New baffle walls | 24 | EA | $16,700 | $400,800 | |
| | Total | | | | | $1,330,800 |
| | | | | | | |
| 2 | **Aeration Basin 7** | | | | | |
| | | | | | | |
| | Construction | 1 | LS | $3,933,631 | $3,933,631 | |
| | Total | | | | | $3,933,631 |
| | | | | | | |
| 3 | **MBR Screen Facility** | | | | | |
| | | | | | | |
| | Demolition of Existing Filter Structure | 1 | LS | $100,000 | $100,000 | |
| | Construction of New MBR Facility | 1 | LS | $3,072,467 | $3,072,467 | |
| | Automatic Rotary Fine Screens | 4 | EA | $383,545 | $1,534,178 | |
| | Instalation of Screens | 1 | LS | $14,458 | $14,458 | |
| | Total | | | | | $3,172,467 |
| | | | | | | |
| | **ITEM NO. 1, 2, & 3 SUBTOTAL** | | | | | **$8,436,898** |
| | | | | | | |
| 4 | **Allowances** | | | | | |
| | | | | | | |
| | Yard Piping & Site Civil Allowance | 10 | % | | $843,690 | |
| | Electrical and Instrumentation Allowance | 20 | % | | $1,687,380 | |
| | Construction Difficulty | 10 | % | | $843,690 | |
| | Total | | | | | $3,374,800 |
| | | | | | | |
| | **SUBTOTAL** | | | | | **$11,811,698** |
| | | | | | | |
| | Estimating Contingency | 30 | % | | | $3,544,000 |
| | **SUBTOTAL** | | | | | **$15,355,698** |
| | | | | | | |
| | General Conditions | 10 | % | | | $1,535,570 |
| | **SUBTOTAL** | | | | | **$16,891,268** |
| | | | | | | |
| | General Contractor Overhead & Profit | 15 | % | | | $1,765,905 |
| | **SUBTOTAL** | | | | | **$17,121,603** |
| | | | | | | |
| | Escalation to June 2018 | 3 | % | | | $513,648 |
| | **SUBTOTAL** | | | | | **$17,635,252** |
| | | | | | | |
| | Sales Tax on 50% of Subtotal Above | 8.75 | % | | | $1,499,000 |
| | **CONSTRUCTION COST SUBTOTAL** | | | | | **$18,620,603** |
| | | | | | | |
| | Engineering, Management, and Legal | 30 | % | | | $5,587,000 |
| | **PROJECT COST** | | | | | **$24,207,600** |

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## CITY OF RIVERSIDE PUBLIC WORKS

### UPDATED INTEGRATED MASTER PLAN FOR WASTEWATER COLLECTION AND TREATMENT FACILITIES PROJECT

**TASK :** 7 - CIP and Financial Plan and User Rates and Fees
**JOB # :** 10495A.00
**LOCATION :** Riverside, CA
**TITLE :** ATT (Plant 2) ENR Upgrade

**ESTIMATE PREPARATION DATE :** 12/15/2017
**PREPARED BY :** WME
**REVIEWED BY :**

| ITEM NO. | DESCRIPTION | QTY | UNIT | UNIT COST | SUBTOTAL | TOTAL |
|---|---|---|---|---|---|---|
| | | | | | | |
| 1 | **Aeration Basin 1-6** | | | | | |
| | | | | | | |
| | Mechanical mixers | 24 | EA | $25,000 | $600,000 | |
| | Installation of Mixers | 1 | LS | $180,000 | $180,000 | |
| | Existing Baffle wall demo | 6 | LS | $25,000 | $150,000 | |
| | New baffle walls | 18 | EA | $16,700 | $300,600 | |
| | Total | | | | | $1,230,600 |
| | | | | | | |
| 2 | **RWQCP Existing Filters** | | | | | |
| | | | | | | |
| | Demolition | 1 | LS | $800,000 | $800,000 | |
| | Retrofit New Filters | 1 | LS | $6,683,349 | $6,683,349 | |
| | Installation | 1 | LS | $4,678,344 | $4,678,344 | |
| | | | | | | |
| | Total | | | | | $12,161,694 |
| | | | | | | |
| | | | | | | |
| | **ITEM NO. 1, & 2 SUBTOTAL** | | | | | **$13,392,294** |
| | | | | | | |
| 3 | **Allowances** | | | | | |
| | | | | | | |
| | Yard Piping & Site Civil Allowance | 10 | % | | $1,339,229 | |
| | Electrical and Instrumentation Allowance | 20 | % | | $2,678,459 | |
| | Construction Difficulty | 10 | % | | $1,339,229 | |
| | Total | | | | | $5,356,900 |
| | | | | | | |
| | **SUBTOTAL** | | | | | **$18,749,194** |
| | | | | | | |
| | Estimating Contingency | 30 | % | | | $5,625,000 |
| | **SUBTOTAL** | | | | | **$24,374,194** |
| | | | | | | |
| | General Conditions | 10 | % | | | $2,437,419 |
| | **SUBTOTAL** | | | | | **$26,811,613** |
| | | | | | | |
| | General Contractor Overhead & Profit | 15 | % | | | $2,803,032 |
| | **SUBTOTAL** | | | | | **$27,177,226** |
| | | | | | | |
| | Escalation to June 2018 | 3 | % | | | $815,317 |
| | **SUBTOTAL** | | | | | **$27,992,543** |
| | | | | | | |
| | Sales Tax on 50% of Subtotal Above | 8.75 | % | | | $2,379,000 |
| | **CONSTRUCTION COST SUBTOTAL** | | | | | **$29,556,226** |
| | | | | | | |
| | Engineering, Management, and Legal | 30 | % | | | $8,867,000 |
| | **PROJECT COST** | | | | | **$38,423,200** |

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 7

## Wastewater Treatment System

## Tertiary Treatment

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City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 7: TERTIARY TREATMENT

FINAL | June 2019

City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 7: TERTIARY TREATMENT

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:21:55-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 7

# TERTIARY TREATMENT

## 7.1 Purpose

The purpose of this chapter is to describe tertiary treatment alternatives associated with the ACT treatment train (Plant 2) and estimate overall capital cost of improvements through 2037. This chapter includes recommendations based on initial investigations. Final decisions about specific equipment types or layouts should be determined during preliminary and final design stages of each associated project.

## 7.2 Conclusions and Recommendations

- Existing filters are maintenance and operationally intrusive and are therefore expensive and difficult to operate.
- Membrane filters are approximately three times the cost of the other alternatives.
- Five alternatives were presented for consideration. Due to cost, alternatives pertaining to tertiary membranes were eliminated. The three remaining alternatives were considered for detailed evaluation.
- Although the alternative to retrofit existing tankage with cloth disk filter units resulted in the least expensive net present value, the cost difference between it and packaged cloth disk filter units was not enough to justify the inherent risk associated with a retrofit project.
- Packaged cloth disk filters are recommended to be installed in phases, with Phase 1 installed by 2022 and Phase 2 installed past this planning period. During Phase 1 pre-design, further consideration should be given to capacity phasing of Tertiary filters based on the plan for operation of the ACT treatment train (Plant 2) plant and influent flows at the time.

## 7.3 Background

There are 16 dual media filters operating in two separate trains. Both trains consist of a 24-in layer of anthracite above a 15-in layer of silica sand. Filters 1 through 10 feed directly to CCB 1, which in turn feeds to CCB 3 to achieve the necessary Title 22 contact time. Filters 11 through 16 are preceded by a chemical flocculation step through Alumer™ injection that aggregates very small particles so they can be efficiently removed in the filters. Currently, flocculation basins are out of service. However, Alumer™ is dosed continuously to meet Title 22 standards. Filters 11 through 16 discharge to CCB 3 directly. Table 7.1 presents design criteria for existing tertiary treatment facilities on the ACT treatment train (Plant 2) side.

Table 7.1 Existing Tertiary Facilities

| Description | Value |
|---|---|
| Filter Influent Pumps | |
| Filters 1 through 10 | |
| Number | 2 duty + 1 standby |
| Capacity, each gpm | 13,050 |
| Horsepower | 250 |

Table 7.1 Existing Tertiary Facilities (continued)

| Description | Value |
|---|---|
| Filters 11 through 16 | |
| Number | 2 duty + 1 standby |
| Capacity, each gpm | 8,000 |
| Horsepower | 125 |
| Tertiary Filtration | |
| Number of Filters | 16 |
| Anthracite Depth, in | 24 |
| Silica Sand Depth, in | 15 |
| Filters 1 through 10 | |
| Surface Area, each, sq. ft. | 552 |
| Average Loading Rate, gpm/ sq. ft. | 3.33 |
| Maximum Loading Rate, gpm/ sq. ft. | 5.00 |
| Capacity, gpm/filter (Average/Max.) | 1,838/2,760 |
| Filters 11 through 16 | |
| Surface Area, each, sq. ft. | 650 |
| Average Loading Rate, gpm/ sq. ft. | 3.33 |
| Maximum Loading Rate, gpm/ sq. ft. | 5.00 |
| Capacity, gpm/filter (Average/Max.) | 2,165/3,250 |
| Flocculation Basins(1) | |
| Number | 10 |
| Number of Stages, each | 2 |
| Volume per basin, gal | 13,740 |
| Main Backwash Pumps (Filters 1 through 10/Filters 11 through 16) | |
| Number | 2 duty + 1 standby |
| Capacity, gpm | 14,100 |
| Horsepower | 150 |
| Max. filter backwash rate, ft. rise/min | 3.41/3.48 |
| Max. filter backwash rate, gpm/sq. ft. (unit) | 25.5/26.0 |
| Max. filter backwash rate, gpm/filter | 14,082/16,900 |
| Capacity, gpm | 1,650 |
| Subsurface Wash Pumps (Filters 1 through 10/Filters 11 through 16) | |
| Number | 1 duty + 1 standby |
| Horsepower | 150 |
| Max. filter wash rate, ft. rise/min | 0.4/0.3 |
| Max. filter wash rate, gpm/sq. ft. (unit) | 3.0/2.54 |
| Max. filter wash rate, gpm/filter | 1,650 |
| Waste Backwash Pumps | |
| Number | 1 duty + 1 standby |
| Capacity, gpm | 5,220 |
| Horsepower | 100 |
| Backwash Storage Tanks | |
| Number | 2 |
| Volume, each, MG | 0.66 |
| Average Alumer Dosage, mg/L | 0.2 |

Notes:
(1) Currently, flocculation basins are not fully utilized. Instead, Alumer™ is continuously dosed to meet Title 22 requirements.

## 7.4 ACT Treatment Train (Plant 2) Tertiary Treatment Design Criteria

Based on preliminary process modeling, the ACT treatment train (Plant 2) has a maximum treatment capacity of approximately 18 mgd average flow. Upstream equalization facilities (i.e., Primary Influent EQ Basins and Secondary Effluent EQ Basins) provide significant reduction to peak event flows. Therefore, the following design considerations utilize a 1.2 peaking factor, resulting in a 21.6 mgd average max month design flow. For comparison, the MBR side has a design capacity of 26 mgd average flow, but only includes the Primary Influent EQ Basins. Under peak event conditions, the design utilizes a 1.5 peaking factor for all downstream processes.

Furthermore, Title 22 requires that treatment capacity be met (i.e., 21.6 mgd) with one unit out-of-service. Discharge requirements for media and cloth filters are 5 mg/L maximum effluent TSS and 2 NTU effluent turbidity. Currently, Filters 1 through 10 are required to operate in order to provide filtered water to the backwash supply tanks. Consequently, Filters 11 through 16 were selected for rehabilitation or retrofit projects necessary to allow the ACT treatment train (Plant 2) side to remain operational during construction. It was assumed that during rehabilitation or retrofit, the existing backwash supply system would be revised as necessary to provide sufficient backwash water; thus, allowing Filters 1 through 10 to be fully decommissioned upon completion of the selected rehabilitation or retrofit project.

## 7.5 Technology Considerations

Three tertiary treatment technologies were evaluated. The first alternative considered a full rehabilitation of existing dual media Filters 11 through 16 to provide new equipment within the existing tankage. Cloth disk filters and membrane filters were also considered. The following sections discuss the treatment technologies and the advantages and disadvantages of each.

### 7.5.1 Replacement in Kind

While Volume 6, Chapter 1 of this update to the Master Plan provides a detailed description of modifications necessary to update existing filters, subsequent sections consider only the equipment within the tanks for equal comparison of technologies. Therefore, it is assumed that the entire underdrain system, media, and all applicable appurtenances would need to be replaced.

Although this alternative requires few modifications to existing tankage, the City has indicated that the existing system is at the end of its useful life. In terms of maintenance and operational activities, the technology is old and intrusive. Very few of the operations staff are familiar with the technology, making filter operation extremely difficult and time consuming. For this alternative to be an improvement to plant operations, filter operation would need to be more streamlined and less intensive in terms of maintenance and operational requirements. Furthermore, based on the condition assessment performed and detailed in Volume 6, Chapter 1, structural rehabilitation may be required. Advantages and disadvantages of this alternative are presented in Table 7.2

Table 7.2 Rehabilitate and Replace in Kind Advantages and Disadvantages

| Advantages | Disadvantages |
|---|---|
| Little necessary modification to the existing facilities (i.e., replace and rehab in kind) | The technology is outdated |
| | The process is intrusive in terms of maintenance and operational activities |

### 7.5.2 Cloth Disk Filters

Cloth Disk Filters have grown in popularity over the last few years. AquaDisk™ has been at the forefront of this development, and as such, was selected as the design basis for this alternative comparison. However, many other manufacturers (e.g., Five Star Filtration, Alfa Laval, and Nexom) produce cloth disk filters and should be considered during preliminary design.

For this application, cloth disk filters would be submerged in secondary effluent drawing water through the membrane, out through the center of the disk, to the effluent collection system. The AquaDisk™ system features spinning disks covered in a cloth membrane much like a carpeted sleeve over a disk. A stationary backwash arm acts like a vacuum cleaner pushing against the disk to clean off any accumulated solids. Each cloth disk filter unit comes with multiple disks on a single drive shaft allowing two disks to backwash while the rest remain in service. Additionally, each disk can be isolated and removed from service for maintenance while the system remains in operation.

Cloth disk filters require a less intensive backwash and less power and maintenance than dual media filters while operating at a lower head loss profile. However, the cloth membranes are susceptible to upstream chemical treatments, particularly chlorine and polymer. Per the manufacturer, AquaDisk™ filters can handle a continuous chlorine dose of 5 mg/L with peak doses of up to 10 mg/L without damage. Furthermore, polymer, Alumer™, and ferric chloride dosing precipitate solids out of suspension increasing SLR and backwash rate reducing overall performance. The AquaDisk™ manufacturer recommends careful control of all upstream chemical additions and further investigation during preliminary design. Table 7.3 shows the advantages and disadvantages of this technology.

Table 7.3 Summary of Cloth Disk Filter Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| Less intensive backwash, power, and maintenance compared to dual media filters | Upstream chemical addition is not recommended by manufacturers |
| Low head loss profile | A cloth disk filter system would be more expensive than the rehabilitation of the existing dual media filter system |
| Filtration during backwash is possible | |

### 7.5.3 Membrane Filters

For this application, membrane filters provide a higher level of tertiary water quality than other technologies discussed. The membranes provide high removal of viruses, parasites, bacteria, and turbidity allowing for significant potential beneficial reuse. They can provide log removal credits for 4 log Cryptosporidium and 4 log Giardia. Effluent quality from membrane filters could allow for direct treatment by an RO system. Membrane filters would bring the ACT treatment train (Plant 2) effluent quality up to match MBR treatment train (Plant 1) effluent quality.

Membrane skids can either be submerged or pressurized units. However, they are more susceptible to outside contaminants (e.g., leaves, twigs, etc.) than other tertiary technologies and as a result, require upstream screening to at least 500 microns. For these filters to be installed in existing tankage, the existing structure would require significant modifications. Additionally,

membrane filters are more expensive than either technology considered above. Table 7.4 shows the advantages and disadvantages of this technology.

Table 7.4 Summary of Membrane Filter Advantages/Disadvantages

| Advantages | Disadvantages |
|---|---|
| High removal of virus, parasites, bacteria, and turbidity | More susceptible to outside contaminants (e.g., twigs, leaves, etc.) |
| High reuse potential with log removal credits for 4 log cryptosporidium and 4 log Giardia | Requires upstream screenings to at least 500 micron |
| Microfiltration or ultrafiltration suitable for direct RO treatment | Extensive modifications to existing structure for retrofit |
| | Most expensive technology considered |

### 7.5.4 Non-Economic Technology Comparison

To adequately assess the technologies as a whole, Table 7.5 shows a comparison across various operational metrics.

Table 7.5 Non-Economic Comparison of Tertiary Technologies

| | Replace In-Kind | Cloth Disk Filters | Membranes |
|---|---|---|---|
| Operating Head Loss | 10 ft | 3 ft | Up to 70 ft Discharge Pressure |
| Operational Reliability | Difficult to Operate | Reliable and Simple | More Components, but Reliable and Simple |
| Resistance to Chemical Addition | Adequate Resistance | Susceptible to high chlorine and polymer | Adequate Resistance |
| Backwash Downtime | Entire Unit is Offline | Disk Backwashes Separately, Full Production | Short Backwash, Full Production |
| Backwash Horsepower | Heavy Duty Pumps | Smaller Pumps | Smaller Reverse Filtration Pumps |
| Clean-In-Place | None | Annual Chlorine Clean | Monthly |
| Maintenance Requirement | Difficult to Maintain | Few/Standard Components | More/Standard Components |
| Air Scour Blower Requirement | Heavy Duty Blower | Not Applicable | Smaller Blowers/Air Compressors |

## 7.6 Tertiary Alternative Configurations

In addition to the technology alternatives, several installation options were considered for this update to the Master Plan. One alternative is replacement in-kind for Filters 11 through 16 with new Leopold underdrain and dual-media, rapid filter materials. To use existing tankage, Filters 11 through 16 were evaluated for both retrofit with cloth disk filter technologies and submerged membrane filters. Standalone package installations of cloth disk filters and membrane filters were also considered at a nearby site. The subsequent sections describe each alternative in more detail.

### 7.6.1 Rehabilitation or Retrofit Alternatives Cost Basis

Although Volume 6, Chapter 1 lists quite a few rehabilitation projects that would be required for full operation of existing filters, it is assumed that all upgrades outside the tanks themselves and exclusive to Filters 11 through 16 or joint ancillary functions would be applicable to all retrofit alternatives. Therefore, $396,300 estimated direct costs, extracted from the near-term rehabilitation costs described in Volume 6, Chapter 1, will be added to all rehabilitation or retrofit alternatives presented.

The estimated direct cost applicable to rehabilitation or retrofit projects includes the following improvements:

- Add an actuator to the air scour blower cross-over interconnection.
- Add an actuator to the backwash supply piping valve that isolates the east and west sides.
- Configure drains for the air blow-off valves in the galleries to eliminate standing water.
- Replace instrumentation that is older than 10 years.
- Rehabilitate the backwash system valves.
- Modify the backwash system to eliminate the need to run the east filters continuously.
- Complete a backwash tank seismic evaluation and an interior condition assessment.
- Investigate the cause of the stair crack described previously.

### 7.6.2 Replacement In-Kind with new Leopold Underdrain and Tri-Media Filter

In this alternative, it is assumed that existing Filters 11 through 16 would be completely emptied of equipment and materials. A thorough structural evaluation and rehabilitation of these tanks is required, although not included in the following pricing due to the unknown extent of any damage and the resulting variability in possible cost. Once the concrete is fully rehabilitated, a new Leopold Type XA underdrain system, air header piping, wash troughs, agitators, ancillary equipment, and dual-media, rapid filter material (12-in Gravel, 15-in Silica Sand, and 24-in Anthracite) would be installed. Table 7.6 provides design criteria for this alternative, which has an estimated project cost of $4,336,000.

Table 7.6 Replacement In-Kind with Leopold Underdrain Design Criteria

| Description | Value |
|---|---|
| Rehabilitated Filter Cells | 11 through 16 |
| Underdrain | Leopold Type XA |
| Air Header Piping | 1 Assembly per Cell |
| Wash Troughs | 3 Troughs per Cell |
| Agitators and Ancillary | 6 Agitators per Cell |
| Media | 12 in Gravel<br>15 in Silica Sand<br>24 in Anthracite Coal |
| Total Filter Area Provided | 3,900 $ft^2$ |
| Hydraulic Loading with All Units In Service | Average (18 mgd): 3.21 $gpm/ft^2$<br>Max (21.6 mgd): 3.85 $gpm/ft^2$ |
| Hydraulic Loading with One Unit Out Of Service | Average (18 mgd): 3.85 $gpm/ft^2$<br>Max (21.6 mgd): 4.62 $gpm/ft^2$ |
| Effluent Water Quality | Turbidity ≤ 2 NTU |

### 7.6.3 Retrofit with AquaDisk™ Filter System

In this alternative, walls between Filters 13 and 15 and between Filters 14 and 16 would be demolished to form two longer filter cells. To meet the full 21.6 mgd design flow, a total of three filter units would be installed in the two longer filter cells. Each filter unit would be equipped with 20 disks with an approximate surface area per disk of 108 $ft^2$. The total filter area provided would be 6,456 $ft^2$. For this application, Aqua Aerobics recommends OptiFiber PES-14 media cloth with a 5 micron pore size. Table 7.7 provides the design criteria for this alternative, which has an estimated project cost of $7,074,000.

Table 7.7 Retrofit with AquaDisk™ Filter System Design Criteria

| Description | Value |
|---|---|
| Required Tank Modifications | Demolish Walls Between Filters 13 and 15 and Between Filters 14 and 16 |
| Install Filter Units for Full 21.6 Mgd Capacity | 3 Filter Units |
| Disks Per Unit | 20 |
| Area Per Disk | ~108 $Ft^2$ |
| Total Filter Area Provided | 6,456 $ft^2$ |
| Hydraulic Loading with All Units In Service | Average (18 mgd): 1.94 gpm/$ft^2$<br>Max (21.6 mgd): 2.90 gpm/$ft^2$ |
| Hydraulic Loading with One Unit Out Of Service | Average (18 mgd): 2.32 gpm/$ft^2$<br>Max (21.6 mgd): 3.49 gpm/$ft^2$ |
| Effluent Water Quality | Turbidity ≤ 2 NTU<br>Total Phosphorus ≤ 0.1 mg/L |

### 7.6.4 Retrofit with Submerged Membrane Filters

In this alternative, walls between Filters 12, 14, and 16 would be demolished. New walls would be constructed to subdivide the footprint of Filters 12, 14, and 16 into five new, smaller tanks. Filters 11, 13, and 15 would be modified to house ancillary membrane system equipment. Because submerged membranes are susceptible to external debris (e.g., leaves, twigs, etc.), upstream pre-screenings to a minimum of 500 microns would be required with covers installed above the membranes. A concrete distribution feed channel, backpulse tank, and CIP tank would also be required. For this update to the Master Plan, GE ZeeWeed 1000 membranes were considered. To meet the full 21.6 mgd design flow, five treatment trains would be installed in the five created filter cells. Each train would have 467 modules installed in the 576 slots available. Table 7.8 provides design criteria for this alternative, which has an estimated project cost of $20,737,000.

Table 7.8 Retrofit with GE Submerged ZeeWeed™ 1000 System Design Criteria

| Description | Value |
|---|---|
| Required Tank Modifications | Demolish walls between Filters 12, 14, and 16; Modify Filters 11, 13, and 15 to house ancillary equipment |
| Install Filter Units for Full 22.8 mgd Capacity | 5 Filter Units |
| Installed Modules per Unit | 467 |
| Available Modules per Unit | 576 |
| Surface Area per Module | 500 $ft^2$ |

Table 7.8 Retrofit with GE Submerged ZeeWeed™ 1000 System Design Criteria (continued)

| Description | Value |
|---|---|
| Membrane Flux with All Trains in Service | Average (19 mgd): 17.6 gpd/ft$^2$<br>Max (22.8 mgd): 21.1 gpd/ft$^2$ |
| Membrane Flux with One Train Out of Service | Average (19 mgd): 22.0 gpd/ft$^2$<br>Max (22.8 mgd): 26.4 gpd/ft$^2$ |
| Effluent Water Quality | Turbidity ≤ 0.2 NTU<br>Total Phosphorus ≤ 1 mg/L |

### 7.6.5 Standalone Packaged AquaDisk™ Filter System

This alternative provides the same equipment as the retrofit equivalent; however, instead of using existing tankage, the AquaDisk™ system would be provided as packaged units. To meet the full 21.6 mgd design flow, a total of three packaged filter units would be installed at a site adjacent to the existing tertiary filter facility. Figure 7.1 shows a proposed facility location. At the Tertiary Treatment Workshop, it was discovered that an existing high-voltage power line was in the proposed location for this standalone facility. This line would need to be relocated during construction. Table 7.7 provides design criteria for this alternative, which has an estimated project cost of $7,929,000.

Figure 7.1 Proposed Location of Standalone Packaged Tertiary Filtration

### 7.6.6 Standalone Pressurized Membrane Filters

In this alternative, pressurized membranes would be installed in a new building adjacent to the existing tertiary filter facility, as shown on Figure 7.1. Again, the high-voltage power line would need to be rerouted. Pressurized membranes are different from submerged membranes in that pressurized membranes are fully enclosed in sealed units. However, upstream pre-screening would still be required. To meet the full 21.6 mgd design flow, a total of 10 racks would be installed.

Each rack can hold 210 modules. Table 7.9 provides design criteria for this alternative, which has an estimated project cost of $32,525,000.

Table 7.9 Standalone Pressurized Membrane Filters Design Criteria

| Description | Value |
|---|---|
| Required Building Footprint | 117′ by 73′ |
| Install Filter Racks for Full 22.8 mgd Capacity | 10 Filter Racks (9 duty + 1 standby) |
| Installed Modules per Unit | 210 |
| Surface Area per Module | 500 $ft^2$ |
| Membrane Flux with All Trains in Service | Average (19 mgd): 18.1 $gpd/ft^2$<br>Max (22.8 mgd): 21.7 $gpd/ft^2$ |
| Membrane Flux with One Train Out-of-Service | Average (19 mgd): 20.1 $gpd/ft^2$<br>Max (22.8 mgd): 24.1 $gpd/ft^2$ |
| System Recovery | 93.2% |
| Effluent Water Quality | Turbidity ≤ 0.2 NTU |

### 7.6.7 Economical Comparison of Installation Alternatives

Five installation alternatives were considered as described above: 1) Replacement In-Kind, 2) Retrofit with AquaDisk™ Cloth Disk Filters, 3) Retrofit with Membrane Filters, 4) Install Package Cloth Disk Filters, and 5) Install Pressurized Membrane Filters. As a whole, membrane filter alternatives are significantly more expensive than the other alternatives, but provide excellent water quality to rival that of the MBR treatment train (Plant 1) effluent. The project cost comparison along with other comparative metrics are summarized in Table 7.10 below.

Table 7.10 Economic Comparison of Installation Alternatives

| Technology Alternative | Project Cost ($ Million) | Effluent Quality (NTU) | O&M Requirement | Operator Attention | Backwash % |
|---|---|---|---|---|---|
| Replacement In-Kind | $4.34 | 2 | Medium | High | 10 |
| Cloth<br>Retrofit Package | $7.07<br>$7.93 | 2 | Low | Low | 3 |
| Membrane<br>Submerged Pressurized | $20.7<br>$32.5 | 0.2 | High | Low | 7 |

Given the information presented, these alternatives were discussed with the City at the Tertiary Treatment Workshop. Although water quality of membrane effluent would be the highest of the presented technologies, membrane alternatives are approximately three times the cost of the others and was therefore not considered for further evaluation.

## 7.7 Life-Cycle Analysis of Alternatives

Upon further discussion, the City requested a lifecycle cost evaluation be conducted on the remaining three alternatives. As discussed in Volume 2, Chapter 3, the Basis of Cost Estimates chapter, the net present value of all life-cycle analyses assumed a 6-percent discount rate and only included costs up to and through 2037.

### 7.7.1 Project Implement Schedule Assumptions

Given current condition of existing filters, the replacement in-kind alternative assumes project completion in 2019. Both Phase 1 cloth disk filters alternatives assume project completion in 2022.

It is further assumed that until the rehab project is completed, O&M costs (electrical and consumables) will be equivalent to replacement in-kind costs. Specifically, between 2017 and 2022 when the cloth disk filters would be put into service, it is assumed that existing filters will continue to operate at the calculated replacement in-kind O&M costs.

### 7.7.2 Capital Cost Assumptions

The City would like to consider a phased implementation approach for the cloth disk filter alternatives. They would also like to plan for Phase 1 to be operational within the next 5 years providing 13.3 mgd average (16 mgd max month) of design treatment capacity. Phase 2 would be operational in the future as ACT treatment train (Plant 2) flows increased to 18 mgd average (21.6 mgd max month). The City has indicated the intent to operate the MBR treatment train (Plant 1) plant to its fullest capacity. The MBR treatment train (Plant 1) side is currently designed for 26 mgd average (39 mgd max month). Based on Volume 2, Chapter 3, this update to the Master Plan assumes a 39 mgd average flow basis at the end of the study period. Assuming the City operates the MBR treatment train (Plant 1) facility at 26 mgd average in 2037, the average flow to the ACT treatment train (Plant 2) side should not exceed design conditions of Phase 1 cloth disk filter alternatives. Therefore, Phase 2 of the cloth disk filter alternatives would not be required within the life-cycle evaluation period.

### 7.7.3 Operations and Maintenance Personal and Consumables Assumptions

Due to operational complexity of the replacement in-kind alternative, the City has requested that staffing requirements be assumed to include one additional operator for all three shifts and one additional maintenance worker for one shift per day in comparison to the cloth disk filter alternatives. Base O&M costs such as parts and consumables on ancillary equipment such as pumps, valves, etc., are assumed to be equal for all three alternatives. Phase 1 cloth disk filter alternatives have an additional $143,700/yr parts replacement cost associated with the disks themselves.

### 7.7.4 Chemical Assumptions

Considering the three alternatives evaluated do not have any substantial chemical costs associated with operation, it is assumed that any chemical treatment of the process units would be the same for all alternatives. Because this analysis is only intended to determine which alternative is better, these costs are not included in the following life-cycle evaluation.

### 7.7.5 Power Assumptions

Based on the existing pumping system, which is sized for 23 mgd peak flow, the replacement in-kind alternative assumes two of the 8,000 gpm raw feed pumps and one of each ancillary pump/blower is required for standard operation. Although regular flow through the ACT treatment train (Plant 2) side is not expected to exceed 13 mgd given the current operational strategy during the planning period, power consumption assumes peak design conditions to

provide worst case scenario costs for comparing alternatives. Table 7.11 summarizes the replacement in-kind equipment and energy consumption estimates used in this analysis.

Table 7.11 Replacement In-Kind Energy Consumption

| Description | hp | kW | Hrs/day | kWh/day |
|---|---|---|---|---|
| Raw Water Pump #5 | 125 | 93 | 24 | 2,237 |
| Raw Water Pump #6 | 125 | 93 | 24 | 2,237 |
| Backwash Supply Pump #1 | 75 | 56 | 4 | 224 |
| Air Scour Blower #1 | 150 | 112 | 4 | 447 |
| Subsurface Wash Pump #1 | 150 | 112 | 4 | 447 |
| Waste Backwash Pump #1 | 100 | 75 | 4 | 298 |
| **TOTAL ENERGY CONSUMPTION PER YEAR** | | | | **1,605,865** |

For the cloth disk filter alternatives, it is assumed new, smaller pumps would be installed to meet Phase 1 flow requirements (i.e., 16 mgd or 11,111 gpm). For the retrofit alternative, the pump(s) would be required to pump against 30 ft of head loss. Assuming 75-percent efficiency, the pump(s) would require a 125 hp motor and would likely operate for 24 hrs/day. This would result in an annual energy consumption of 816,600 kWh/yr. For the standalone package alternative, the pump(s) would only be required to pump against 20 ft of head loss. Using the same assumptions, the pump(s) would require a 100 hp motor and would result in 653,300 kWh/yr energy consumed. In addition to the influent pumps, the manufacturer indicates that the cloth disk filter system requires approximately 15,540 kWh/yr to operate including drive mechanisms and backwash/solids waste pumps. Table 7.12 summarizes the energy consumption calculations for both cloth disk filter alternatives.

Table 7.12 Cloth Disk Filter Alternatives Energy Consumption

| Description | Phase 1 Retrofit with Cloth Disk Filters | Phase 1 Package Cloth Disk Filters |
|---|---|---|
| Influent Feed Pumps | | |
| Pump Head | 30 ft | 20 ft |
| Flow | 11,111 gpm | 11,111 gpm |
| Assumed Efficiency | 75% | 75% |
| Required hp | 113 hp | 75 hp |
| Provided hp | 125 hp | 100 hp |
| Operational hrs/day | 24 hrs/day | 24 hrs/day |
| Energy Consumption | 816,600 kWh/yr | 653,300 kWh/yr |
| Cloth Disk Filter System | | |
| Energy Consumption(1) | 15,540 kWh/yr | 15,540 kWh/yr |
| **TOTAL ENERGY CONSUMPTION** | **696,040 kWh/yr** | **559,940 kWh/yr** |

Notes:
(1) Includes drive mechanism and backwash/solids waste pump cost as well as associated instrumentation.

### 7.7.6 Life-Cycle Alternatives Comparison

Table 7.13 presents the summarized life-cycle cost and the net present value of each alternative. Given the costs presented in the table as well as the age of the existing concrete and the inherent complexity of retrofit projects, installing package cloth disk filters by 2022 is recommended. The

cost difference between the retrofit and packaged option is not significant and modifications to the existing structure could be more challenging and intensive than estimated here. The replacement in-kind alternative is not recommended at all due to operational complexity and the net present value shown in Table 7.13 below. During pre-design, further consideration should be given to the phasing of the tertiary filters based on plans for ACT treatment train (Plant 2) plant operation and influent flow conditions.

Table 7.13 Life-Cycle Cost Comparison of Alternatives

| | Replacement In-Kind | Phase 1 Retrofit Cloth Disk Filters | Phase 1 Package Cloth Disk Filters |
|---|---|---|---|
| Year of Completion | 2019 | 2022 | 2022 |
| Phase 1 Capital Cost | $4,336,000 | $5,617,000 | $6,079,000 |
| Phase 2 Capital Cost(1) | $0 | $2,240,000 | $3,065,000 |
| Operational Cost | | | |
| Electrical | | | |
| 2017 to Year of Completion | $258,000 | $258,000 | $258,000 |
| Year of Completion to 2037 | $258,000 | $99,900 | $80,300 |
| Operational Cost (continued) | | | |
| O&M (staffing and consumables) | | | |
| 2017 to Year of Completion | $591,400(2) | $591,400(2) | $591,400(2) |
| Year of Completion to 2037 | $591,400(2) | $143,700(3) | $143,700(3) |
| **Net Present Value (Cost)** | **$13,633,000** | **$9,377,000** | **$9,555,000** |

Notes:
(1) Phase 2 capital costs are shown for reference. However, as discussed in Section 7.7.2 above, Phase 2 is not needed during the planning period and is not included in the net present value determination.
(2) O&M costs specifically allocated to personnel.
(3) O&M costs specifically associated with annual parts replacement.

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 8

## Wastewater Treatment System

## Advanced Water Treatment

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Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 8: ADVANCED WATER TREATMENT

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Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:23:07-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 8

# ADVANCED WATER TREATMENT

## 8.1 Purpose

The purpose of this chapter is to describe AWT alternatives needed at some point in the future to reduce effluent TDS concentrations. This chapter includes recommendations based on initial investigations. However, final decisions about specific technologies or layouts should be determined during the preliminary and final design stages of each associated project.

## 8.2 Conclusions and Recommendations

- The 2015 Salinity Study commissioned by the City indicated that the most effective method of TDS mitigation was source control methodologies. Therefore, the City intends to work with RPU and the CSDs to manage TDS in potable source waters below 400 mg/L and in influent wastewater streams to meet the effluent TDS discharge limit of 650 mg/L.
- The City has implemented pretreatment program ordinances to limit the wastewater TDS concentrations from each of the CSDs to the agreed permit levels.
- This chapter provides a comparison of technical alternatives for end-of-pipe treatment if potable water and wastewater source control strategies become unable to satisfy effluent requirements in the future.
- For this Master Plan, it is assumed that, if an end-of-pipe project were implemented, it will reduce effluent TDS by 50 mg/L. This target should provide an adequate amount of time between any subsequent project.
- To meet TDS removal goals, RO is selected as the primary AWT technology. Other technologies for comparable results are available and should be evaluated during preliminary design.
- Both on-site and off-site RO treatment were considered. However, off-site RO treatment was eliminated early in the analysis due to the excessive project costs in comparison with on-site RO alternatives and other potential regulatory challenges.
- On-site RO treatment alternatives evaluated seven brine disposal alternatives. Alternatives featuring RO treatment and hauling of brine (Alternatives 1A and 1B) result in too many trucks per day to be feasible. RO treatment followed by brine concentration and varying brine disposal methods (Alternatives 2A, 2B, and 2C) are feasible and reasonably cost-effective. Piped disposal of RO brine without concentration (Alternative 3) would be cost prohibitive. RO treatment followed by brine softening (Alternative 4) would result in both a solid and liquid waste stream for disposal increasing complexity and cost; the City did not consider this alternative further. Alternatives 2A, 2B, and 2C were evaluated for life-cycle costs. Alternative 2C, RO treatment followed by EDR brine concentration and piped disposal of brine, is the most expensive of the three

alternatives, but eliminates concerns associated with truck hauling. For planning purposes, Alternative 2C is used in this Master Plan.

- The City should conduct pilot testing of the Alternative 2C configuration to confirm performance of the treatment process.

## 8.3 Background

In 2015, the City commissioned Carollo to complete a Salinity Study to recommend the most feasible and cost-effective method for the City to achieve future reliable NPDES effluent permit compliance for TDS. During that time, the study concluded that source control of TDS was the most cost-effective method for minimizing effluent TDS concentrations. To that end, the City is working with RPU and the CSDs to manage source TDS concentrations and will continue to monitor effluent TDS concentrations closely. In addition, the City recently implemented ordinances to limit the wastewater TDS concentrations from each of the CSDs to their agreed permit levels. Should it become apparent in the future that source control strategies are incapable of keeping up with increasing TDS concentrations, end-of-pipe treatment alternatives will be considered. This chapter discusses possible technologies for that end-of-pipe treatment specifically targeting brine disposal methods.

## 8.4 Historical TDS Trends and Future Projections

Reviewing effluent TDS data from December 1985 to October 2018 shows considerable variation in TDS when using a 1-year rolling average. However, the City is in the process of securing a revised NPDES permit using a waste load allocation compliance matrix based on a 10-year rolling average discharge limit that more accurately reflects the regional drought cycles. Figure 8.1 shows a 1-year rolling average in grey along with a 10-year rolling average in blue.

The City intends to manage future effluent TDS concentrations with potable and wastewater source control strategies in collaboration with RPU and the CSDs. Therefore, it is impossible to estimate future projections based on historical data. For that reason, no projections will be made by this Master Plan. The City will actively monitor TDS concentrations and intends to act promptly if and when an end-of-pipe alternative is needed.

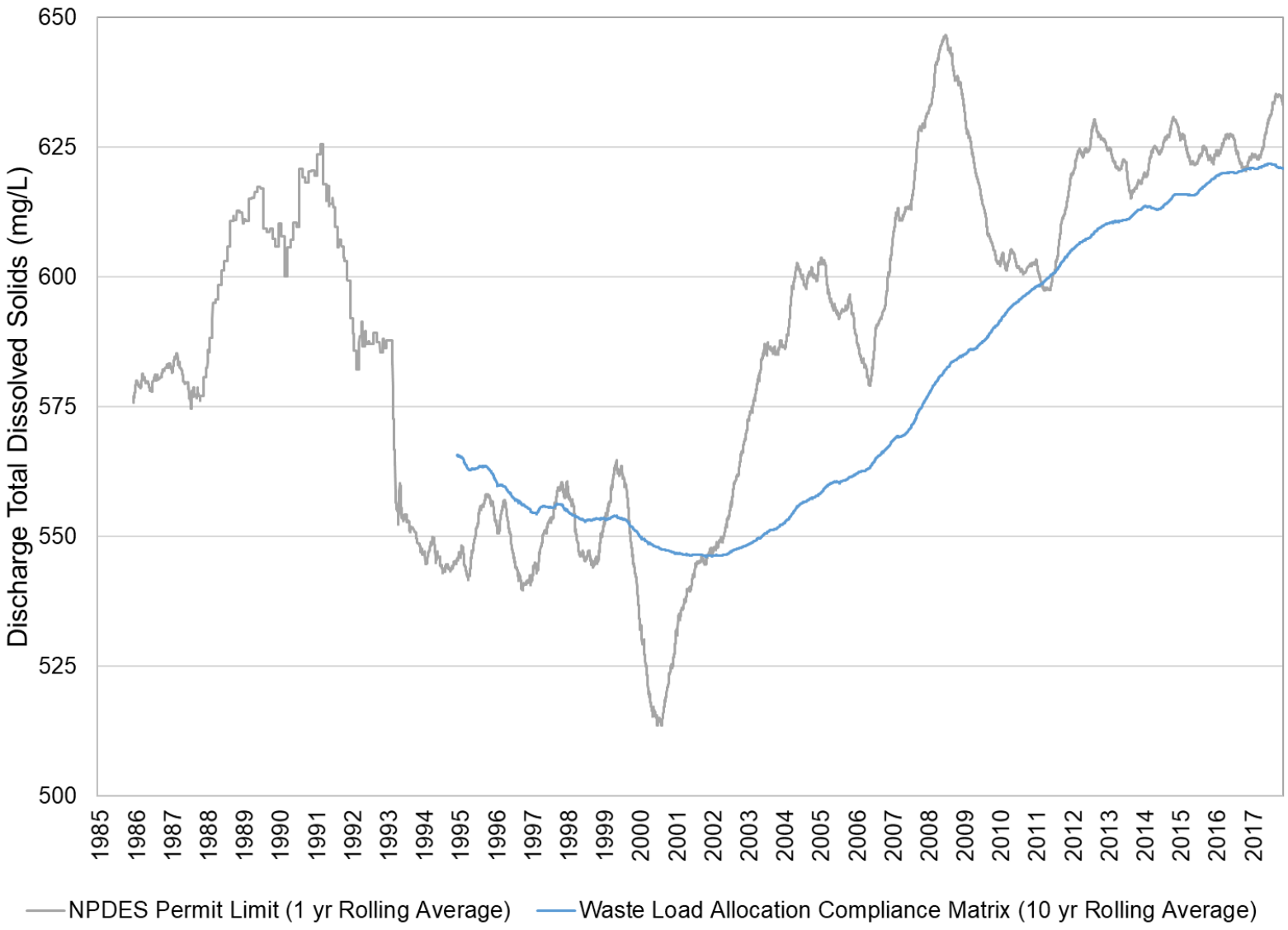


Figure 8.1 TDS Historical 1-Year and 10-Year Rolling Average Trends

## 8.5 Basis of Comparison

There are many technologies available that can remove TDS from MBR effluent. For this Master Plan effort, RO was selected as the primary treatment technology for TDS removal because it is the most commonly used salinity removal process in wastewater treatment. Given the wastewater effluent quality, specifically silica and calcium concentrations, the RO system alone is assumed to be capable of achieving a recovery of 82 percent. Figure 8.2 shows the process flow diagram of the RO train that would result in a blended effluent concentration to meet the target TDS reduction of 50 mg/L.

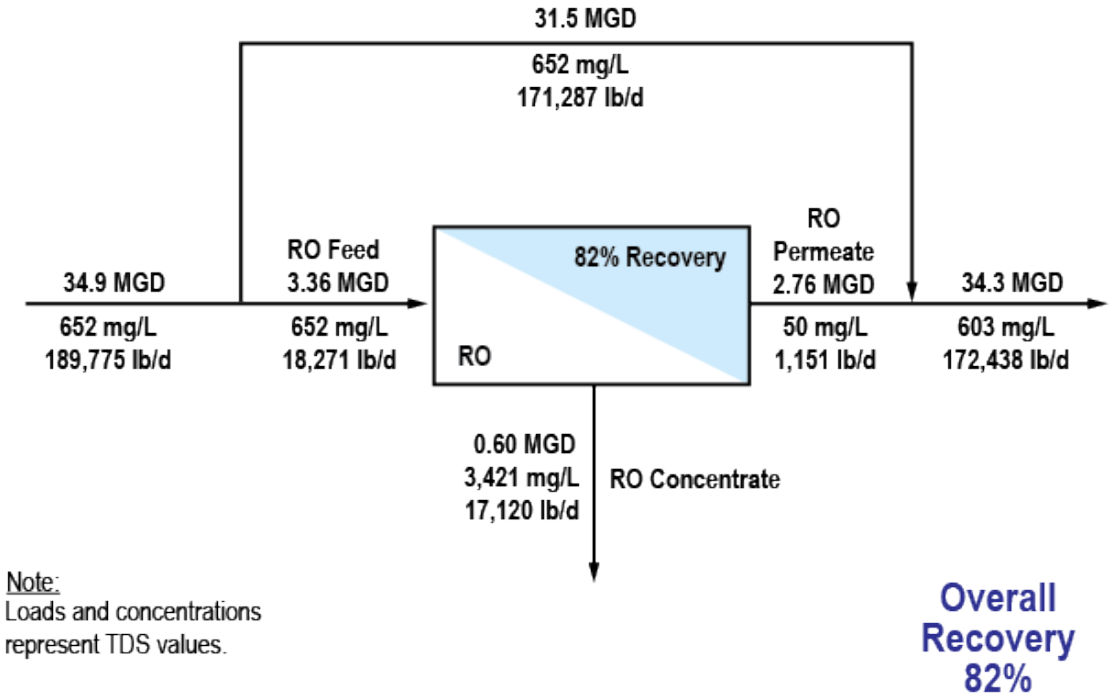


Figure 8.2 Process Flow Diagram of RO System for TDS Reduction in Effluent Water Quality

For demonstration purposes, the following analyses will use a plant effluent flow of 34.9 mgd and effluent TDS concentration of 652 mg/L. This translates to 189,775 ppd of TDS in the effluent flow stream. Because the goal is to only remove 50 mg/L of TDS, a substantial portion of the plant effluent flow would bypass the RO system. In Figure 8.2 above, 31.5 mgd of flow would bypass the RO system. The RO system would treat 3.36 mgd of flow and produce 2.76 mgd of low TDS product water (permeate), based on 82-percent recovery. The RO permeate was assumed to have a TDS concentration of 50 mg/L. When blended with the bypass stream, this would result in 34.3 mgd of final plant effluent with a blended effluent TDS concentration of 603 mg/L.

The reject water, or concentrate from the RO process, would total 0.6 mgd and would be discharged as a waste stream. The TDS concentration in this waste stream would be approximately 3,420 mg/L representing a TDS load of 17,120 ppd. This means that the use of a 3.36 mgd RO system is expected to remove 17,120 ppd of TDS from the RWQCP plant effluent.

Figure 8.3 shows the proposed location of the RO treatment facility in red. RO permeate would be blended with MBR treatment train (Plant 1) permeate at the influent channel of Chlorine Contact Basin 2.

Figure 8.3 Proposed Location of On-Site RO Treatment

## 8.6 Brine Disposal Alternatives

The most challenging aspect of implementing RO treatment is the disposal of the RO concentrate stream, or brine. This stream typically has high concentrations of TDS, silica, calcium, etc. and as a result has a high potential to form scale within tanks, pipes, and equipment. For the City, the most cost-effective disposal method available for brine is the IEBL. The IEBL is a pipeline running from approximately San Bernardino to Orange County. There are various member agencies that own capacity in the IEBL. Currently, all capacity in the IEBL is allocated, however, it is likely that one of these agencies would be willing to sell the City rights to discharge into the IEBL. In addition to the need to purchase capacity in the IEBL, the City would be required to purchase treatment capacity from the OCSD because the IEBL delivers flow to their facilities where it receives treatment before being discharged to the ocean. The remainder of this Chapter focuses on methods of disposing and concentrating this waste stream to determine the most cost-effective alternative.

The following alternatives were considered and are presented below:

- Alternative 1A: On-Site RO with Total Brine Hauling.
- Alternative 1B: On-Site RO with Partial Evaporation of Brine and Hauling.
- Alternative 2A: On-Site RO with EDR Brine Concentration and Total Brine Hauling.
- Alternative 2B: On-Site RO with EDR Brine Concentration and Partial Evaporation of Brine and Hauling.
- Alternative 2C: On-Site RO with EDR Brine Concentration and Piped Discharge to the IEBL.

- Alternative 3: On-Site RO and Piped Discharge to the IEBL.
- Alternative 4: On-Site RO with Near ZLD and Total Hauling.

Additionally, off-site RO alternatives were evaluated and were presented to the City at the AWT Workshop. These alternatives assumed that it would be possible to locate a remote treatment facility near the IEBL which would also be in close proximity to the Jurupa CSD sewer main that transports flow to the RWQCP. Intercepting and extracting wastewater from the sewer would provide the source water for treatment. This remote facility would require the following:

- Procurement of a site.
- Preliminary Treatment.
- Primary Treatment.
- Secondary Treatment.
- Microfiltration.
- RO.
- UV/AOP.
- Remineralization.

The resulting water would be of a very high quality. Instead of discharging and/or blending this water with RWQCP effluent flow, this water would be put to better use in a groundwater replenishment application. However, the cost of such remote facilities would be high and therefore cost prohibitive. Preliminary cost estimates showed that off-site treatment alternatives were more than double the cost of on-site treatment alternatives and presented significantly more challenges. For example, in addition to requiring a site location near both the IEBL and the Jurupa CSD truck sewer, the site would need to be located within proximity to a product water discharge location (i.e., a groundwater recharge basin or the Santa Ana River). In addition, the City's discharge permit would need to be revised to ensure that the City was given credit for the TDS removal if the remote facility product water was not directly blended with RWQCP plant effluent. Also, a remote facility would require City staff to travel off-site for operations, sampling, and maintenance tasks. Despite the benefit of freeing up treatment capacity at the RWQCP, the costs and complexities of such a project were considered too high and for these reasons, the off-site RO alternatives were not evaluated further.

### 8.6.1 Alternative 1A: On-Site RO with Total Brine Hauling

In this alternative, brine would be temporarily stored in a holding tank before being pumped into trucks with two liquid tanks (6,000 gallons each) and disposed of into the IEBL at the Chino I Desalter facility located approximately 10 miles from the RWQCP. Figure 8.4 shows the process flow diagram for this alternative. Given the waste flow of 0.6 mgd, 50 trucks per day, each transporting 12,000 gallons, would be required to dispose of this flow at the Desalter. The number of trucks coupled with plant security and vehicular access constraints make this alternative infeasible.

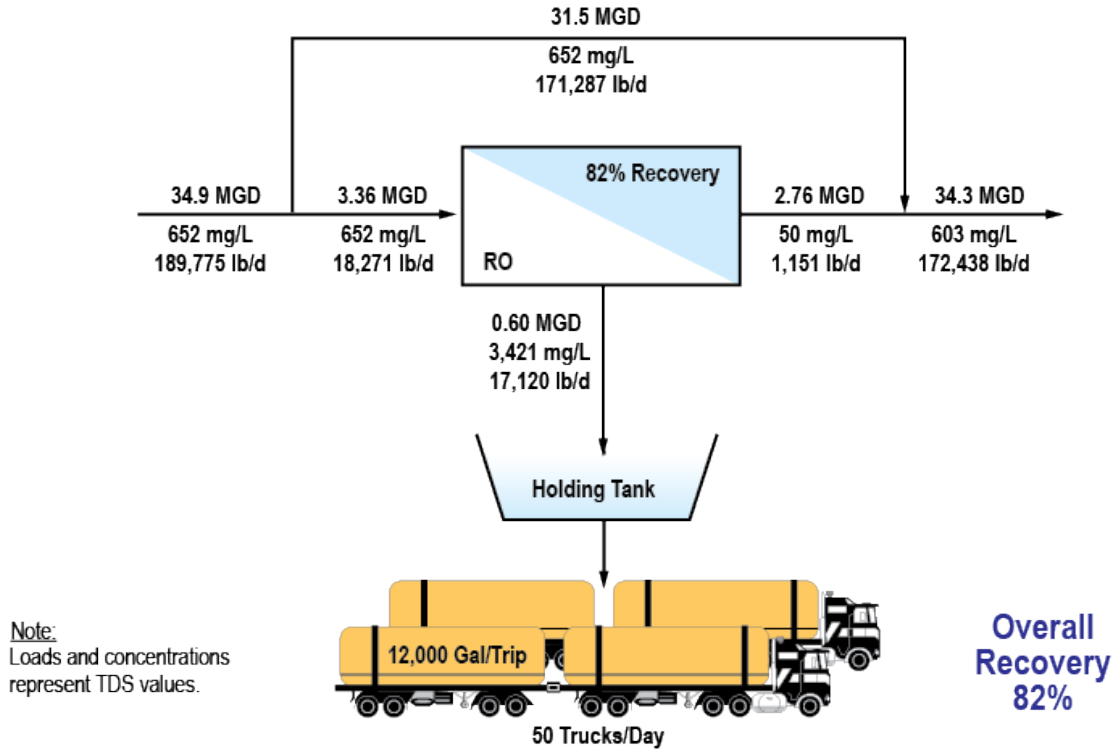


Figure 8.4 Alternative 1A: On-Site RO with Total Hauling

### 8.6.2 Alternative 1B: On-Site RO with Partial Evaporation of Brine and Hauling

This alternative is similar to Alternative 1A except instead of a holding tank, the brine is stored in triple-lined evaporation ponds to allow a portion of it to be evaporated, thereby reducing the volume that would need to be hauled away. The old sludge drying beds located on the south west corner of the plant make up approximately 8 acres of "free space" at the RWQCP. Converting these beds to 8 acres of lined evaporation ponds would allow for significant storage and surface area. Figure 8.5 shows the process flow diagram for this alternative while Figure 8.6 shows the proposed location of the lined ponds in red. Based on historical evaporation rates from 1948 to 2005, approximately 45,000 gpd could evaporate from the lined ponds. However, this approach only reduces the number of double-tanker truck loads (12,000 gallons per load) necessary for disposal from 50 to 46 trucks per day. For the same reasons as Alternative 1A, this alternative is not feasible.

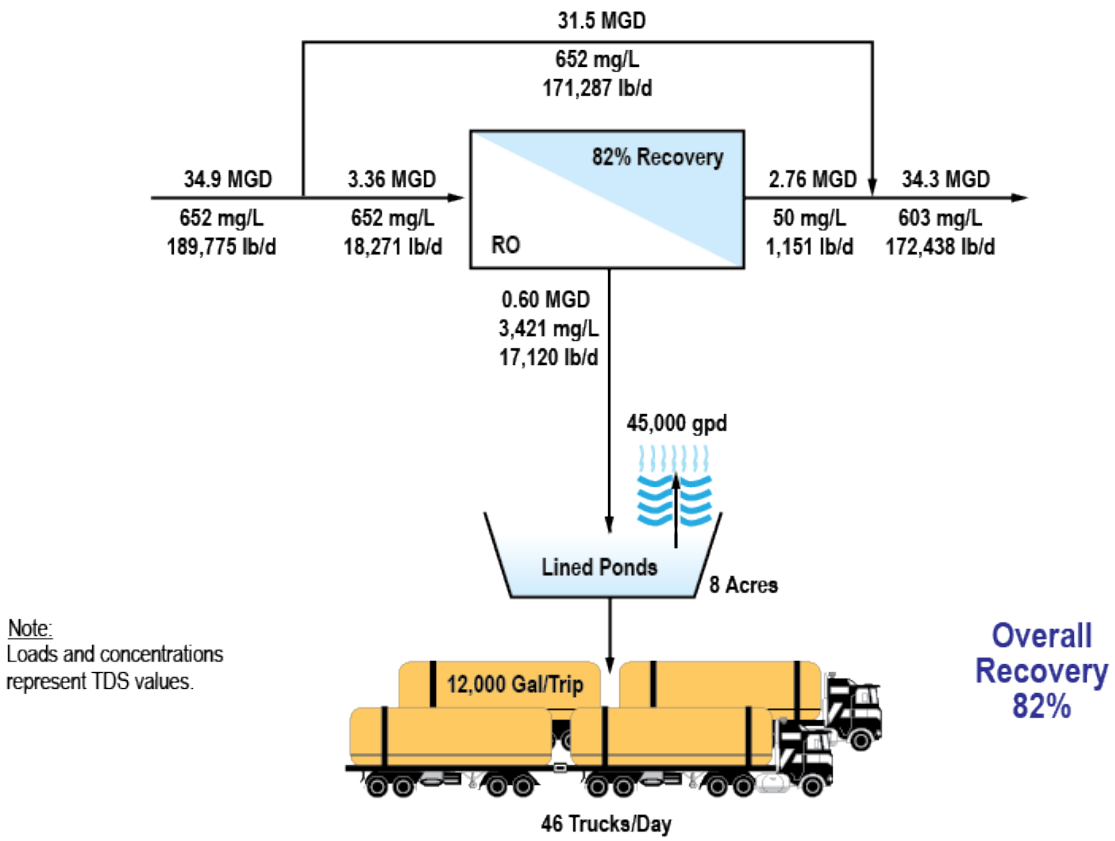


Figure 8.5 Alternative 1B: On-Site RO with Partial Evaporation of Brine and Hauling

Figure 8.6 Proposed Location of Lined Evaporation Ponds

### 8.6.3 Alternative 2A: On-Site RO with EDR Brine Concentration and Total Brine Hauling

In order to reduce the quantity of brine beyond that possible with evaporation ponds on-site, brine concentration would be required. While there are various available technologies, this alternative considers EDR because it can achieve concentration without producing a solid byproduct. EDR technology separates charged ionic species (i.e., sodium, calcium, etc.) into a more concentrated brine stream returning higher quality water back for blending. This would allow the brine flow rate to be reduced but the TDS concentration would increase. Given the wastewater effluent quality, specifically silica and calcium concentrations, the EDR system alone is assumed to meet a recovery of 75 percent. This increases the system recovery from 82 percent (with RO alone) to 95.5 percent (with RO and EDR) reducing the brine from 0.60 mgd to 0.15 mgd. Figure 8.7 shows the process flow diagram for this alternative. In addition, this alternative assumes that trucks with two liquid tanks (6,000 gallons each) could be used to dispose of the brine. This reduces the number of truck loads from 46 down to 12 per day, which is a feasible number for the RWQCP.

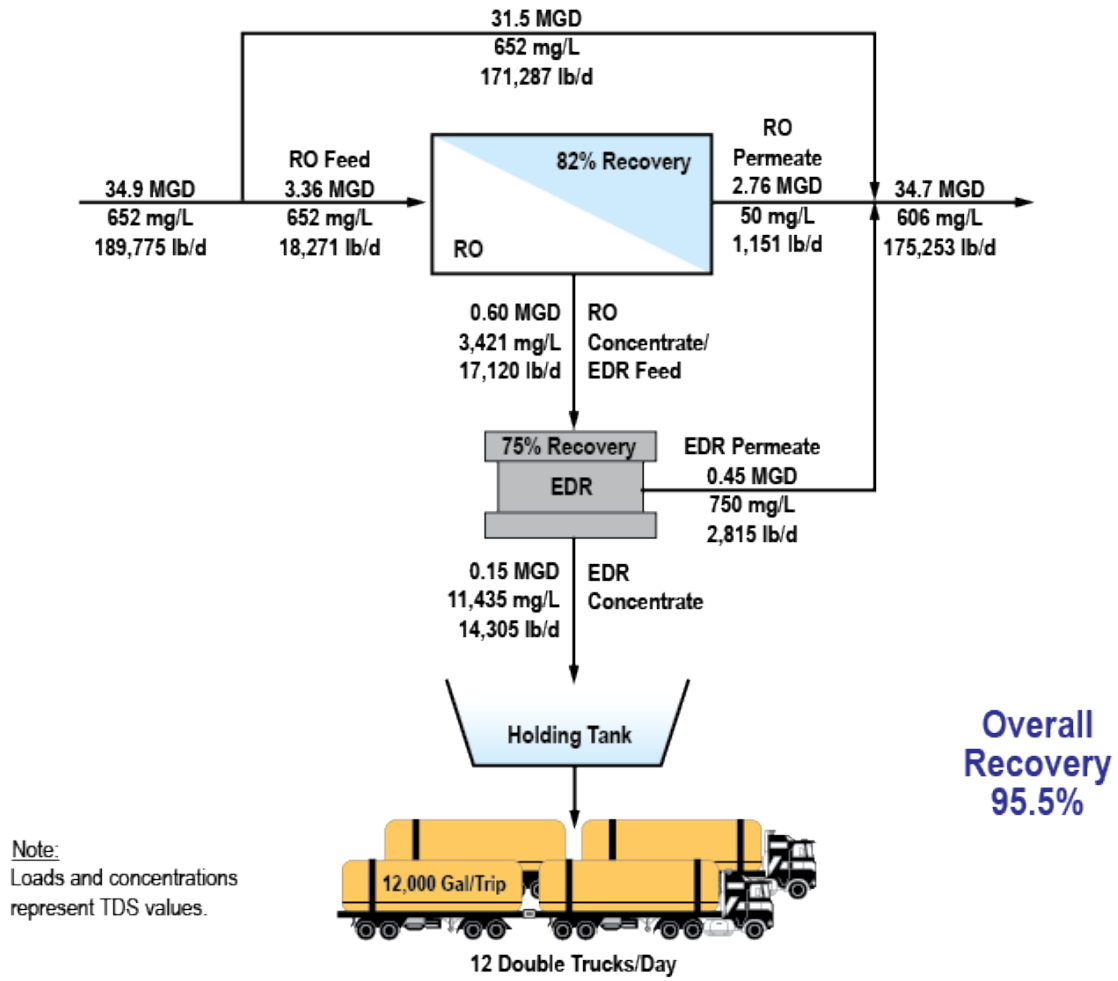


Figure 8.7 Alternative 2A: On-Site RO with EDR Brine Concentration and Total Hauling

### 8.6.4 Alternative 2B: On-Site RO with EDR Brine Concentration and Partial Evaporation of Brine and Hauling

This alternative is a combination of Alternative 1B and Alternative 2A. As shown in the Figure 8.8, it involves on-site RO treatment with EDR concentration and 8 acres of triple lined evaporation ponds to reduce the brine volume further. In this case, the anticipated brine volume for discharge to the IEBL would drop to about 105,000 gpd. In comparison to Alternative 2A, this approach reduces the number of double-tanker truck loads (12,000 gallons per load) necessary for disposal from 12 to 8 trucks per day; also a workable number of truck loads.

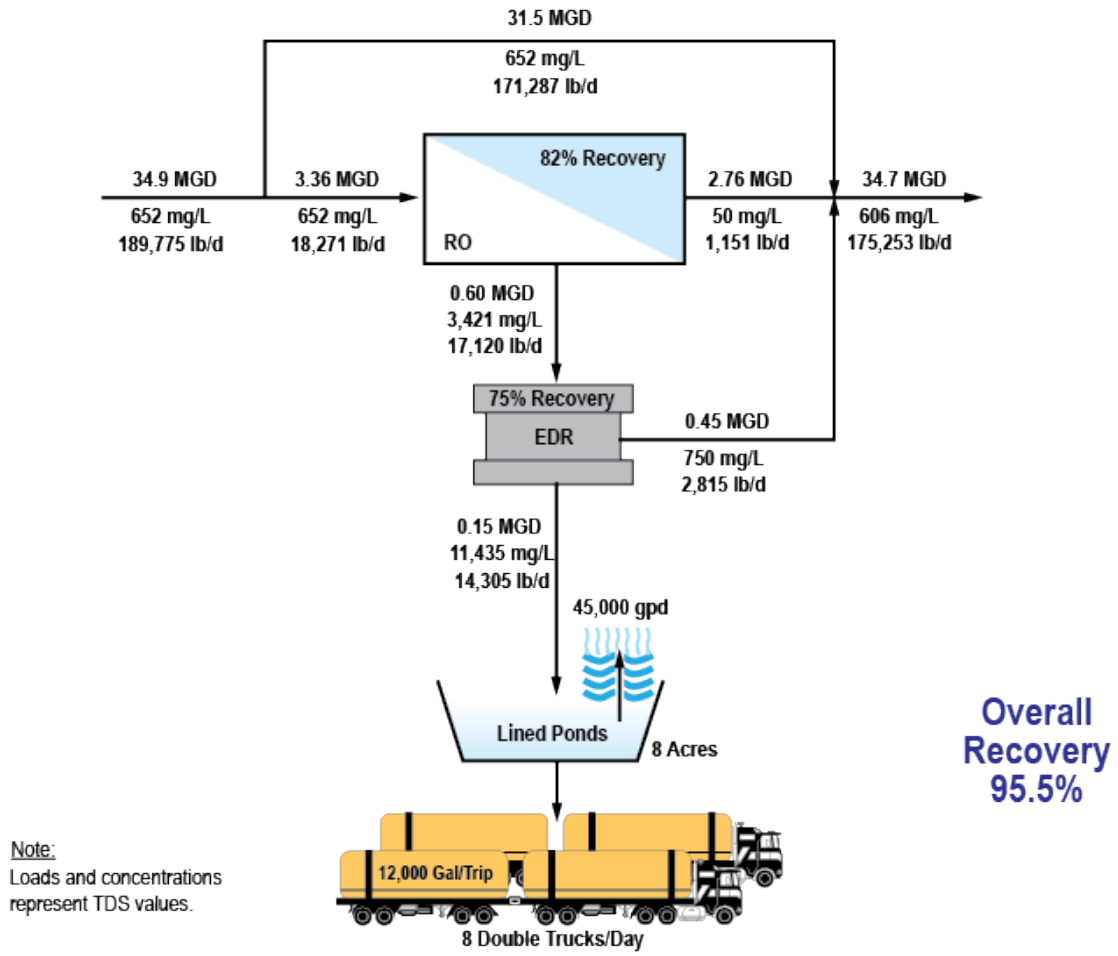


Figure 8.8 Alternative 2B: On-Site RO using EDR and Partial Evaporation for Brine Concentration with Hauling

### 8.6.5 Alternative 2C: On-Site RO with EDR Brine Concentration and Piped Discharge to the IEBL

This alternative again utilizes on-site RO treatment with EDR brine concentration, but instead of trucks, brine is discharged through a 4-mile-long pipeline to the IEBL. This pipe would carry flow from the RO facility across the Santa Ana River and along Camino Real to Jurupa Road. However, the brine stream would be expected to have a TDS of approximately 11,500 mg/L and a high scaling potential. As a result, regular maintenance would be required to maintain the pipelines to minimize scale formation. For this reason, this alternative assumes that brine would be stored and

then pumped to the IEBL for 8 hours a day. Batching the brine pumping in this way allows higher velocities to be used in the pipeline and also allows for flushing to prevent scale buildup. A 1-hour flush cycle with recycled water would be used to clean the pipe after every use. Additionally, due to the inherent scaling potential, this alternative assumes 2 identical pipes would be installed in the same trench. This approach would provide redundancy in the case of pipe failure or blockage, and also allows for duty/standby operation of the pipes to allow for period acid cleaning. While this alternative eliminates the plant security and traffic issues associated with the previously mentioned alternatives, it does pose significant challenges to pipeline maintenance. The City would need to regularly inspect the RWQCP brine line for scale formation and maintain the lines with periodic acid washes. Figure 8.9 shows the process flow diagram for this alternative.

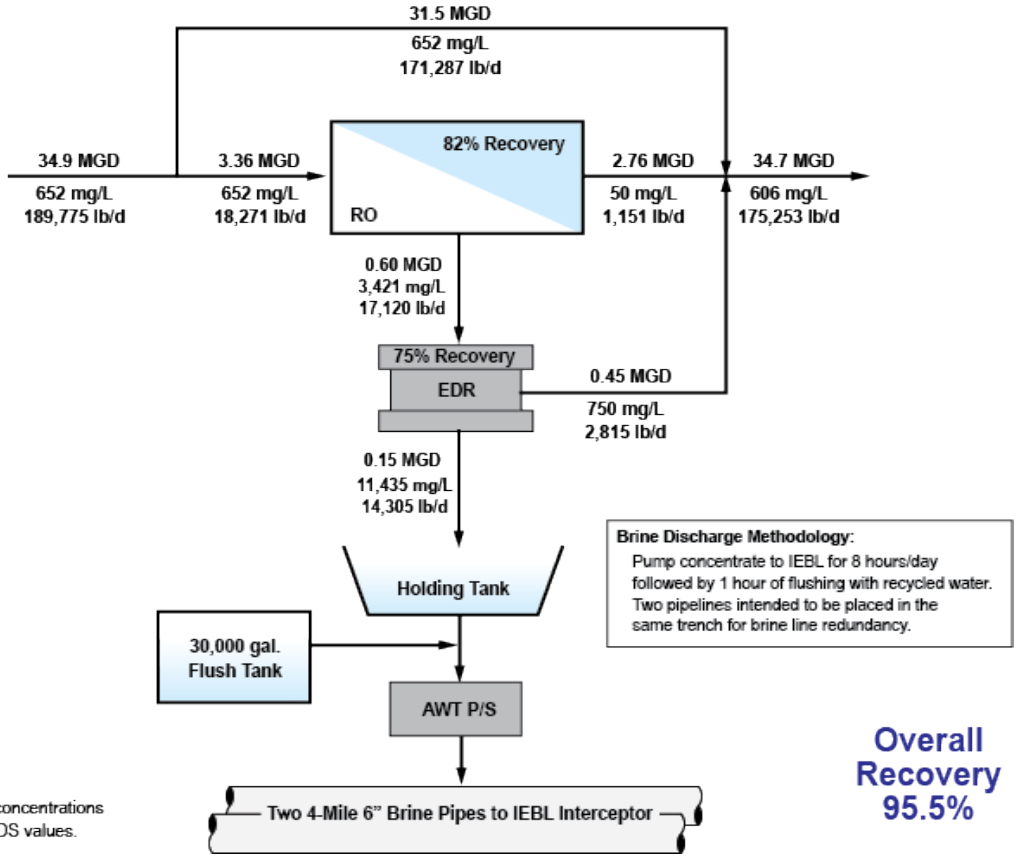


Figure 8.9 Alternative 2C: On-Site RO with EDR Brine Concentration and Piped Discharge to the IEBL

It is important to note that when brine is exposed to the atmosphere, such as in the holding tank shown in Figure 8.9, this creates conditions to allow scale to form. This would need to be considered during the design phase should this alternative be considered for implementation.

### 8.6.6 Alternative 3: On-Site RO with Piped Discharge to the IEBL

This alternative includes discharge of 0.60 mgd of RO brine directly to the IEBL without concentration using the double pipeline approach. It is similar to Alternative 2C. Due to the larger brine flow, the pipelines would increase in size from 6-inch diameter to 10-inch diameter. Figure 8.10 shows the process flow diagram for this alternative.

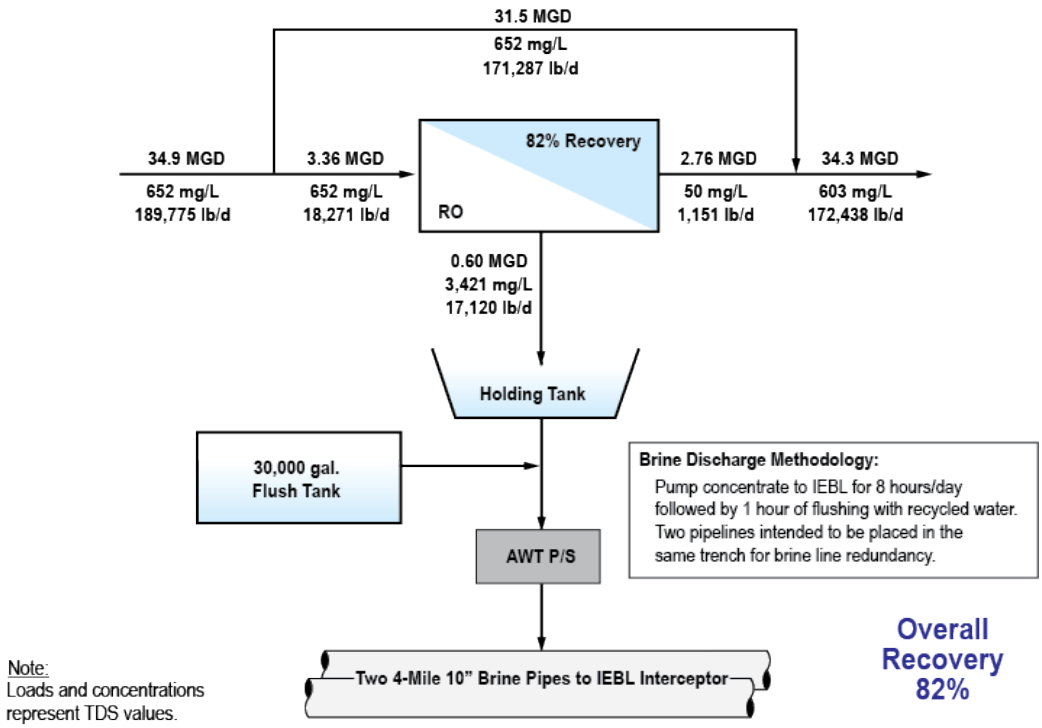


Figure 8.10 Alternative 3: On-Site RO with Piped Discharge to the IEBL

### 8.6.7 Alternative 4: On-Site RO with Near ZLD and Total Hauling

This alternative presents another option to concentrate RO brine and takes the recovery to near ZLD conditions. There are several treatment configurations that could be considered for this alternative that could include proprietary processes such as Hero or VSEP for example. For this analysis, we have considered a generic softening technology such as lime/soda ash softening to remove calcium, magnesium, and silica from the liquid stream. The overall results would be similar for other treatment arrangements. The softening process will produce a solids byproduct that will need to be disposed of. For this analysis, it is assumed that approximately 7 tons of solids would be removed per day from the softener. Because the softener is designed to precipitate solids out of solution, a polishing filter is required to prevent solids carry over to the next process. The purpose of the softening step is to remove the constituents that limit water recovery in the RO process (calcium and silica). With the concentration of these constituents reduced, the water can be treated in a secondary RO unit to further concentrate the brine. The RO unit is assumed to operate at an 80 percent recovery, increasing the overall system recovery to 96.4 percent, slightly higher than that of Alternative 2C. From the process flow diagram shown in Figure 8.11, 0.48 mgd

of 100 mg/L TDS product water would be returned for blending. Brine disposal for the 0.12 mgd brine would still require 10 double-tanker trucks a day.

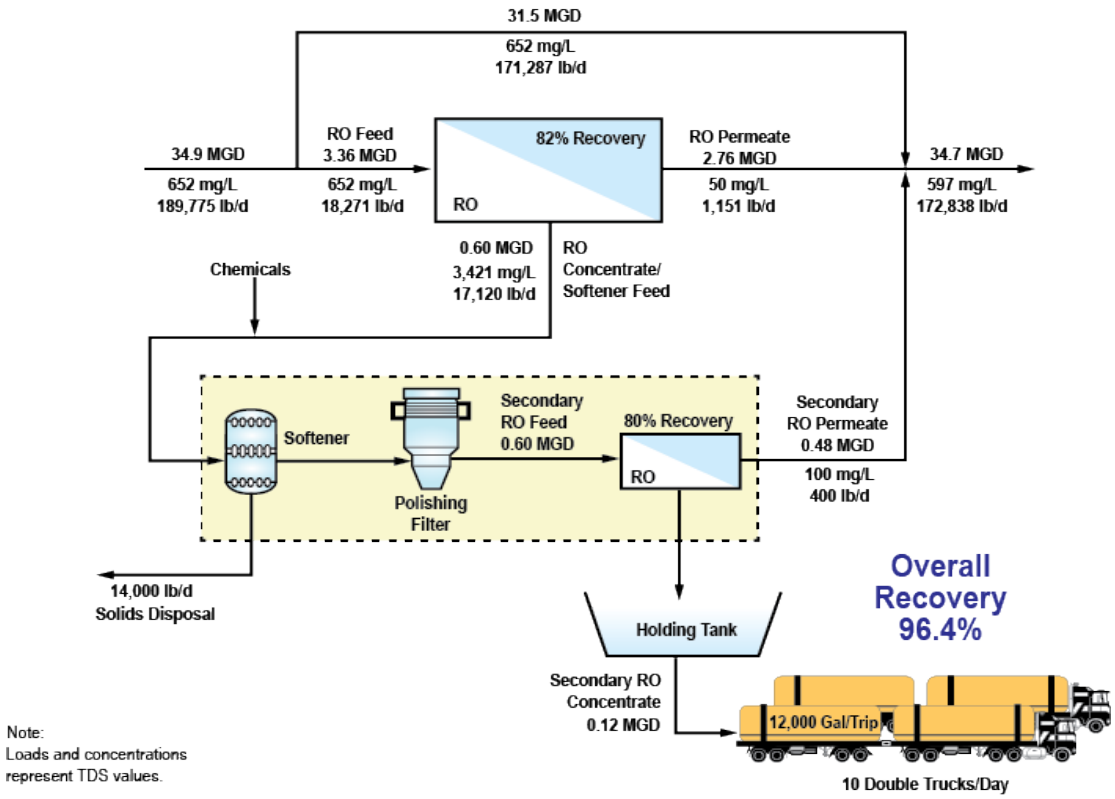


Figure 8.11 Alternative 4: On-Site RO with Near ZLD and Total Hauling

## 8.7 Economic Comparison of Disposal Alternatives

The following alternatives were considered in the economic comparison:

- Alternative 1A: On-Site RO with Total Brine Hauling.
- Alternative 1B: On-Site RO with Partial Evaporation of Brine and Hauling.
- Alternative 2A: On-Site RO with EDR Brine Concentration and Total Brine Hauling.
- Alternative 2B: On-Site RO with EDR Brine Concentration and Partial Evaporation of Brine and Hauling.
- Alternative 2C: On-Site RO with EDR Brine Concentration and Piped Discharge to IEBL.
- Alternative 3: On-Site RO and Piped Discharge to IEBL.
- Alternative 4: On-Site RO with Near ZLD and Total Hauling.

Project costs were calculated for all alternatives based on planning level estimates and include facilities costs, site development and yard work, electrical, instrumentation, and control. Also included is the cost to procure capacity in the IEBL and treatment capacity at OCSD. In addition, the various contingencies and allowances described in Volume 2, Chapter 4, Basis of Cost Estimates of this Master Plan are also included. The project cost comparison along with other comparative metrics are summarized in Table 8.1 below.

Table 8.1 shows the projected annual disposal costs, which includes the IEBL discharge fees, the OCSD treatment fees, and the hauling costs for both liquid and solids streams where applicable.

Trucking contracts can sometimes be volatile and variable meaning that the City would need to be diligent in establishing a broad portfolio of hauling contracts for all applicable alternatives described in this Master Plan in order to ensure economical hauling disposal. For this Master Plan, solids hauling is assumed to be $50/ton while liquid hauling is estimated at approximately $43/12,000 gallon trip.

Given the information presented in Table 8.1, these alternatives were discussed with the City at the AWT Workshop. Alternatives 1A and 1B were immediately classified as non-feasible. Alternatives 2A, 2B, and 4 were classified as non-ideal due to the trucking requirements, however they are considered feasible. Additionally, Alternative 4 was further discounted due to the solids handling requirement of 7 tons per day in addition to 10 double-tanker trucks for liquids disposal. While the RWQCP staff is accustomed to handling biosolids, softener solids waste is significantly different and presents its own challenges. Note that Alternative 4 could be modified to include a pipeline for brine disposal but that would increase the capital cost further. Alternatives 2C and 3 may be able to reduce project costs by repurposing abandoned sewer lines as carrier pipes for the brine disposal pipeline. However, the practicality of this would need to be investigated during preliminary design. Alternative 3 was considered to be cost prohibitive.

## 8.8 Life-Cycle Analysis of Alternatives

To help determine the most cost efficient solution, the three most feasible alternatives with the lowest costs, Alternatives 2A, 2B, and 2C, were further evaluated on a life-cycle basis.

### 8.8.1 Project Implement Schedule Assumptions

Based on the 4 mg/L per year TDS increase projection, the RWQCP is expected to reach the current effluent TDS limit of 650 mg/L in 2023. As the temporary basis of this analysis, all three alternatives are assumed to be fully constructed in 2023. Although a second AWT project would be required during the planning period as shown in Figure 8.1, this analysis is solely intended to distinguish between the three alternatives being considered. Therefore, the second AWT project is not included in the following evaluation. However, once revised TDS projections from the City are available, this analysis will be revised accordingly.

### 8.8.2 Capital Cost Assumptions

The capital costs shown in Table 8.1 above were assumed to apply in 2023. Prior to 2023, no capital costs are attributed to this analysis. Additionally, the lined ponds in Alternative 2B are assumed to provide sufficient capacity throughout the planning period. Additional capital costs would be required beyond the planning period for Alternative 2B to build another pond while the first is dredged for accumulated solids. Such additional costs have not been included in this analysis.

Annual cost of capital was calculated assuming a 30-year bond period and a 6-percent discount rate, as described in Volume 2, Chapter 4. Because the cost of capital is assumed to take place in 2023, the net present value of the capital cost is calculated for use in determining the annual cost of capital.

### 8.8.3 O&M Personnel and Consumables Assumptions

All alternatives being considered assume RO followed by EDR treatment. As a result, the personnel and consumables cost are assumed to be identical for all three alternatives and are therefore not included in this analysis.

Table 8.1 Economic Comparison of Disposal Alternatives for On-site RO Treatment

| Technology Alternative | Project Cost ($ Million) | Annual Disposal Fees ($k)(1) | O&M Requirement | Operator Attention | System Recovery | Technical Feasibility | Comments |
|---|---|---|---|---|---|---|---|
| 1A) Total Hauling | $36.5 | $1,215 | Low | Low | 82% | Not Feasible | 50 double-tanker truck (12,000 gallons per trip) loads per day |
| 1B) Partial Evaporation and Hauling | $38.1 | $969 | Low | Low | 82% | Not Feasible | 46 double-tanker truck (12,000 gallons per trip) loads per day |
| 2A) EDR with Total Hauling | $24.1 | $457 | Medium | Medium | 95.5% | Possibly Feasible | 12 double-tanker truck (12,000 gallons per trip) loads per day and lowest project cost |
| 2B) EDR with Partial Evaporation and Hauling | $26.2 | $379 | Medium | Medium | 95.5% | Possibly Feasible | 8 double-tanker truck (12,000 gallons per trip) loads per day and second lowest project cost |
| 2C) EDR with Piped Discharge | $38.7 | $270 | High [2] | Medium | 95.5% | Feasible | No hauling but higher project cost due to pipelines |
| 3) Piped Brine Discharge | $62.9 | $433 | Medium [2] | Low | 82% | Feasible | Highest cost due to pipelines and purchase of IEBL and OCSD capacity |
| 4) NZLD with Total Hauling | $40.4 | $543 | High | High | 96.4% | Possibly Feasible | Cost is high and includes hauling brine and solids |

Notes:
(1) Includes disposal of brine and solids as appropriate. Brine disposal costs include IEBL and OCSD annual fees based on the 2017-18 Fiscal Year rate structure.
(2) Piped disposal of brine will inherently have scaling issues requiring higher O&M of the pipeline.

### 8.8.4 Chemical Cost Assumptions

Alternatives 2A and 2B do not require any additional chemical treatment apart from those associated with RO and EDR treatment. These costs are assumed to be consistent for all three alternatives since each assumes the same treatment configuration. Alternative 2C requires regular pipeline maintenance in the form of chemical cleaning. For the purposes of this analysis, it is assumed that annual pipe flushes with 2-percent citric acid solution will be used to remove residual scale buildup within the brine pipelines. The 30,000-gallon flushing tank would be filled with recycled water and 5,100 lbs of citric acid powder would be dissolved in the tank to create the appropriate solution. The associated cost for this annual acid flush would be approximately $10,000 per year.

### 8.8.5 Power Assumptions

For all alternatives, the power requirements associated with the treatment processes would be the same and were not included in this analysis. For Alternatives 2A and 2B, it is assumed that the tanker trucks will be equipped with onboard suction pumps and that no additional facilities will be required to transfer brine from the holding tank or lined ponds to the tanker trucks. Alternative 2C, however, requires a small pump station to pump the brine to the IEBL 4 miles away. Preliminary analyses indicate a 60 hp motor would be required. Operating 9 hours a day, as discussed in Section 8.6.5, this pump would require 403 kWh/day or 146,977 kWh/yr. Power was assumed to cost $0.12/kWh.

### 8.8.6 Life-Cycle Alternatives Comparison

Table 8.2 presents the summarized life-cycle cost analysis of the alternatives. The costs in Table 8.2 indicate that Alternative 2A is expected to have the lowest annual cost. Adding evaporation ponds (Alternative 2B) reduces the brine volume for disposal slightly, but the higher capital cost offsets any saving in brine disposal costs. Alternative 2B has a slightly higher overall cost than Alternative 2A; about 3 percent higher, which could be considered about the same in terms of the accuracy of this analysis. Alternative 2C results in an annual cost that is about 40 percent greater than Alternative 2A. The additional cost is all attributed to the cost of installing the brine pipelines and pump station.

Table 8.2 Life-Cycle Cost Comparison of Alternatives

| | Alternative 2A: EDR + Total Hauling | Alternative 2B: EDR + Partial Evaporation and Hauling | Alternative 2C: EDR + Piped Discharge |
|---|---|---|---|
| Capital Cost Estimate | $24,005,000 | $26,101,000 | $38,616,000 |
| Annual Cost of Capital | $1,645,200 | $1,789,000 | $2,647,000 |
| Annual Electrical Cost [1] | $0 | $0 | $17,600 |
| Annual Chemical Cost [1] | $0 | $0 | $10,000 |
| Annual Disposal Cost [2] | $457,300 | $378,500 | $269,600 |
| Number of Trucks Daily | 12 | 8 | 0 |
| Hauling Cost | $187,700 | $125,100 | $0 |
| IEBL and OCSD Annual Cost | $269,600 | $253,400 | $269,600 |
| **Comparative Annual Cost** | **$2,102,500** | **$2,167,500** | **$2,944,200** |

Notes:
(1) Additional cost over and above the baseline cost for the treatment processes of RO and EDR.
(2) Annual disposal cost includes hauling, discharge, and treatment fees. IEBL and OCSD costs are based on the 2017-18 Fiscal rate structure.

While the two hauling alternatives, 2A and 2B have the lowest cost and are feasible, additional considerations are required, namely plant security concerns and vehicular access constraints. Furthermore, for any truck disposal alternative to be successful, contracts would need to be developed to ensure disposal. For example, a contract would need to be developed between the Chino Desalter Authority and the City to ensure access to the brine discharge station. Hauling contracts represent the most variability in the annual disposal cost and the City would need to be particularly vigilant when negotiating the contract to ensure a fixed price over a determined time period. For planning purposes Alternative 2C was selected. It has the highest capital cost ($14 million higher than Alternative 2A), but the lowest annual operating cost. During preliminary design, the City can decide whether to install the brine pipelines or dispose of the brine via hauling, which would effectively be the implementation of Alternative 2A. The brine pipelines could then be constructed as part of a future project.

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 9

## Wastewater Treatment System

## Disinfection

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City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 9: DISINFECTION

FINAL | June 2019

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City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 9: DISINFECTION

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.08.05 15:26:19-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 9

# DISINFECTION

## 9.1 Purpose

The purpose of this Chapter is to describe and discuss the Riverside RWQCP disinfection processes as they relate to meeting the discharge limitations established by RWQCB and show conformance with the DDW design and reliability requirements of Water Recycling Criteria or Title 22, Division 4 of the California Administrative Code. This Chapter describes the existing facilities following the Phase I Plant Expansion project, and proposed modifications to the disinfection process based on an on-site CT Study commissioned by the City. This study evaluated the capacity of the existing CCBs and conversion of the disinfection process to a free chlorine residual system. The Title 22 Report was updated as part of this process.

## 9.2 Conclusion and Recommendations

- CCB 2 was increased in size during the MBR Phase I Plant Expansion.
- A tracer study on CCB 2 recovered more than 98 percent of the tracer, indicating that CCB 2 has a high degree of plug flow characteristics.
- A CT study showed that the chlorine demand of MBR Plant effluent and ACT tertiary filter effluent was about the same and that free chlorine was able to meet the required bacterial and virus removal rates to satisfy the Title 22 regulations during bench testing.
- A full-scale on-site test at an average flowrate of 47-mgd showed that bacterial removal requirements could be comfortably achieved with a free chlorine residual.
- A draft updated Title 22 Report was submitted to the SWRCB requesting the use of free chlorine for disinfection. The City was granted conditional acceptance by the State in a letter dated May 6, 2019. The conditional acceptance includes operating CCB 2 at a free chlorine CT of 30 mg-min/L and a modal contact time of 29 mins at 46-mgd.
- The City is working through the requirements from the State to make the switch to free chlorine and to issue a Final Title 22 Report. Relatively minor modifications to the plant are required to make the switch to free chlorine disinfection. As such, no projects have been included in the CIP for disinfection improvements as part of the update of the Master Plan.

## 9.3 Background

The City Public Works is responsible for the collection and treatment of wastewater generated within the City and the community of Highgrove. Public Works is also responsible for the treatment and discharge of the wastewater from the Community Service Districts of Edgemont, Jurupa, and Rubidoux.

Wastewater treatment is performed at the RWQCP, which provides primary, secondary, and tertiary treatment. The RWQCP has undergone several expansion projects, and as part of the MBR Phase I Plant Expansion completed in 2017, the treatment capacity was increased from 40 to 46 mgd ADWF and expanded CCB 2.

## 9.4 Disinfection Criteria

California Title 22 standards are used for the evaluation in this Chapter. However, as of May 2019 an updated disinfection criteria was approved by the state. Per the existing NPDES Permit, the RWQCP is required to discharge disinfected tertiary effluent per Title 22 to meet the following criteria:

1. The filtered wastewater has been disinfected by either:
   a. 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 30 mg-min/L at all times (note that the previous Title 22 requirement by the City was 900 mg-min/L) with a modal contact time of at least 29 mins (previously 90 mins), based on peak dry weather design flow; or
   b. 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. A virus that is at least as resistant to disinfection as polio virus may be used for purposes of the demonstration.
2. The median concentration of total coliform bacteria measured in the disinfected effluent does not exceed an MPN of 2.2 per 100 mL 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 mL in more than one sample in any 30-day period. No sample shall exceed an MPN of 240 total coliform bacteria per 100 mL.

## 9.5 Existing Disinfection Facilities

As discussed in Volume 4, Chapter 1, Existing Facilities, the RWQCP has three CCBs for chlorination. The location of the CCBs are highlighted on Figure 9.1 and their inter-connection with the treatment processes is shown in the process flow diagram on Figure 9.2. Generally, effluent from the MBR treatment train (Plant 1) is treated in CCB 2 (see Figure 9.3 for normal operation configuration), and effluent from the tertiary filters 1 through 10 in the ACT treatment train (Plant 2) flows to CCB 1, and filtrate from filters 11 through 16 flows to CCB 3. Flow from CCB 1 can be directed to CCB 3. Sodium hypochlorite is dosed to all three CCBs to produce chloramines for disinfection. However, since a majority of the flow goes through the MBR side, CCB 3 has been taken offline. Flow from the ACT treatment train (Plant 2) flows through CCB 1 (no chlorination) and is comingled with MBR treatment train (Plant 1) effluent before entering CCB 2 for chlorination. In the future when flows increase CCB 3 will be brought online (Figure 9.3).There is no provision to add ammonia to the effluent streams. Formation of chloramines relies on residual ammonia from the MBR treatment train (Plant 1) and ACT treatment train (Plant 2) activated sludge treatment processes. Sodium bisulfite is used to dechlorinate the flow that is discharged to the Santa Ana River.

The existing disinfection facilities are listed in Table 9.1.

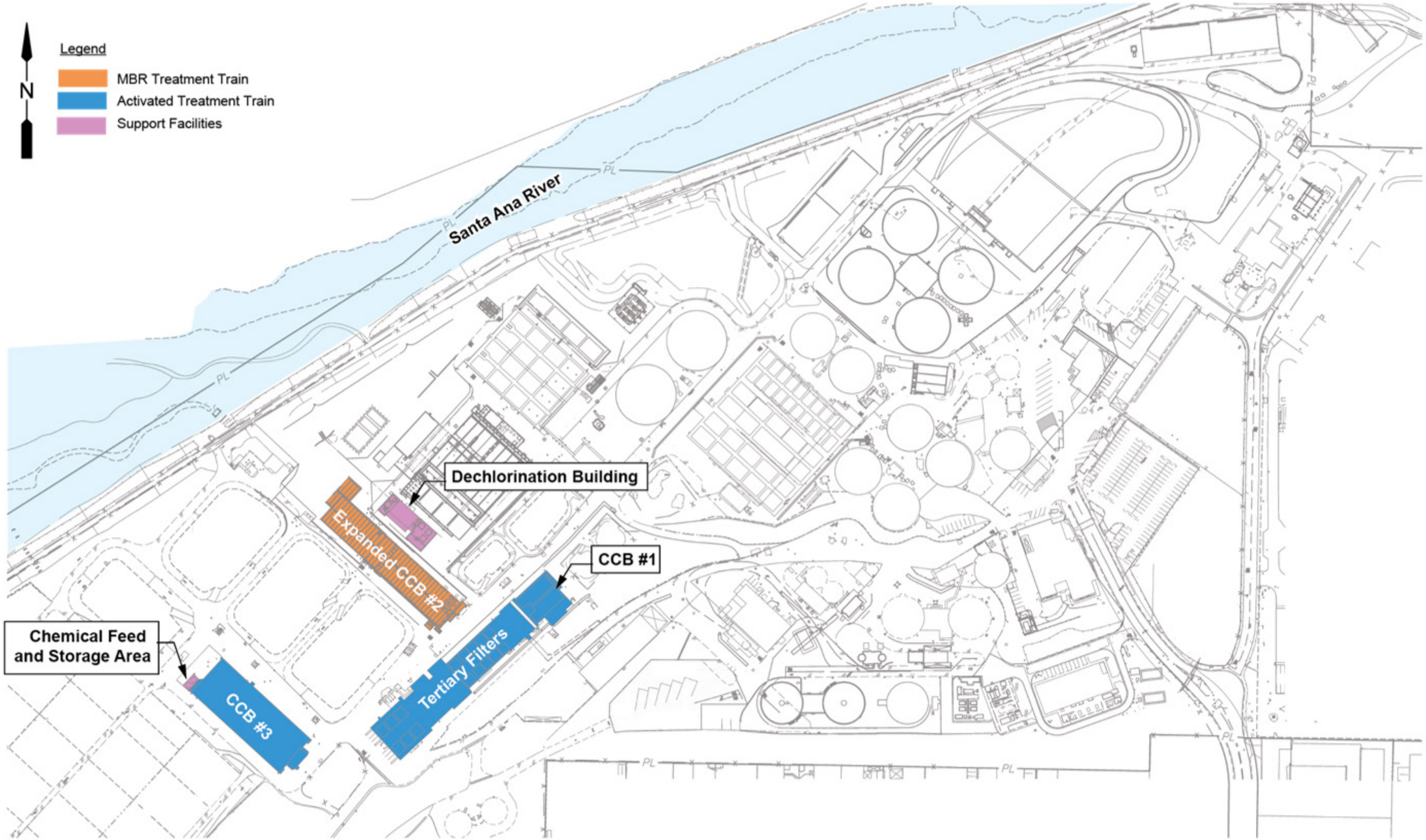


Figure 9.1 2017 Existing Facilities Layout

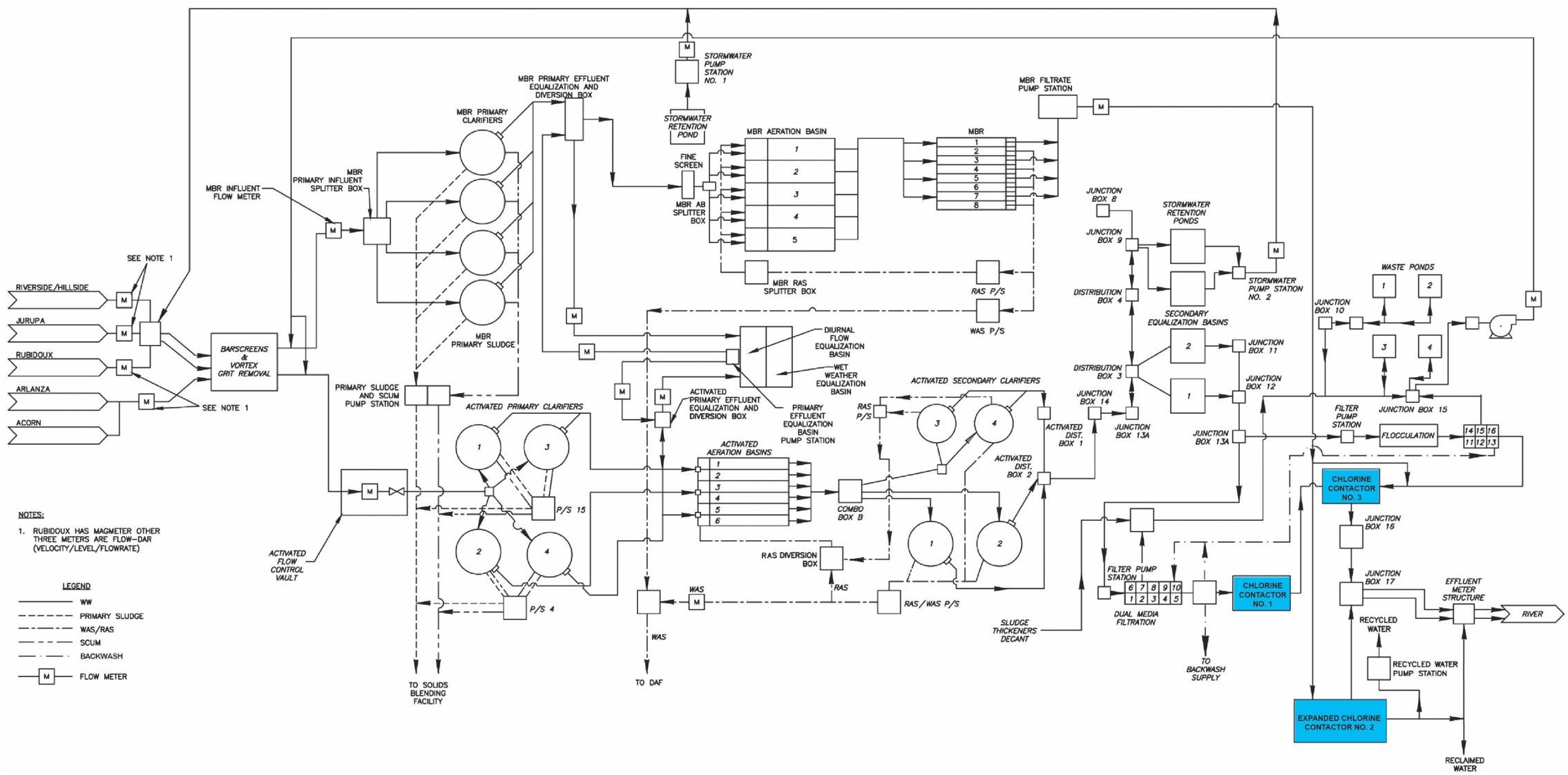


Figure 9.2 Process Flow Schematic

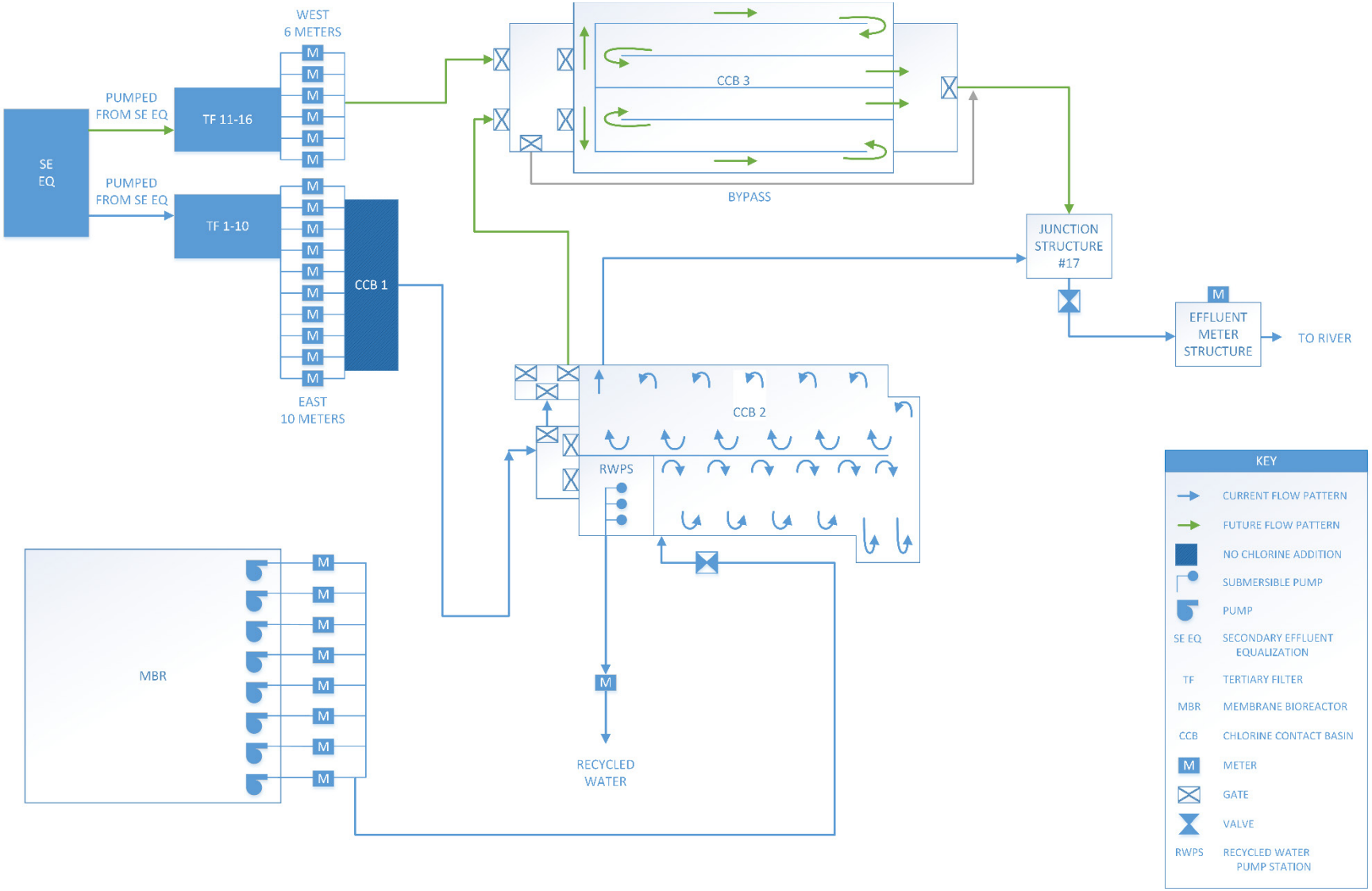


Figure 9.3 Normal Operational RWQCP Flow Path

Table 9.1 Existing Disinfection Facilities

| Description | Value | | |
|---|---|---|---|
| **Chlorination** | | | |
| Disinfection Chemical | Sodium Hypochlorite (NaOCl) | | |
| Current Chemical Dose | 5 to 8 mg/L | | |
| Chlorine Contact Basin | CCB 1 | CCB 2[(1)] | CCB 3 |
| Length to Width Ratio | 18.5:1 | 422:1 | 48:1 |
| Volume | 448,320 gal | 3,240,000 gal | 2,900,000 gal |
| Total Volume | 6.55 MG | | |
| Theoretical Contact Time | 204 min | | |
| Modal Contact Time (Required) | 29 min | | |
| Existing Capacity | 46 mgd (ADF) | | |
| **Dechlorination** | | | |
| Dechlorination Chemical | Sodium Bisulfite ($NaSHO_3$) | | |

Notes:
(1) As part of the MBR Phase I Plant Expansion, CCB 2 was expanded by about 50-ft to allow CCB 2 to achieve a 29-min modal contact time at 46 mgd.

## 9.6 CT Study

In order to fine tune its disinfection strategy by moving to a free chlorine disinfection residual and combining CCB operation, the City undertook a CT Study in 2018. Two operational strategies were considered as part of the CT Study:

- Use CCB 2 to treat only MBR Plant effluent.
- Use CCB 2 to treat all plant effluent (MBR and ACT Plant effluent).

These strategies would allow CCB 3 to be taken off-line. Flow would still need to pass through CCB 1 to get to CCB 2, but no chemicals would normally be needed at CCB 1.

A contact time tracer study test was performed on CCB 2. The plant stored flow and was able to feed 46-mgd to CCB 2 as part of the test. The test was run for 248 mins (two theoretical detention times). Tracer recovery of 98.2 percent was achieved, indicating a very high degree of plug flow in CCB 2.

Chlorine demand tests were carried out at chlorine doses of 3.0, 5.0, 8.0 and 9.0 mg/L for MBR effluent, ACT tertiary filter effluent, and a 70:30 blend of MBR:ACT tertiary filter effluent. The results for all three effluents were similar, and the chlorine demand was generally in the range of 2 to 3 mg/L. Laboratory bacterial and virus studies showed good dose/response results for free chlorine residuals.

Based on the positive laboratory test results, the City requested approval from the Regional Board to conduct a full-scale "low CT" test using CCB 2. Approval was given and the test was carried out in August 2018. The plant again stored flow prior to the test and then brought in flow to CCB 2 at close to 46-mgd. Free chlorine residuals were targeted at 1.5, 3.0 and 5.0 mg/L at the closest sampling point to the chlorine dosing location. The mixture of effluent during the test was 32-mgd of MBR Plant effluent and 14-mgd of ACT tertiary filter effluent. Flow was monitored throughout the test and varied between 44 and 49-mgd, with an average of 47-mgd.

The results of the full-scale test were good and confirmed that the free chlorine residual provides disinfection to meet the total coliform permit requirements. Based on the results of the CT Study, it was recommended that CCB 2 be operated at a free chlorine CT of 30 mg-min/L with a modal contact time of 29 mins, corresponding to a flow rate of 46-mgd. Lower flows would result in a higher CT value.

## 9.7 Title 22 Report Summary

In order to obtain approval from the Regional Board for the proposed modification to the disinfection procedure at the RWQCP, an updated Title 22 Report was prepared.

The updated draft Title 22 Report includes the full CT Study as an Appendix to that report, and is included in Appendix 9A. The draft Title 22 report was submitted and received conditional acceptance from the SWRCB, DDW, allowing the switch to free chlorine and the use of CCB 2 to treat the total plant flow. Conditional acceptance was received from the State in a letter dated May 6, 2019, which is included in Appendix 9B. The summary of the RWQCP capacity as presented in the updated draft Title 22 Report is shown in Table 9.2.

Table 9.2 Updated Title 22 Report Summary of Plant Capacities (2018)

| Process | MBR Plant | | ACT Plant | | Total Capacity(1) (mgd) |
|---|---|---|---|---|---|
| | Operating Condition | Flow(1) (mgd) | Operating Condition | Flow(1) (mgd) | |
| **Primary Treatment Capacity** | | | | | |
| | AUIS | 36 | AUIS | 23 | 59 |
| **Secondary Treatment Capacity** | | | | | |
| | MBR treatment train (Plant 1) OOS | 26 | SC AUIS | 20 | 46 |
| | MBR treatment train (Plant 1) OOS | 26 | SC OOS | 17.5 | 43.5 |
| **Tertiary Treatment Capacity** | | | | | |
| Disinfection | CCB 2 | | CCB 2 | | 46 |
| Filtration (Installed) | | | | 45.1(2) | |
| Filtration (per Title 22) | | | | 67.7(3) | |
| Filtration (Reliable) | | | | 42.2(4) | |

Notes:
(1) Per BioWin modeling. OOS – Out of Service, AUIS – All units in service.
(2) Total installed capacity at a plant filter loading of 3.33 gpm per square foot.
(3) Total installed capacity at a Title 22 loading rate of 5 gpm per square foot.
(4) Filter capacity with one cell out of service at a loading rate of 3.33 gpm per square foot.

The draft updated Title 22 Report (Appendix 9A) can be referenced for the results showing modal contact time and CT requirements for CCB 2.

## 9.8 Implementation of Disinfection Changes

Following conditional acceptance of the switch to free chlorine, the City has started adjustments to the plant to convert CCB 2 to a free chlorine disinfection system and meet the additional requirements of DDW, including finalizing the Title 22 Report. CCB 2 currently has two chlorine residual analyzers that measure total chlorine. The first analyzer is located between sampling point 1 (SP 1) and sampling point 2 (SP 2) and the second analyzer is located at

sampling point 6 (SP 6). SP 6 is the current compliance sampling point. An unused chlorine analyzer from CCB 3 will be moved to the location between SP 1 and SP 2 for free chlorine measurement. This will allow the plant to monitor both free and total combined chlorine. The free chlorine analyzer between SP 1 and SP 2 would be used to monitor compliance free chlorine residual.

Since no new analyzers are required in order to accomplish the proposed modifications to disinfection with free chlorine, there are no CIP needs for the disinfection process that need to be included in the update of the Master Plan.

## 9.9 Discharge Goals and Reuse

RPU has plans underway to expand the existing recycled water system. An increase in recycled water use will enable more efficient use of potable water supplies in the region. Currently, the RWQCP has a 25,000 AFY environmental commitment to the Santa Ana River. After subtracting this commitment from the RWQCP capacity, Public Works has the potential to transfer approximately 4,150 AFY of reclaimed water based on 2016 effluent flows, which equates to 3.7 mgd. As influent flow into the plant increases, the completed MBR Phase I Expansion Project can produce up to 21,000 AFY for future recycle water projects.

Public Works transfers the recycled water to RPU which is responsible for the distribution to its customers. The major recycled water uses in RPUs distribution system are irrigation and non-irrigation uses, construction, habitat development, maintenance, and recreational uses. Groundwater recharge with recycled water may be considered by RPU in the future. RPU will be taking responsibility of distributing all recycled water.

Appendix 9A

# FINAL DRAFT - TITLE 22 ENGINEERING REPORT

City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

## CHAPTER 9: DISINFECTION

# APPENDIX 9A: FINAL DRAFT - TITLE 22 ENGINEERING REPORT

City of Riverside

# TITLE 22 ENGINEERING REPORT

FINAL DRAFT | November 2018

carollo

City of Riverside

# TITLE 22 ENGINEERING REPORT

FINAL DRAFT | November 2018

Carollo Engineers, Inc.
Project No:

10903A.00

This document is released for the purpose of information exchange review and planning only under the authority of Nicola Fontaine, November 8, 2018, California 76863.

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# Abbreviations

| | |
|---|---|
| µg/L | micrograms per liter |
| ACT | Activated Treatment Train |
| ADWF | average dry weather flow |
| AFY | acre-feet per year |
| BOD | Biochemical Oxygen Demand |
| CCB | chlorine contact basin |
| cf | cubic feet |
| cfs | cubic feet per second |
| City | City of Riverside |
| County | County of Riverside |
| CT | concentration-contact time |
| DBPs | disinfection byproducts |
| DDW | Division of Drinking Water |
| DMR | Discharge Monitoring Report |
| ft | feet |
| gpd | gallons per day |
| gpm | gallons per minute |
| in | inch |
| MBR | Membrane Bioreactor |
| MG | million gallons |
| mgd | million gallons per day |
| mg/L | milligrams per liter |
| mg-min/L | milligram-minutes per liter |
| mL | milliliter |
| MPN | most probable number |
| NTU | nephelometric turbidity units |
| PFD | process flow diagram |
| PS | pump station |
| psi | pounds per square inch |
| psig | pounds per square inch |
| RPU | Riverside Public Utilities |
| RWQCB | Regional Water Quality Control Board |
| RWQCP | Riverside Regional Water Quality Control Plant |
| SCADA | Supervisory Control and Data Acquisition |
| SLR | solids loading rate |
| SMF | solids mass flux |
| SOR | surface overflow rate |

| | |
|---|---|
| sq/ft | square foot |
| SVI | sludge volume index |
| TDS | total dissolved solids |
| TSS | total suspended solids |
| WDO | waste discharge order |

# Chapter 1
# INTRODUCTION

## 1.1 Background

The City of Riverside (City) Department of Public Works (Public Works) is responsible for the collection and treatment of wastewater generated within the City and the community of Highgrove. Public Works is also responsible for the treatment only of the wastewater from the Community Service Districts of Edgemont, Jurupa, and Rubidoux. Public Works operates water and wastewater systems to provide potable water, wastewater services (collection, treatment, and disposal), and recycled water to Riverside Public Utilities (RPU) and one business within the City. Water operations and recycled water distribution are controlled by RPU and wastewater operations are controlled by the Sewer Division of the City's Department of Public Works. Currently, the Sewer Division sells a small amount of recycled water to one customer and the rest is transferred to RPU for distribution.

Wastewater treatment is performed at the Riverside Regional Water Quality Control Plant (RWQCP), which provides primary, secondary, and tertiary treatment. The RWQCP began serving the City in 1947. In 1978, the RWQCP transitioned into a regional facility by expanding its service district to include the Rubidoux, Jurupa, and Edgemont Community Service Districts and later, in 2015, Highgrove. As part of the 1978 expansion, primary, secondary, and tertiary treatment capacity was constructed to accommodate the increased plant flows. Subsequent projects have added capacity and upgraded existing primary, secondary, tertiary, solids handling, and storage facilities to increase the treatment capacity and meet water quality standards and goals at the RWQCP. The latest expansion completed in 2017 increased the treatment capacity from 40 to 46 million gallons per day (mgd) average dry weather flow (ADWF).

Given the growing scarcity of water resources and increasing costs of imported water, communities and agencies have looked to maximize the use of recycled water. The City, through a partnership between RWQCP and RPU, is committed to a recycled water program to enhance the City's water supply portfolio by providing a local, reliable, and sustainable source of water.

## 1.2 Purpose

The State of California Water Recycling Criteria or Title 22 requires the submission of an engineering report to the California Regional Water Quality Control Board (RWQCB) and the California Division of Drinking Water (DDW) before recycled water projects are implemented. The City has not previously submitted an engineering report for the RWQCP. The RWQCP currently uses an interim approval granted by the RWQCB (October 2010) to provide recycled water.

The purpose of this engineering report is to describe and discuss the plant's treatment processes as they relate to meeting the discharge limitations established by the RWQCB and show conformance with the DDW design and reliability requirements of Water Recycling Criteria or

Title 22, Division 4 of the California Administrative Code. This report presents contingency plans for containment or treatment and disposal of effluent during process upsets or outages; and shows that untreated or inadequately treated wastewater will not be delivered, used, or transferred to recycled water users. This report covers only the end users that receive tertiary disinfected effluent directly from Public Works which is currently only Toro Turf. The roles of all entities involved in the transfer of recycled water to Toro Turf and RPU are also described in this report.

## 1.3 Facility Description

The RWQCP operates as two individual plants with a small volume of the final combined effluent distributed to RPU and Toro Turf. The majority of the remaining effluent is discharged to the Santa Ana River. Wastewater comes into the RWQCP at the headworks through four trunklines; Acorn/Arlanza, Jurupa, Riverside, and Rubidoux and proceeds through screening and grit removal. From the headworks processes, flow splits into the two separate trains via a flow control vault upstream of the Activated Treatment Train (ACT), formerly referred to as Plant 2,and a meter vault upstream of the Membrane Bioreactor (MBR) train, formerly referred to as Plant 1..

The MBR Plant includes primary clarifiers, primary effluent equalization, aeration basins, membrane tanks, and chlorine disinfection. MBR permeate is introduced to Chlorine Contact Basin (CCB) 2 where it is chlorinated to meet the RWQCB interim approval concentration-contact time (CT) requirements of 900 milligram-minutes per liter (mg-min/L) and dechlorinated prior to combining with the ACT Plant CCB3 effluent. The MBR installation was completed in the summer of 2017 with an ADWF design rated treatment capacity of 26 mgd. The membrane tanks were designed for 32 mgd, but the system was equipped with enough membrane area to operate at an average capacity of 26 mgd. In addition to the installation of the MBR system, CCB2 volume was also increased to 3.24 million gallons (MG). A CT analysis of CCB2 has been conducted as part of this regulatory effort and is attached as Appendix A. The goal of the analysis was to drop the interim approval contact time requirements and CT requirements below the currently mandated value of 90-minute modal and 900 mg-min/L values, respectively.

The ACT Plant includes primary clarifiers, primary effluent equalization, secondary treatment, and tertiary treatment using traditional sand/gravel filter media. Filter effluent is introduced to CCB3 where it is chlorinated and then dechlorinated for discharge to the Santa Ana River. The CCB3 volume is 2.9 MG. The ACT Plant has an ADWF design rated treatment capacity of 20 mgd.

There is a diurnal flow equalization structure downstream of primary treatment for both plants that is intended to receive primary effluent flow from either or both Plants. The equalization basin normalizes flow through the two Plants by scalping excess flows independently from each train by use of flow meters and control valves. The diurnal flow equalization basin pump station (PS) can pump primary effluent stored in the equalization basin to either of the two train diversion structures for further treatment in either the MBR Plant or the ACT Plant. The RWQCP is designed to produce disinfected tertiary effluent per Title 22 requirements.

The RWQCP is looking to fine tune its operational strategy and streamline efficiencies which may consist of taking CCB3 offline and commingling the MBR and ACT Plant flows prior to CCB2. The two operational plans for disinfection in CCB2 at a reduced free chlorine CT are:

- CCB2 only treats MBR Plant effluent
- CCB2 treats all MBR Plant effluent and ACT Plant effluent

Recycled water pumping facilities have been designed for 10 recycled water pumps. Currently three recycled water pumps are installed. Each installed pump has a capacity of 2,400 gallons per minute (gpm). The installed pumps provide recycled water to RPU and the Public Works customer.

## 1.4 Discharge Goals and Reuse

RPU has plans underway to expand the existing recycled water system. An increase in recycled water use will enable more efficient use of potable water supplies in the region. Currently, the RWQCP has a 25,000 acre-feet per year (AFY) environmental commitment to the Santa Ana River. After subtracting this commitment from the RWQCP capacity, Public Works has the potential to transfer approximately 4,150 AFY of reclaimed water based on 2016 effluent flows, which equates to 3.7 mgd. As influent flow into the plant increases, the completed Phase I upgrade and expansion project can produce up to 21,000 AFY for future recycle water projects.

Public Works transfers the recycled water to RPU which is responsible for the distribution to its customers. The major recycled water uses in RPUs distribution system are irrigation and non-irrigation uses, construction, habitat development, maintenance, and recreational uses. Groundwater recharge with recycled water may be considered by RPU in the future. RPU will be taking responsibility of distributing all recycled water, even to Toro Turf.

Chapter 2

# DISCHARGE REQUIREMENTS AND WATER QUALITY

## 2.1 Wastewater Quality Criteria

The following sections describe the water quality criteria used at the RWQCP for conformity to the effluent discharge limitations issued by the California RWQCB as well as Title 22 requirements for disinfected tertiary recycled water.

### 2.1.1 Discharge Requirements

Operation of the RWQCP is regulated by local, state, and federal agencies. These agencies provide standards and requirements that establish plant reliability and the required quality of liquid, gas, and solids streams discharged from the plant.

Discharge of treated effluent from RWQCP is governed by Waste Discharge Order (WDO) No. R8-2013-0016, issued by the California RWQCB, Santa Ana Region. The numerical limits prescribed in the order are consistent with local Basin Plan objectives in place at the time, and are set to protect designated beneficial uses, public health, and the environment.

### 2.1.2 Waste Discharge Order No. R8-2013-0016

In November, 2013, Santa Ana RWQCB adopted WDO No. R8-2013-0016. This order prescribed numerical limitations for discharge of treated effluent from RWQCP, as well as referencing Title 22 as governing the treatment and reuse of recycled water.

Table 2.1 and Table 2.2 summarize the discharge limitations contained in WDO No. R8-2013-0016. The entire permit is located in Appendix B. RWQCP currently has four discharge points as follows:

1. Santa Ana River at discharge point 001.
2. Santa Ana River at discharge point 002.
3. Irrigation of Toro Turf.
4. Transfer to RPU for distribution.

Chapter 4 presents specific information on the Toro Turf use site.

Table 2.1 RWQCB Waste Discharge Order No. R8-2013-0016 Requirements

| Constituent | Unit | Daily Maximum | Weekly Average | Monthly Averge | Instantaneous Minimum | Instantaneous Maximum |
|---|---|---|---|---|---|---|
| Biochemical Oxygen Demand ($BOD_5$ at $20^0$ C) | mg/L(2) | - | 30/45 | 20/30 | | |
| BOD Percent Removal | % | | 85 | | | |
| Total Suspended Solids (TSS) | mg/L(2) | - | 30/45 | 20/30 | | |
| Total Dissolved Solids (TDS) | mg/L(3) | - | - | 650 | | |
| Total Inorganic Nitrogen | mg/L(3) | - | - | 10 | | |
| Ammonia Nitrogen | mg/L | - | - | 5 | | |
| Nitrite-N (as N) | mg/L | - | - | 0.9 | | |
| Cyanide | ug/L | 8.5 | - | 4.2 | | |
| pH | std units(4) | | | | 6.5 | 8.5 |

Notes:
mg/L = milligrams per liter.
ug/L = micrograms per liter.
(1) Refer to Appendix B for complete permit requirements.
(2) Without 20:1 dilution/with 20:1 or more dilution in the receiving water.
(3) 12 months weighted running average.
(4) Time outside of range not to exceed 7 hours and 26 minutes. Individual excursion from range not to exceed 60 minutes.

Table 2.2 Title 22 Recycled Water Quality Requirements for Filtered, Disinfected Tertiary Effluent

| Constituent | Unit | Maximum |
|---|---|---|
| **Turbidity** | | |
| Not exceeded average within any 24-hour period | NTU | 2 |
| Not exceeded more than 5 percent of the time in a 24-hour period | NTU | 5 |
| Not exceeded more than 1 minute at any time | NTU | 10 |
| **Coliform Bacteria** | | |
| Median of the previous 7 days shall not exceed this amount | MPN/100 mL | 2.2 |
| Not exceeded in more than one sample in 30 days | MPN/100 mL | 23 |
| Not exceeded in any sample | MPN/100 mL | 240 |
| **Chlorine Disinfection** | | |
| CT (interim approval) | mg-min/L | 900(1) |
| Modal Contact Time (interim approval) | minutes | 90 |
| MS2 Bacteriophage or Poliovirus | percentage | 0.001(2) |

Notes:
mL = milliliter.
MPN = most probable number.
NTU = nephelometric turbidity units.
(1) Interim permit to operate CCB2. Appendix A contains CCB2 CT testing results that reduces CT and contact time requirements.
(2) Equivalent to 99.999 percent removal.

## 2.2 Monitoring and Reporting

To verify compliance with the recycled water criteria, RWQCP monitors its effluent for the following parameters:

- Total coliform bacteria are analyzed daily.
- Turbidity is continuously monitored and recorded.
  - Compliance with the daily average operating filter effluent turbidity of 2 NTUs is determined from the continuous turbidity analyzer.
  - MBR turbidity limits are 0.2 NTU more than 5 percent of the time within a 24-hour period and 0.5 NTU at any time.
- Chlorine residual is continuously monitored and recorded. Compliance with CT requirements is determined daily.

In addition to turbidity of filtered effluent, turbidity of the filter influent will be monitored to aid in process control and in setting chemical feed rates.

### 2.2.1 Reporting by RWQCP to RWQCB and DDW

Per the requirements of section 60329 of the Title 22, operating records and reports should be maintained at a reclamation plant for all analyses specified in the reclamation criteria; records of operational problems, plant and equipment breakdowns, and diversions to emergency storage or disposal, corrective or preventative action taken, process or equipment failures triggering any alarms, time and cause of failure, and corrective actions taken.

A monthly treatment plant operations summary report to demonstrate compliance with filtration and disinfection requirements must be submitted to RWQCB and DDW.

In order to comply with the above monitoring requirements, occurrence of the following failures would initiate a plant shutdown or diversion of inadequately treated wastewater. Notification to the RWQCB, DDW, and local health officer would be made within 24 hours:

- Failure of chlorination process to meet the minimum CT as outlined in the CT study attached as Appendix A.
- Effluent total coliform bacteria MPN greater than 23 per 100 mL in more than one sample in a 30-day period or greater than 240/100 mL in any sample.
- Turbidity greater than 10 NTU.
- Any known direct cross-connection between recycled and potable water.

The RWQCP reports the following in the monthly Discharge Monitoring Report (DMR) as required by the current discharge permit:

- Total coliform bacteria - daily, running a 7-day median and daily maximum reading for the previous month.
- Chlorine residual - minimum and maximum daily.
- Effluent turbidity - daily maximum, results of 4-hour readings, average, and noting any excursions to Title 22 compliance requirements.
- CT - daily compliance determinations.

## 2.3 Influent Wastewater Characteristics

Influent to the RWQCP is primarily from residential and commercial sources. Table 2.3 presents values of influent wastewater characteristics for the RWQCP for the period of January 2017 through December 2017. Summary plots showing the statistical significance of the values used and the spot and moving average data of influent characteristics data are included as Appendix C.

Table 2.3 Influent Wastewater Characteristics for RWQCP 2017

| Parameter | Annual Median | 95th Confidence Interval | Annual Average |
|---|---|---|---|
| Influent Flow (mgd) | 27.5 | 27.42 – 27.75 | 27.59 |
| BOD (mg/L) | 352 | 354.97 – 365.27 | 360.12 |
| TSS (mg/L) | 258 | 262.95 – 299.16 | 281.06 |
| Ammonia (mg/L) | 46.35 | 45.82 – 48.59 | 47.21 |

Notes:
(1) Wastewater characteristics for RWQCP for the January 2017 through December 2017 period.

## 2.4 Effluent Characteristics

Table 2.4 presents values for the Plant's tertiary effluent as monthly average values for December 2016 through December 2017. The current WDO stipulates the effluent limitations for both the two discharge points at the Santa Ana River and the discharge points to RPU and Toro Turf. The effluent limitations are as shown in Table 2.1 and include monthly and weekly averages and percent removal of BOD and TSS. All measured values in Table 2.4 are compliant with the RWQCB permitted values.

Table 2.4 Summary of Monthly Average Tertiary Effluent Characteristics

| Parameter | 2016 | 2017 | | | | | | | | | | | Monthly Average |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Dec | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | |
| Flow (mgd) | 26.47 | 29.31 | 27.63 | 26.89 | 26.56 | 26.62 | 26.63 | 26.15 | 26.08 | 26.40 | 26.28 | 26.36 | 26.78 |
| BOD (mg/L) | 1.75 | 2.74 | 2.00 | 2.13 | 2.07 | 3.42 | 5.00 | 9.19 | 3.61 | 2.90 | 3.26 | 2.46 | 3.38 |
| TSS (mg/L) | 0.87 | 1.36 | 1.27 | 0.64 | 0.81 | 1.15 | 1.12 | 2.18 | 2.42 | 3.15 | 1.93 | 1.44 | 1.53 |
| Turbidity (NTU) | 0.73 | 0.84 | 0.96 | 1.01 | 0.69 | 0.46 | 0.70 | 1.01 | 0.92 | 0.86 | 1.22 | 0.89 | 0.86 |
| Ammonia-Nitrogen (mg/L) | 0.15 | 0.5 | 0.21 | 0.11 | 0.10 | 0.11 | 0.32 | 0.26 | 0.47 | 0.45 | 0.7 | 0.25 | 0.3 |
| Total Coliform (MPN/100 mL) | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 |
| pH (max units) | 7.14 | 7.09 | 7.08 | 7.09 | 7.05 | 7.04 | 6.98 | 6.95 | 7.02 | 7.01 | 7.12 | 6.82 | 7.03 |
| TDS (mg/L) | 624 | 624 | 625 | 627 | 627 | 627 | 625 | 623 | 621 | 621 | 621 | 623 | 624 |
| Total Residual Chlorine (mg/L) | 0.00 | 0.01 | 0.00 | 0.02 | 0.00 | 0.01 | 0.02 | 0.02 | 0.01 | 0.03 | 0.01 | 0.00 | 0.01 |
| Total Inorganic Nitrogen (mg/L) | 4.88 | 5.38 | 4.52 | 4.32 | 4.73 | 4.62 | 3.87 | 3.76 | 2.94 | 4.57 | 5.33 | 5.00 | 4.49 |
| Nitrite-N (as N) (mg/L) | 0.00 | 0.05 | 0.01 | 0.00 | 0.00 | 0.00 | 0.03 | 0.01 | 0.03 | 0.00 | 0.03 | 0.00 | 0.01 |
| Cyanide (ug/L) | 3.8 | 4.1 | 1.9 | 1.1 | 1.2 | 1.3 | 1.9 | 0 | 0 | 1.1 | 1.0 | 1.1 | 1.5 |

Chapter 3

# TREATMENT PROCESS

## 3.1 General

This chapter presents an overview of the unit processes currently installed RWQCP. Following the summary is a discussion of how each of the applicable liquid stream processes meets the reliability criteria required by Title 22. There are no specific requirements for the solids handling processes in Title 22, and they are not discussed in this report. The section also includes an evaluation of the plant's reliable capacity and a contingency plan to prevent the discharge or reuse of inadequately treated effluent.

## 3.2 Treatment Process Summary

RWQCP treats its liquid stream with existing MBR Plant and the ACT Plant facilities. The two plants share the following preliminary treatment facilities:

- Influent screening
- Grit removal
- Primary effluent equalization
- Primary effluent equalization pumping

The existing unit processes in the MBR Plant are as follows:

- Primary clarification
- Aeration (biological treatment)
- Membrane Tanks
- Chlorine Disinfection (CCB2)

The existing unit processes in the ACT Plant are as follows:

- Primary clarification
- Aeration (biological treatment)
- Secondary clarification
- Filter influent pumping
- Dual media filtration
- Chlorine Disinfection (CCB1/CCB3)

The process flow diagram (PFD) for the facility is provided on Figures 3.1 and a site plan for the facility is shown on Figure 3.2. The future operation of the plant disinfection includes two operating strategies as follows, which assumes CCB3 is taken offline:

- CCB2 only treats MBR Plant effluent
- CCB2 treats all MBR Plant effluent and the ACT Plant

Table 3.1 and Table 3.2 summarize design criteria for primary/secondary and tertiary treatment, respectively. The criteria for the new facilities are used in the BioWin numerical process models attached as Appendix D.

## 3.3 Compliance with Title 22 - Reliability

Title 22 contains two basic categories of requirements for treatment and reclamation of wastewater. The first category prescribes the required degrees of wastewater treatment and the permissible qualities for various uses of recycled water. The second category establishes the degree of reliability required and the acceptable methods for providing it in the design and operation of reclamation facilities.

The RWQCP currently treats all of its influent to meet Title 22 requirements for "Disinfected Tertiary Recycled Water" prior to being transferred to RPU for distribution to RPU users, directly by RWQCP to Toro Turf, and to the Santa Ana River.

### 3.3.1 Requirements for Reliability

Title 22 generally recognizes several means to provide the reliability required for reclamation plants. Depending on the process involved, these include the following:

- Multiple process units or equipment capable of providing adequate treatment with one unit not in operation.
- Short-term retention or disposal provisions coupled with standby replacement equipment.
- Long-term storage or alternative disposal provisions.
- Other methods, demonstrated to the satisfaction of the DDW, which ensure reliability equal to the above.

All of the above methods are used at the RWQCP. The primary method of providing reliability is the availability of multiple process units. Short-term storage is also available for storage of non-compliant water.

Reliability through secondary treatment is provided by mechanical redundancy to provide continuous treatment in case of failure of individual equipment. The MBR Plant and the ACT Plant provide additional reliability through their parallel treatment trains. The secondary effluent equalization pond upstream of the ACT Plant provides temporary storage in the event that secondary treatment facilities are out of service. In the event the MBR Plant fails, return activated sludge can be returned to the primary effluent equalization basin for storage.

Reliability through the tertiary treatment process is provided by the use of storage for out-of-compliance effluent in the storm water retention ponds that can be sent back to the head of the plant for retreatment once process upsets are corrected. In addition, multiple filtration units can treat the design flow with one filter not in service.

Under Section 60341, regarding storage, retention, or disposal provisions, short term is defined as at least a 24-hour period. Long-term storage or disposal is considered to be a minimum of 20 days.

Reliability for individual processes is discussed below.

| KEY | |
|---|---|
| 1. | Influent Flow Monitoring |
| 2. | Final Effluent Metering Flow Control Outflow |
| 3. | Headworks Facility |
| 4. | Headworks Odor Control Facility |
| 5. | Primary Clarifier |
| 6. | Aeration Basin |
| 7. | Secondary Clarifier |
| 8. | Tertiary Filter |
| 9. | Chlorine Contact Basin |
| 10. | Primary Clarifiers Odor Control Facility |
| 11. | Primary Sludge and Scum Pump Station |
| 12. | RAS/WAS Pump Station |
| 13. | RAS Pump Station |
| 14. | Diurnal Flow Equalization Basin |
| 15. | Wet Weather Equalization Basin |
| 16. | MBR Fine Screen Facility |
| 17. | MBR Filtration Units |
| 18. | MBR Train Influent Flow Meter |
| 19. | MBR Air Scour/Blower Building |
| 20. | Blower Building |
| 21. | Chemical Storage and Feed Area |
| 22. | Dechlorination Building |
| 23. | Digester |
| 24. | Digester Control Building |
| 25. | Digested Sludge Pump Station |
| 26. | Digested Sludge Storage Tank |
| 27. | Digester Gas Storage Facility |
| 28. | Digester Flares |
| 29. | Dissolved Air Flotation Thickeners |
| 30. | Solids Metering Facility |
| 31. | Bio-Solids Blending Facility |
| 32. | Bio-Solids Dewatering Facility |
| 33. | Bio-Solids Silos/Loading Facility |
| 34. | TWAS Disintegration Facility |
| 35. | FOG Receiving Station |
| 36. | Secondary Effluent Equalization Pond |
| 37. | Stormwater Retention Pond |
| 38. | Waste Backwash Lagoon |
| 39. | Backwash Storage Tank |
| 40. | RWQCP Plant Effluent Outfall |
| 41. | Flow Monitoring or Flow Control Structure |
| 42. | Splitter Box or Diversion Structure |
| 43. | Pump Station/Switchgear Room |
| 44. | Sampling Station |
| 45. | Electrical or MCC Structure |
| 46. | Power Generation or Standby Generators |
| 47. | Fuel Cell |
| 48. | Cogeneration |
| 49. | Administration Building |
| 50. | Cooling Tower |
| 51. | Maintenance, Storage, or Misc. |
| 52. | Material Storage Bed |
| 53. | Native American Rock Reserve Area |
| 54. | Recycled Water Pump Station |

Figure 3.1 Process Flow Diagram

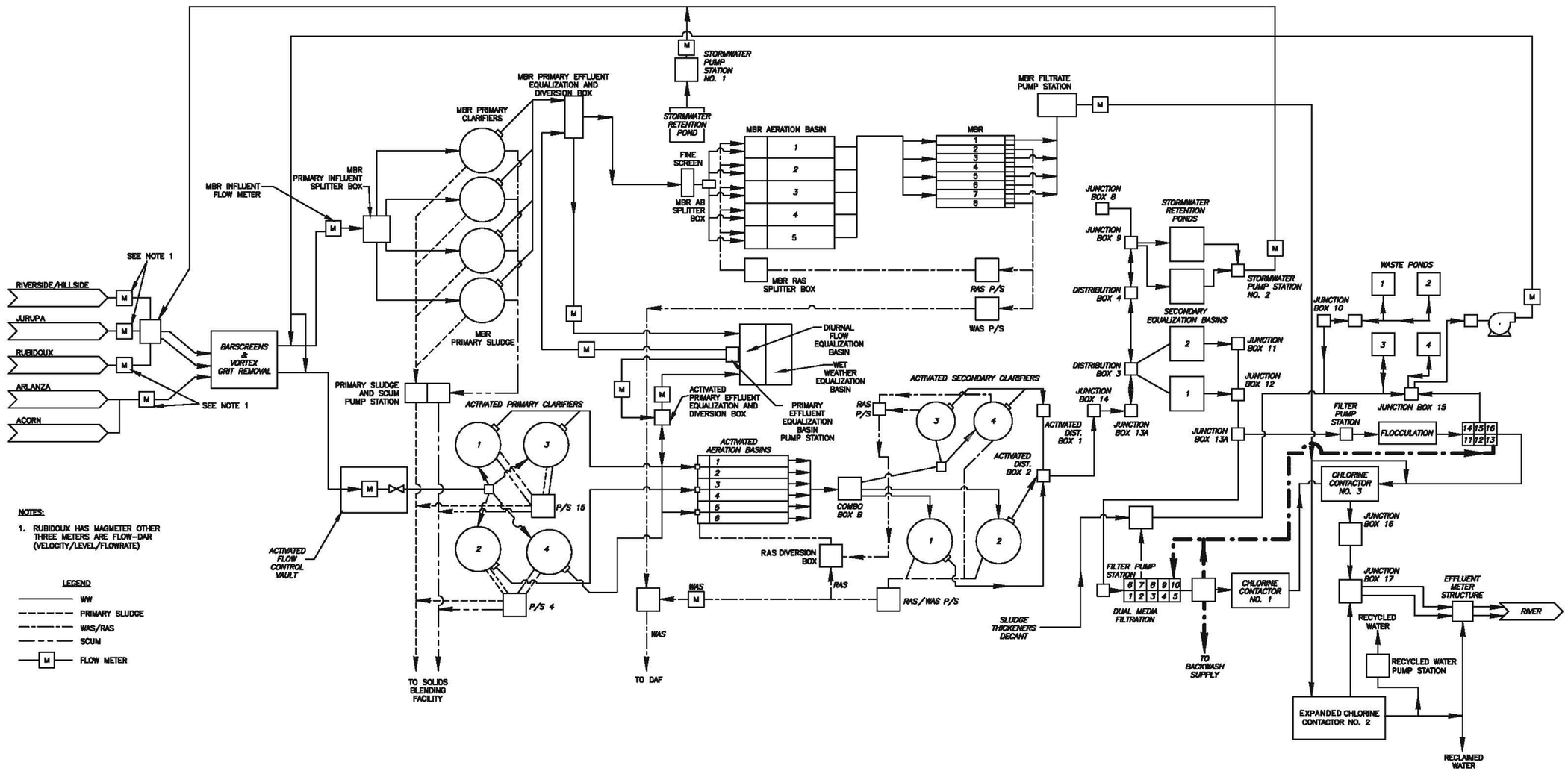


Figure 3.2 Site Plan

Table 3.1 Primary and Secondary Treatment Design Criteria at RWQCP

| Process and Equipment | Units | Value | | |
|---|---|---|---|---|
| | | MBR | ACT | Joint or Combined |
| **Influent Flow** | | | | |
| ADWF/Maximum Month | mgd | - | - | 46.0/52.9[1] |
| **Bar Screens** | | | | |
| Number of Channels | - | - | - | 4 |
| Mechanical Bar Screens | | | | |
| Number of Bar Screens | Duty/Standby | - | - | 3+1 |
| Bar Screen Channel Width | ft | - | - | 3.5 |
| Opening Width | ins | - | - | 0.5 |
| Type | - | - | - | Climber |
| **Grit Removal** | | | | |
| Type | - | - | - | Vortex |
| Number | - | - | - | 2 |
| Diameter | ft | - | - | 20 |
| Capacity, each | mgd | - | - | 50 |
| Grit Removal Efficiency >50 mesh | percent | - | - | 95 |
| Grit Removal Efficiency >100 mesh | percent | - | - | 65 |
| **Grit Pumps** | | | | |
| Type | - | - | - | Recessed Impeller |
| Number | - | - | - | 2 |
| Pumping Capacity | gpm | - | - | 250 |

Table 3.1 Primary and Secondary Treatment Design Criteria at RWQCP (Continued)

| Process and Equipment | Units | Value | | |
|---|---|---|---|---|
| | | MBR | ACT | Joint or Combined |
| **Grit Classifier** | | | | |
| Type | - | - | - | Hydraulic Vortex |
| Number of Classifiers | - | - | - | 2 |
| Capacity | gpm | - | - | 250 |
| Diameter | in | - | - | 42 |
| **Grit Dewatering Equipment** | | | | |
| Number | | - | - | 2 |
| Belt width | in | - | - | 18 |
| **Primary Clarifiers** | | | | |
| Type | | Circular | Circular | - |
| Number | | 4 | 4 | 8 |
| Diameter | ft | 120 | 95 | - |
| Total Volume | MG | 4.1 | 1.9 | 6.0 |
| Side Water Depth | ft | 18 | 9 | - |
| Surface Overflow Rate (SOR) | gpd/sf | 707 | 705 | - |
| Detention Time | hours | 3 | 2.54 | - |
| **Secondary Effluent Flow Equalization Ponds** | | | | |
| Number of Ponds | - | 2 | | 1 |
| Volume, each | MG | 1.5 | | |
| **MBR Fine Screens** | | | | |
| Number (Duty + Standby) | | 3 +1 | - | - |
| Type | | In-channel Rotary Drum Screen | - | - |
| Channel Width | ft | 8.5 | - | - |

Table 3.1 Primary and Secondary Treatment Design Criteria at RWQCP (Continued)

| Process and Equipment | Units | Value | | |
|---|---|---|---|---|
| | | MBR | ACT | Joint or Combined |
| Screen Perforation | in (mm) | 0.08(2) | - | - |
| Nominal Screening Basket Diameter | in | 102 | - | - |
| Nominal Screenings auger diameter | in | 20 | - | - |
| Flow per screen Ave/Max | mgd | 10.7/16 | - | - |
| Washwater Flowrate | gpm | 65 | - | - |
| Washwater Supply Pressure | psi | 60 | - | - |
| **MBR Aeration Basins** | | | | |
| Number | | 5 | | |
| Volume in Service, MG | | 5.04 | | |
| Anoxic Faction, percent | | 25 | | |
| Air flow, scfm | | 5,000 | | |
| Type | | Fine Bubble | | |
| Number | | 395 | | |
| **Activated Sludge Aeration Basins** | | | | |
| Number of Basins | - | - | 6 | - |
| Anoxic Fraction | ft | - | 25 | - |
| Available Volume | MG | - | 7.85 | |
| **Diffused Aeration** | | | | |
| Air flow | scfm | - | 5,000 | |
| Diffuser Type | | - | Fine Bubble | |
| Number, each | | - | 694 | |

Table 3.1 Primary and Secondary Treatment Design Criteria at RWQCP (Continued)

| Process and Equipment | Units | Value | | |
|---|---|---|---|---|
| | | MBR | ACT | Joint or Combined |
| **Aeration Blowers** | | | | |
| Number of Blowers | | 3+1 common standby | 2+1 common standby | 6 |
| Type | | Single Stage Centrifugal, Vertical Split | Single Stage Centrifugal, Vertical Split | - |
| Capacity (each) | sfcm | 9,000 | 12,500 | - |
| Discharge Pressure | psig | 8.8 | 9.2 | - |
| **Membrane Tanks** | | | - | |
| Number of Tanks | - | 8 | - | |
| Membrane Area per Basin | $ft^2$ | 248,625 | - | |
| Membrane Cassettes per Tank | - | 14 | | |
| Capacity with 1 Tank Out of Service (2) | mgd | 26 | - | |
| **Secondary Clarifiers** | | | | |
| Type | | | Circular | |
| Number | | | 2 / 2 | 14 |
| Diameter | ft | | 130 / 100 | |
| Width | ft | | - / - | |
| Side Water Depth | ft | | 14 / 10.3 | |
| Inflow per Clarifier | mgd | | 3.1 / - | |

Table 3.1 Primary and Secondary Treatment Design Criteria at RWQCP (Continued)

| Process and Equipment | Units | Value | | |
|---|---|---|---|---|
| | | MBR | ACT | Joint or Combined |
| SOR | gpd/sf | | 494 | - |
| Hydraulic Detention Time | hours | | 5.0 | 3.7 |
| Weir Loading Rate | gpd/ft | | 4,300 | - |
| Solids Loading Rate (SLR) | lb/ft$^2$-d | | 15 | - |

Notes:
ft = feet
gpd = gallons per day.
psi = pounds per square inch.
psig = pounds per square inch gauge.
(1) Average Dry Weather Flow/Maximum Month.
(2) The membrane tanks were designed for 32 mgd, but the system was equipped with enough membrane area to operate at an average capacity of 26 mgd.

Table 3.2 Tertiary Treatment Design Criteria at RWQCP

| Process and Equipment | Units | Value | | | |
|---|---|---|---|---|---|
| | | MBR | ACT | | Joint or Combined |
| **Filter Influent Pump Station** | | | | | |
| Filters | | - | Filter 1-10 | Filter 11-16 | |
| Pump Type | - | - | Vertical Turbine | Vertical Turbine | - |
| Number of Pumps | No. | - | 2+1 | 2+1 | 4+2 |
| Pump Capacity (each) | gpm | - | 13,050 | 8,000 | - |
| Horsepower | hp | | 250 | 125 | |
| Total Reliable Capacity | mgd | - | 37.6 | 23 | 60.6 |
| **Flocculation Basins** | | | | | |
| Number of Basins | - | - | 10 | | - |
| Cells per Basin | - | - | 2 | | - |
| Volume per Basin | gal | - | 13,740 | | - |
| **Tertiary Filters** | | | | | |
| Number of Units | - | - | 16 | | 16 |
| Type of Filter | - | - | Media Filters | | - |
| Filters | | | Filter 1-10 | Filter 11-16 | - |
| Surface Area (per cell) | sf | - | 552 | 650 | - |
| Hydraulic Loading Rate(1) | gpm/sq.ft | - | 3.33 | 3.33 | - |
| Maximum Loading Rate | | - | 5 | 5 | - |
| Total Installed Capacity(2) | mgd | - | 26.4(5) | 18.7 | 45.1 |
| Title 22 Capacity(3) | mgd | - | 39.7 | 28 | 67.7 |
| Total Reliable Capacity(4) | mgd | - | 23.8 | 15.6 | 41.8 |
| **CCBs** | | CCB2 | CCB1 | | CCB3 |
| Number of Basins | No. | 1 | 1 | 1 | - |
| Length to Width Ratio | No. | 422:1 | 18.5:1 | 48:1 | - |

Table 3.2 Tertiary Treatment Design Criteria at RWQCP (Continued)

| Process and Equipment | Units | Value | | | |
|---|---|---|---|---|---|
| | | MBR | ACT | | Joint or Combined |
| Side Water Depth at ADWF | ft | 19.2 | 11 | 18 | - |
| Modal Contact Time | min | - | - | | Appendix A[6] |
| CT | mg-min/L | | | | 30 |
| Total Capacity at Modal Time | mgd | 46 | 26 | | - |
| **Sodium Hypochlorite Storage** | | | | | |
| Number of Storage Tanks | | 3 | | | - |
| Tank Capacity | gal | 20,000 | | | - |
| Days of Storage | days | 29 | | | - |
| Number of Pumps | | 1 + 1 (CCB 2) | 1 duty (CCB3) 1 duty (CCB 1) 1 shared standby | | 3+ 2 |
| Type | | Diaphragm | | | - |
| **Sodium Bisulfite** | | | | | |
| Number of Storage Tanks | | 2 | | | - |
| Tank Capacity | gal | 12,000 | | | - |
| Days of Storage | days | 33 | | | - |
| Number of Pumps | | 3 + 1 | | | |
| **Storm Water Retention Ponds** | | | | | |
| Number of Ponds | - | 2 | | | 1 |
| Volume, each | MG | 1.5 | | | |

Notes:
ft = feet.
sq/ft = square feet.
(1) Loading rate based on historical operation of the filters at the plant.
(2) Capacity based on loading rate of 3.33 gpm/sf.
(3) Capacity based on Title 22 loading rate of 5 gpm/sf.
(4) Reliable capacity (one filter out of service) based on plant filter loading rate.
(5) Total filter capacity limited to ACT Plant capacity of 26 mgd.
(6) Revised CT and compliance point in Appendix A. Flows to be combined prior to treatment in CCB2.

#### 3.3.1.1 Alarms

Under Section 60335, Title 22 requires that alarm devices be provided to alert operations personnel to various plant conditions. The required alarm functions include:

- Loss of power from the normal supply
- Failure of the biological treatment process
- Failure of the coagulation process
- Failure of the filtration process
- Failure of the disinfection process
- Any other failures that the regulatory agency may require

The plant has a supervisory control and data acquisition (SCADA) system that transmits numerous status indications, variables, and alarms to the plant's control building. Most alarms activate when equipment fails to respond to a start/stop signal or indicates electrical or mechanical failure. Alarms also activate on high or low process variables. The incoming data related to the above Title 22 requirements include:

- Loss of commercial power
- Engine generator failure
- Primary recirculation pumps low pressure
- Storm water pumps status and failure
- Primary clarifier status and failure
- Primary clarifier sludge pump failure
- MBR level measurement failure
- MBR PC flow transmitter fault
- MBR Primary Effluent flow valve failure
- Activated Sludge Primary Effluent flow valve failure
- Diurnal flow equalization basin level transmitter failure
- Wet weather equalization basin level failure
- Activated Sludge and MBR Flow equalization pumps failure
- MBR Equalization Basin Return flow valve failure
- MBR Primary Equalization Effluent flow valve failure
- MBR Equalization Basin Return flow failure
- Activated Sludge Equalization Basin Return flow control valve failure
- MBR rotary fine screens and lube pumps status and failure
- MBR fine screen inlet and outlet gates status and failure
- MBR anoxic zone mixers status and failure
- MBR aeration basins ORP transmitters failure
- MBR aeration basins do transmitters failure.
- MBR aeration basin no. 5 mixed liquor recycle pump status and failure
- Aeration blowers status and failure
- MBR DEOX zone mixers status and failure
- MBR RAS pumps status and failure
- MBR air scour blowers
- Air scour blowers discharge header pressure alarm
- Air scour blowers discharge header flowmeter

- CCB2 and JB-17 chlorine mixers status and failure
- CCB2 level indication fault alarm
- CCB2 influent chlorine residual transmitter fault alarm
- CCB2 effluent chlorine residual transmitter fault alarm
- MBR CIP sodium hypochlorite feed pumps status and failure
- Sodium hypochlorite flow transmitters
- MBR CIP citric acid storage tank level
- MBR CIP citric acid feed pumps status and failure
- Citric acid flow transmitters
- MBR CIP/backpulse citric acid feed valve
- Sodium hypochlorite storage tank level
- CCB sodium hypochlorite feed pumps status and alarm
- Sodium hypochlorite flow transmitters
- Disinfection pumps sump level
- Sodium bisulfite storage tank level
- Effluent sodium bisulfite feed pumps
- Secondary clarifier status and failure
- Coagulation feed failure
- Flocculation equipment status and failure
- Filter influent pumps status and failure
- High and low tertiary filter water level
- High filter influent turbidity
- Tertiary filter status and failure
- Individual filter effluent turbidity level
- Low CT product for disinfection
- Effluent pumps status and failure

As required by Title 22, the above alarms are independent of the normal power supply at the plant. In addition to the alarms specific to comply with Section 60335, the facility monitors and records a large number of other alarms, process conditions, and equipment status.

#### 3.3.1.2 Power Supply Reliability

Section 60337 of the Title 22 criteria requires that the RWQCP be provided with one of the following power reliability features:

- Alarm and standby power source
- Alarm and automatically actuated short-term retention or disposal provisions as specified in Section 60341
- Automatically actuated long-term storage or disposal provisions as specified in Section 60341

RWQCP has alarm and standby power source. In the event of power failure, an alarm will be activated and the standby generator will automatically start to supply power for the continuous operation of critical processes within the plant until the normal power has been restored. RWQCP has three generator systems one for the MBR, a second for the aeration blowers and a third for the rest of the plant. Each of the three systems consists of a duty and standby generator for reliability.

### 3.3.2 Treatment Process Reliability

This subsection describes the manner by which each treatment process provides reliability.

#### 3.3.2.1 Preliminary Treatment

Although Title 22 lists no specific criteria for reliability of preliminary treatment and pumping, reliability is necessary and is provided to ensure flow of wastewater through the plant. The headworks facility consists of climber bar screens with 1/2-inch (in) bar spacing installed in the four screening channels to remove solids in the influent flow. The existing screens operate at full capacity during peak flow. The update to the Integrated Master Plan for the Wastewater Collection and Treatment Facilities recommended addition of a screening bypass channel to the headworks to bypass excess flows should one of the screening channels be out of service. Screening and grit removal systems include screenings conveyance, washing and compaction, grit removal, and grit pumping. The plant can be operated without preliminary treatment for short periods without significant negative effects. Existing alarms also indicate failure of pumps, motors, and other critical equipment.

#### 3.3.2.2 Primary Sedimentation

Section 60343 requires that primary treatment processes be provided with one of the following reliability features:

- Multiple primary units capable of producing primary effluent with one unit not in operation
- Standby primary treatment process
- Long-term storage or disposal provisions

*MBR Plant*

The primary treatment for the MBR Plant employs four primary clarifiers. In the event one basin is out of service, the remaining basins can adequately treat the primary influent flow to the MBR Plant. A portion of the primary influent flow from the MBR Plant can be diverted to the ACT Plant if needed.

*ACT Plant*

The primary treatment for the ACT Plant employs four primary clarifiers. In the event one basin is out of service, the remaining basins can adequately treat the primary influent. In the event of failure of a primary treatment in the ACT Plant, a portion of that plant's inflow can be diverted for treatment in the MBR Plant.

#### 3.3.2.3 Biological Treatment

Section 60345 requires that biological treatment processes be provided with one of the following reliability features:

- Alarm and multiple biological treatment units capable of producing oxidized effluent when one unit is not in service
- Alarm, short-term retention or disposal provisions, and standby replacement equipment
- Alarm and long-term storage or disposal provisions
- Automatically actuated long-term storage or disposal provisions

*MBR Plant*

The MBR Plant has five aeration basins and eight MBR treatment train membrane tanks. The membrane tanks were designed for 32 mgd, but the system was equipped with enough membrane area to operate at an average capacity of 26 mgd. The MBR Plant biological treatment capacity is 26 mgd with one unit out of service. In the event that one basin is taken out of service, excess flow may be routed to the ACT Plant.

The MBR Plant has three single-stage centrifugal aeration blowers. In the event that one aeration blower fails, the standby unit will provide for uninterrupted air to the process. Flow can also be stored at the diurnal flow equalization and wet weather equalization basin and then pumped back to the MBR Plant or the ACT Plant aeration basins. Alarms are provided to indicate failure of blowers, RAS pumps and mixers, and to indicate low and high dissolved oxygen levels.

*ACT Plant*

The ACT Plant has six aeration basins. The capacity of the ACT Plant biological treatment is 17.5 mgd with one secondary clarifier out of service and 20 mgd with all units in service. In the event that one zone of one basin is taken out of service, flow in excess of 17.5 mgd can be treated at the MBR Plant.

The ACT Plant has three turbo aeration blowers. In the event that any of the two duty blowers fail, the standby unit will provide for uninterrupted flow of air to the process. Flow can also be stored at the diurnal flow equalization and wet weather equalization basin and then pumped back to the MBR Plant or the ACT Plant aeration basins. Alarms are provided to indicate failure of blowers, RAS pumps, and mixers and to indicate low and high dissolved oxygen levels.

#### 3.3.2.4 Secondary Sedimentation

Section 60347 requires that secondary sedimentation unit processes be provided with one of the following reliability features:

- Multiple sedimentation units capable of treating the entire flow with one unit not in operation
- Standby sedimentation unit process
- Long-term storage or disposal provisions

The MBR Plant has 8 MBR basins with a current capacity of 26 mgd ADWF with one basin out of service. The ACT Plant has four circular secondary clarifiers with total capacity of 20 mgd ADWF and a reliable capacity of 17.5 mgd with one secondary clarifier unit out of service. The combined secondary treatment capacity if the two plants with one unit out of service at each plant is 46 mgd.

#### 3.3.2.5 Flow Equalization and Filter Influent Pumping

Title 22 contains no specific requirements for equalization or pumping reliability. Both the MBR Plant and the ACT Plant have filter influent pumps with status and failure alarms and standby units.

#### 3.3.2.6 Pre-Filtration Chemical Feed and Coagulation

Section 60349 requires coagulation facilities to be provided. Coagulant is continuously fed at the secondary clarifier structure. Flow from the secondary clarifier goes to the equalization basin prior to tertiary filtration. Turbidity is continuously monitored and has alarm capabilities.

Tertiary effluent that does not meet the discharge requirements would be automatically routed to the storm water retention ponds and thereafter pumped back into the process for treatment.

#### 3.3.2.7 Flocculation

Title 22 does not have specific reliability requirements for the flocculation process. There are flocculation basins upstream of filters 11 – 16 that discharge into CCB3.

#### 3.3.2.8 Filtration

Section 60351 requires that filtration processes be provided with one of the following reliability features:

- Alarm and multiple filter units capable of treating the entire flow with one unit non-operational
- Alarm and short-term retention or disposal provisions and standby replacement equipment
- Alarm and long-term storage or disposal provisions
- Automatically actuated long-term or disposal provisions
- Alarm and standby filtration unit process

Primary reliability for the filtration process is provided by alarm and multiple filtration units that can treat the design influent flow with one unit out of service. Out-of-compliance effluent can be diverted to storm water retention ponds and pumped back into the process for retreatment.

The design criteria for the tertiary filters and media are shown in Table 3.2.

The combined filter effluent turbidity is continuously monitored immediately upstream of the CCBs. Filter influent turbidity is monitored and coagulant feed started automatically if the turbidity is greater than 5 NTU for 15 continuous minutes. All turbidity readings required by Title 22 are indicated in the control building.

*Disinfection*

Section 60353 requires that disinfection unit processes, where chlorine is used as the disinfectant, shall be provided with the following features for uninterrupted chlorine feed:

- Standby chlorine supply
- Manifold systems to connect chlorine cylinders
- Chlorine scales
- Automatic devices for switching to full-chlorine cylinders

The facility uses liquid sodium hypochlorite as the disinfection chemical rather than gaseous chlorine, hence only Requirement 1, above, directly applies. The plant stores sodium hypochlorite in three tanks with five diaphragm hypochlorite feed pumps feeding all three CCBs. Two of the five pumps are standby pumps. The required standby supply is provided by storing up

to 20,000 gallons of sodium hypochlorite which provides 29 days of storage. All storage tanks have level sensors and low-level alarms.

Control is provided locally and remotely though the SCADA Automatic residual control (via a compound control loop of flow pacing and trim), which measures and records chlorine residual.

In addition, Title 22 requires that chlorine disinfection processes be provided with one of the following reliability features:

- Alarm and standby chlorinator
- Alarm, short-term retention or disposal provisions, and standby replacement equipment
- Alarm and long-term storage or disposal provisions
- Automatically actuated long-term storage or disposal provisions
- Alarm and multiple point chlorination, each with independent power source, separate chlorinator, and separate chlorine supply

The use of liquid sodium hypochlorite necessitates modifying the wording of the requirements. In this case, a standby feed pump fills the need for the standby chlorinator. The standby feed pump is automatically started on failure of a duty feed pump. The chlorination system is connected to a backup power generation system. There is a chlorine residual analyzer at the influent and effluent of CCB2. An alarm is activated if the chlorine residual is too high or too low based on the CT monitoring.

## 3.4 Contingency Plan

Overall planning to prevent the discharge of inadequately treated effluent in the event of process outages or upsets is an integral part of plant reliability. Whenever a problem, or potential problem that would adversely affect effluent quality is noted, some action must be taken to prevent improper discharge. The first actions are to assess the nature of the problem and its potential severity and to determine if corrections can readily be made. If it is determined that process adjustments or repairs cannot be made quickly enough, the following general procedures would be followed:

1. The unit experiencing problems would either be taken out of service, or depending on the process involved or type of problem, have its loading temporarily reduced by increasing the loading to other units or processes. As discussed earlier in this chapter, all processes have sufficient redundancy to operate with one unit, treatment bay, or item of major equipment out of service. The problem would then be corrected by either repairing failed equipment or adjusting process variables such as recirculation rates, airflow rates and patterns, chemical feed rates, etc.:
    a. If it is determined that equipment failure is the problem, the plant would be brought back to normal operating mode as soon as the failure is repaired and tested.
    b. If the cause results from process upset or changes to influent flow or quality, the operating staff would make adjustments to the process to accommodate the changes.
2. If it is judged that process adjustments cannot be made quickly enough to prevent the discharge of inadequately treated water to the recycled water system, pumps conveying tertiary effluent to the recycled water system would be stopped, and flow diverted to the storm water retention ponds. Similarly, if inadequately treated effluent is detected,

via either low effluent chlorine residual or high effluent turbidity, diversion to storm water retention ponds would occur in the same manner.

3. A third option is to discharge flow to the Santa Ana River.

## 3.5 Capacity Analysis

To determine a reliable treatment capacity for the plant, Carollo Engineers prepared analyses and mass balances using a BioWin computer process model under future plant influent flow and load conditions to identify the process capacity of each unit process. Results of the evaluation are discussed below. The spreadsheet output is included in Appendix D.

## 3.6 Process Model Application

The calibrated process model was used to simulate RWQCP performance under corresponding peak month plant influent flow conditions at an ADWF rate of 46 mgd. Peak month peak influent loads were based on current constituent concentrations and peaking factors.

The flow split between the MBR Plant and the ACT Plant was 1.3:1.0 (i.e., 26 mgd: 20 mgd), based on existing design data. An aerobic SRT of 7.5 days was assumed for both the MBR Plant and the ACT Plant to ensure reliable nitrification at the minimum wastewater temperature of 20 degrees C.

### 3.6.1 Process Reliability Criteria

The individual and combined capacities of the primary and secondary treatment trains were evaluated at the conditions below:

- One process unit from the MBR Plant out of service
- One secondary clarifier from the ACT Plant out of service

The evaluation assumed that only one of the process units from either the MBR Plant or the ACT Plant would be out of service at a time per interpretation of the Title 22 requirements. In this case, taking one of the MBR process units out of service is the most restrictive capacity assumption for the RWQCP, thus that is what the secondary treatment capacity was based upon.

### 3.6.2 MBR Treatment Train

The MBR treatment train process capacity was estimated using five criteria: primary clarifier SOR, MBR aeration tank air demand, MBR hydraulic flux, and MBR solids mass flux (SMF), and chlorine contact tank hydraulic residence time. The MBR hydraulic flux and SMF are calculated assuming one MBR tank out of service. The minimum chlorine contact tank hydraulic residence time of 120 minutes (calculated based on the entire tank volume) is based on the Title 22 minimum modal contact time of 90 minutes, assuming a conservative modal contact time: hydraulic residence time ratio of 0.75.

A separate test has been completed to determine the hydraulic residence time and CT value for CCB No. 2. This will reduce the amount of chlorine and /or residence time that is required. The report is attached as Appendix A.

The MBR treatment train capacity was determined to be to 26 mgd ADF, with one MBR tank out of service. The MBR membrane tanks were designed with space to allow additional membrane cassettes to be added, which would increase the filtration capacity to 32 mgd.

### 3.6.3 Activated Treatment Train (ACT)

Secondary clarifier SLR limits were calculated using statepoint analysis and a design sludge volume index (SVI) of 120 mL/g based on historical operations and performance data. The secondary clarifiers are critically loaded at 17.5 mgd ADF with one of the larger clarifiers out of service, when operating with an SVI of 120 mL/g (residence time of 3.5 hours).

The ACT process capacity was estimated using six criteria: primary clarifier SOR, activated sludge aeration tank air demand, secondary clarifier SOR, secondary clarifier SLR, filter hydraulic loading rate, chlorine contact tank hydraulic residence time. The secondary clarifier SOR and SLR were calculated assuming that one of the larger clarifiers is out of service. The maximum filter hydraulic loading rate of 3.33 gpm/sq ft is based on historical operation. The minimum chlorine contact tank hydraulic residence time of 120 minutes (calculated based on the entire tank volume) was based on the Title 22 minimum modal contact time of 90 minutes, assuming a conservative modal contact time: hydraulic residence time ratio of 0.75.

The secondary clarifier SLR limits the activated sludge treatment train capacity to 17.5 mgd ADF with one of the larger clarifiers out of service. This is closely followed by the aeration air capacity (approximately 20 mgd).

Table 3.3 summarizes the treatment capacities of the MBR Plant and the ACT Plant under different operating conditions.

Table 3.3 Summary of Plant Capacities

| Process | MBR Plant | | ACT Plant | | Total Capacity[1] (mgd) |
|---|---|---|---|---|---|
| | Operating Condition | Flow[1] (mgd) | Operating Condition | Flow[1] (mgd) | |
| **Primary Treatment Capacity** | | | | | |
| | AUIS | 36 | AUIS | 23 | 59 |
| **Secondary Treatment Capacity** | | | | | |
| | MBR Train OOS | 26 | SC AUIS | 20 | 46 |
| | MBR Train OOS | 26 | SC OOS | 17.5 | 43.5 |
| **Tertiary Treatment Capacity** | | | | | |
| Disinfection | CCB2 | | CCB2 | | 46 |
| Filtration (Installed) | | | | 45.1[2] | |
| Filtration (Title 22) | | | | 67.7[3] | |
| Filtration (Reliable) | | | | 42.2[4] | |

Notes:
(1) Per BioWin modeling. OOS – Out of Service, AUIS – All units in service.
(2) Total installed capacity at a plant filter loading of 3.33 gpm per square foot.
(3) Total installed capacity at a Title 22 loading rate of 5 gpm per square foot.
(4) Filter capacity with one cell out of service at a loading rate of 3.33 gpm per square foot.

As discussed in Chapter 1, the RWQCP is looking to fine tune its operational strategy and streamline efficiencies which consist of taking CCB3 offline and commingling the MBR Plant and the ACT Plant flows prior to CCB2. The two operating strategies will be as follows:

- CCB2 only treats MBR Plant effluent
- CCB2 treats all MBR Plant effluent and ACT Plant effluent

In an effort to reduce disinfection byproducts (DBPs) formed during chlorination, and in an effort to improve overall operational efficiency, Public Works requires modifications to their existing chlorination operations along with detailed "Title 22" permit modifications and documentation. The goal is to drop the contact time requirements and CT requirements below the currently mandated value of 90 minute modal and 900 mg-min/L values, respectively.

The contact time testing has been conducted on the existing CCB2 at peak flow conditions with the understanding that contact time will be longer at lower flow values. The results showing modal contact time and CT requirements for CCB2 are attached as Appendix A.

In conclusion, the combined capacity of the two plants with one MBR train out of service and all other limitations put into consideration is 46 mgd.

Chapter 4

# RECYCLED WATER TRANSMISSION AND DISTRIBUTION AND USE AREAS

## 4.1 Transmission and Distribution Systems

Disinfected tertiary effluent meeting Title 22 requirements is transferred to RPU for distribution to RPU customers and to Toro Turf, a Public Works customer. RPU is responsible for the customers they serve and the associated transmission and distribution piping. The major recycled water uses by RPU customers at this time are irrigation and non-irrigation uses, construction, habitat development, maintenance, and recreational uses. Groundwater recharge with recycled water may be considered in the future by RPU.

In the future, the use of recycled water will increase as treated flows increase and as facilities to convey recycled water from the plant to new areas are constructed. Currently, any unused recycled water is discharged into the Santa Ana River.

## 4.2 Recycled Water Users and Use Areas

The RPU distributes the treated effluent from RWQCP to its end users under its own WDO R8-2013-0028. RPU has Rules and Regulations for its customers. This report covers only the end users that receive tertiary disinfected effluent directly from Public Works, which is currently only Toro Turf.

The daily average recycled flow to Toro Turf for the year 2017 was 54,000 gpd. The Toro Turf irrigation boundaries are shown on Figure 4.1.

The RWQCP plans to increase its recycled water supply to the RPU as influent flow increases, but there are no plans to add any other customers receiving recycled water directly from Public Works. There are plans to have RPU take over distribution of recycled effluent to Toro Turf. Therefore all future recycled water customers will receive water from the RPU not Public Works.

## 4.3 Use Area Inspections and Monitoring

Public Works has a user agreement with the Toro Company and the County of Riverside (County) dated December 1985 which is attached as Appendix E. The primary use of the treated effluent by Toro is irrigation of grass for testing of their mowing equipment. The agreement outlines the responsibility of each of the three parties during and after the development of the pipeline project delivering the irrigation water to Toro. The agreement required the County to be primarily responsible for the construction of the pipeline and for obtaining any required permits. It also requires all recycled water to be metered. Public Works should inform the County and the Toro Company of any treatment shutdown, failure, or discharge of effluent not meeting the required standards within 24 hours, but is not obligated to provide an alternative water source.

## 4.4 Supplemental Water Supply

Section 60315 of Title 22 does not permit potable water supply as a backup to recycled water unless the connection between the two systems is protected by an air gap separation.

There are no proposed potable water cross connections specific to the Toro use area. Due to high recycled water demands, there are future proposed projects that would install interconnections between the potable water system and the recycled water system. All proposed interconnections would be in the RPU recycled water distribution area and hence the responsibility of the RPU. Any interconnection would include an air gap separation to meet the above requirements.

Section 60310 of Title 22 does not permit irrigation with disinfected tertiary recycled water within 50 feet of any domestic water supply unless all of the following conditions have been met:

- A geological investigation demonstrates that an aquitard exists at the well between the uppermost aquifer from which it is drawn and the ground surface.
- The well contains an annular seal that extends from the surface into the aquitard.
- The well is housed to prevent any recycled water spray from coming into contact with the wellhead facilities.

There are no potable water wells in the Toro Turf use area.

Figure 4.1 End User Area

Appendix A

# CCB2 CT TESTING REPORT

City of Riverside
Chlorine CT and Modal Contact Time Study

# CONTACT TIME TESTING AND CT DEMONSTRATION RESULTS

FINAL | November 2018

carollo

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Carollo Engineers, Inc.
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Digitally signed by Nicola A. Fontaine
Contact Info: Carollo Engineers, Inc.
Date: 2018.11.09 14:32:45-08'00'

REGISTERED PROFESSIONAL ENGINEER
NICOLA A. FONTAINE
No. 76863
CIVIL
STATE OF CALIFORNIA

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# Abbreviations

| | |
|---|---|
| ACS | American Chemical Society |
| BOD | Biological Oxygen Demand |
| Carollo | Carollo Engineers, Inc. |
| CCB | Chlorine Contact Basin |
| City | City of Riverside |
| DBP | Disinfection Byproduct |
| DDW | Division of Drinking Water |
| DO | Dissolved Oxygen |
| MBR | Membrane Bioreactor |
| MG | million gallons |
| mg/L | milligrams per liter |
| mg-min/L | milligram-minutes per liter |
| mgd | million gallons per day |
| MPN | most probable number |
| RWQCB | Regional Water Quality Control Board |
| RWQCP | Riverside Regional Water Quality Control Plant |
| RWPS | Recycled Water Pump Station |
| SP | Sample Point |
| SWRCB | State Water Resources Control Board |
| TDS | Total Dissolved Solids |
| TDT | Theoretical Detention Time |
| TOC | Total Organic Carbon |
| USEPA | United States Environmental Protection Agency |

Section 1

# CONTACT TIME TESTING AND CT DEMONSTRATION

## 1.1 Purpose

In an effort to reduce disinfection byproducts (DBPs) formed during chlorination, and to improve overall operational efficiency at the Riverside Regional Water Quality Control Plant (RWQCP), the City of Riverside (City) Department of Public Works (Public Works) would like to modify their existing chlorination operations which would require associated "Title 22" permit modifications and documentation.

The goal of this work was to drop the contact time requirements and CT (chlorine residual measured at the end of the contact time multiplied by the contact time) requirements to below the currently mandated value of 90-minute modal contact time and 900 milligram-minutes per liter (mg-min/L) values, respectively. The current CT of 900 mg-min/L is calculated using the total chlorine residual. Work within our water reuse industry has demonstrated disinfection compliance (both bacteria and virus) at CT values much less than 450 mg-min/L, provided that the chlorine residual component of the CT is always free chlorine. To achieve this goal Carollo Engineers, Inc. (Carollo) and Public Works conducted:

- Tracer testing to determine modal contact times in Chlorine Contact Basin 2 (CCB2)
- Bench-scale testing to determine:
  - Chlorine demand
  - Seeded virus and indigenous coliform disinfection dose/response with free chlorine
  - Lowest free chlorine disinfection CT that will meet both Title 22 virus requirements (5-log reduction) and achieve total coliform permit compliance with a degree of reliability
- Full-scale testing to demonstrate that the reduced free chlorine CT provides disinfection to meet total coliform permit requirements

While the RWQCP does have other CCBs, CCB2 is the only CCB that would be utilized for "low CT" applications at this time.

Research suggests that the 5-log virus disinfection requirements can be satisfied at contact time and free chlorine CT values as low as 10 minutes and 25 mg-min/L, respectively. The 2.2 most probable number per 100 milliliters (MPN/100 mL) total coliform requirements can also be satisfied at values much lower than the current permit (900 mg-min/L) and the conventional Title 22 requirements (450 mg-min/L). Stated once more for clarity, the current 900 mg-min/L system operates based upon an extended contact time and a total chlorine residual (not free chlorine).

The protocol prepared for this testing was submitted to the Regional Water Quality Control Board (RWQCB) and the State Water Resources Control Board (SWRCB) Division of Drinking

Water (DDW) for review and approval prior to the performance of the work. Comments from DDW were incorporated into the testing protocol used to guide the work documented within this report.

## 1.2 Background

Public Works is responsible for the collection and treatment of wastewater generated within the City as well as the community of Highgrove. Public Works is also responsible for the treatment only for the wastewater from the Community Service Districts of Edgemont, Jurupa, and Rubidoux. The RWQCP provides primary, secondary, and tertiary treatment at a rated capacity of 46 million gallons per day (mgd). The RWQCP operates as two individual plants with a final combined effluent which is discharged to the Santa Ana River. Figure 1 shows the RWQCP process flow schematic. The Membrane Bioreactor (MBR) Plant is shown with dark lines and the Activated Treatment Train (ACT) is shown with grey lines.

The MBR Plant includes primary treatment and MBRs. MBR permeate is then introduced to CCB2 where it is chlorinated to meet the RWQCB interim approval CT requirements of 900 mg-min/L. CCB2 effluent that is not reused is then dechlorinated prior to combining with the ACT Plant Chlorine Contact Basin 3 (CCB3) effluent. The combined flow is discharged to the Santa Ana River. A small flow is transferred as Title 22 recycled water.

The MBR Plant installation was completed in the summer of 2017. The membrane tanks were designed for 32 mgd, but the system was equipped with enough membrane area to operate at an average capacity of 26 mgd. In addition to the installation of the MBR system, CCB2 volume was increased to 3.24 million gallons (MG). Interim approvals from the RWQCB to operate CCB2 were based on design calculations since a tracer test had not been performed. The conservative interim approval CT value is 900 mg-min/L, which is double the Title 22 requirement of 450 mg-min/L.

The ACT Plant includes primary, secondary, and tertiary treatment using traditional anthracite/sand/gravel filter media. Filter effluent is introduced to CCB3 where it is chlorinated/dechlorinated and discharged to the Santa Ana River. The volume of CCB3 is 2.9 MG. The ACT Plant has an average flow rated treatment capacity of 20 mgd.

As mentioned previously, the RWQCP is looking to fine tune its operational strategy and streamline efficiencies which may include taking CCB3 offline and commingling the MBR Plant and the ACT Plant flows prior to CCB2. Going forward, the two operational plans are:

- CCB2 only treats MBR Plant effluent
- CCB2 treats MBR Plant and the ACT Plant effluent

## 1.3 Chlorine Contact Basin 2

CCB2 was constructed in 1981, consisting of 8.0-feet wide serpentine passes 32-feet in length with a total volume of 1.89 MG. In 2017 CCB2 was expanded to 3.24 MG. The expanded portion of the basin has 8.0-feet wide serpentine passes 32-feet in length as well as four passes approximately 50-feet in length. The design water depth is 19.17 feet. CCB2 was expanded to achieve a 90-minute modal contact time at peak flow (46 mgd). Drawings of CCB2 are provided in Appendix A.

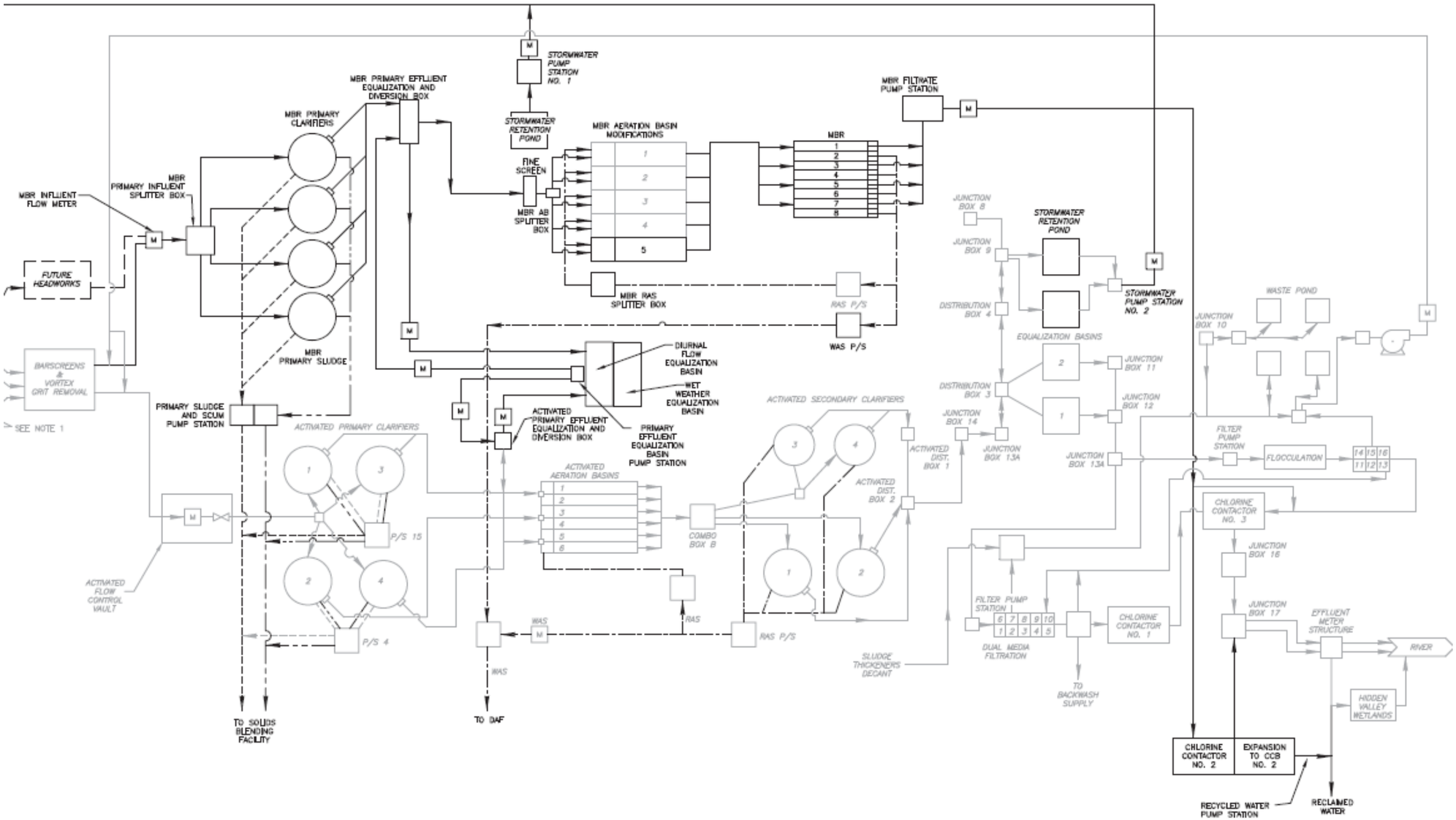



Figure 1 RWQCP Process Flow Schematic

Section 2

# CONTACT TIME TRACER TESTING

To determine the contact time at several locations in CCB2, tracer testing using fluoride was performed. Tracer testing included sampling at six locations in CCB2 to determine the modal contact time and $t_{10}$ (time for 10 percent of the flow to pass through the basin) for each location at peak flow (46 mgd). The goal of selecting the six locations was to evaluate alternative compliance points which would allow for a lower chlorine dose and reduce DBP formation. MBR Plant effluent and ACT Plant effluent were commingled to achieve the peak flow for testing. The MBR Plant provided 32 mgd and the ACT Plant provided 14 mgd to make up the test flow of 46 mgd.

Based on the peak flow and the volume of CCB2 and the recycled water pump station (RWPS) the theoretical detention time is 123 minutes at 46 mgd. Typically tracer studies are run for three theoretical detention times (TDT) to account for unknown hydraulic conditions that may occur within the basin. Alternatively, the testing can be run until all tracer has left the basin and fluoride has returned to background levels. Tracer testing was run for 248 minutes (two TDT) until the fluoride concentrations returned to background levels at the outlet of the basin. This was confirmed through a mass balance recovery analysis.

## 2.1 Tracer and Methods

American Chemical Society (ACS) grade sodium fluoride (>99 percent purity) was used as the tracer chemical. Testing was performed by the slug dose method. In a slug dose test, the tracer chemical is quickly introduced into the CCB influent at the point of chlorination. For a slug dose test to work properly, the tracer must be added nearly instantaneously. The Disinfection Profiling and Bench Marking Guidance Manual, published by the United States Environmental Protection Agency (USEPA) define instantaneous dosing as less than two percent of the TDT for the test. At 46 mgd the dosing time should be 2.46 minutes. Dosing was timed with a stopwatch. It took 42 seconds to dump the fluoride solution into the basin. The amount of fluoride added for the slug dose method took into account the background fluoride concentration, which was 0.7 milligrams per liter (mg/L) during the testing.

## 2.2 Dosing and Sampling Locations

The fluoride solution was dosed into the opening at the chlorine dosing point ahead of the vertical mixer. Samples were collected at six sample points within the basin to measure fluoride concentrations. Sampling Point (SP) 6 is the current compliance point for monitoring total chlorine residual. Therefore, the results obtained at SP6 during the study are representative of the current chemical disinfection modal contact time. Figure 2 presents the dosing point and the sampling locations in CCB2.

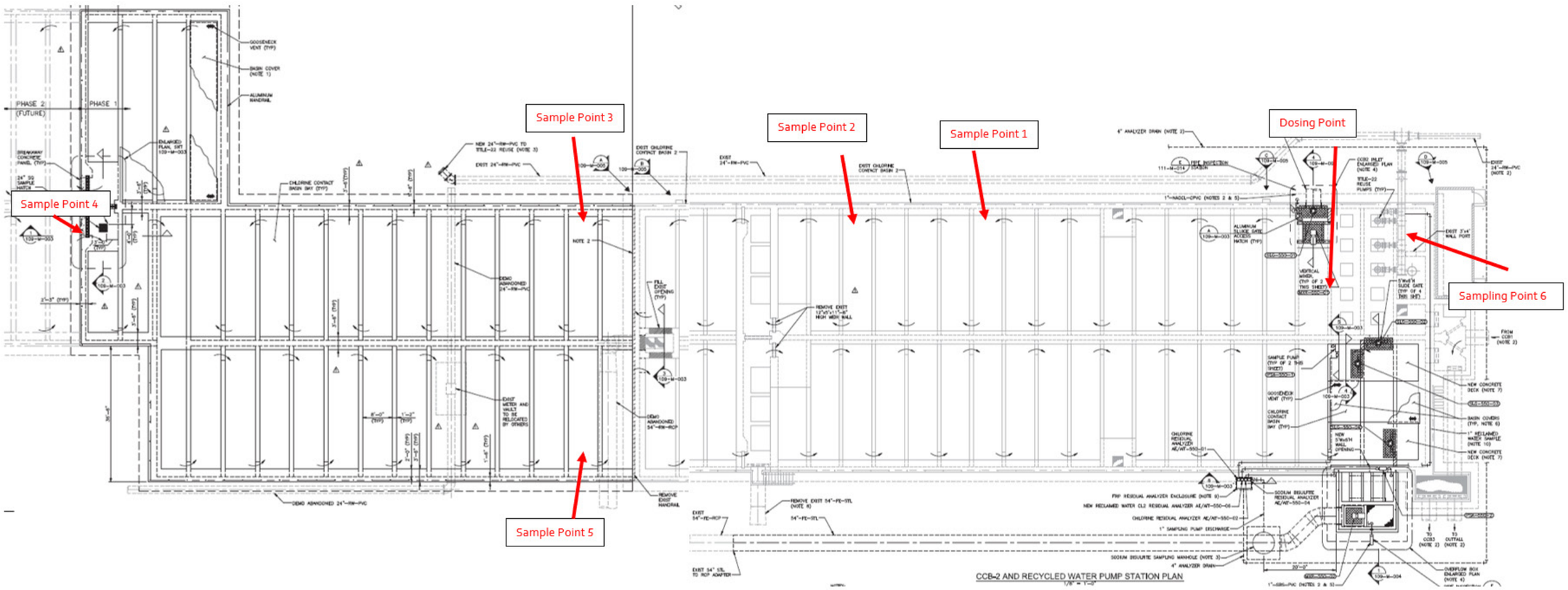



Figure 2 CCB2 Dosing and Sampling Locations

## 2.3 Flow

Flow into CCB2 during the testing was continuously measured by adding the final effluent magmeter value and the recycled water magmeter value, as the recycled water flow is pulled from the CCB effluent before the final effluent magmeter location. Although steady flow during tracer testing is important, variability in the flow during the testing was unavoidable due to the backpulsing in the MBR system. During the testing flow ranged from 44 to 50 mgd with an average of 47 mgd. While these are not ideal conditions for tracer testing they do reflect actual "real-world" operating conditions at the RWQCP. Figure 3 shows the flow variation during the testing.

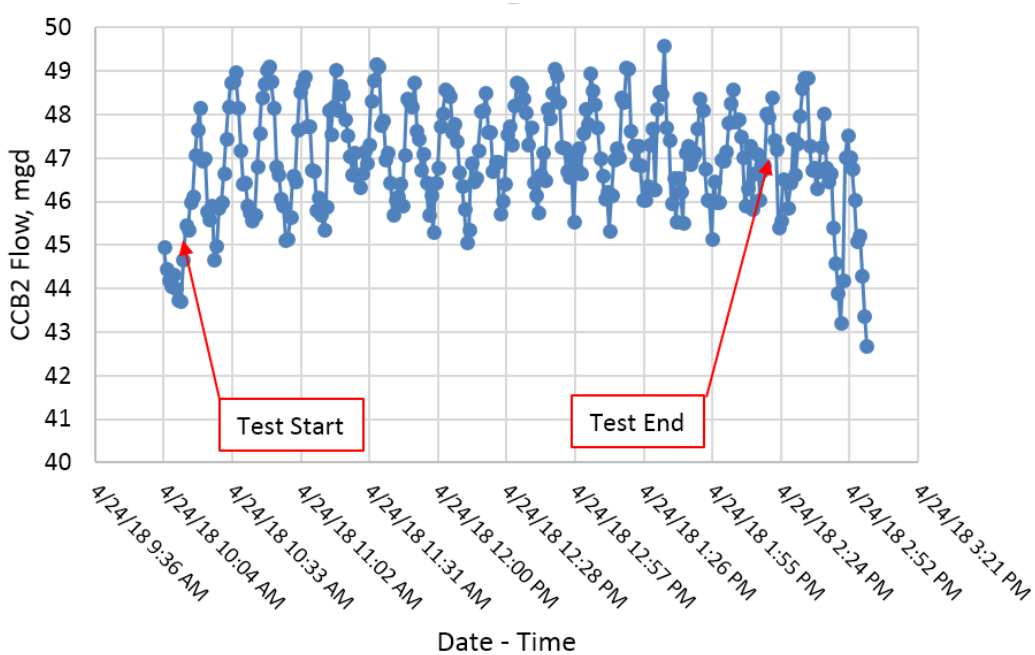



Figure 3 CCB2 Flow during Testing

Figure 4 shows a schematic of the flow path used for the tracer testing. Sample locations and the locations of the magmeters are also shown on the schematic. For the tracer testing the inlet box for CCB3 was used to divert flow from Tertiary Filters 11 -16 to CCB2. Effluent from Tertiary Filters 1-10 flowed from CCB1 to CCB2. Tertiary filter flows commingled with the MBR effluent prior to entering CCB2. Figure 5 shows a schematic of the normal operational flow path.

## 2.4 Sampling and Analysis

Samples were collected before testing to confirm the background fluoride concentration of 0.7 mg/L. After the tracer was added samples were collected every minute until the tracer peak concentration passed SP1 and SP2. At the other SP locations the sampling was begun prior to the TDT of that SP at minute intervals. After the tracer peak had passed, SP samples were collected every five to fifteen minutes until the fluoride concentration returned to background levels. A portion of the samples was analyzed after collection to monitor the tracer peaks.

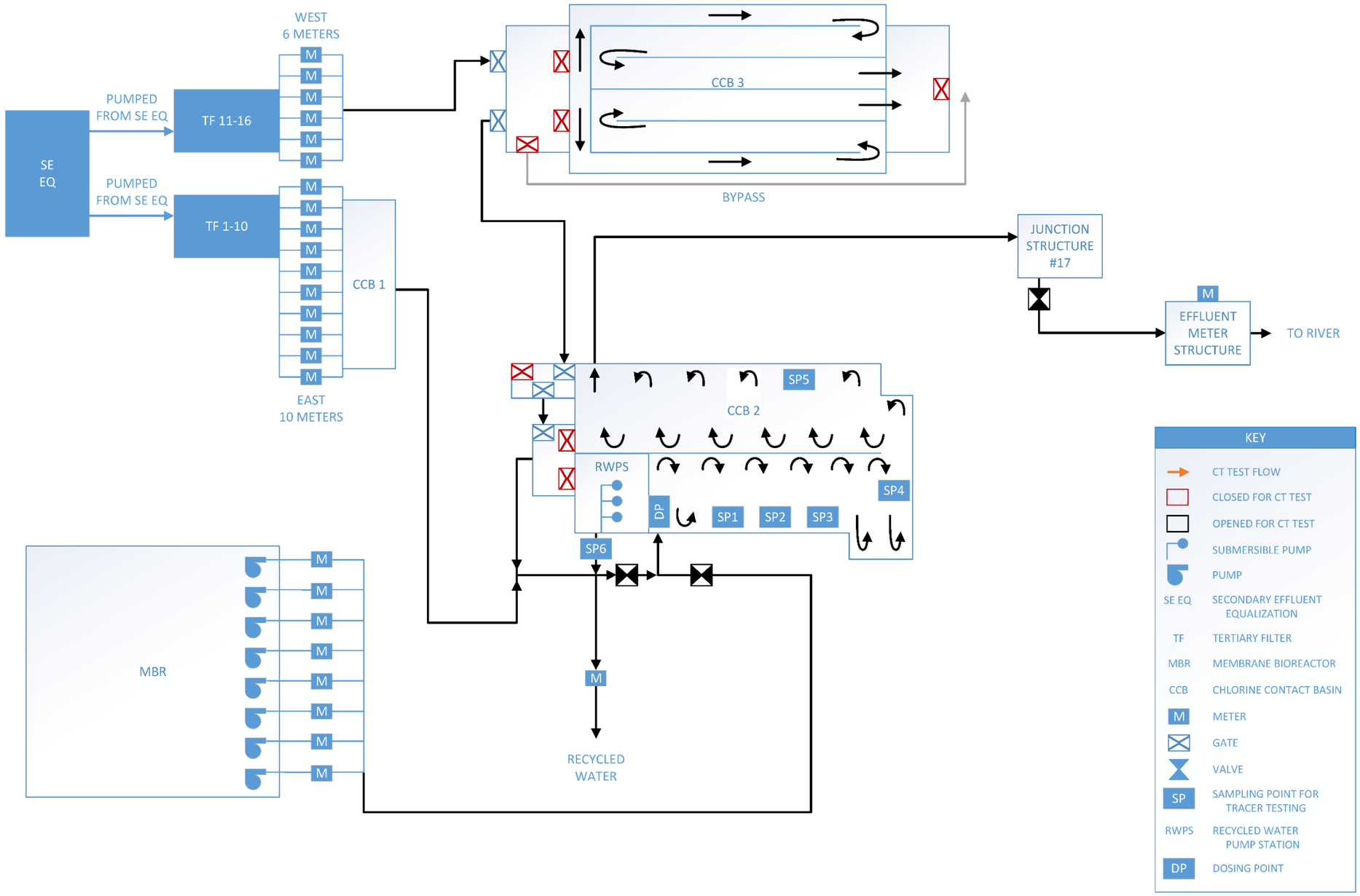



Figure 4 Tracer Testing RWQCP Flow Path

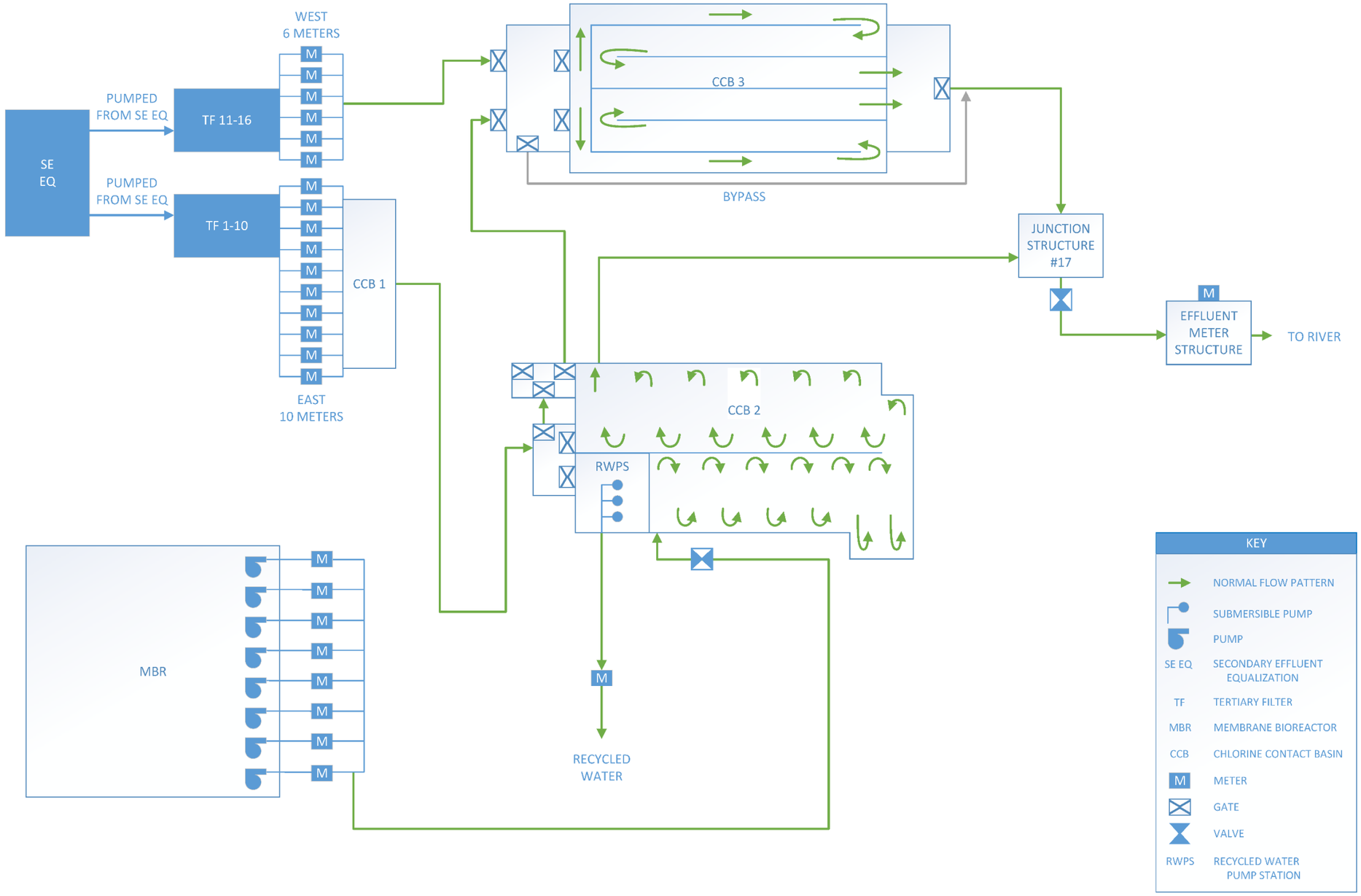



Figure 5 Normal Operational RWQCP Flow Path

All samples were collected in new clean plastic sample containers and labeled by location and time. All samples were stored in a cooler between the time of sample collection and analysis.

Fluoride concentrations were measured using a HACH DR900 handheld colorimeter and the HACH SPADNS 2 method which is equivalent to standard method SM 4500-F D. All of the samples including the ones previously read in the field were measured at the end of the testing once the samples were allowed to reach room temperature. The method recommends that samples are allowed to reach room temperature before analysis to improve accuracy. The data from the samples measured at room temperature was used for analysis and the development of tracer curves and the mass balance recovery rate.

## 2.5 Tracer Testing Results

The tracer testing results were analyzed to determine the modal contact time and $t_{10}$ at all six sample locations. Title 22, 60301.600 defines Modal Contact Time as "the amount of time elapsed between the time that a tracer, such as salt or dye, is injected into the influent at the entrance to the chamber and the time that the highest concentration of the tracer is observed in the effluent of the chamber". The $t_{10}$ is the amount of time for 10 percent of the water to pass through the basin. Note that Title 22 disinfection is based on modal contact time at the sample point, though recent work focusing on reduced CT values considers the use of $t_{10}$. The $t_{10}$ value is typically used in drinking water disinfection.

The data analysis for each of the six SPs included the following steps:

- Measure fluoride concentrations in all samples
- Calculate the actual fluoride tracer concentration (C) for each sample by subtracting the baseline concentration from the measured concentration
- Create a C-curve by plotting C versus time
- Determine the modal contact time
- Convert the data to an F-curve by plotting cumulative recovery versus time
- Determine $t_{10}$
- Calculate the mass of tracer recovered
- Determine the percent recovery rate
- Determine whether the test was valid

Recovery rates greater than 90 percent are indicative of a valid test that did not have interfering chemical reactions and had an adequate sampling schedule. Results between 75 and 90 percent are considered reasonable but may indicate a testing problem or that the sampling was not performed long enough to capture the entire tracer curve back to background concentrations (Tracer Studies in Water Treatment Facilities: A Protocol and Case Studies, AWWARF 1996).

### 2.5.1 Sampling Point 1

The fluoride solution was dosed into the opening at the chlorine dosing point ahead of the vertical mixer. SP1 was located at the second hatch in the basin downstream of the dosing location. Figure 6 shows a photo of the SP1 hatch location and the fluoride dosing location. A chlorine residual analyzer is located after SP1 that is currently used to confirm total chlorine concentrations for the control of chlorine dosing. Based on the range of flow during the testing the TDT at SP1 ranged from 14 to 16 minutes. Figure 7 shows the C-curve for SP1. The flow through CCB2 is included on the graph to illustrate the fluctuation in flow. The peak

concentration representing the modal contact time was sampled at 17 minutes (Figure 7). The $t_{10}$ was determined to be 13 minutes from the F-curve (Figure 8). The recovery rate determined through a mass balance was 98.2 percent.

Figure 6 CCB2 Dosing and Sample Point 1

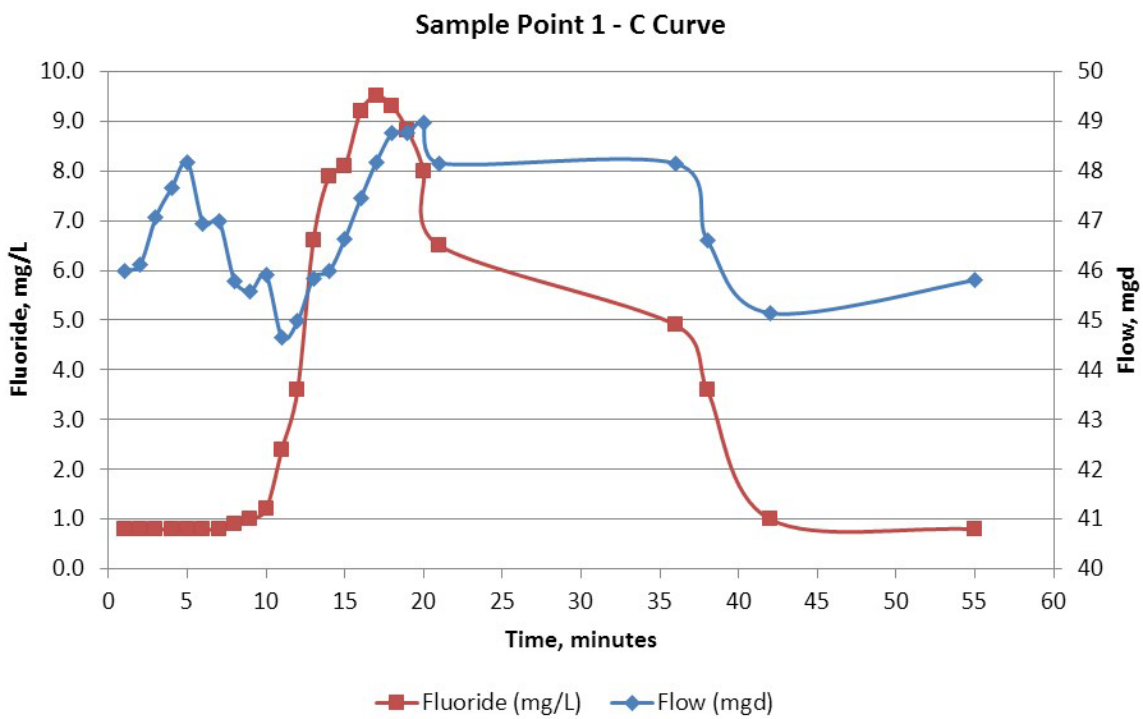



Figure 7 C-Curve for Sample Point 1

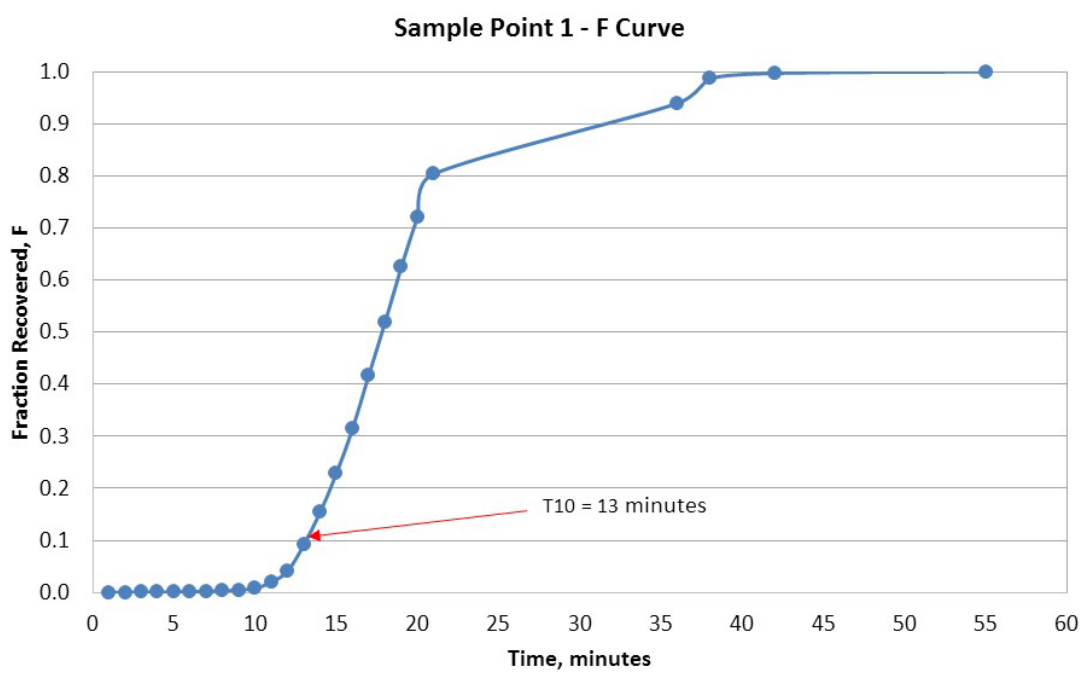



Figure 8 F-Curve for Sample Point 1

### 2.5.2 Sampling Point 2

SP2 was located at the third hatch in the basin downstream of the dosing location. Figure 9 shows a photo of the hatch location that was sampled for SP2 in relation to SP1. The chlorine residual analyzer is located upstream of SP2. Based on the range of flow during the testing the TDT at SP2 ranged from 24 to 27 minutes. Figure 10 shows the C-curve for SP3. The flow through CCB2 is included on the graph to illustrate the fluctuation in flow. The peak concentration representing the modal contact time was sampled at 29 minutes (Figure 10). The $t_{10}$ was determined to be 23 minutes from the F-curve (Figure 11). The recovery rate determined through a mass balance was 98.5 percent.

### 2.5.3 Sampling Point 3

SP3 was located at the first vent on the new part of the basin. The vent (see inset in Figure 12) was removed for sampling. Figure 12 shows the locations of SP2, SP3, and SP4. Based on the range of flow during the testing the TDT at SP3 ranged from 34 to 39 minutes. Figure 13 shows the C-curve for SP3. The flow through CCB2 is included on the graph to illustrate the fluctuation in flow. The peak concentration representing the modal contact time was sampled at 39 minutes (Figure 13). The $t_{10}$ was determined to be 34 minutes from the F-curve (Figure 14). The recovery rate determined through a mass balance was 85.3 percent. The low recovery rate is due to the sampling being stopped prior to returning to the background fluoride level of 0.7 mg/L (Figure 13).

### 2.5.4 Sampling Point 4

SP4 was located at the mid-point of the basin (Figure 12). Samples were collected from a lift hatch. Based on the range of flow during the testing the TDT at SP4 ranged from 56 to 64 minutes. Figure 154 shows the C-curve for SP4. The flow through CCB2 is included on the graph to illustrate the fluctuation in flow. The peak concentration representing the modal contact time was sampled at 68 minutes (Figure 15). The $t_{10}$ was determined to be 59 minutes from the F-curve (Figure 16). The recovery rate determined through a mass balance was 99.0 percent.

### 2.5.5 Sampling Point 5

SP5 was located on the opposite side of the basin across from SP3 (refer to Figure 2). Based on the range of flow during the testing the TDT at SP5 ranged from 77 to 88 minutes. Figure 17 shows the C-curve for SP5. The flow through CCB2 is included on the graph to illustrate the fluctuation in flow. The peak concentration representing the modal contact time was sampled at 90 minutes (Figure 17). The $t_{10}$ was determined to be 76 minutes from the F-curve (Figure 18). The recovery rate determined through a mass balance was 98.3 percent.

### 2.5.6 Sampling Point 6

SP6 was located at the current compliance point at the end of the basin above the RWPS recycled water pumps. Figure 19 shows a photo of the sampling location. Based on the range of flow during the testing the TDT at SP6 ranged from 114 to 130 minutes. Figure 20 shows the C-curve for SP6. The flow through CCB2 is included on the graph to illustrate the fluctuation in flow. The peak concentration representing the modal contact time was sampled at 135 minutes (Figure 20). The $t_{10}$ was determined to be 110 minutes from the F-curve (Figure 21). The recovery rate determined through a mass balance was 99.1 percent.

Figure 9 CCB2 Sample Points 1 and 2

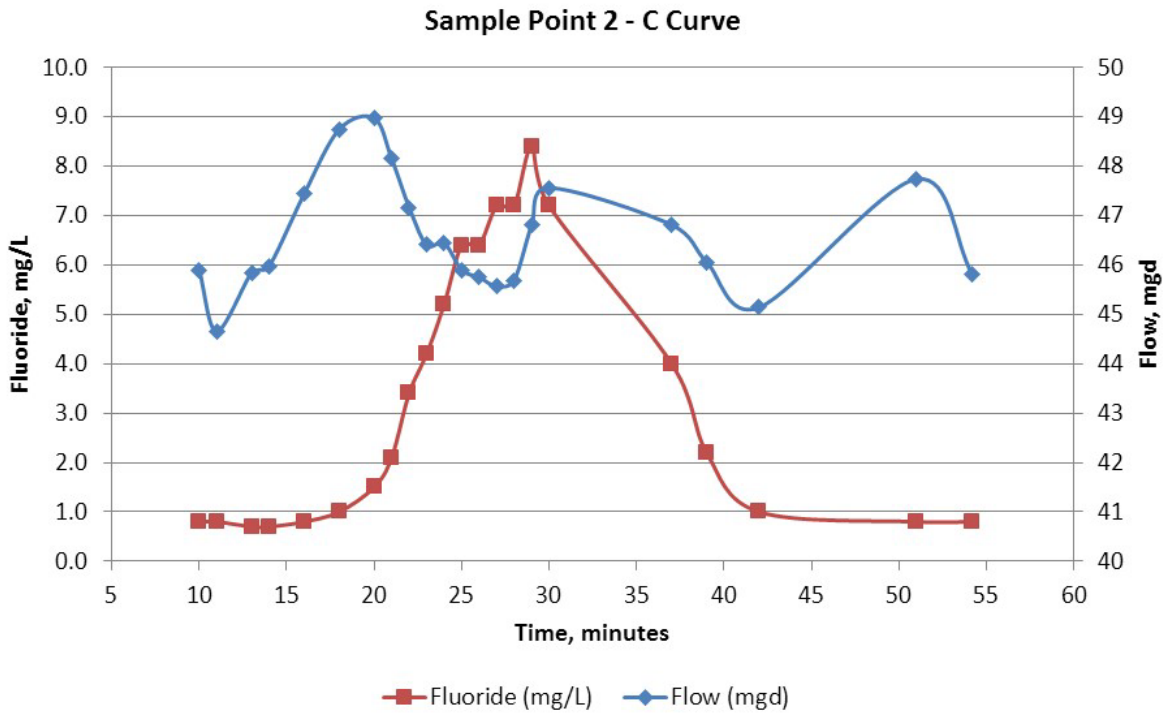



Figure 10 C-Curve for Sample Point 2

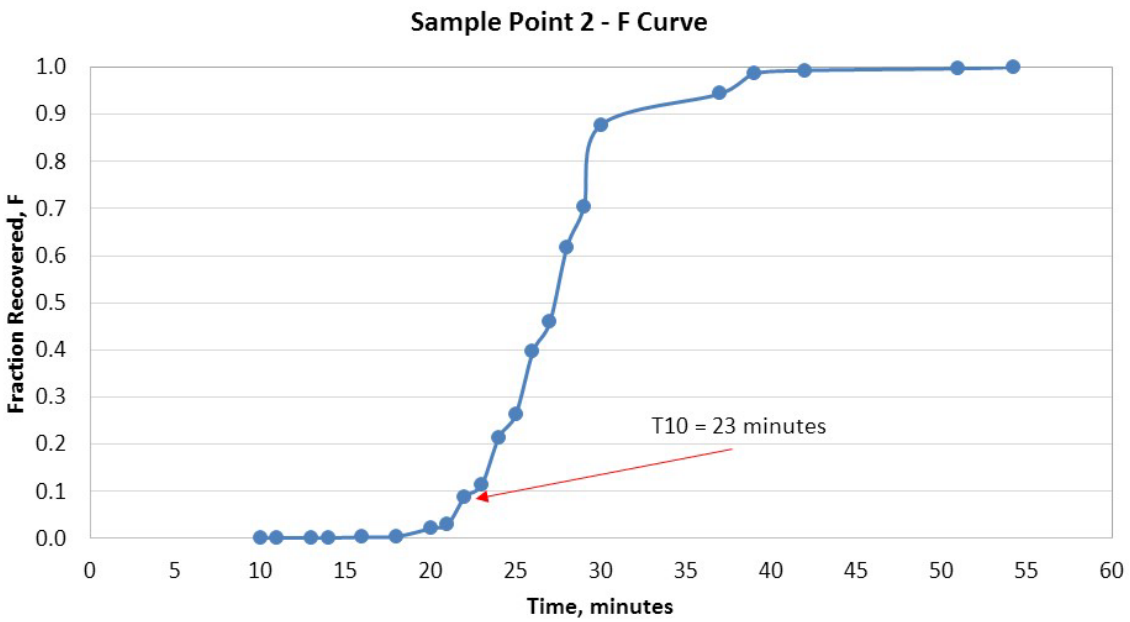



Figure 11 F-Curve for Sample Point 2

Figure 12 CCB2 Sample Points 2, 3, and 4

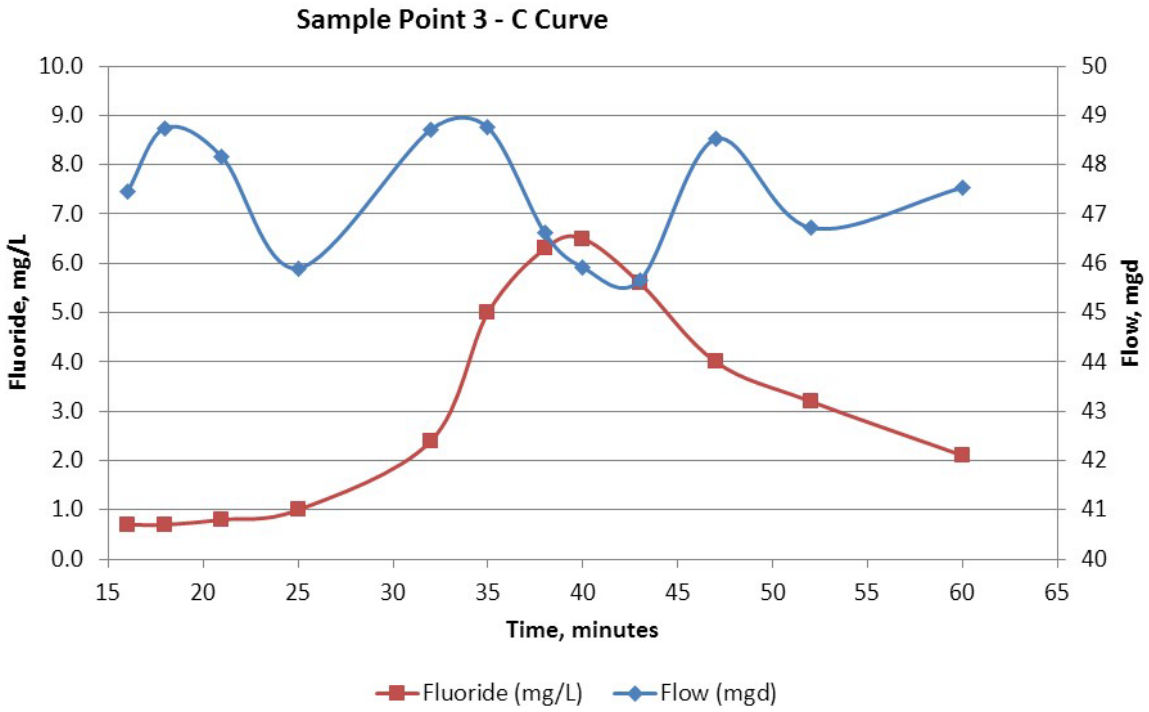



Figure 13 C-Curve for Sample Point 3

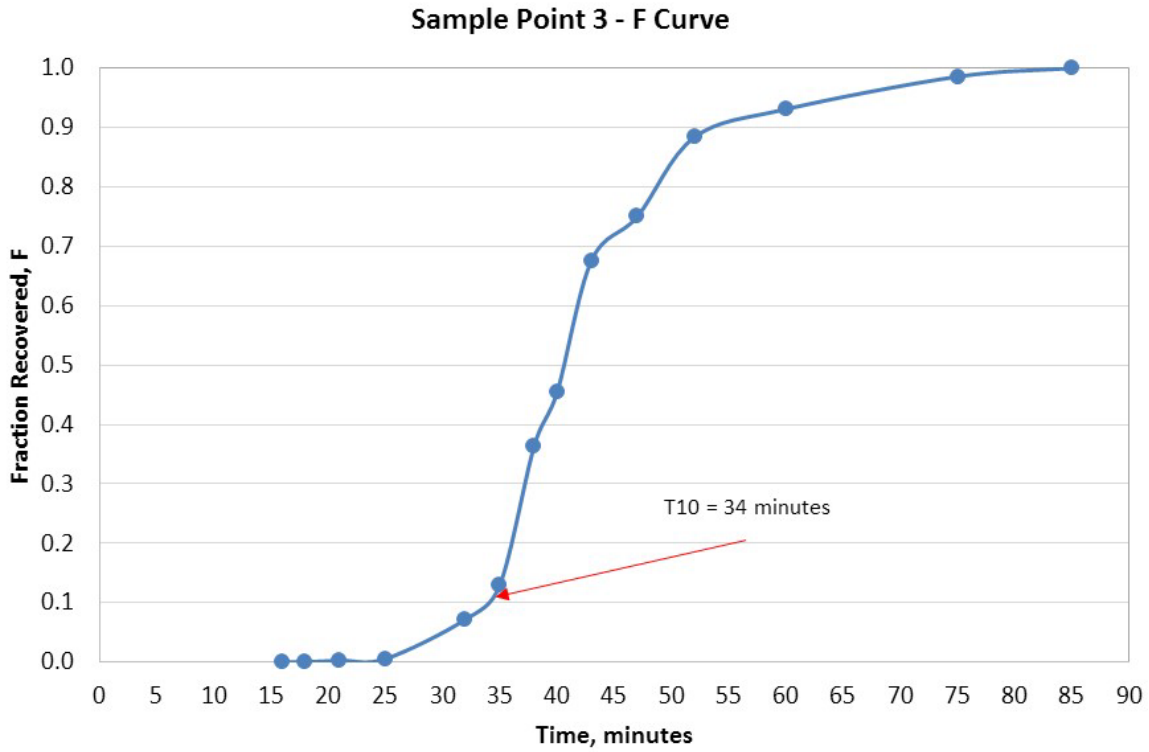



Figure 14 F-Curve for Sample Point 3

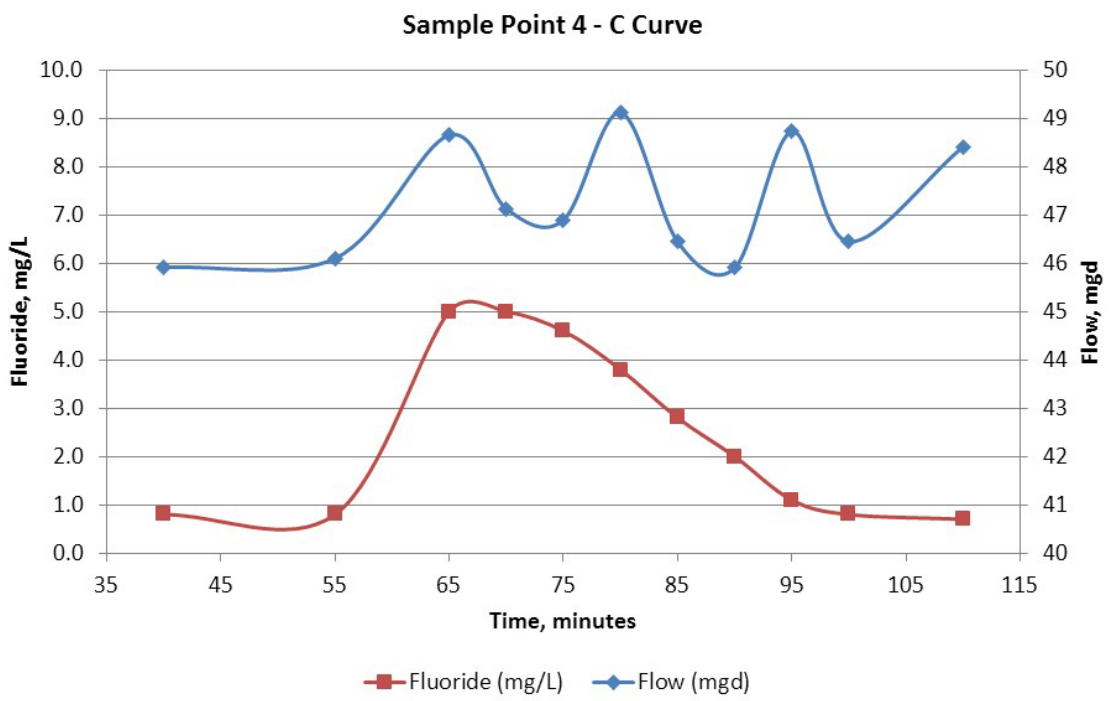



Figure 15 C-Curve for Sample Point 4

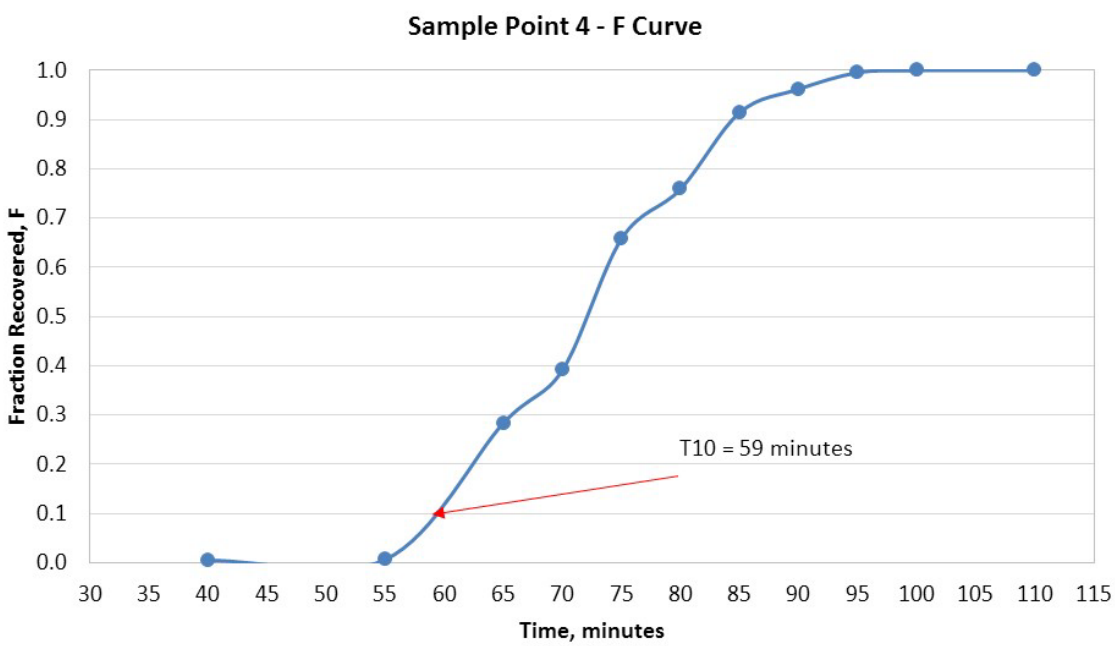



Figure 16 F-Curve for Sample Point 4

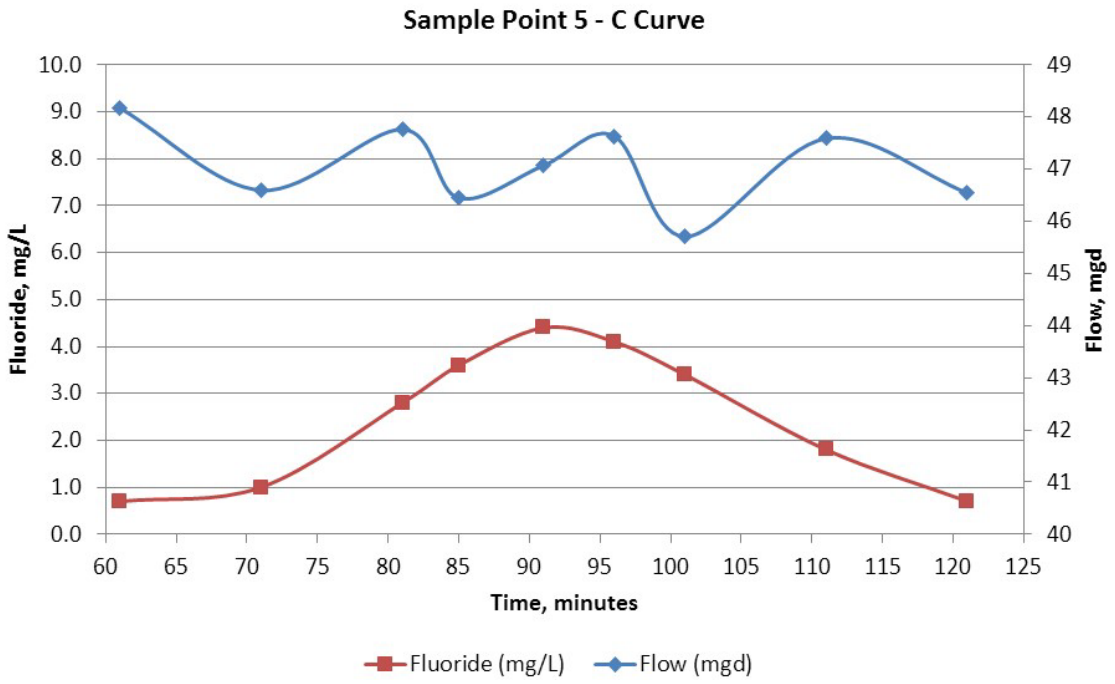



Figure 17 C-Curve for Sample Point 5

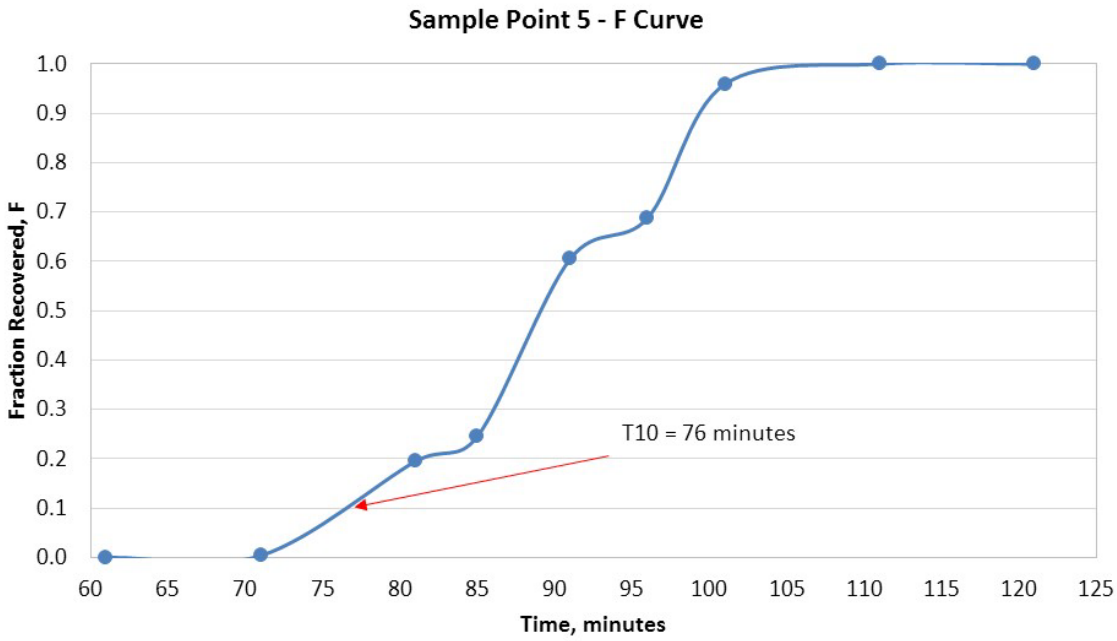



Figure 18 F-Curve for Sample Point 5

Figure 19 CCB2 Sample Point 6

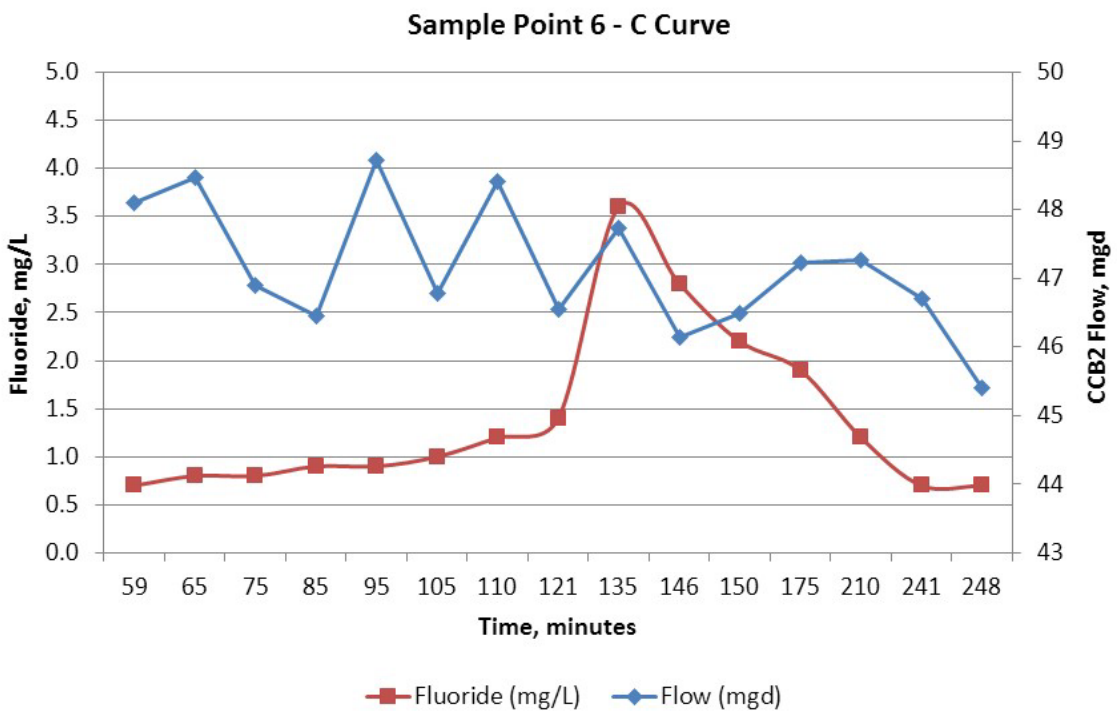



Figure 20 C-Curve for Sample Point 6

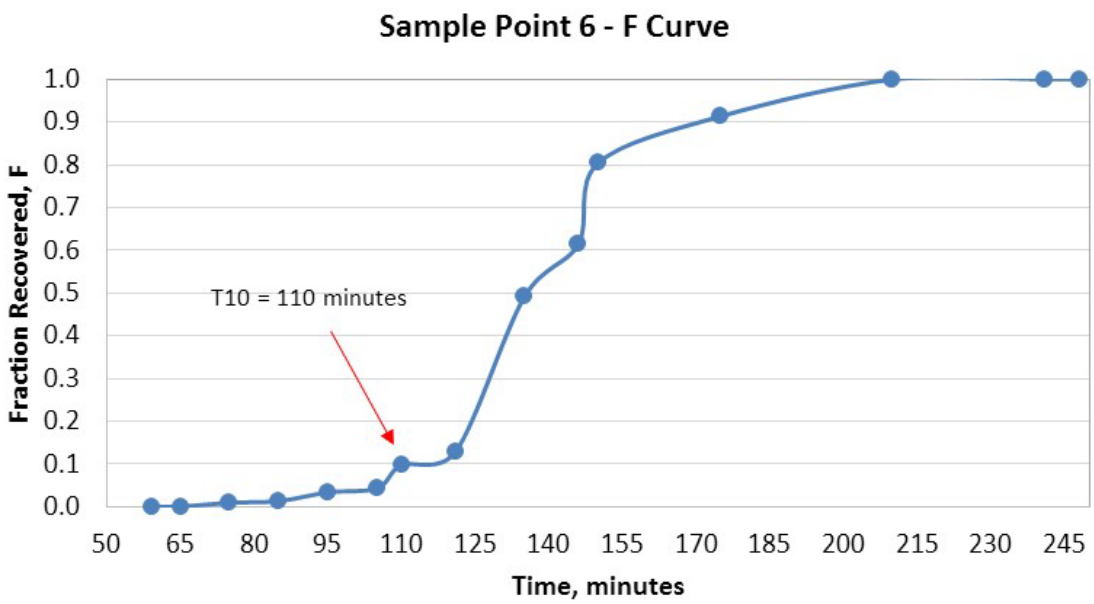



Figure 21 F-Curve for Sample Point 6

## 2.6 Tracer Testing Conclusions

The flow variation, while it reflects real operating conditions at the plant, did impact the tracer testing. However, the recovery rates at all but one sample location were very high ranging from 98.2 to 99.1 percent indicating successful tracer testing. The recovery rate at SP3 was only 85.3 percent because sampling stopped prior to returning to the background fluoride level. As such the curve was not complete on the down slope after the peak had passed SP3, noting that 85.3 percent recovery is acceptable.

Table 1 summarizes the tracer testing results. Differences between theoretical and measured contact times indicate that the actual flows may have been lower than the measured values by the two magmeters. This difference may have been caused by the flow variations due to normal MBR operation in which portions of the membrane tanks are taken off-line periodically for backpulsing, magmeter accuracy (+/- 5 percent) of the magmeters used to determine CCB2 flow, sampling interval, human sampling response time, and/or leaking of isolation gates that were closed for the testing to route flow to CCB2; or a combination of issues. Refer to Figure 3 to see the flow variation during the tracer testing.

Figure 22 shows a summary of the F-curves illustrating the tracer movement through the basin over time.

Table 1 RWQCP Tracer Testing Summary

| Sample Point | Range During Testing Theoretical Detention Time[1], minutes | | | Measured Modal Contact Time, mins | Tracer Recovery Rate, % |
|---|---|---|---|---|---|
| | Max Flow 50 mgd | Ave Flow 47 mgd | Min Flow 44 mgd | | |
| SP1 | 14 | 15 | 16 | 17 | 98.2 |
| SP2 | 24 | 25 | 27 | 29 | 98.5 |
| SP3 | 34 | 36 | 39 | 39 | 85.3[2] |
| SP4 | 56 | 59 | 64 | 68 | 99.0 |
| SP5 | 77 | 82 | 88 | 90 | 98.3 |
| SP6 | 114 | 120 | 130 | 135 | 99.1 |

Notes:
(1) CCB2 flow ranged from 44 to 50 mgd during the tracer testing due to normal MBR operation.
(2) Lower recovery rate because sampling stopped prior to reaching the background fluoride levels.

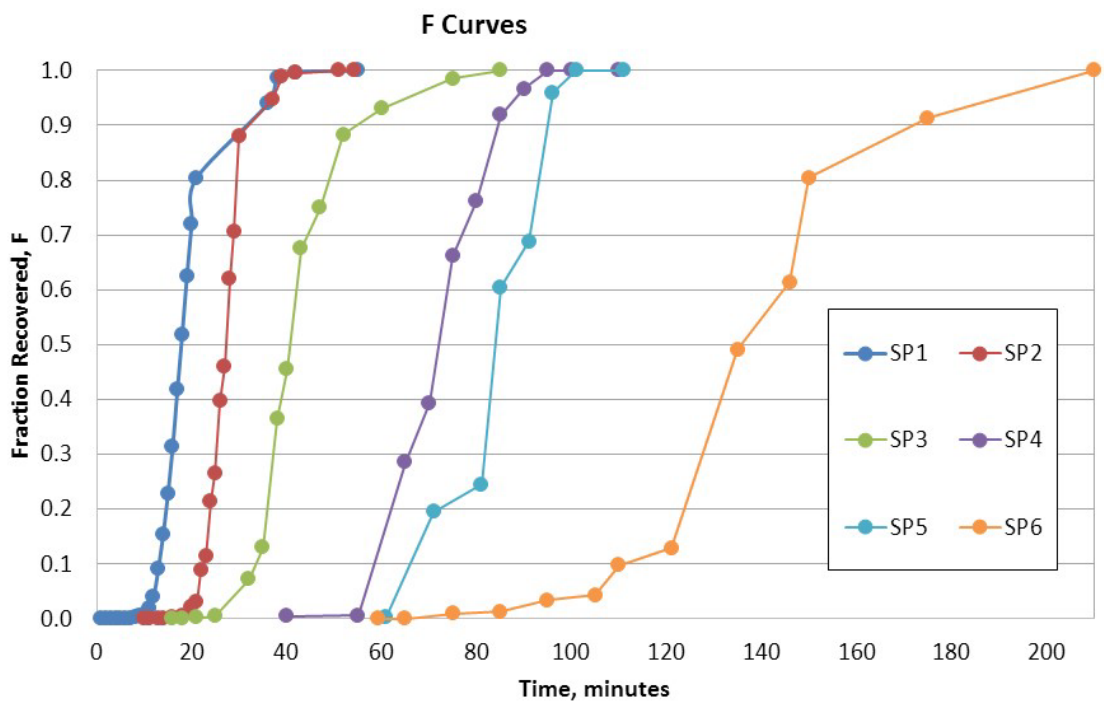



Figure 22 F-Curves for all Sample Points during the Testing

Section 3

# CT DEMONSTRATION

Once the modal contact times were determined for the six locations in CCB2, a detailed bench-scale CT demonstration study was performed to look at chlorine demand and disinfection dose response of seeded virus and indigenous total coliforms over a range of contact times and chlorine doses. The purpose of the bench-scale testing was to determine the lowest free-chlorine CT that met disinfection requirements for virus (5-log inactivation) and permit compliance for total coliforms (2.2 MPN/100mL). Once the bench-scale testing was complete, full-scale testing was performed to demonstrate the total coliform disinfection at low CT values under site specific conditions.

## 3.1 Effluent Samples

Bench-scale testing was performed on the effluents below to evaluate possible operating scenarios at the RWQCP.

- 100 percent MBR effluent
- 30 percent MBR effluent and 70 percent Tertiary Filter effluent
- 100 percent Tertiary Filter effluent

Table 2 contains the water quality characteristics of the effluent samples tested during the bench-scale work. The date on which testing occurred is shown below each of the sample types. All samples were collected downstream of the tertiary process and upstream of the CCBs. The water quality was similar between the two effluent types indicating that there may not be a significant chlorine demand difference. Historical MBR effluent average ammonia and nitrite concentrations are very low, 0.14 mg/L and 0.02 mg/L, respectively (October 2016 to October 2018). Historical tertiary filter effluent average ammonia and nitrite concentrations are also very low, 0.42 mg/L and 0.02 mg/L, respectively (October 2016 to October 2018). Prior to detailed disinfection tests, limited bench-scale tests were done to estimate chlorine demand on the sample blends listed above.

## 3.2 Glassware

Glassware used for the bench-scale testing was pretreated for chlorine demand by soaking in a dilute chlorine solution for approximately an hour and then rinsing with distilled water.

## 3.3 Chlorine Residual Measurement Methods

Total and free chlorine residuals were measured with a HACH DR900 handheld colorimeter by the DPD Method with powder pillows. The DPD Method is equivalent to the USEPA and Standard Method 4500-Cl G for drinking water and wastewater. Two methods were employed to capture the high range and low range residuals. The methods required different DR900 programs, sample cell types, sample volumes, and powder pillows. Due to the frequency of sampling the methods could not be easily interchanged during a specific run. Table 3 contains the methods and the detection range. Doses greater than 2 mg/L were performed with the high range method and doses at or below 2 mg/L with the low range method.

Table 2 RWQCP Effluent Samples Water Quality Characteristics for Bench-Scale Testing

| Parameter | Tertiary Filter Effluent 5-14-18 | Tertiary Filter Effluent 7-2-18 | MBR Effluent 5-14-18 | MBR Effluent 7-2-18 |
|---|---|---|---|---|
| pH | 7.24 | 7.32 | 7.26 | 7.27 |
| Turbidity, NTU | 0.593 | 0.625 | 0.198 | 0.180 |
| Total Dissolved Solids (TDS), mg/L | 658 | 596 | 586 | 586 |
| Ammonia, mg/L | 0.17 | ND | 0.31 | ND |
| Nitrite-N, mg/L | 0.08 | ND | ND | ND |
| Biological Oxygen Demand (BOD), mg/L(1) | 6 | 4 | 6 | 4 |
| Total Organic Carbon (TOC), mg/L | 8.5 | 7.8 | 5.8 | 6.0 |

Notes: ND = non-detect
(1) BOD batch on 5-14-18 had a blank failure, excessive depletion greater than 0.2 mg/L dissolved oxygen (DO).
(2) Nitrite method detection limit is 0.016 mg/L.
(3) Ammonia method detection limit is 0.24 mg/L.

Table 3 Chlorine Residual Measurement Methods

| DPD HACH Method(1) | Detection Range |
|---|---|
| High Range Total Chlorine 10069 | 0.1 to 10.0 mg/L |
| High Range Free Chlorine 10070 | 0.1 to 10.0 mg/L |
| Low Range Total Chlorine 8167 | 0.02 to 2.00 mg/L |
| Low Range Free Chlorine 8021 | 0.02 to 2.00 mg/L |

Notes:
(1) Methods equivalent to USEPA and Standard Method 4500-Cl G for drinking water and wastewater.

All samples were run immediately after collection. A distilled water blank was run before each sample to zero the meter. The sample cells and caps were rinsed three times with distilled water between samples. Prior to taking readings the sample cells were wiped in a downward motion with kimwipes to remove water and/or condensation.

## 3.4 Chlorine Demand

Chlorine demand was evaluated at chlorine doses of 3.0, 5.0, 8.0, and 9.0 mg/L for the three effluent types listed in Section 3.1. Chlorine dosing accuracy was confirmed with distilled water at doses of 6.5 mg/L and 8.0 mg/L. The dilutions used resulted in accurate dosing measurements (within 0.1 mg/L of the target doses). Chlorine solutions were prepared each day of testing in the morning and afternoon depending on the duration of the testing that day.

Figure 23 shows the chlorine demand analysis for the tertiary filter effluent and MBR effluent for a chlorine dose of 8.0 mg/L. The initial chlorine demand of 2 to 3 mg/L occurred quickly within the first minute. After approximately 10 to 15 minutes the total and free chlorine residuals began to stabilize. Note that this exposure time is less than the measured modal contact time at peak flow for the first sample point (SP1) measured during the tracer testing (refer to Table 1). Similar demands were observed for the other chlorine doses evaluated. The exception was for chlorine doses less than 3.0 mg/L where the resulting residuals were near the lower detection limit of the method (0.02 mg/L). All graphs of the chlorine demand analysis are contained in Appendix B.

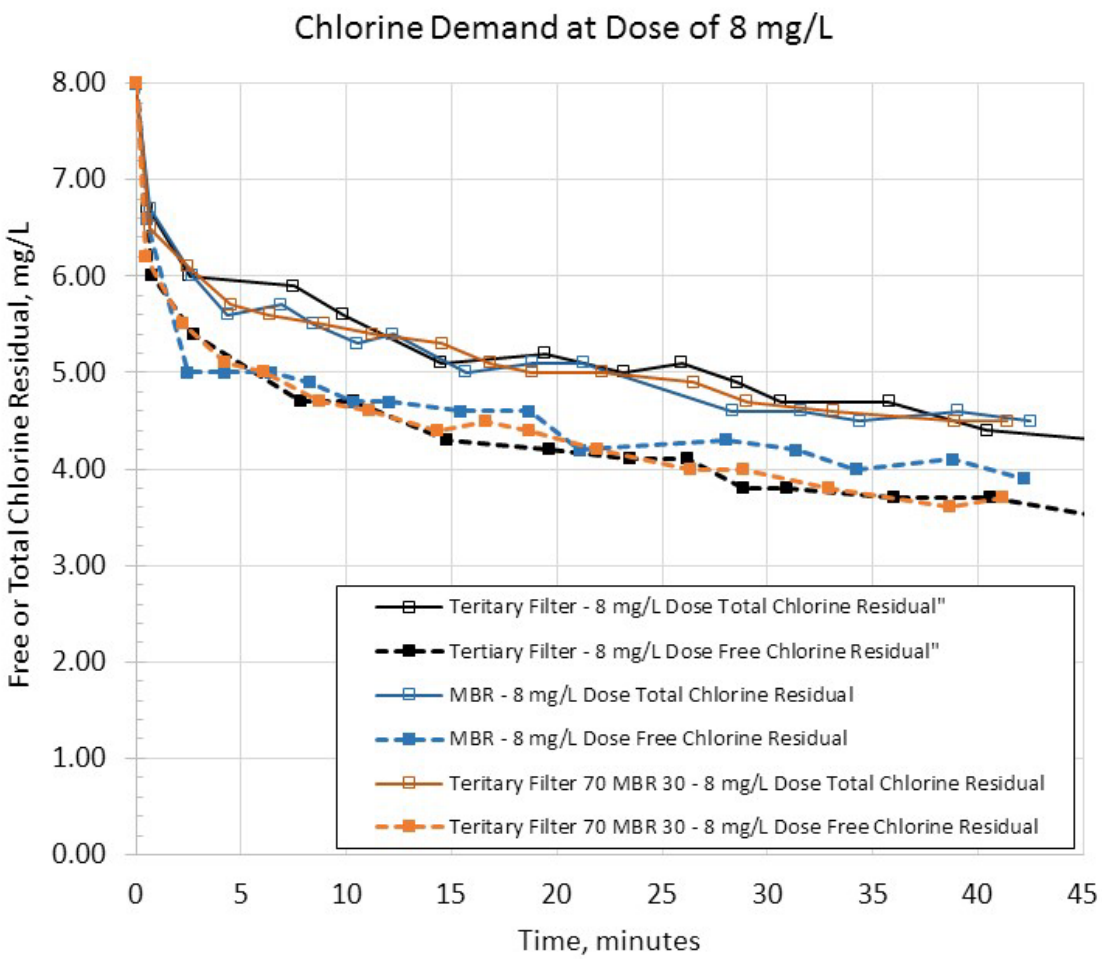



Figure 23 Chlorine Demand Curve for Tertiary Filter and MBR Effluent at a Chlorine Dose of 8 mg/L

## 3.5 Indigenous Total Coliform and Seeded Virus Dose Response

Detailed bench-scale dose response testing was performed in separate runs with each of the three effluent types. Samples were first evaluated for chlorine dose response with indigenous total coliforms and then separate effluent samples were evaluated for chlorine dose response with seeded virus. Male specific bacteriophage (MS2) was used to seed the effluent samples to evaluate virus dose response. Testing was performed separately to eliminate the potential impact to total coliform disinfection dose response due to the injection of large concentrations of MS2.

All containers used for microbial sampling contained quenching agent (sodium thiosulfate) in tablets (total coliform) or solution (MS2) to dechlorinate the samples at the sample time.

### 3.5.1 Indigenous Total Coliform Dose Response

Table 4 contains the bench-scale test matrix for the indigenous total coliform dose response testing. The "X" indicates when a bench-scale run was performed. Total coliforms were enumerated by the multiple tube fermentation method (Standard Method 9221B) which is used for compliance monitoring. This testing was performed with effluent samples collected on

5-14-18. Refer to Table 2 for the water quality of these samples. Background indigenous total coliforms in the effluent samples ranged from 200 to 400 MPN/100mL.

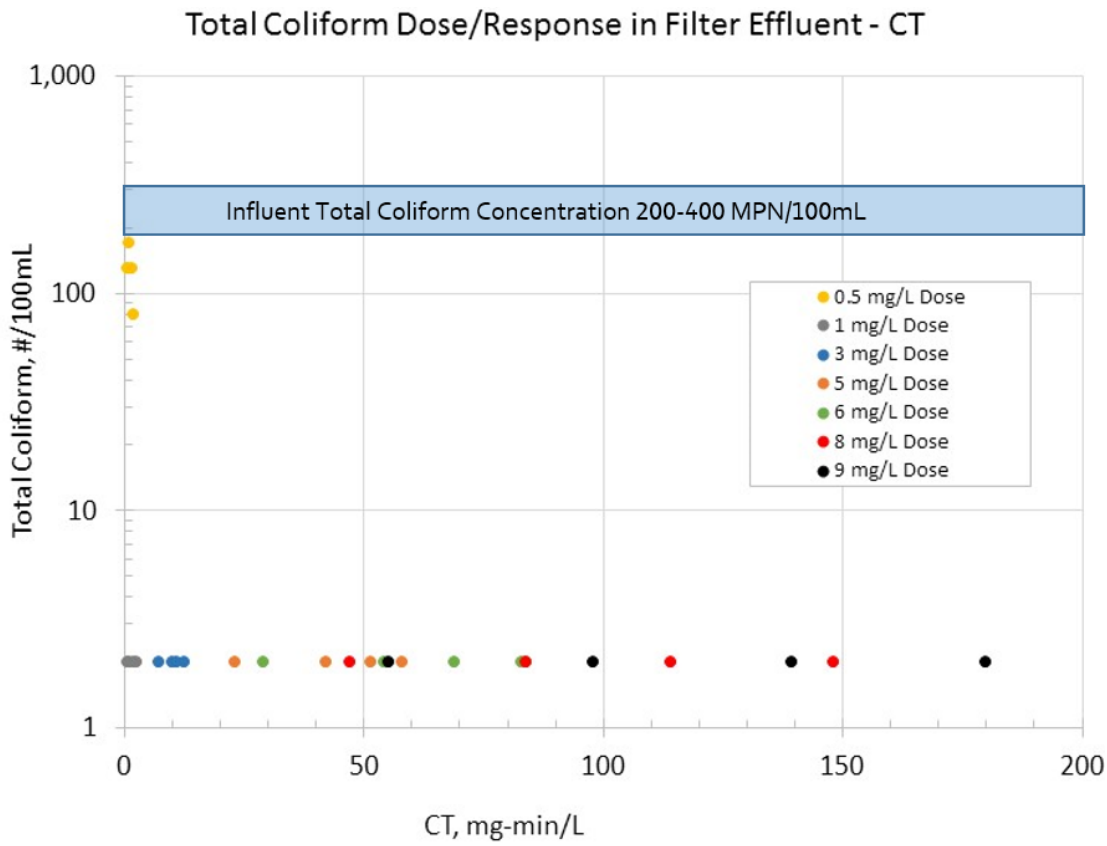
Table 4 Indigenous Total Coliform Chlorine Dose Response Bench-Scale Test Matrix

| Effluent Sample Contact Time | Chlorine Dose, mg/L X = Bench-scale Test Run | | | | | | |
|---|---|---|---|---|---|---|---|
| | 0.5 | 1.0 | 3.0 | 5.0 | 6.0 | 8.0 | 9.0 |
| 100% Tertiary Effluent | | | | | | | |
| 10 mins | X | X | X | X | X | X | X |
| 20 mins | X | X | X | X | X | X | X |
| 30 mins | X | X | X | X | X | X | X |
| 40 mins | X | X | X | X | X | X | X |
| 100% MBR Effluent | | | | | | | |
| 10 mins | X | X | X | X | | X | |
| 20 mins | X | X | X | X | | X | |
| 30 mins | X | X | X | X | | X | |
| 40 mins | X | X | X | X | | X | |
| 70% Tertiary Effluent 30% MBR Effluent | | | | | | | |
| 10 mins | | | X | X | | X | |
| 20 mins | | | X | X | | X | |
| 30 mins | | | X | X | | X | |
| 40 mins | | | X | X | | X | |

Notes:
(1) Total and free chlorine measured at each contact time when total coliform sample was collected.
(2) Containers for total coliform analysis contained quenching agent tablets.

Figure 24 shows the CT results of the indigenous total coliform chlorine dose response testing in the 100 percent Tertiary Filter effluent. CT was calculated as the contact time multiplied by the free chlorine residual at that time. At a chlorine dose of 0.5 mg/L, the free chlorine was consumed and the resulting total coliform concentrations in all samples were greater than the permit compliance target of 2.2 MPN/100mL. All other chlorine doses and contact times sampled resulted in total coliform concentrations less than the target concentration. Target compliance was met at low free chlorine CTs, much less than the current permitted 900 mg-min/L or the Title 22 requirement of 450 mg-min/L. The contact times evaluated (10-40 minutes) were also much lower than the Title 22 requirement of 90 minutes modal. Figure 25 shows the total coliform concentration as a function of the free chlorine residual. Target compliance was met at very low free chlorine residuals.

The matching figures for the MBR effluent and the Tertiary Filter/MBR mix are not included in the body of the report because under all conditions tested (Table 4) the total coliform compliance was met. These results indicated that total coliform compliance could be met at CTs less than 20 mg-min/L at contact times as low as 10 minutes at the RWQCP. Note that this is not a conclusion for permitting or operation, just a conclusion based upon the information presented to this point in the report.




CT = Free Chlorine Residual x Contact Time

Figure 24 Indigenous Total Coliform Bench-Scale Free Chlorine CT Results Filter Effluent

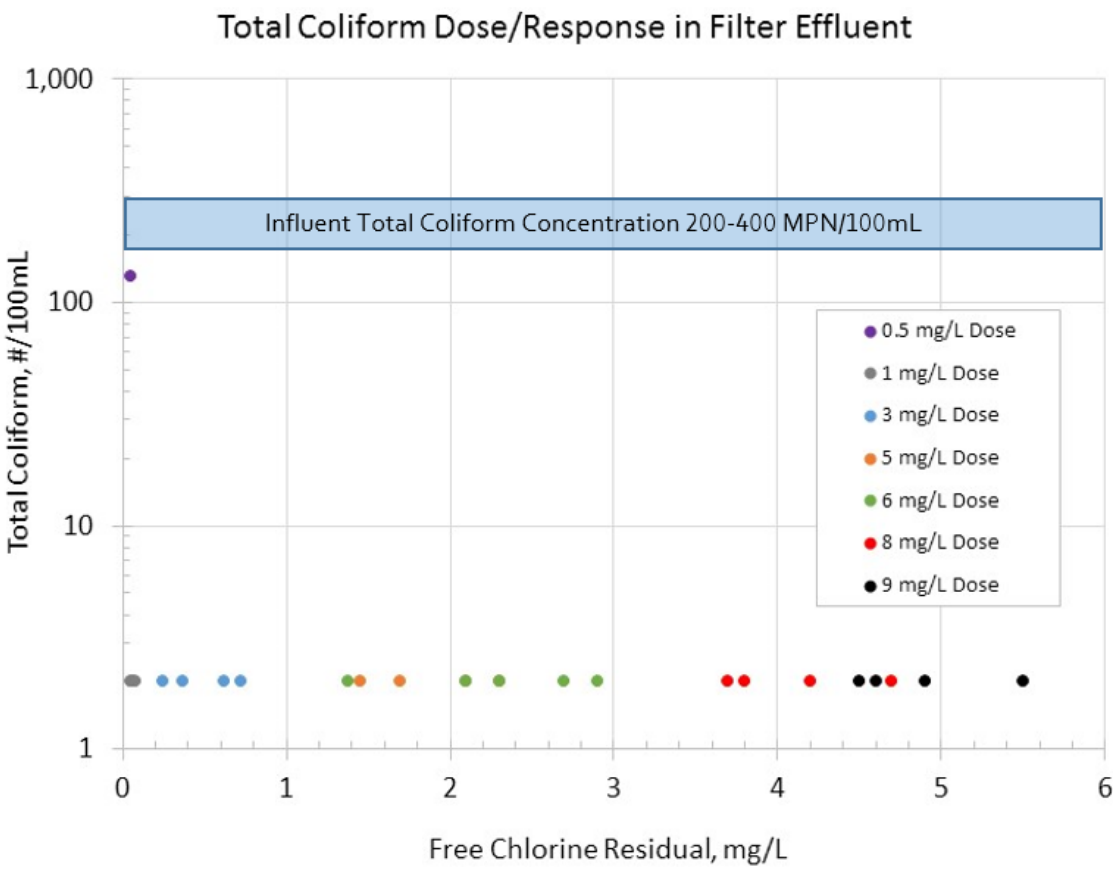



Figure 25 Indigenous Total Colifrm Bench-Scale Free Chlorine Residual Results Filter Effluent

### 3.5.2 Seeded Virus Dose Response

MS2 was seeded into the filtered effluent samples for dose response testing. MS2 was seeded to achieve a pre-disinfection concentration of 7-log to allow accurate enumeration of bacteriophage to demonstrate 5-log inactivation.

The addition of the MS2 solution resulted in an additional chlorine demand of approximately 1 to 2 mg/L. The MS2 is propagated in a tryptic soy broth that is the source of the chlorine demand. The smallest volume of MS2 solution that still allowed accurate enumeration was used for the analysis to minimize the impact on chlorine demand. Total organic carbon (TOC) samples were collected prior to and after adding MS2 to all three effluent types and it was found that the TOC increased by 2.0 to 5.0 mg/L just from the addition of the MS2 solution. Clearly, the added TOC exerts a chlorine demand.

Table 5 contains the bench-scale test matrix for the seeded virus dose response testing. The "X" indicates when a bench-scale run was performed. MS2 was enumerated by the double agar overlay method. This testing was performed initially with effluent samples collected on 5-14-18 at lower chlorine doses and then with effluent samples collected on 7-2-18 at higher chlorine doses. Refer to Table 2 for the water quality of these samples. The first round of virus testing was performed at lower chlorine doses that mimicked the total coliform dose response results. The second round of testing was performed to confirm the results of two chlorine doses (5 and 8 mg/L) and evaluate other high doses.

Table 5 Seeded Virus Chlorine Dose Response Bench-Scale Test Matrix

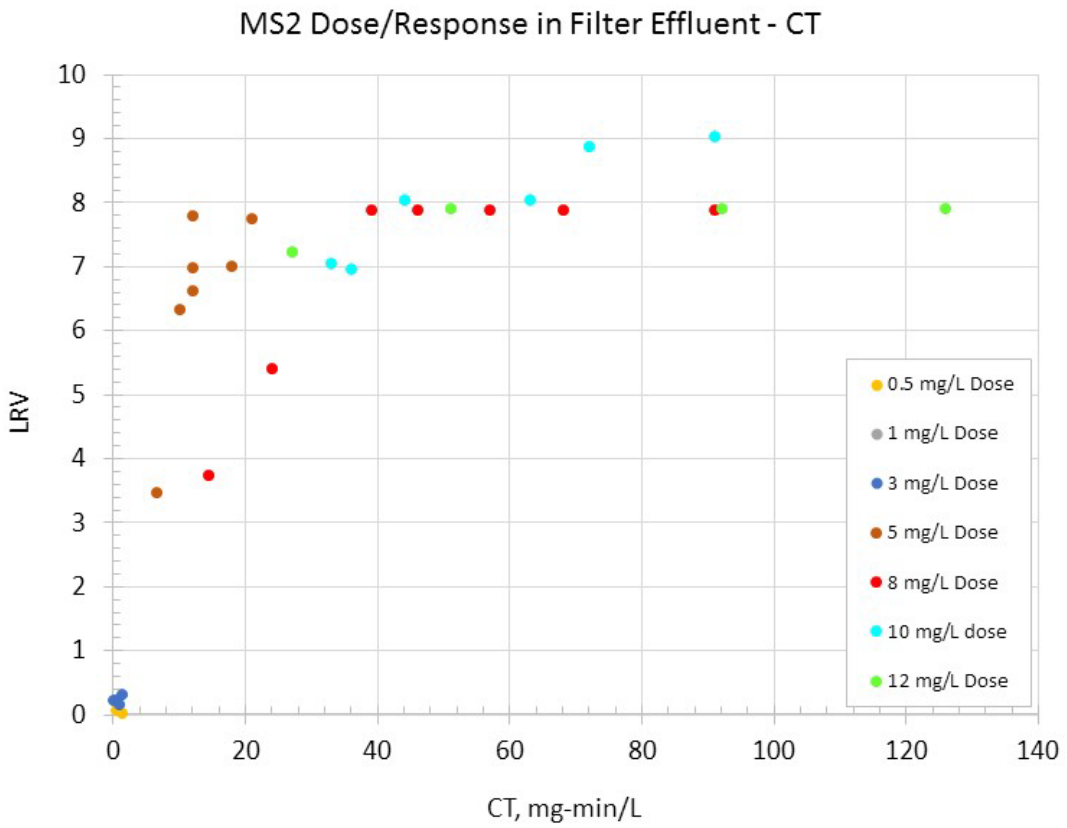
| Effluent Sample Contact Time | Chlorine Dose, mg/L X = Bench-scale Test Run | | | | | | |
|---|---|---|---|---|---|---|---|
| | 0.5 | 1.0 | 3.0 | 5.0 | 8.0 | 10.0 | 12.0 |
| 100% Tertiary Effluent | | | | | | | |
| 5 mins | X | X | X | X | X | X | X |
| 10 mins | X | X | X | X | X | X | X |
| 15 mins | X | X | X | X | X | X | X |
| 20 mins | X | X | X | X | X | X | X |
| 30 mins | X | X | X | X | X | X | X |
| 40 mins | | | | X | X | X | X |
| 70 mins | | | | X | X | X | X |
| 100% MBR Effluent | | | | | | | |
| 5 mins | X | X | X | X | X | | |
| 10 mins | X | X | X | X | X | | |
| 15 mins | X | X | X | X | X | | |
| 20 mins | X | X | X | X | X | | |
| 30 mins | X | | | | X | | |
| 40 mins | | | | | X | | |
| 70 mins | | | | | | | |
| 70% Tertiary Effluent 30% MBR Effluent | | | | | | | |
| 5 mins | | | X | X | X | | |
| 10 mins | | | X | X | X | | |
| 15 mins | | | X | X | X | | |
| 20 mins | | | X | X | X | | |
| 30 mins | | | | | X | | |
| 40 mins | | | | | X | | |
| 70 mins | | | | | X | | |

Notes:
(1) Total and free chlorine measured at each contact time when MS2 sample was collected.
(2) Containers for MS2 analysis contained quenching agent solution.

Figure 26 shows the CT results of the seeded virus chlorine dose response testing in the Tertiary Filter effluent. CT was calculated for a sample as the contact time multiplied by the free chlorine residual at that time. At chlorine doses of 3.0 mg/L or less the free chlorine was consumed and the resulting MS2 log reduction in all samples was less than 0.3-log. All other chlorine doses and contact times sampled resulted in MS2 log reductions between 3.5-log and 9-log. Virus inactivation of 5-logs was reliably met at CTs greater than 24 mg-min/L.

Similar to the total coliform dose response testing, virus inactivation requirements were met at low free chlorine CTs, much less than the current permitted 900 mg-min/L or the Title 22 requirement of 450 mg-min/L. The contact times evaluated (10-70 minutes) were also much lower than the Title 22 requirement of 90 minutes modal. Figure 27 shows the virus log reduction as a function of the free chlorine residual. The virus log reduction requirement was met at low free chlorine residuals. It is important to note that the tests that resulted in less than 5-log

inactivation reflect a 5 minute contact time. Such a low contact time would not occur at full-scale.




CT = Free Chlorine Residual x Contact Time

Figure 26 Seeded Virus Bench-Scale Free Chlorine CT Results Filter Effluent

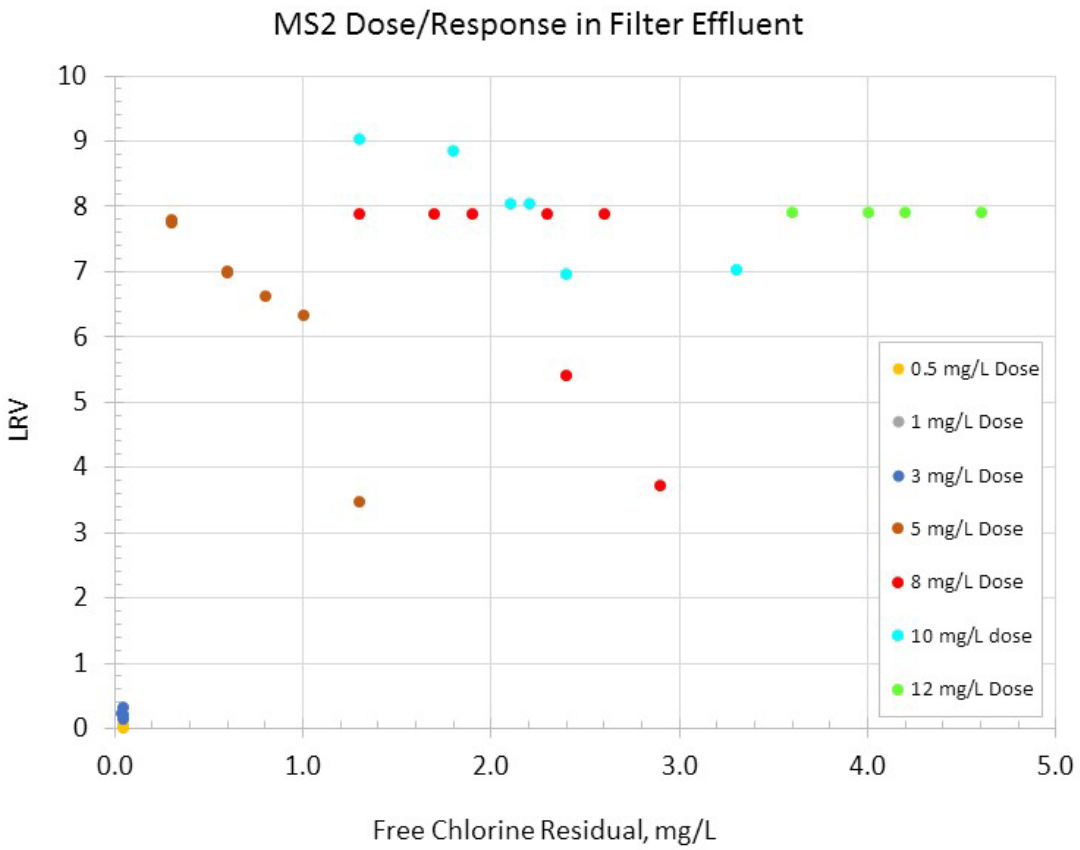



Figure 27 Seeded Virus Bench-Scale Free Chlorine Residual Results Filter Effluent

The matching figures for the MBR effluent and the Tertiary Filter/MBR mix are not included in the body of the report because the virus log reduction requirement was similar to the Tertiary Filter effluent results. The 5-log virus reduction in the MBR effluent was met at a CT of 26 mg-min/L (15 minute contact time). The virus log reduction in the Tertiary Filter effluent and MBR effluent mixture (70/30) exceeded 8-log reduction at a CT of 27 mg-min/L (10 minute contact time). The Tertiary Filter Effluent was more challenging to disinfect at low chlorine doses but still resulted in compliance at very low CTs. These results indicated that virus compliance could be met at a CT of 30 mg-min/L at contact times between 10 to 15 minutes. Note that this is not a conclusion for permitting or operation, just a conclusion based upon the information presented to this point in the report.

## 3.6 Full-scale Low CT Testing

Full-scale testing was performed at the RWQCP on 8-1-18 to confirm that a low CT could meet total coliform compliance at full-scale. The City received approval from the RWQCB for a single day of testing at a CT less than the permitted 900 mg-min/L. During the full-scale testing, the chlorine dose was reduced to three target free chlorine residuals and samples were collected at the six sample points sampled during tracer testing. The target free chlorine residuals at SP1 were 1.5, 3.0, and 5.0 mg/L. These were selected based on the previous bench-scale testing and the modal contact times determined during the tracer testing so that a range of reduced CTs could be evaluated at the six sample locations within CCB2.

The testing was conducted at peak flow (46 mgd) to match the tracer testing modal contact times determined for the six sample points. To achieve the peak flow the CCB influent was a mixture of MBR and Tertiary Filter effluent (32 mgd MBR effluent/14 mgd filter effluent). During the full-scale testing the average flow was 46 mgd (range 44 to 49 mgd). This flow variation was similar to that experienced during the tracer testing where the average flow was 47 mgd and the range was 44 to 50 mgd. This indicates that the measured modal contact times from the tracer testing at peak flow can be used for the analysis of data collected during the full-scale low CT testing since the samples were collected at the same sample points.

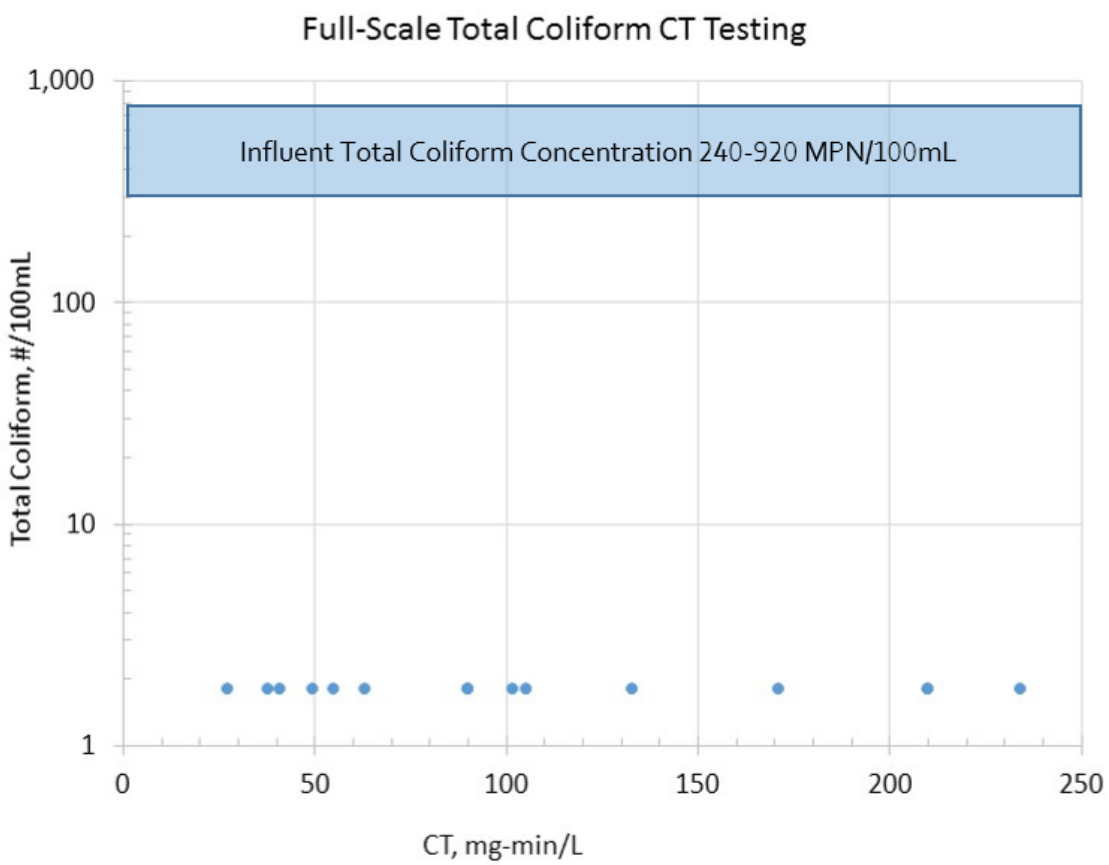
Table 6 contains the water quality characteristics for the CCB2 influent on 8-1-18. The water quality from the full-scale testing was similar to the water quality during the bench-scale effluent testing, indicating that a significant change in chlorine demand would not be anticipated during the full-scale low CT testing. The average chlorine demand from the dosing point to the first chlorine analyzer between SP1 and SP2 during the full-scale testing was approximately 4 mg/L total chlorine.

Figure 28 shows the total coliform concentration as a function of CT during the full-scale testing. The total coliform concentration for all samples collected throughout the CCB at all sample points was <1.8 MPN/100mL during the full-scale testing. The average influent total coliform concentration was 310 MPN/mL (range 240 to 920 MPN/100mL). The lowest free chlorine CT tested was 27 mg-min/L. The sample points sampled during the full-scale testing matched the tracer testing sample locations (Figure 2). The results in Figure 28 include the results at all six sample points.

Table 6 RWQCP CCB2 Influent Water Quality Characteristics During Full-Scale Testing

| Parameter | 8-1-18<br>10:50 am | 8-1-18<br>12:25 pm | 8-1-18<br>1:50 pm |
|---|---|---|---|
| Ammonia, mg/L(1) | ND | ND | 0.19 |
| Nitrite-N, mg/L(2) | ND | ND | ND |
| Nitrate-N, mg/L | 4.4 | 4.6 | 4.6 |
| TOC, mg/L | 6.0 | 6.3 | 5.9 |

Notes:
(1) Ammonia method detection limit is 0.24 mg/L.
(2) Nitrite method detection limit is 0.016 mg/L.




CT = Free Chlorine Residual x Contact Time

Figure 28 Full-scale Total Coliform CT Testing Results

Section 4

# CT TESTING CONCLUSIONS AND RECOMMENDATIONS

In an effort to reduce DBPs formed during chlorination, and to improve overall operational efficiency at the RWQCP, the City desires to modify their existing chlorination operations, which would require associated "Title 22" permit modifications and documentation.

The goal of the contact time testing and CT demonstration is to substantially drop the contact time requirements and CT requirements to below the currently mandated value of 90-minute modal and 900 mg-min/L values, respectively. This could be achieved by moving the compliance point up in CCB2 and reducing the CT to reasonably low values documented in this report. To achieve this goal the following was conducted:

- Tracer testing to determine modal contact times in Chlorine Contact Basin 2 (CCB2)
- Bench-scale testing to determine
  - Chlorine demand
  - Seeded virus and indigenous coliform chlorine disinfection dose/response with free chlorine
  - Lowest free chlorine disinfection CT that will meet both Title 22 virus requirements (5-log reduction) and achieve bacterial permit compliance with a degree of reliability
- Full-scale testing to demonstrate the reduced free chlorine CT provides disinfection to meet total coliform permit requirements

The following preliminary conclusions from the testing include:

- Tracer testing determined modal contact times for six points in CCB2 at 46 mgd.
- 2.2 MPN/100mL total coliform requirements were met at a minimum free chlorine CT of less than 20 mg-min/L and minimum contact time of 10 minutes at bench-scale.
- 5-log reduction virus requirements were met at a minimum free chlorine CT of 30 mg-min/L and minimum contact time of 15 minutes at bench-scale.
- 2.2 MPN/100mL total coliform requirements were met at a minimum free chlorine CT of 27 mg-min/L and minimum contact time of 17 minutes at full-scale.
- The results of the tracer testing and full-scale reduced CT testing represent real world conditions at the RWQCP with an MBR and a conventional activated sludge treatment plant in operation.

## 4.1 Recommendation

Based on the results presented in this report, it is recommended that CCB2 be operated at a free chlorine CT of 30 mg-min/L with compliance sampling at SP2 to meet virus and total coliform requirements.

At peak flow (46 mgd) SP2 had a measured modal contact time of 29 minutes. Note that at flows lower than 46 mgd the modal contact time at SP2 would increase thereby increasing the free chlorine CT. Operation would be based on maintaining a free chlorine residual of 1.0 mg/L at SP2.

Figure 29 shows a schematic of CCB2 and the recommended location of the new compliance point at SP2.

## 4.2 Implementation

The CCB2 currently has two chlorine residual analyzers that measure total chlorine. The first analyzer is located between SP1 and SP2 and the second analyzer is located at SP6. SP6 is the current compliance sampling point. An unused chlorine analyzer from CCB3 will be moved to the location between SP1 and SP2 for free chlorine measurements. This will allow Public Works to monitor both free and total combined chlorine. The free chlorine analyzer between SP1 and SP2 would be used to monitor compliance free chlorine residual.

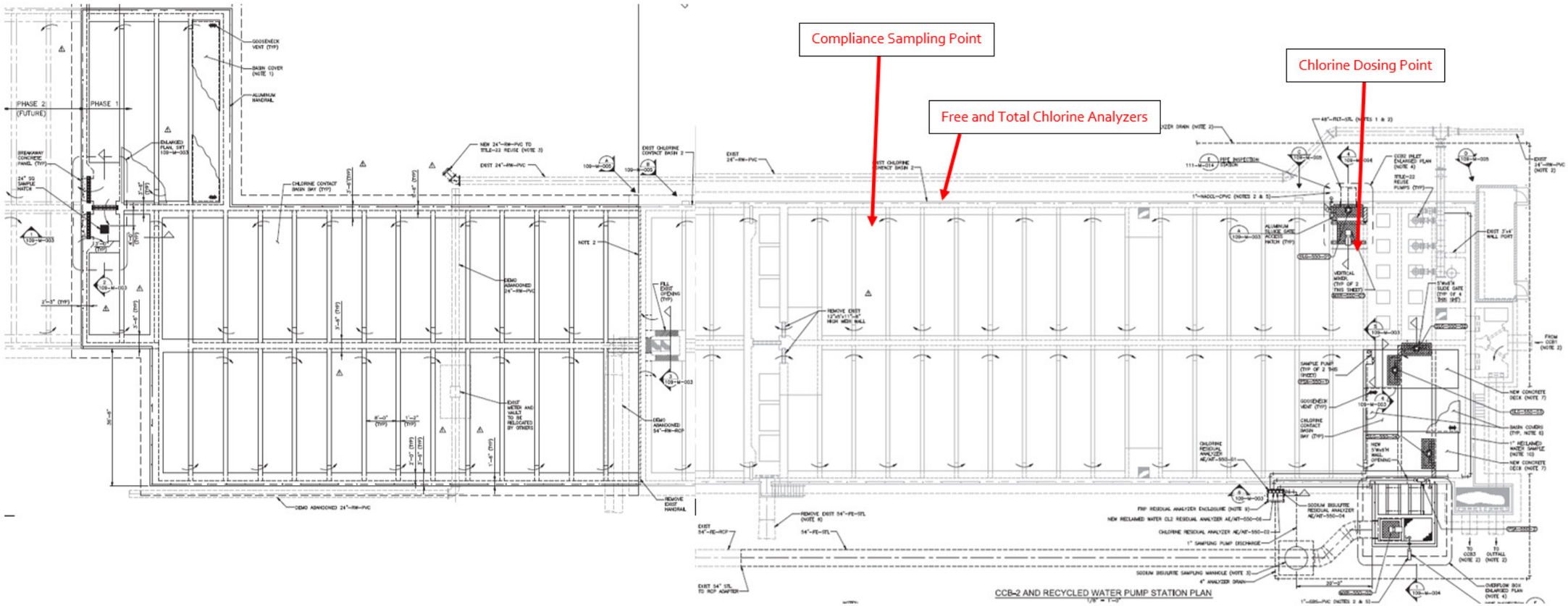



Figure 29 Recommended CCB2 New Compliance Location

Section 5

# SIMILAR LOW CT PROJECTS

Recent work in Los Angeles, Sacramento, Australia, and San Francisco clearly document the ability to drop CT values to as low as 15 mg-min/L and drop contact times down to as low as 10 minutes. Per Carollo discussions with the DDW, limited testing is necessary to gain the lower CT approval, as long as free chlorination is maintained at all times. Information on these successful projects is provided below for perspective and future comparison.

## 5.1 Los Angeles County Sanitation Districts

LACSD was the first utility in California to pursue reduced CT and contact time requirements with the DDW for disinfection of filtered secondary effluent. Their work demonstrated reliable 5-log disinfection of virus at a CT of 9 mg-min/L, with a minimum free chlorine residual of 1 mg/L at the end of the contact basin, and with a minimum modal contact time of 4 minutes (Huitric et al., 2013, 2014). LACSD was granted approval for the reduced CT values from the DDW in August of 2013 (CDPH, 2013), with approval of the aforementioned CT, residual, and contact time values.

## 5.2 Sacramento Regional County Sanitation District

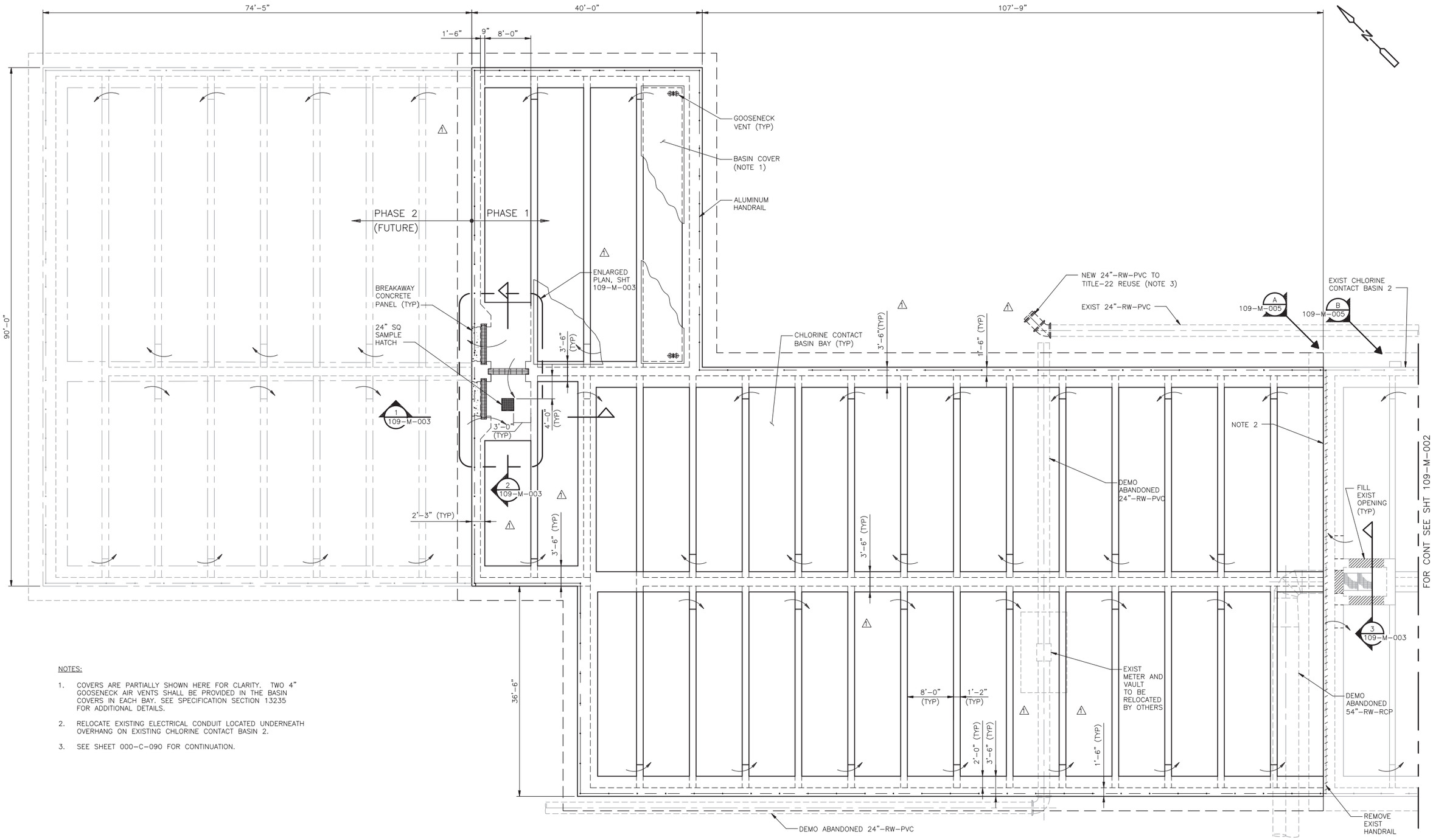
Similar to LACSD, the Sacramento Regional County Sanitation District (Regional San) performed extensive bench and pilot-scale analysis of free chlorine virus kill on filtered secondary effluent. That work demonstrated 5-log virus kill at CT values of 16 mg-min/L and higher, at contact times of 11 and 24 minutes. Because reliably attaining 2.2 MPN/100 mL required CT values of at least 100 mg-min/L, Regional San made the conservative decision to design their new 217 mgd net filtered effluent system at a CT of 162.5 mg-min/L with a minimum modal contact time of 32.5 minutes, which was permitted by the DDW on October 12, 2015.

## 5.3 Australia

The Australian Smart Water Fund with Keegan et al. (2012) and WaterSecure (2017) have now extensively documented low CT performance for pathogen kill in recycled water, matching the results from LACSD and Sacramento that are described above. Those results clearly document 5-log virus kill at CT values of 15 mg-min/L and above. In discussions with the DDW, in particular between Andrew Salveson of Carollo and Brian Bernados of the DDW, the DDW has indicated their willingness to approve low CT projects in California based upon limited testing and the guarantee of free chlorination.

Appendix A

# CCB2 DRAWINGS




NOTES:

1. COVERS ARE PARTIALLY SHOWN HERE FOR CLARITY. TWO 4" GOOSENECK AIR VENTS SHALL BE PROVIDED IN THE BASIN COVERS IN EACH BAY. SEE SPECIFICATION SECTION 13235 FOR ADDITIONAL DETAILS.
2. RELOCATE EXISTING ELECTRICAL CONDUIT LOCATED UNDERNEATH OVERHANG ON EXISTING CHLORINE CONTACT BASIN 2.
3. SEE SHEET 000-C-090 FOR CONTINUATION.

## CCB-2 EXPANSION PLAN

1/8" = 1'-0"

BLACK & VEATCH Building a world of difference.

LINE IS 2 INCHES AT FULL SIZE (IF NOT 2"-SCALE ACCORDINGLY)

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|
| 1 | 8/1/12 | CKM | JHC | CONFORMED (ADD # 4) |

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

DESIGNED BY: A. BAUMGARDNER
DRAWN BY: F. CARAVEO
SHEET CHK'D BY: D. GAY
CROSS CHK'D BY: B. HEMKEN
APPROVED BY: J. CLARK
DATE: SEPTEMBER 2011

REGISTERED PROFESSIONAL ENGINEER JAMES HOWARD CLARK C31200 Exp. 9/30/12 CIVIL STATE OF CALIFORNIA

CDM In Association with B&V and WEBB ASSOCIATES

Camp Dresser & McKee Inc.
9220 Cleveland Avenue, Suite 100
Rancho Cucamonga, CA 91730
Phone: 909-980-5185
consulting • engineering • construction • operations

CITY OF RIVERSIDE

Department of Public Works
Riverside Regional Water Quality Control Plant
Phase I Plant Expansion

CONFORMED DOCUMENT AUGUST 2012
THE ADDENDA ISSUED DURING THE BIDDING PERIOD ARE UNIFIED IN THIS DOCUMENT. IF DISCREPANCIES BETWEEN THE UNIFIED SET AND THE BIDDING SET INCLUDING THE ADDENDA ARE FOUND, THE BIDDING SET WITH THE ORIGINAL ADDENDA SHALL GOVERN.

CCB-2 AND RECYCLED WATER PUMP STATION
PLAN 1 OF 2

T-46
PROJECT NO. 2012-73196
FILE NAME: M001CCPL.dwg
DRAWING NO. 109-M-001
SHEET NO. 646 OF 1476

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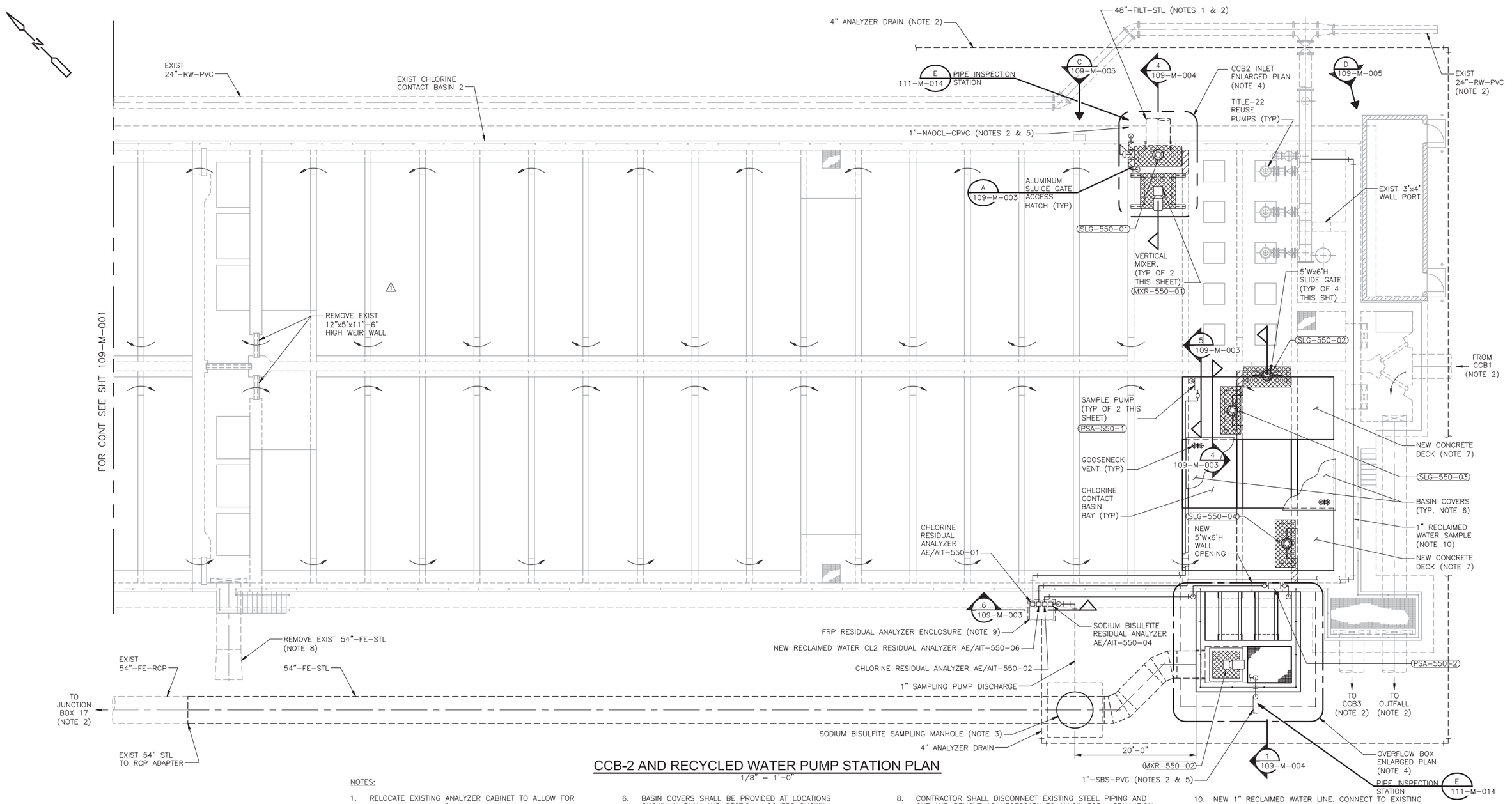


## CCB-2 AND RECYCLED WATER PUMP STATION PLAN

1/8" = 1'-0"

NOTES:

1. RELOCATE EXISTING ANALYZER CABINET TO ALLOW FOR INSTALLATION OF 48"-FILT-STL PIPE AT NEW INLET FOR CCB2.
2. SEE YARD PIPING SHEET 000-C-091 FOR CONTINUATION.
3. SEE SHEET 111-M-010 FOR DETAILS.
4. SEE SHEET 109-M-004 FOR DETAILS.
5. BURIED CHEMICAL FEED PIPING SHALL BE DOUBLE-WALL PIPING PER SPECIFICATION SECTION 02626.
6. BASIN COVERS SHALL BE PROVIDED AT LOCATIONS SHOWN IN PLAN AND SECTION. COVERS SHOWN PARTIALLY HERE FOR CLARITY. TWO 4" GOOSENECK AIR VENTS SHALL BE PROVIDED IN THE BASIN COVERS IN EACH BAY.
7. EXISTING COVER SYSTEM NOT SHOWN. EXISTING COVER SYSTEM SHALL BE REMOVED AS NECESSARY TO ALLOW FOR INSTALLATION OF NEW EQUIPMENT. EXISTING EDGE OF COVER SYSTEM SHALL BE RELOCATED TO FORM NEW EDGE AFTER INSTALLATION OF NEW EQUIPMENT AND NEW CONCRETE DECK.
8. CONTRACTOR SHALL DISCONNECT EXISTING STEEL PIPING AND CUT AND REMOVE AS NECESSARY TO ALLOW FOR INSTALLATION OF NEW 54"-FE-STL PIPING. REMAINDER OF EXISTING STEEL PIPING SHALL BE ABANDONED IN PLACE.
9. 10' WIDE x 8' DEEP x 7' HIGH FRP PREFABRICATED ENCLOSURE BY GLOBAL TREAT, INC. TO HOUSE RESIDUAL ANALYZERS. PROVIDE WITH VENTILATION FAN, LIGHTING, AND DOOR CONFIGURATION TO MATCH EXISTING CITY ENCLOSURE. CONTRACTOR TO MOUNT/ SUPPORT RESIDUAL ANALYZERS AND REAGENT BOTTLES, ETC. ON BACK WALL.
10. NEW 1" RECLAIMED WATER LINE. CONNECT TO EXISTING RECLAIMED WATER HEADER TAP. FIELD-VERIFY EXISTING TAP LOCATION. 1
11. REPLACE EXISTING CCB-2 COVERS WITH COVERS SPECIFIED FOR CCB-2 EXPANSSION. SEE NOTE 1 ON SHEET 109-M-002. DESIGN COORDINATION DETAILS TO BE DEVELOPED AND SUBMITTED BY CONTRACTOR THROUGH THE SUBMITTAL PROCESS.
12. REMOVE EXISTING CCB-2 HAND RAILS EXCEPT FOR PERIMITER HANDRAILS REQUIRED FOR FALL PROTECTION FROM THE STRUCTURE.

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LINE IS 2 INCHES AT FULL SIZE (IF NOT 2"-SCALE ACCORDINGLY)

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|
| 1 | 8/1/12 | CKM | JHC | CONFORMED (ADD # 4) |

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

DESIGNED BY: A. BAUMGARDNER
DRAWN BY: F. CARAVEO
SHEET CHK'D BY: D. GAY
CROSS CHK'D BY: B. HEMKEN
APPROVED BY: J. CLARK
DATE: SEPTEMBER 2011

REGISTERED PROFESSIONAL ENGINEER JAMES HOWARD CLARK C31200 Exp. 9/30/12 CIVIL STATE OF CALIFORNIA

CDM In Association with B&V and WEBB & ASSOCIATES
Camp Dresser & McKee Inc.
9220 Cleveland Avenue, Suite 100
Rancho Cucamonga, CA 91730
Phone: 909-980-5185
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Department of Public Works
Riverside Regional Water Quality Control Plant
Phase I Plant Expansion

CONFORMED DOCUMENT AUGUST 2012
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CCB-2 AND RECYCLED WATER PUMP STATION
PLAN 2 OF 2

T-46

PROJECT NO. 2012-73196
FILE NAME: M002CCPL.dwg
DRAWING NO. 109-M-002
SHEET NO. 647 OF 1476

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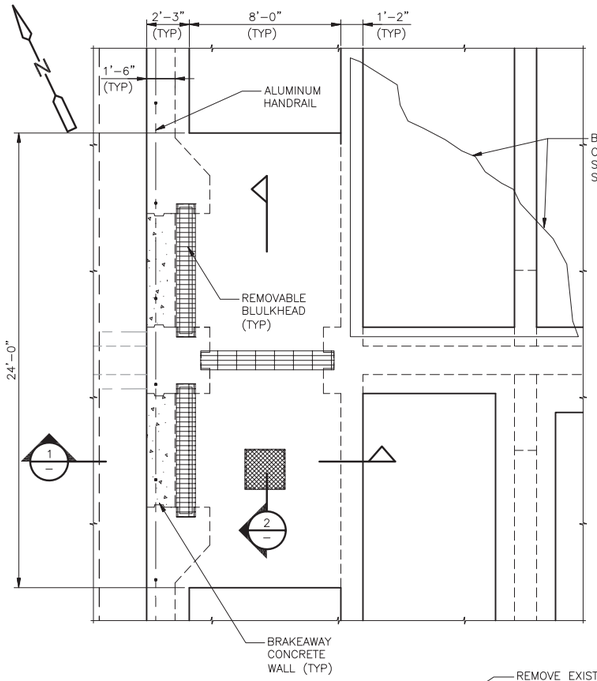
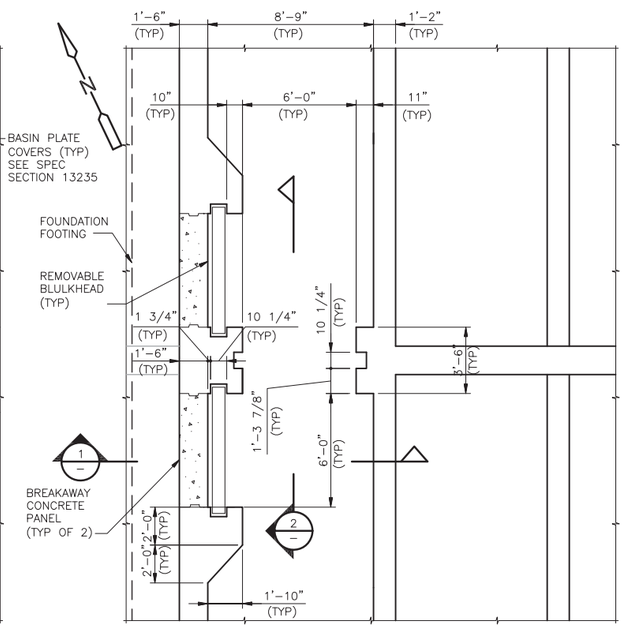
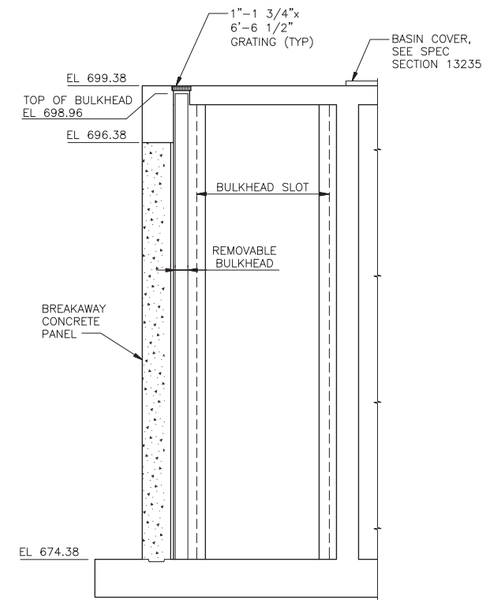


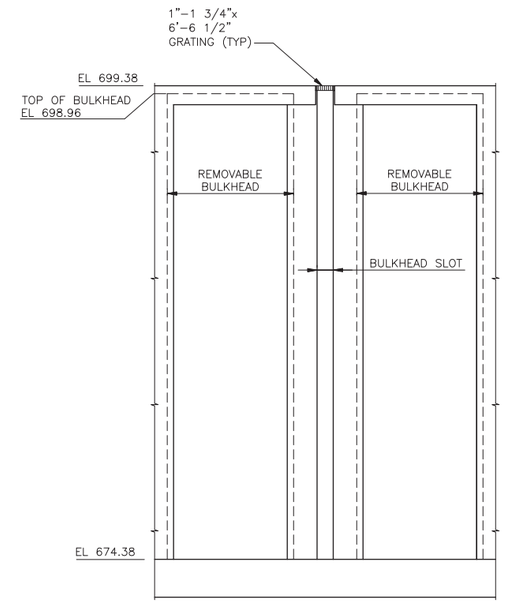

## ENLARGED TOP PLAN
1/4" = 1'-0"



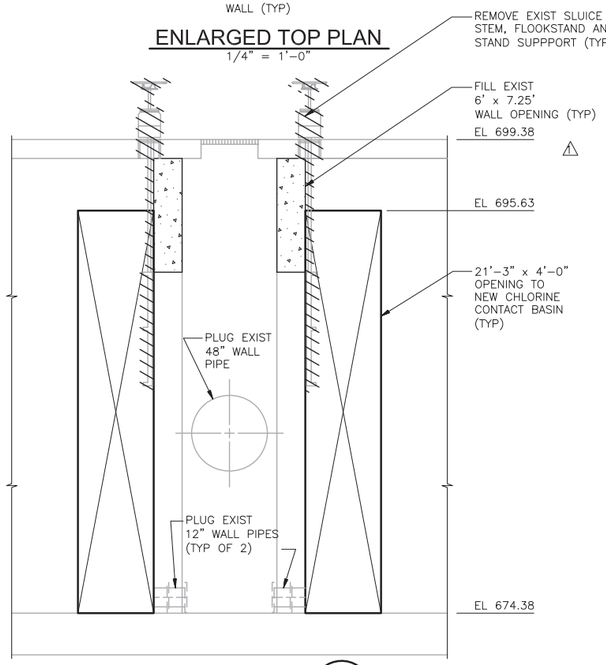

## ENLARGED BOTTOM PLAN
1/4" = 1'-0"



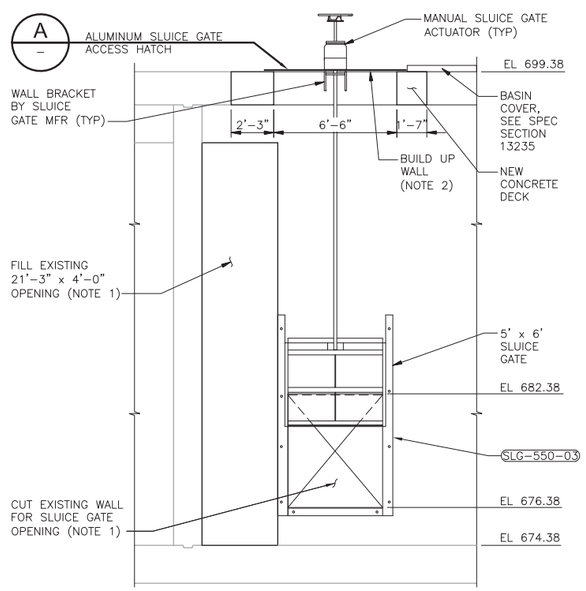

## SECTION 1
1/4" = 1'-0"



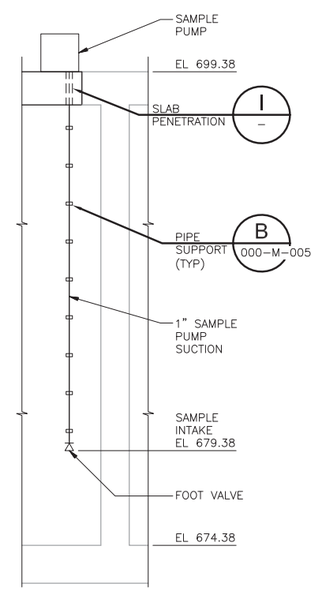

## SECTION 2
1/4" = 1'-0"



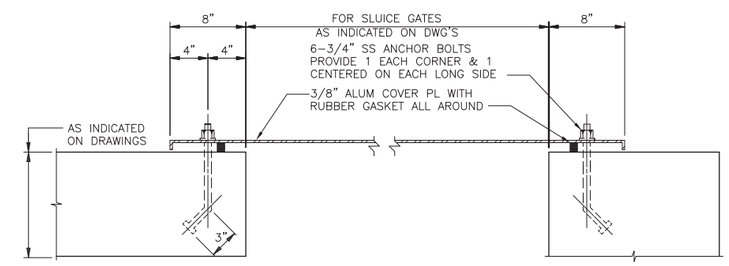

## SECTION 3
1/4" = 1'-0" 109-M-001



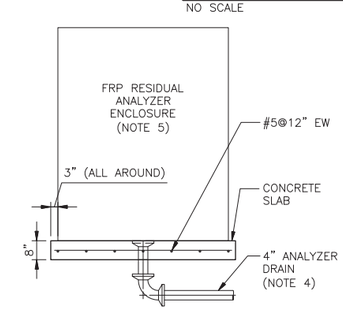

## SECTION 4
1/4" = 1'-0" 109-M-002




## SECTION 5
1/4" = 1'-0" 109-M-002




## SLUICE GATE ACCESS HATCH
NO SCALE




## SECTION 6
NO SCALE 109-M-002

NOTES:

1. FILL EXISTING 21'-3"x4'-0" WALL OPENING AS SHOWN. CUT EXISTING WALL AS SHOWN FOR NEW 5'-0" X 6'-0"OPENING FOR SLUICE GATE. SEE STRUCTURAL DRAWINGS FOR DETAILS.
2. BUILD UP CONCRETE WALL TO PERMIT INSTALLATION OF NEW MANUAL SLUICE GATE ACTUATOR AND NEW BASIN COVERS. TOP OF NEW WALL EXTENSION SHALL MATCH TOP OF EXISTING WALKWAY.
3. INSTALLATION OF SLUICE GATE SLG-550-04 SIMILAR.
4. ANALYZER DRAIN SHALL HAVE 1% SLOPE FROM ANALYZER ENCLOSURE TO MANHOLE ON 000-C-091.
5. FRP PREFABRICATED ENCLOSURE TO HOUSE RESIDUAL ANALYZERS. PROVIDE WITH VENTILATION FAN AND LIGHTING. MATCH TO EXISTING CITY ENCLOSURE MANUFACTURER, SIZE AND DOOR CONFIGURATION.

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| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|
| 1 | 8/1/12 | CKM | JHC | CONFORMED (ADD # 4) |

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

DESIGNED BY: J. CORBIN
DRAWN BY: F. CARAVEO
SHEET CHK'D BY: D. GAY
CROSS CHK'D BY: B. HEMKEN
APPROVED BY: J. CLARK
DATE: SEPTEMBER 2011

REGISTERED PROFESSIONAL ENGINEER JAMES HOWARD CLARK C31200 Exp. 9/30/12 CIVIL STATE OF CALIFORNIA

CDM In Association with B&V and WEBB

Camp Dresser & McKee Inc.
9220 Cleveland Avenue, Suite 100
Rancho Cucamonga, CA 91730
Phone:909-980-5185
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Department of Public Works
Riverside Regional Water Quality Control Plant
Phase I Plant Expansion

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CHLORINE CONTACT BASIN 2

ENLARGED PLANS, SECTIONS AND DETAILS

T-46
PROJECT NO. 2012-73196
FILE NAME: M001CCSC.dwg
DRAWING NO. 109-M-003
SHEET NO. 648 OF 1476

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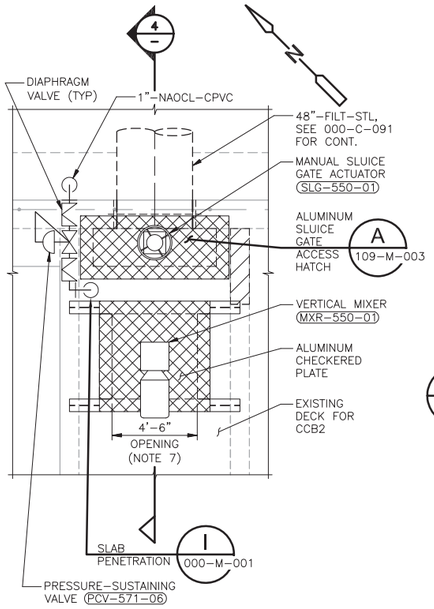
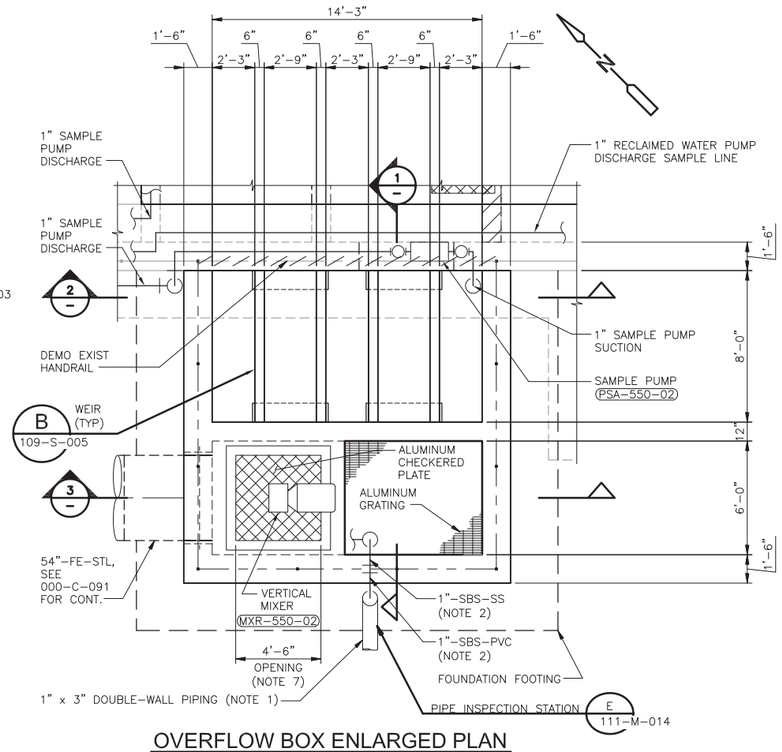


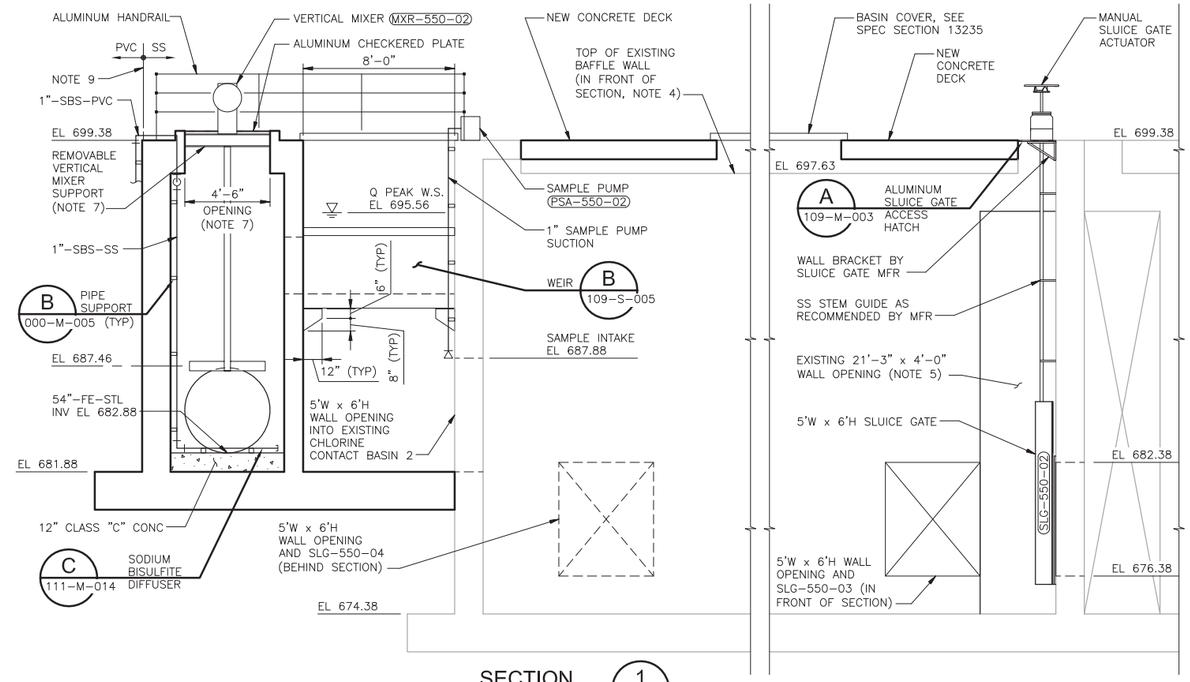

## CCB2 INFLUENT MIXER ENLARGED TOP PLAN

1/4" = 1'-0"




## OVERFLOW BOX ENLARGED PLAN

1/4" = 1'-0"

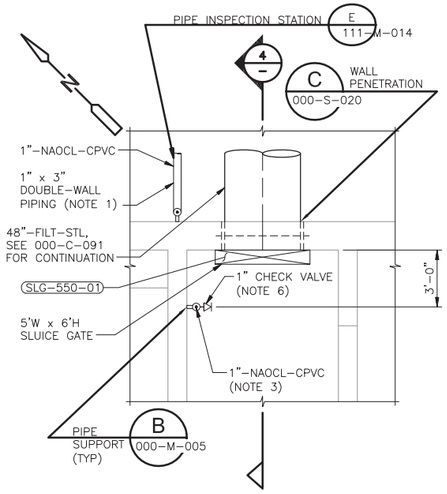



## SECTION 1

1/4" = 1'-0" 109-M-002

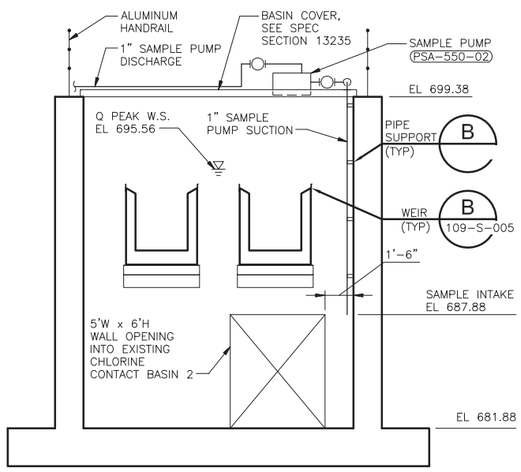
NOTES:

1. BURIED CHEMICAL FEED PIPING SHALL BE DOUBLE-WALL PIPING PER SPECIFICATION SECTION 02626.
2. SODIUM BISULFITE FEED PIPING SHALL BE S.S. FROM THE TOP OF CCB2 WALL TO THE END OF THE DIFFUSER IN THE MIXING CHAMBER.
3. SODIUM HYPOCHLORITE PIPING WITHIN CCB2 SHALL BE SCHEDULE 120 CPVC. PIPE SHALL BE SUPPORTED ON WALL OF CCB2 INLET CHANNEL AND TERMINATE WITH AN ELBOW TOWARDS THE CENTER AT THE ELEVATION SHOWN IN SECTION 4 OF THIS SHEET. PIPE SUPPORTS SHALL BE SPACED EVERY 2'-0" CENTER TO CENTER.
4. RAISE TOP OF EXIST BAFFLE WALL TO MATCH TOP OF CCB2 DECK TO ALLOW FOR INSTALLATION OF NEW CONCRETE DECK AND BASIN COVERS. SEE STRUCTURAL DRAWINGS FOR DETAILS.
5. FILL EXISTING 21'-3" x 4'-0" WALL OPENING AS SHOWN. SEE SECTION 4 ON 109-M-003 AND STRUCTURAL DRAWINGS FOR DETAILS.
6. CONTRACTOR SHALL PROVIDE 1" TIDEFLEX SERIES 35 CHECK VALVE. CHECK VALVE MATERIAL OF CONSTRUCTION SHALL BE VITON.
7. OPENING FOR VERTICAL MIXER SHALL BE 4'-6" SQUARE AND STRUCTURAL SUPPORTS SHALL BE REMOVABLE TO PERMIT REMOVAL OF VERTICAL MIXER FROM THE MIXING CHAMBER WITHOUT DISASSEMBLING ITS IMPELLER.
8. PIPE SUPPORTS SHALL BE STAINLESS STEEL BELOW THE TOP DECK.
9. CONTRACTOR SHALL PROVIDE PRESSURE-SUSTAINING VALVE (PCV-575-5) WITH TWO ISOLATION BALL VALVES AT TRANSITION IN PIPE MATERIAL.



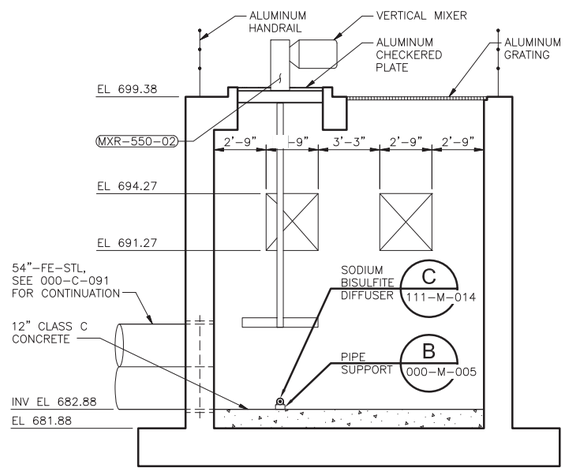

## CCB2 INFLUENT MIXER ENLARGED BOTTOM PLAN

1/4" = 1'-0"




## SECTION 2

1/4" = 1'-0" –




## SECTION 3

1/4" = 1'-0" –




## SECTION 4

1/4" = 1'-0" 109-M-002

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| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|
| 1 | 8/1/12 | CKM | JHC | CONFORMED |

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

DESIGNED BY: A. BAUMGARDNER
DRAWN BY: T. DRIZ
SHEET CHK'D BY: D. GAY
CROSS CHK'D BY: B. HEMKEN
APPROVED BY: J. CLARK
DATE: SEPTEMBER 2011

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# CHLORINE CONTACT BASIN 2 ENLARGED PLANS AND SECTIONS

T-46

PROJECT NO. 2012-73196
FILE NAME: M002CCSC.dwg
DRAWING NO. 109-M-004
SHEET NO. 649 OF 1476

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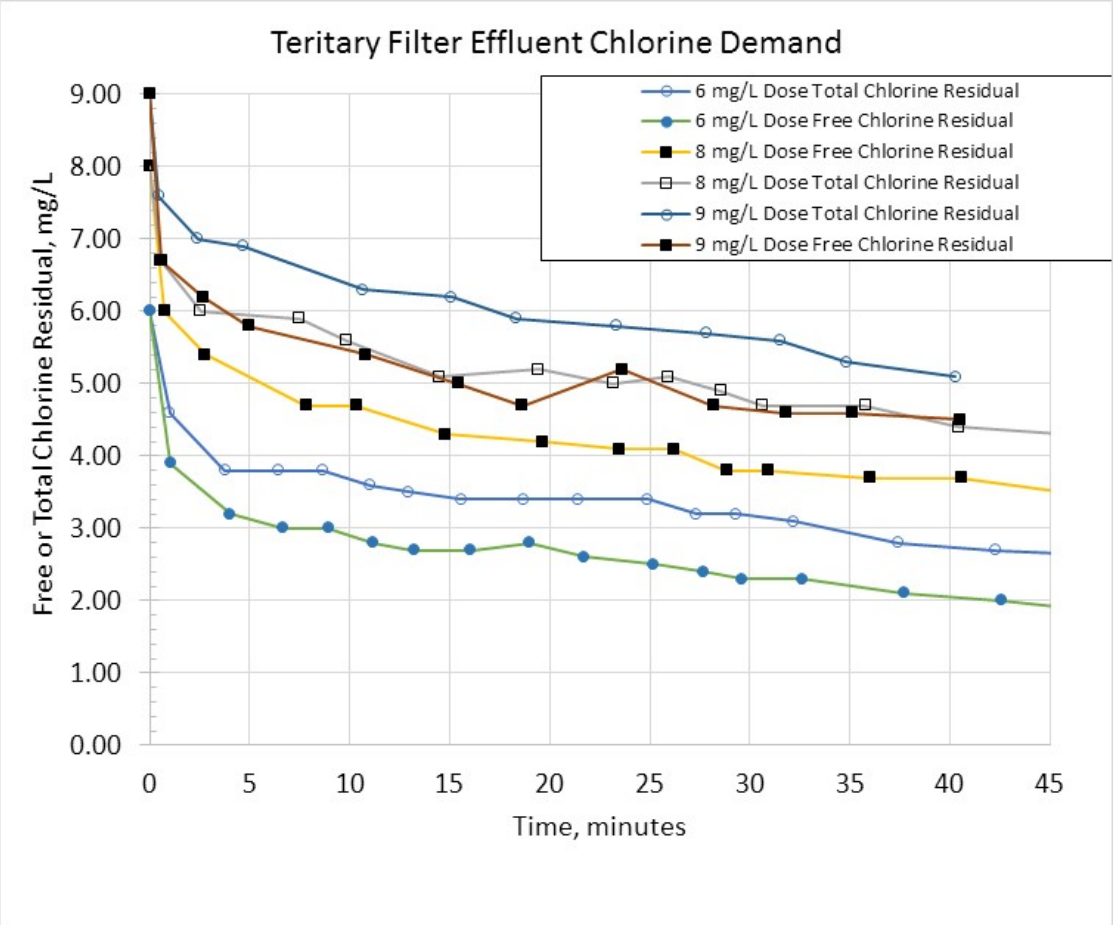
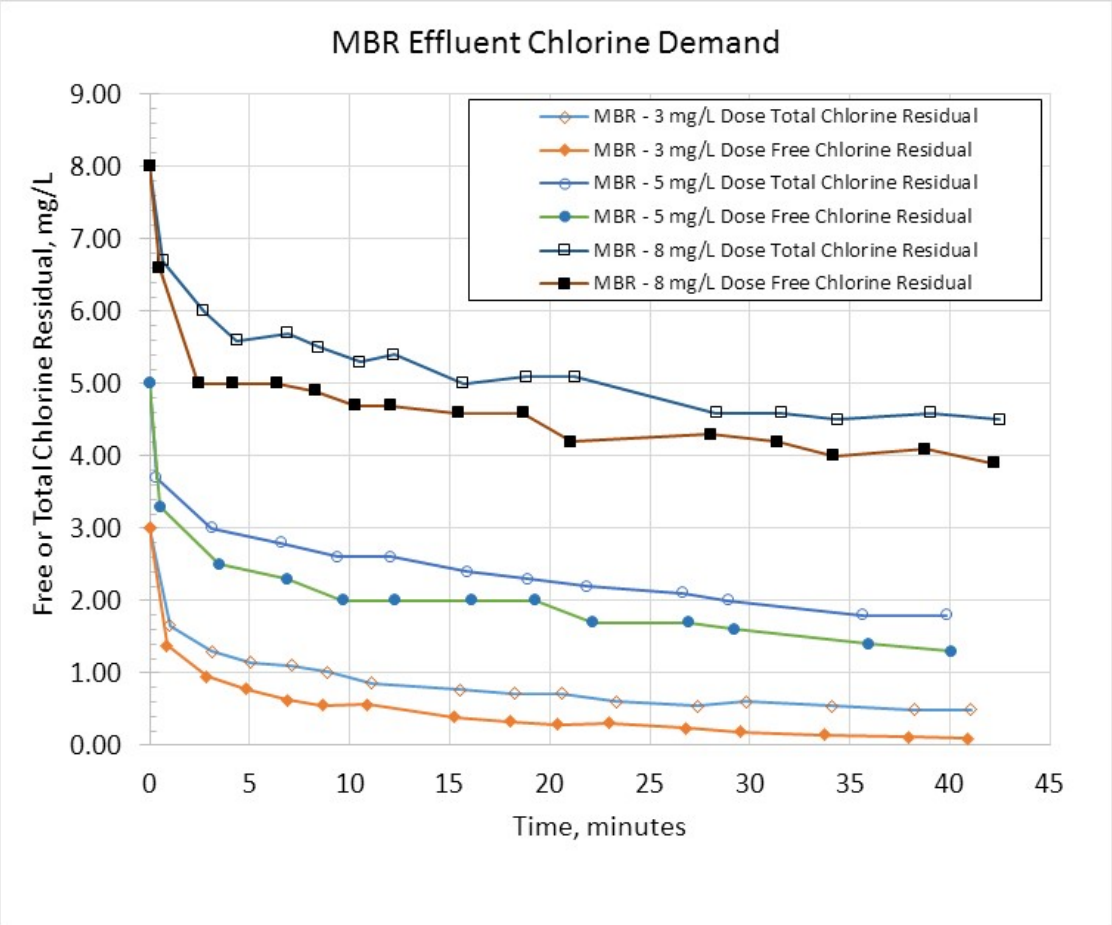
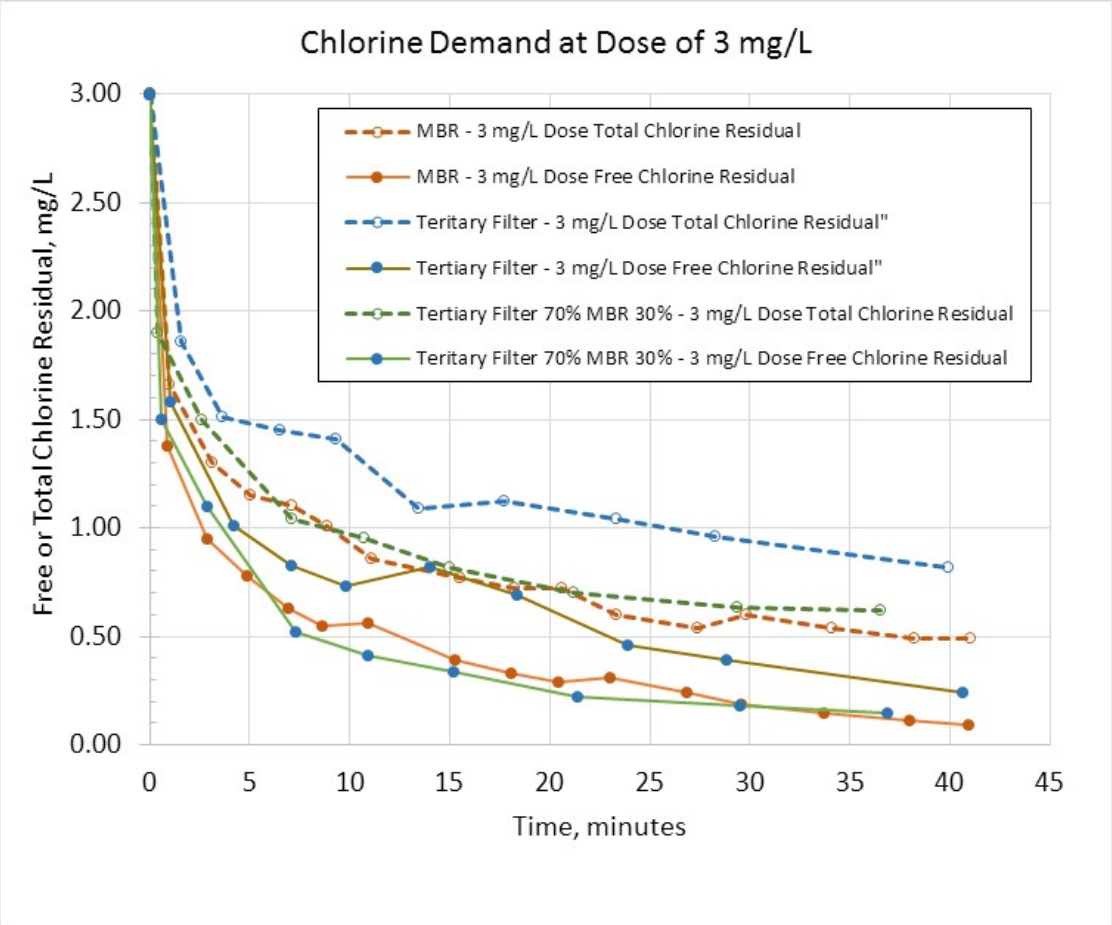
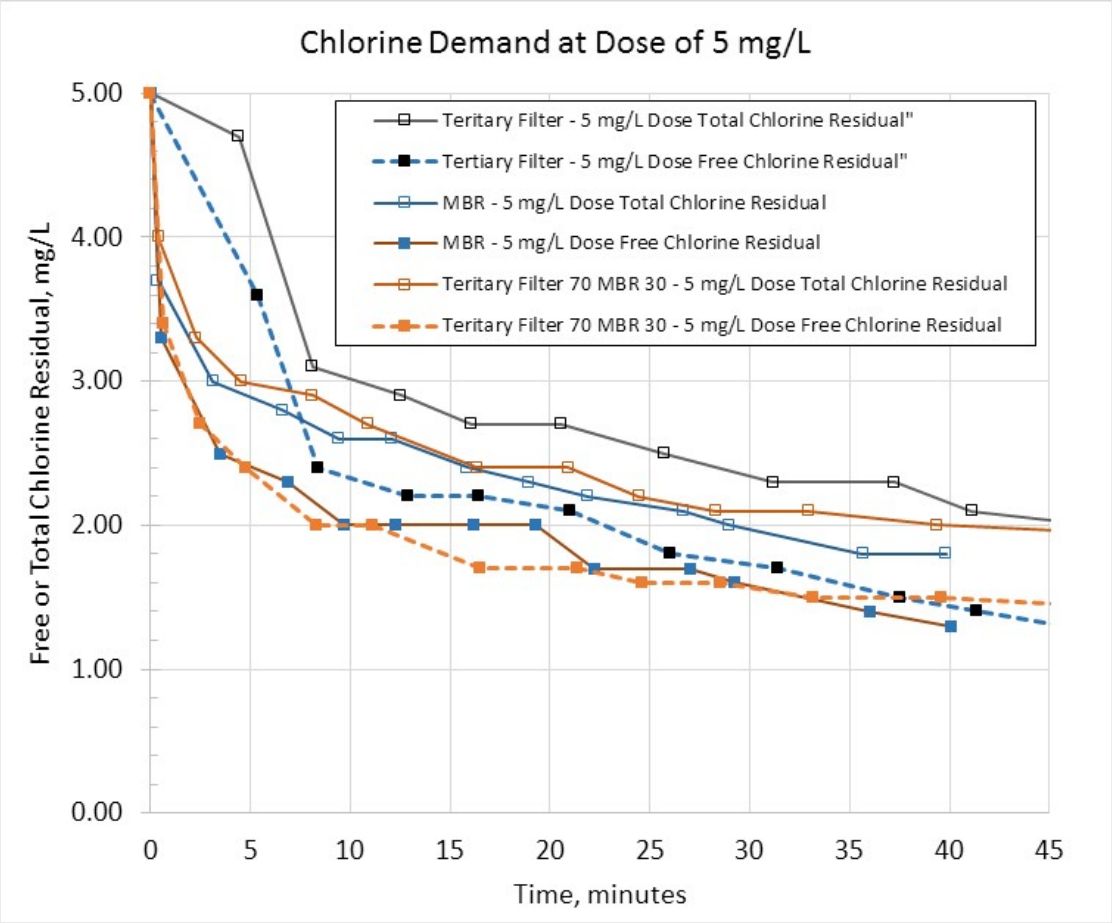
Appendix B

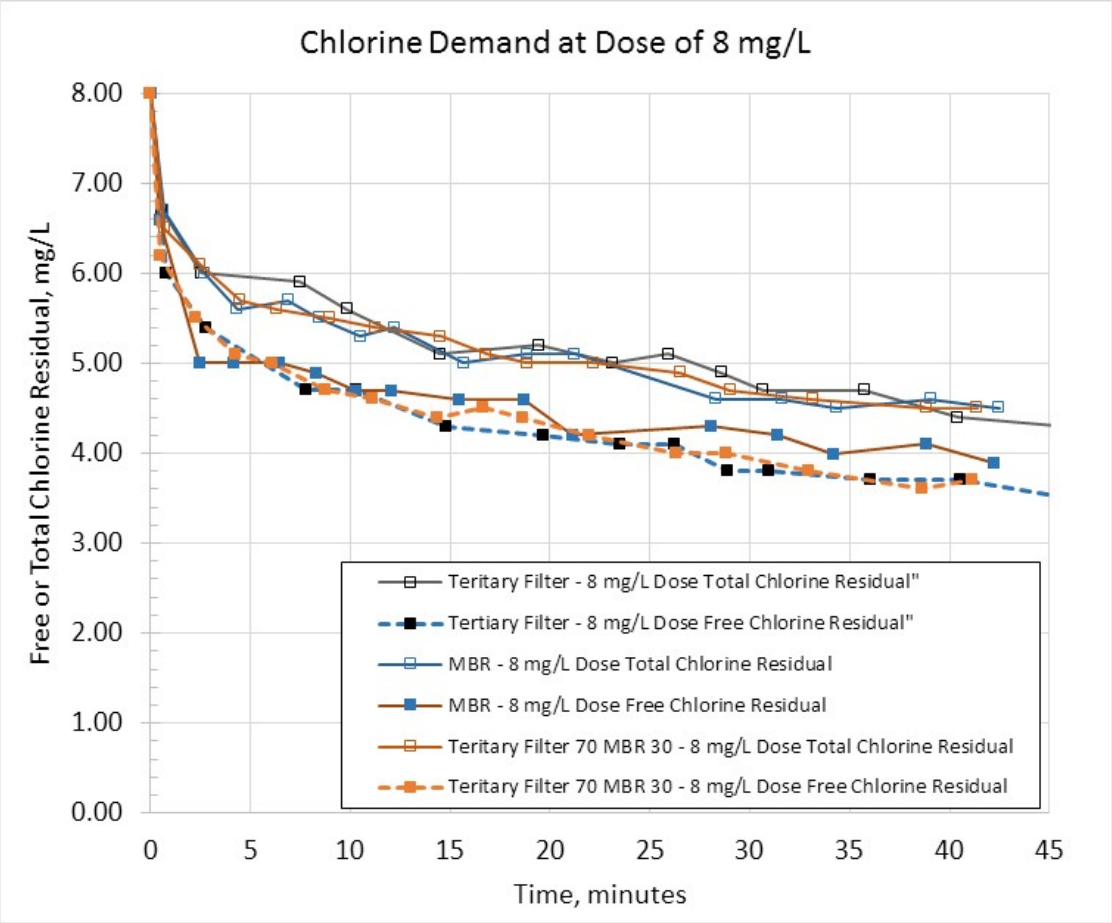
# TESTING DATA

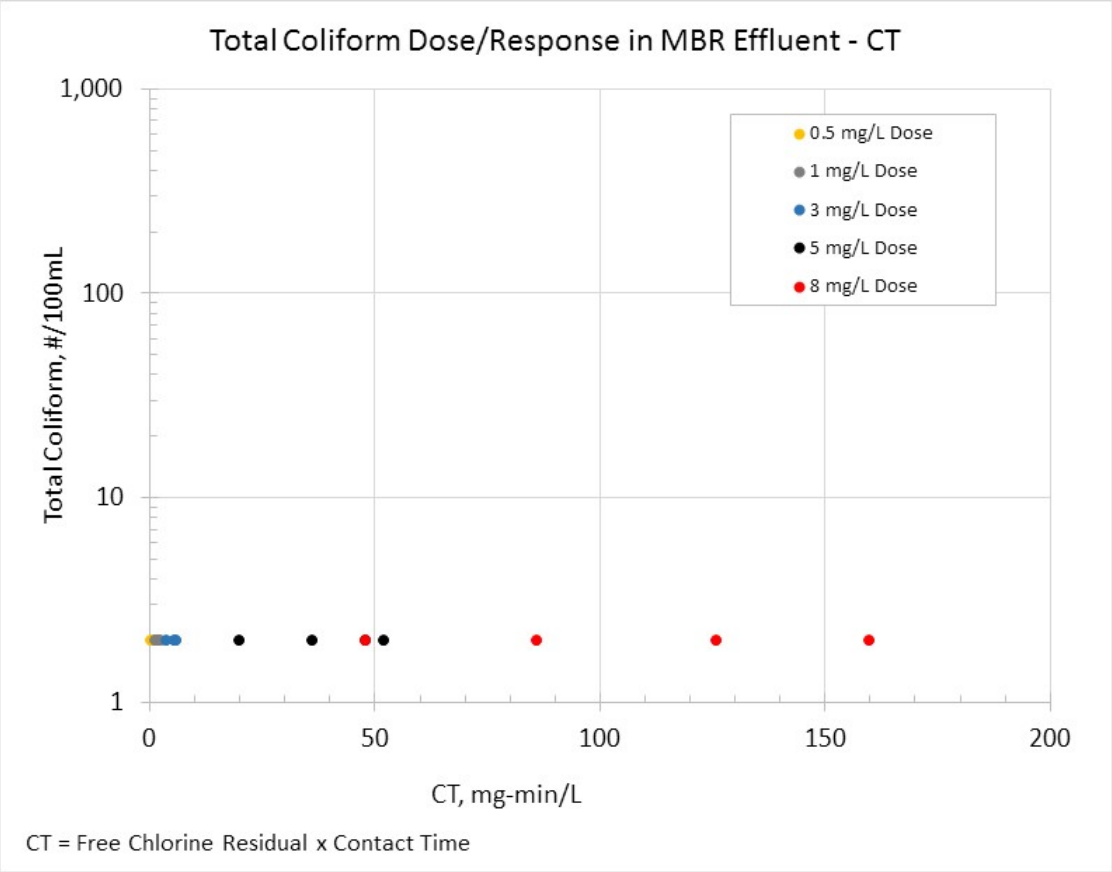

Teritary Filter Effluent Chlorine Demand
6 mg/L Dose Total Chlorine Residual
6 mg/L Dose Free Chlorine Residual
8 mg/L Dose Free Chlorine Residual
8 mg/L Dose Total Chlorine Residual
9 mg/L Dose Total Chlorine Residual
9 mg/L Dose Free Chlorine Residual
Free or Total Chlorine Residual, mg/L
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
5
10
15
20
25
30
35
40
45
Time, minutes

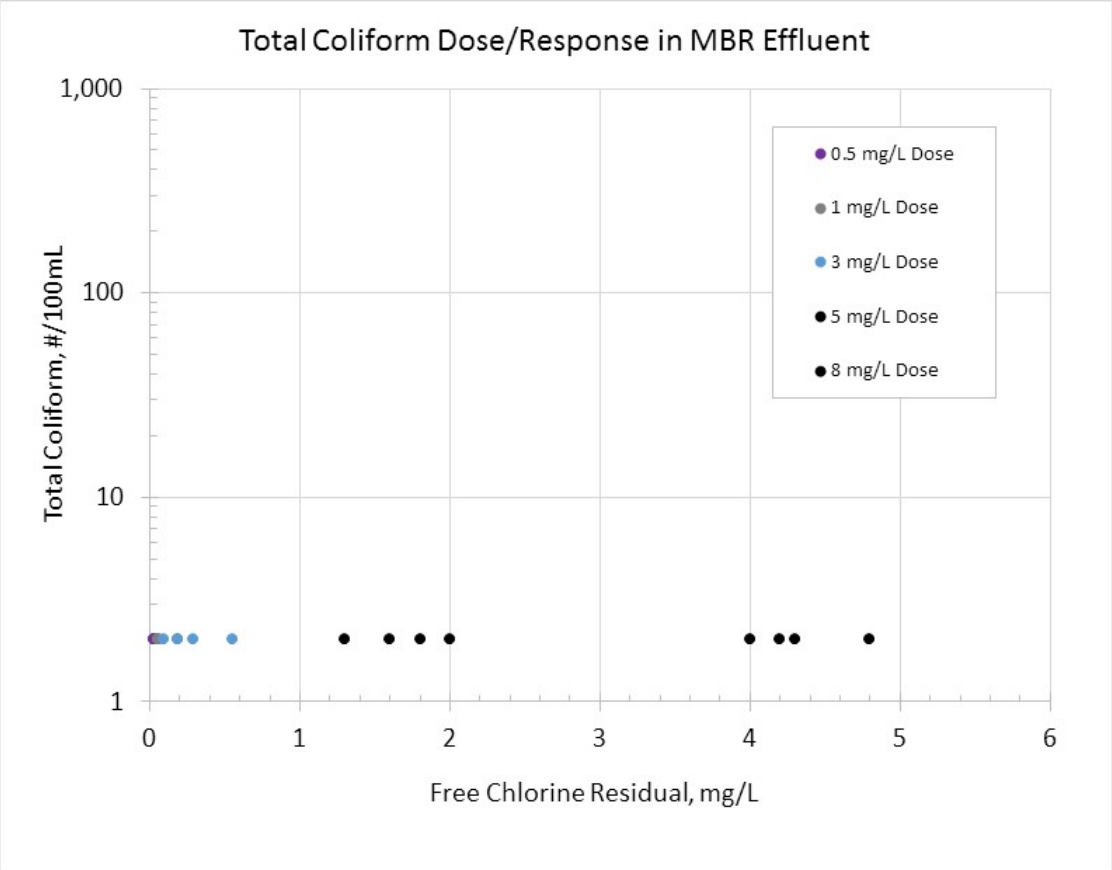

MBR Effluent Chlorine Demand
Free or Total Chlorine Residual, mg/L
Time, minutes
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
5
10
15
20
25
30
35
40
45
MBR - 3 mg/L Dose Total Chlorine Residual
MBR - 3 mg/L Dose Free Chlorine Residual
MBR - 5 mg/L Dose Total Chlorine Residual
MBR - 5 mg/L Dose Free Chlorine Residual
MBR - 8 mg/L Dose Total Chlorine Residual
MBR - 8 mg/L Dose Free Chlorine Residual

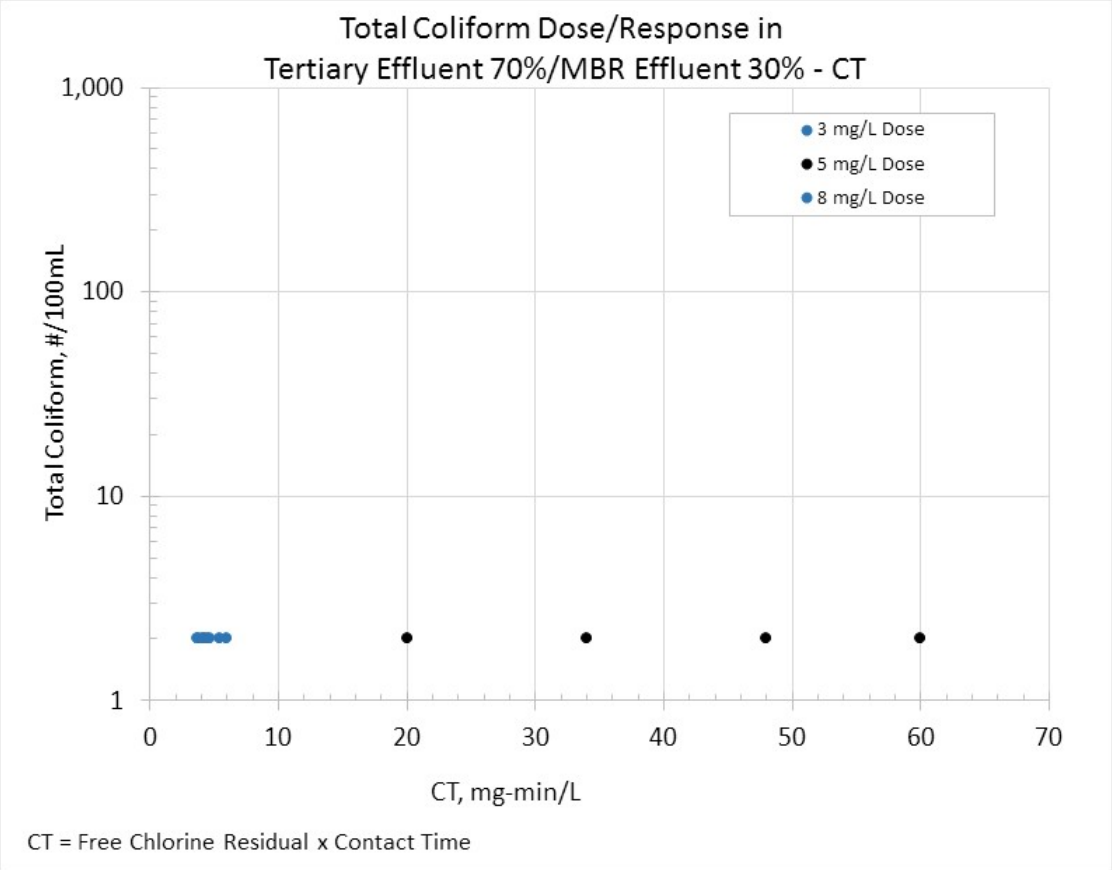

Chlorine Demand at Dose of 3 mg/L
Free or Total Chlorine Residual, mg/L
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0
5
10
15
20
25
30
35
40
45
Time, minutes
MBR - 3 mg/L Dose Total Chlorine Residual
MBR - 3 mg/L Dose Free Chlorine Residual
Teritary Filter - 3 mg/L Dose Total Chlorine Residual"
Tertiary Filter - 3 mg/L Dose Free Chlorine Residual"
Tertiary Filter 70% MBR 30% - 3 mg/L Dose Total Chlorine Residual
Teritary Filter 70% MBR 30% - 3 mg/L Dose Free Chlorine Residual

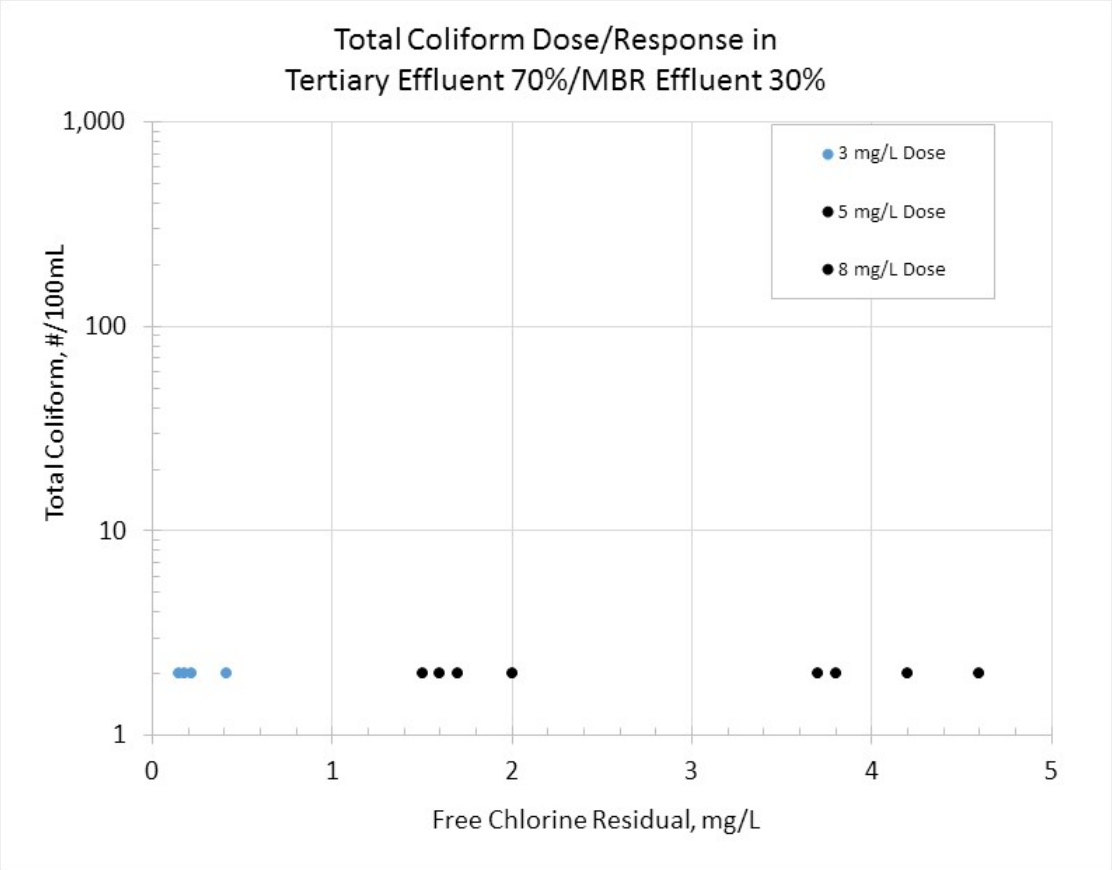

Chlorine Demand at Dose of 5 mg/L
Free or Total Chlorine Residual, mg/L
Time, minutes
5.00
4.00
3.00
2.00
1.00
0.00
0
5
10
15
20
25
30
35
40
45
Teritary Filter - 5 mg/L Dose Total Chlorine Residual"
Tertiary Filter - 5 mg/L Dose Free Chlorine Residual"
MBR - 5 mg/L Dose Total Chlorine Residual
MBR - 5 mg/L Dose Free Chlorine Residual
Teritary Filter 70 MBR 30 - 5 mg/L Dose Total Chlorine Residual
Teritary Filter 70 MBR 30 - 5 mg/L Dose Free Chlorine Residual

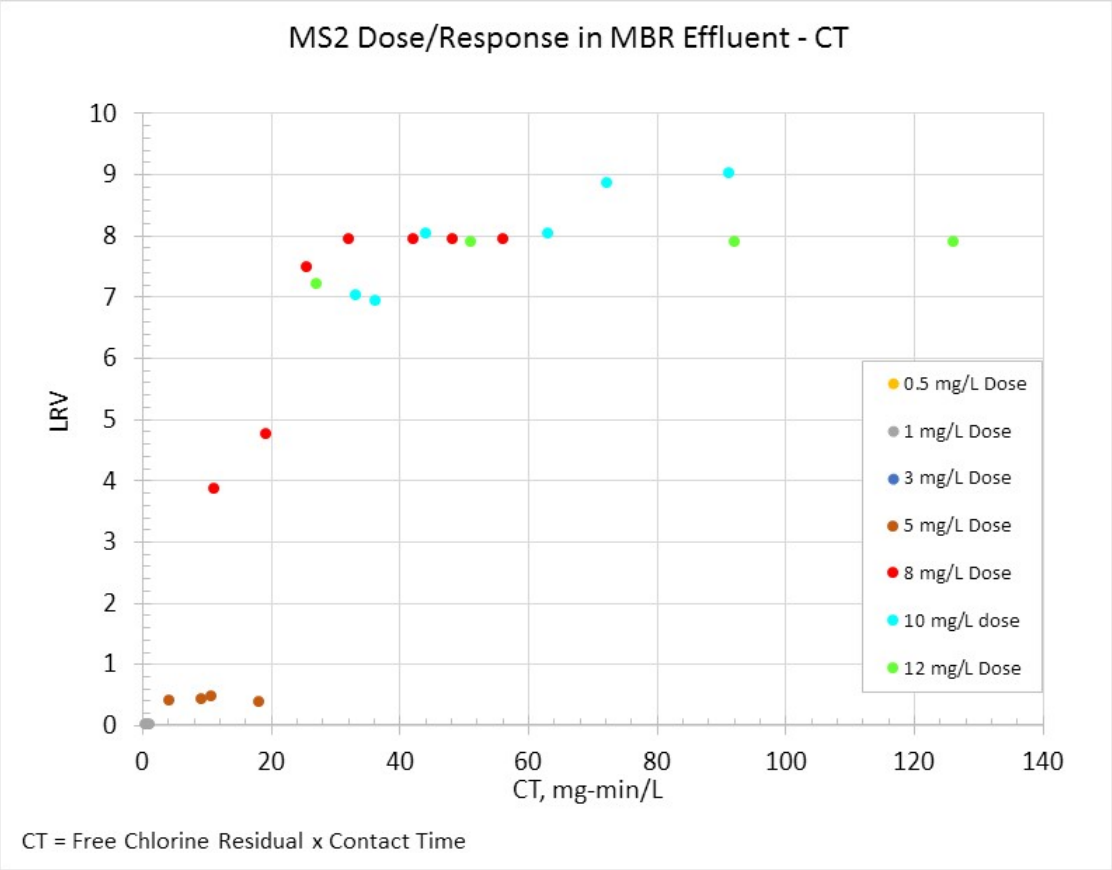

Chlorine Demand at Dose of 8 mg/L
Free or Total Chlorine Residual, mg/L
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
5
10
15
20
25
30
35
40
45
Time, minutes
Teritary Filter - 8 mg/L Dose Total Chlorine Residual"
Tertiary Filter - 8 mg/L Dose Free Chlorine Residual"
MBR - 8 mg/L Dose Total Chlorine Residual
MBR - 8 mg/L Dose Free Chlorine Residual
Teritary Filter 70 MBR 30 - 8 mg/L Dose Total Chlorine Residual
Teritary Filter 70 MBR 30 - 8 mg/L Dose Free Chlorine Residual

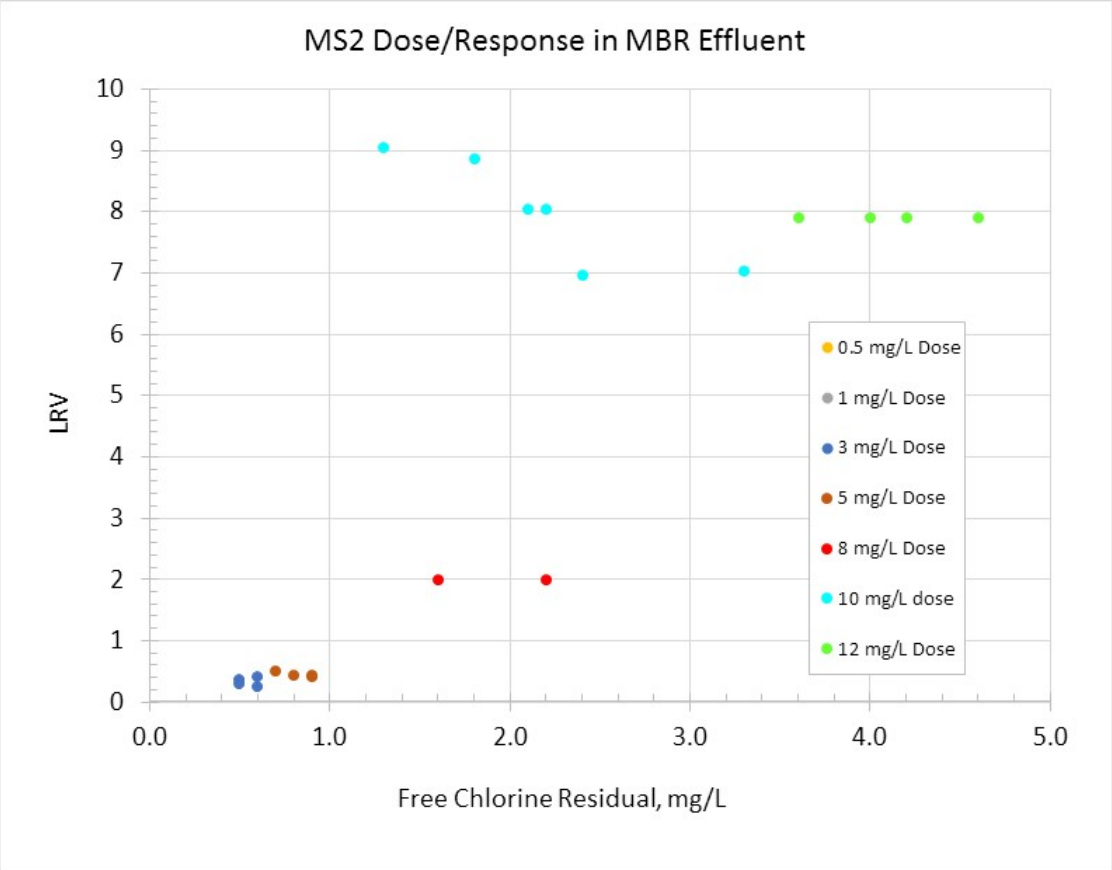

Total Coliform Dose/Response in MBR Effluent - CT
1,000
100
10
1
Total Coliform, #/100mL
0
50
100
150
200
CT, mg-min/L
0.5 mg/L Dose
1 mg/L Dose
3 mg/L Dose
5 mg/L Dose
8 mg/L Dose
CT = Free Chlorine Residual x Contact Time

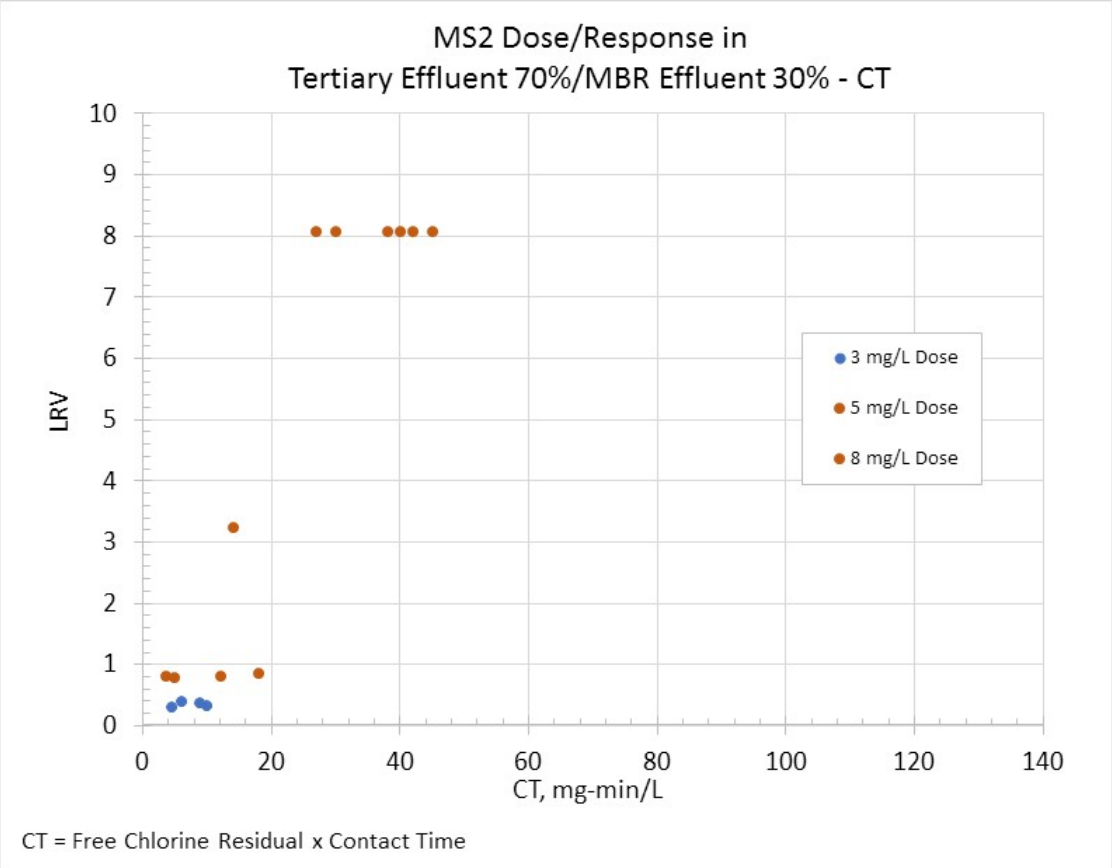

Total Coliform Dose/Response in MBR Effluent
1,000
100
10
1
Total Coliform, #/100mL
0
1
2
3
4
5
6
Free Chlorine Residual, mg/L
0.5 mg/L Dose
1 mg/L Dose
3 mg/L Dose
5 mg/L Dose
8 mg/L Dose

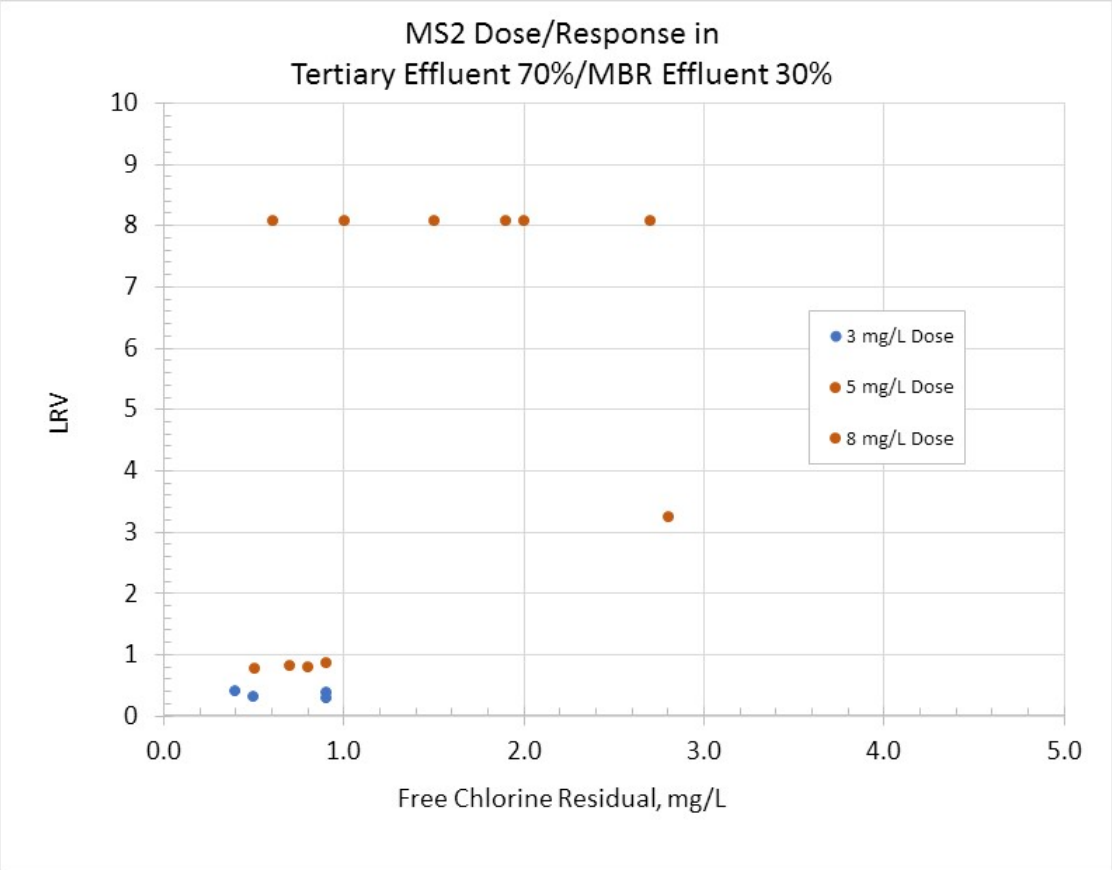

Total Coliform Dose/Response in
Tertiary Effluent 70%/MBR Effluent 30% - CT
1,000
100
10
1
Total Coliform, #/100mL
0
10
20
30
40
50
60
70
CT, mg-min/L
3 mg/L Dose
5 mg/L Dose
8 mg/L Dose
CT = Free Chlorine Residual x Contact Time


Total Coliform Dose/Response in
Tertiary Effluent 70%/MBR Effluent 30%
1,000
100
10
1
Total Coliform, #/100mL
0
1
2
3
4
5
Free Chlorine Residual, mg/L
3 mg/L Dose
5 mg/L Dose
8 mg/L Dose


MS2 Dose/Response in MBR Effluent - CT
LRV
CT, mg-min/L
0
1
2
3
4
5
6
7
8
9
10
0
20
40
60
80
100
120
140
0.5 mg/L Dose
1 mg/L Dose
3 mg/L Dose
5 mg/L Dose
8 mg/L Dose
10 mg/L dose
12 mg/L Dose
CT = Free Chlorine Residual x Contact Time


MS2 Dose/Response in MBR Effluent
LRV
Free Chlorine Residual, mg/L
0
1
2
3
4
5
6
7
8
9
10
0.0
1.0
2.0
3.0
4.0
5.0
0.5 mg/L Dose
1 mg/L Dose
3 mg/L Dose
5 mg/L Dose
8 mg/L Dose
10 mg/L dose
12 mg/L Dose


MS2 Dose/Response in
Tertiary Effluent 70%/MBR Effluent 30% - CT
LRV
CT, mg-min/L
0
1
2
3
4
5
6
7
8
9
10
0
20
40
60
80
100
120
140
3 mg/L Dose
5 mg/L Dose
8 mg/L Dose
CT = Free Chlorine Residual x Contact Time


MS2 Dose/Response in
Tertiary Effluent 70%/MBR Effluent 30%
LRV
10
9
8
7
6
5
4
3
2
1
0
0.0
1.0
2.0
3.0
4.0
5.0
Free Chlorine Residual, mg/L
3 mg/L Dose
5 mg/L Dose
8 mg/L Dose

Appendix B

# WASTE DISCHARGE ORDER R8-2013-0016

California Regional Water Quality Control Board
Santa Ana Region

November 1, 2013

**ITEM:** ***8**

**SUBJECT:** Renewal of Waste Discharge and Producer/User Reclamation Requirements for the City of Riverside's Regional Water Quality Control Plant, Order No. R8-2013-0016, NPDES No. CA0105350, Riverside County

**DISCUSSION:**

See attached Fact Sheet

**RECOMMENDATIONS:**

Adopt Order No. R8-2013-0016, NPDES No. CA0105350 as presented.

**COMMENT SOLICITATION:**

Comments were solicited from the discharger and the following agencies:

U.S. Environmental Protection Agency (WTR-5) – Peter Kozelka, Ph.D
U.S. Army District, Los Angeles, Corps of Engineers - Regulatory Branch
U.S. Fish and Wildlife Service, Carlsbad
State Water Resources Control Board, Office of the Chief Counsel – David Rice
State Department of Fish and Wildlife, Ontario
California Department of Health Services, San Diego – Steve Williams
Riverside County Environmental Health Services – Sandy Bunchek
Riverside County Flood Control and Water Conservation District – Jason Uhley
Santa Ana Watershed Project Authority – Celeste Cantu
Orange County Water District - Nira Yamachika, Marsha Westropp
Inland Empire Waterkeeper - Meagan Brousseau
Orange County Coastkeeper - Garry Brown
Lawyers for Clean Water – Daniel Cooper

## Table 2 - Discharge Locations

| Discharge Points | Effluent Description | Discharge Point (Latitude) | Discharge Point (Longitude) | Receiving Water |
|---|---|---|---|---|
| 001 | Tertiary wastewater discharged directly to the Santa Ana River | 33° 57' 55"N | 117° 27' 28"W | Reach 3 of Santa Ana River |
| 002 | Tertiary wastewater commingled with Hole Lake effluent and then to the Santa Ana River | 33° 57' 44"N | 117° 29' 7"W | Reach 3 of Santa Ana River |
| 003 | Recycled water for irrigation of Toro turf. | 33° 57' 47"N | 117° 27' 41"W | Chino-South Groundwater Management Zone |
| 004 | Recycled water for Riverside Public Utilities distribution system | 33° 57' 43"N | 117° 27' 42"W | Various Groundwater Management Zones |

## Table 3 - Administrative Information

| | |
|---|---|
| This Order was adopted by the Regional Water Board on: | November 1, 2013 |
| This Order shall become effective on: | November 1, 2013 |
| This Order shall expire on: | October 31, 2018 |
| The U.S. Environmental Protection Agency (U.S. EPA) and the Regional Water Board have classified this discharge as a **major** discharge. | |
| The Discharger shall file a Report of Waste Discharge in accordance with Title 23, California Code of Regulations, not later than 180 days in advance of the Order expiration date as application for issuance of new waste discharge requirements. | |

IT IS HEREBY ORDERED, that this Order supersedes Order No. R8-2006-0009 except for enforcement purposes, and, in order to meet the provisions contained in Division 7 of the California Water Code (CWC) and regulations adopted thereunder, and the provisions of the federal Clean Water Act (CWA) and regulations and guidelines adopted thereunder, the Discharger shall comply with the requirements in this Order.

I, Kurt V. Berchtold, Executive Officer, do hereby certify that Order No. R8-2013-0016 with all attachments is a full, true, and correct copy of an Order adopted by the California Regional Water Quality Control Board, Santa Ana Region, on November 1, 2013.

_____

**Kurt V. Berchtold, Executive Officer**

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## LIST OF ATTACHMENTS

Attachment A – Definitions
Attachment B – Plant Location Map
Attachment C – Flow Schematic
Attachment D – Standard Provisions
Attachment E – Monitoring and Reporting Program
Attachment F – Fact Sheet
Attachment G – EPA Priority Pollutant List
Attachment H – Minimum Levels
Attachment I – Triggers for Monitoring of Priority Pollutants

## I. DISCHARGER/FACILITY INFORMATION

The following Discharger is subject to waste discharge requirements as set forth in this Order:

**Table 4 – Discharger/Facility Information**

| | |
|---|---|
| **Discharger** | City of Riverside-Department of Public Works |
| **Discharger Contact** | Tom Boyd, Public Works Director (951) 826-5575 |
| **Mailing Address** | 3900 Main St., Riverside, CA 92522 |
| **Facility** | Riverside Regional Water Quality Control Plant |
| **Facility Address** | 5950 Acorn Street, Riverside, CA 92504 |
| **Facility Contacts** | Gary Valladao, Wastewater Systems Manager, (951) 351-6140<br>Edward Filadelfia, Regulatory Affairs & Compliance, (951) 351- 6080<br>Kevin Street, Regulatory Programs and Compliance Manager, (951) 351- 6007 |
| **Mailing Address** | Same as Facility Address |
| **Type of Facility** | Publicly Owned Treatment Works (POTW) |
| **Facility Design Flow** | 46 million gallons per day |

## II. FINDINGS

The California Regional Water Quality Control Board, Santa Ana Region (hereinafter Regional Water Board), finds:

A. **Background.** The City of Riverside, Department of Public Works (hereinafter Discharger) is currently discharging pursuant to Order No. R8-2006-0009 and National Pollutant Discharge Elimination System (NPDES) Permit No. CA0105350. The Discharger submitted a Report of Waste Discharge, dated September 27, 2010, and applied for a NPDES permit renewal to discharge from the Riverside Regional Water Quality Control Plant (hereinafter RWQCP, or Facility) up to 46 million gallons per day (mgd) of tertiary treated wastewater to Reach 3 of the Santa Ana River. The application was deemed complete on August 18, 2011.

For the purposes of this Order, references to the "Discharger" or "permittee" in applicable federal and State laws, regulations, plans, or policy are held to be equivalent to references to the Discharger herein.

B. **Facility Description**. The Discharger operates the Riverside Regional Water Quality Control Plant (RRWQCP). The RRWQCP is a municipal wastewater treatment plant located on a 121-acre site at 5950 Acorn Street in the City of Riverside, south of the Santa Ana River near the intersection of Van Buren Boulevard. The RRWQCP discharges tertiary treated wastewater to Reach 3 of the Santa Ana River. The RRWQCP is designed to tertiary treat 46 million gallons per day (mgd) of wastewater. The annual average daily flow for 2012 was 29 mgd.

C. **Legal Authorities.** This Order serves as Waste Discharge Requirements (WDRs) pursuant to Article 4, Chapter 4 of California Water Code (CWC) commencing with Section 13260. This Order shall also serve as an NPDES permit pursuant to Section 402 of the federal Clean Water Act (CWA) and implementing regulations adopted by the U.S. Environmental Protection Agency (USEPA) and Chapter 5.5, Division 7 of the CWC for point source discharges from this facility to the surface waters.

D. **Background and Rationale for Requirements.** The Regional Water Board developed the requirements in this Order based on information submitted as part of the application, through monitoring and reporting programs, and other available information. The Fact Sheet (Attachment F), which contains background information and rationale for Order requirements, is hereby incorporated into this Order and constitutes part of the Findings for this Order. Attachments A through E and G through I are also incorporated into this Order.

E. **Pretreatment Requirements.** The Discharger has established an approved regional pretreatment program. The approved pretreatment program and its components and control mechanisms, among others, are hereby made an enforceable condition of this Order.

F. **California Environmental Quality Act (CEQA).** Under CWC section 13389, the action to adopt these waste discharge requirements is exempt from the provisions of CEQA, Public Resources Code section 21000 et seq. (County of Los Angeles v. California State Water Resources Control Board (2006) 143 Ca. App.4th 985. Mod. (Nov 6, 2006, B184034) 50 Cal.Rptr.3d 619, 632-636). This action also involves the re-issuance of waste discharge requirements for an existing facility that discharges treated wastewater and as such, is exempt from the provisions of the California Environmental Quality Act (Commencing with Section 21100)pursuant to Title 14 of the California Code of Regulations Section 15301.

G. **Technology-based Effluent Limitations.** Section 301(b) of the CWA and implementing USEPA permit regulations at 40 CFR[1] 122.44, require that permits include conditions meeting applicable technology-based requirements at a minimum, and any more stringent effluent limitations necessary to meet applicable water quality standards. The discharge authorized by this Order must meet minimum federal technology-based requirements based on Secondary Treatment Standards at 40 CFR Part 133 and/or Best Professional Judgment (BPJ) in accordance with 40 CFR 125.3. A detailed discussion of the technology-based effluent limitations development is included in the Fact Sheet (Attachment F).

H. **Water Quality-Based Effluent Limitations.** Section 301(b) of the CWA and 40 CFR 122.44(d) require that permits include limitations more stringent than applicable federal technology-based requirements where necessary to achieve applicable water quality standards. This Order contains requirements, expressed as a technology equivalence requirement, more stringent than secondary treatment requirements that are necessary to meet applicable water quality standards. The rationale for these requirements, which consist of tertiary treatment or equivalent requirements or other provisions, is discussed in the Fact Sheet.

40 CFR 122.44(d)(1)(i) mandates that permits include effluent limitations for all pollutants that are or may be discharged at levels that have a reasonable potential to cause or contribute to an exceedance of a water quality standard, including numeric and narrative objectives within a standard. Where reasonable potential has been established for a pollutant, but there is no numeric criterion or objective for the pollutant, water quality-based effluent limitations (WQBELs) must be established using: (1) USEPA criteria guidance under CWA section 304(a), supplemented where necessary by other relevant information; (2) an indicator parameter for the pollutant of concern; or (3) a calculated numeric water quality criterion, such as a proposed state criterion or policy interpreting the state's narrative criterion, supplemented with other relevant information, as provided in 40 CFR 122.44(d)(1)(vi).

I. **Water Quality Control Plans.** The Regional Water Board adopted a revised Water Quality Control Plan for the Santa Ana Region (hereinafter Basin Plan) that became effective on January 24, 1995. The Basin Plan designates beneficial uses, establishes water quality objectives, and contains implementation programs and policies to achieve those objectives for all waters in the Santa Ana Region addressed through the plan.

[1] Title 40 of the Code of Federal Regulations

More recently, the Basin Plan was amended significantly to incorporate revised boundaries for groundwater subbasins, now termed "management zones", new nitrate-nitrogen and TDS objectives for the new management zones, and new nitrogen and TDS management strategies applicable to both surface and ground waters. The Regional Water Board adopted this Basin Plan Amendment on January 22, 2004. The State Water Resources Control Board (State Water Board) and Office of Administrative Law (OAL) approved the Amendment on September 30, 2004 and December 23, 2004, respectively. EPA approved the surface water standards components of the N/TDS Amendment on June 20, 2007.

In addition, the Basin Plan implements State Water Resources Control Board (State Water Board) Resolution No. 88-63, which established State policy that all waters, with certain exceptions, should be considered suitable or potentially suitable for municipal or domestic supply. Based on the criteria specified in the State Water Board Resolution, the Basin Plan specifies that Reach 3 of the Santa Ana River and downstream reaches, as well as most of its tributaries, are excepted from the municipal and domestic supply beneficial use.

As discussed in detail in the Fact Sheet (Attachment F), beneficial uses applicable to Reach 3 of Santa Ana River are as follows:

**Table 5 - Basin Plan Beneficial Uses**

| Discharge Point | Receiving Water Name | Beneficial Use(s) |
|---|---|---|
| 001 and 002 | Reach 3 of Santa Ana River, near Van Buren Blvd Bridge | Present or Potential:<br>Agricultural supply, groundwater recharge, water contact recreation, non-contact water recreation, warm freshwater habitat, wildlife habitat, and rare, threatened or endangered species. |
| 001, 002, and 003 | Chino-South Groundwater Management Zone | Present or Potential:<br>Municipal and domestic supply, agricultural supply, industrial service supply, and industrial process supply. |
| 004 | Chino-South; Arlington; Riverside-A, D, E, and F; and Temescal Groundwater Management Zones | Present or Potential:<br>Municipal and domestic supply, agricultural supply, industrial service supply, and industrial process supply. |

Requirements of this Order implement the applicable provisions of the Basin Plan.

J. **National Toxics Rule (NTR) and California Toxics Rule (CTR).** USEPA adopted the NTR on December 22, 1992, and later amended it on May 4, 1995 and November 9, 1999. About forty criteria in the NTR were applicable in California. On May 18, 2000, USEPA adopted the CTR, which incorporated the NTR criteria that were applicable in California. The CTR was amended on February 13, 2001. The NTR and the CTR contain water quality criteria for priority pollutants.

K. **State Implementation Policy.** On March 2, 2000, the State Water Board adopted the Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays, and Estuaries of California (State Implementation Policy or SIP). The SIP became effective on April 28, 2000 with respect to the priority pollutant criteria promulgated for California by the USEPA through the NTR and to the priority pollutant objectives established by the Regional Water Board in the Basin Plan. The SIP became effective on May 18, 2000 with respect to the priority pollutant criteria promulgated by the USEPA through the CTR. The State Water Board adopted amendments to the SIP on February 24, 2005 that became effective on July 13, 2005. The SIP establishes implementation provisions for priority pollutant criteria and objectives and provisions for chronic toxicity control. Requirements of this Order implement the SIP.

L. **Alaska Rule.** On March 30, 2000, USEPA revised its regulation that specifies when new and revised State and Tribal water quality standards (WQS) become effective for CWA purposes (40 CFR 131.21, 65 FR 24641, April 27, 2000). Under the revised regulation (also known as the Alaska Rule), new and revised standards submitted to USEPA after May 30, 2000, must be approved by USEPA before being used for CWA purposes. The final rule also provides that standards already in effect and submitted to USEPA by May 30, 2000, may be used for CWA purposes, whether or not approved by USEPA.

M. **Antidegradation Policy.** 40 CFR 131.12 requires that State water quality standards include an antidegradation policy consistent with the federal policy. The State Water Board established California's antidegradation policy in State Water Board Resolution No. 68-16. Resolution No. 68-16 incorporates the federal antidegradation policy where the federal policy applies under federal law. Resolution No. 68-16 requires that existing quality of waters be maintained unless degradation is justified based on specific findings. The Regional Water Board's Basin Plan implements, and incorporates by reference, both the State and federal antidegradation policies. As discussed in detail in the Fact Sheet the permitted discharge is consistent with the antidegradation provision of 40 CFR Section 131.12 and State Water Board Resolution No. 68-16.

N. **Anti-Backsliding Requirements.** Sections 402(o)(2) and 303(d)(4) of the CWA and federal regulations at 40 CFR 122.44(l) prohibit backsliding in NPDES permits. These anti-backsliding provisions require effluent limitations in a reissued permit to be as stringent as those in the previous permit. As discussed in the Fact Sheet, all effluent limitations in this Order are at least as stringent as the effluent limitations in the previous Order. Therefore, this Order conforms to the anti-backsliding requirements of the CWA.

O. **Monitoring and Reporting.** Water Code Sections 13267 and 13383 authorize the Regional Water Board to require technical and monitoring reports. 40 CFR 122.48 requires that all NPDES permits specify requirements for recording and reporting monitoring results. The Monitoring and Reporting Program establishes monitoring and reporting requirements to implement State and federal requirements. This Monitoring and Reporting Program is provided in Attachment E.

P. **Biosolids Requirements.** On February 19, 1993, the USEPA issued a final rule for the use and disposal of sewage sludge, 40 CFR, Part 503. This rule requires that producers of sewage sludge meet certain reporting, handling, and disposal requirements. The State of California has not been delegated the authority to implement this program, therefore, the U.S. Environmental Protection Agency is the implementing agency. However, this Order includes biosolids monitoring requirements.

Q. **State General Waste Discharge Requirements for Sanitary Sewer Systems.** The State Water Board issued General Waste Discharge Requirements for Sanitary Sewer Systems, Water Quality Order No. 2006-0003 on May 2, 2006, requiring public agencies that own sanitary sewer systems, comprised of more than one mile of pipes or sewer lines, to enroll for coverage under the General Order. The General Order requires agencies to develop sanitary sewer management plans (SSMPs) and report all sanitary sewer overflows (SSOs). This Order requires the Discharger to continue its enrollment under the General Order.

R. **Standard and Special Provisions.** Standard Provisions, which apply to all NPDES permits in accordance with 40 CFR Sections 122.41 and additional conditions applicable to specified categories of permits in accordance with 40 CFR 122.42, are provided in Attachment D. The Discharger must comply with all standard provisions and with those additional conditions that are applicable under 40 CFR 122.42. The Regional Water Board has also included in this Order special provisions applicable to the Discharger. A rationale for the special provisions contained in this Order is provided in the attached Fact Sheet (Attachment F).

S. **Provisions and Requirements Implementing State Law.** The provisions/requirements in subsections IV.B. and V.B. of this Order are included to implement state law only. These provisions/requirements are not required or authorized under the federal CWA; consequently, violations of these provisions/requirements are not subject to the enforcement remedies that are available for NPDES violations.

T. **Notification of Interested Parties.** The Regional Water Board has notified the Discharger and interested agencies and persons of its intent to prescribe Waste Discharge Requirements for the discharge and has provided them with an opportunity to submit their written comments and recommendations. Details of notification are provided in the Fact Sheet (Attachment F) of this Order.

U. **Consideration of Public Comment.** The Regional Water Board, in a public meeting, heard and considered all comments pertaining to the discharge. Details of the Public Hearing are provided in the Fact Sheet (Attachment F) of this Order.

## III. DISCHARGE PROHIBITIONS

**A.** The discharge of wastewater at a location or in a manner different from those described in this Order is prohibited.

**B.** The bypass or overflow of untreated wastewater or wastes to surface waters or surface water drainage courses is prohibited, except as allowed in Standard Provisions (Attachment D), section I. G.

**C.** The discharge of any substances in concentrations toxic to animal or plant life is prohibited.

**D.** The discharge of any radiological, chemical, or biological warfare agent or high level radiological waste is prohibited.

**E.** The injection of fats, oil, and grease and food processing waste directly into an anaerobic digester is prohibited, unless the discharger has developed and is implementing standard operating procedures for this activity. The standard operating procedures shall address spill prevention, spill response, vector control, traffic control, feed quality control, and process control. The discharger shall provide training to its staff on these standard operating procedures.

# IV. EFFLUENT LIMITATIONS AND DISCHARGE SPECIFICATIONS

## A. Effluent Limitations – Discharge Points 001 and 002

Unless otherwise specifically specified herein, compliance with the following effluent limitations is measured at monitoring location M-001A as described in the attached Monitoring and Reporting Program (Attachment E).

### 1. Effluent Limitations for Discharges Under Conditions Without 20:1 Dilution in the Receiving Water

Except as specified in Effluent Limitation IV.A.2., below, the Discharger shall comply with the following at Discharge Points 001 and 002:

a. Effluent Limitations:

**Table 6 - Effluent Limitations Without 20:1 Dilution**

| Parameter | Units | Effluent Limitations | | |
|---|---|---|---|---|
| | | Average Monthly | Average Weekly | Maximum Daily |
| Biochemical Oxygen Demand 5-day @ 20°C | mg/L | 20 | 30 | -- |
| Total Suspended Solids | mg/L | 20 | 30 | -- |
| Ammonia-Nitrogen | mg/L | 5.0 | -- | -- |
| Cyanide (Free) | µg/l | 4.2 | -- | 8.5 |

b. Percent Removal:

The average monthly percent removal of BOD 5-day at $20^0$C and total suspended solids shall not be less than 85 percent.

c. Total Dissolved Solids:

1) The 12-month flow weighted running average total dissolved solids constituent concentration shall not exceed 650 mg/l, unless the discharger demonstrates to the satisfaction of the Regional Board's Executive Officer that:

i. Discharges in excess of the TDS limits are due to the quality of water supply sources utilized in the Discharger's service area, and that all reasonable steps, as agreed upon by Executive Officer, have been taken to ensure that the best quality supplies are obtained and utilized in the Dischargers service area; and/or

ii. Discharges in excess of the TDS limits are due to chemical additions in the treatment process needed to meet waste discharge requirements, and the Discharger has taken all reasonable steps to optimize chemical additions so as to minimize the increases; and

iii. The Discharger implements a plan, with the approval of the Executive Officer, to offset TDS discharges in excess of the 650 mg/L limit.

2) The 12-month flow weighted running average total dissolved solids concentration shall not exceed the 12-month flow weighted running average total dissolved solids concentration in the water supply by more than 250 mg/l, unless the Discharger demonstrates to the satisfaction of the Regional Board's Executive Officer that TDS discharge in excess of 250 mg/l mineral increment are due solely to chemical additions in the treatment process needed to meet waste discharge requirements, and the discharger has taken all reasonable steps to optimize chemical additions so as to minimize the TDS increases.

d. Total Inorganic Nitrogen (TIN):

The 12-month flow weighted running average TIN concentration shall not exceed 10 mg/l. unless the Discharger implements a plan, with the approval of the Executive Officer, to offset TIN discharges in excess of the TIN limits.

e. Disinfected Tertiary Treated Wastewater:

The discharge shall at all times be adequately oxidized, filtered, and disinfected tertiary treated wastewater and shall meet the following limitations:

1) Turbidity:
   a) When filtration is through natural undisturbed soils or a bed of filter media, the turbidity of the filter effluent shall not exceed any of the following:
      i. Average of 2 Nephelometric Turbidity Unit (NTU) within any 24-hour period;
      ii. 5 NTU more than 5 percent of the time in any 24-hour period; and
      iii. 10 NTU more than one minute.

   b) When filtration is through microfiltration, the turbidity of the filter effluent shall not exceed any of the following:
      i. 0.2 Nephelometric Turbidity Unit (NTU) more than 5 % of the time within a 24-hour period; and
      ii. 0.5 NTU at any time.

2) Disinfection:

   a) When a chlorine disinfection process is utilized following filtration, 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[2], based on peak dry weather design flow[3]; shall be provided[4].

   b) When a disinfection process combined with the filtration process is utilized, the combined process shall demonstrate inactivation and/or removal of 99.999 percent of the plaque-forming 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.

[2] The modal contact time requirement is applicable only to the use of recycled water and not to surface water discharges, provided the receiving water provides a 1:1 dilution. The receiving water considered here shall exclude upstream POTW effluent flow.
[3] "Peak Dry Weather Flow" means the arithmetic mean of the maximum peak flow rates sustained over some period of time (for example three hours) during the maximum 24-hour dry weather period. Dry weather period is defined as period of little or no rainfall.
[4] Modal contact time and CT shall be calculated based on the minimum one-hour average value in a 24-hr period.

c) Where ultraviolet (UV) disinfection is solely used for disinfection, UV disinfection shall meet the requirements specified in the Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse, published by the National Water Research Institute, Second Edition, unless otherwise approved by the California Department of Public Health. The Discharger's UV disinfection system shall not be utilized as the sole source of disinfection except under emergency conditions unless it is re-commissioned under CDPH oversight.

3) Coliform:

a) The weekly average concentration of total coliform bacteria shall not exceed a Most Probable Number (MPN) of 2.2 total coliform bacteria per 100 milliliters (ml).
To comply with this limit, the 7-day median MPN must not exceed 2.2 per 100 milliliters on any day during a calendar week. However, only one violation is to be recorded for each calendar week, even if the 7-day median MPN value is greater than 2.2 for more than one day in the week.

b) The MPN of total coliform bacteria shall not exceed 23 per 100 ml in more than one sample during in any 30-day period.

c) No total coliform bacteria sample shall exceed an MPN of 240 per 100 ml.

4) Total Chlorine Residual:

a) The total time during which the total chlorine residual values are above 0.1 mg/L (instantaneous maximum value) shall not exceed 7 hours and 26 minutes in any calendar month;

b) No individual excursion above 0.1 mg/L shall exceed 5 minutes; and

c) No individual excursion shall exceed 5.0 mg/L.

f. pH:

The pH of the discharge shall be within 6.5 to 8.5 pH units at all times. Compliance with pH limits shall be determined as follows:

1) The total time during which the pH is outside the range of 6.5 - 8.5 pH units shall not exceed 7 hours and 26 minutes in any calendar month; and
2) No individual excursion from the range of 6.5 – 8.5 pH units shall exceed 60 minutes.

## 2. Effluent Limitations for Discharges Under Conditions With 20:1 or More Dilution in the Receiving Water

Whenever the flow[5] in Santa Ana River, as measured at monitoring location R-001U results in a dilution of 20:1 (receiving water flow : wastewater flow) or more, the Discharger shall comply with the following effluent limitations in lieu of those specified in Effluent Limitation IV.A.1., above :

a. Physical/Biological Limitations:

**Table 7 - Effluent Limitations With 20:1 Dilution**

| Parameter | Units | Effluent Limitations | |
|---|---|---|---|
| | | Average Monthly | Average Weekly |
| Biochemical Oxygen Demand 5-day @ 20°C | mg/L | 30 | 45 |
| Total Suspended Solids | mg/L | 30 | 45 |

b. Percent Removal:

The average monthly percent removal of BOD 5-day at 20°C and total suspended solids shall not be less than 85 percent.

c. Coliform:

The weekly average number of coliform bacteria does not exceed a median of 23 per 100 milliliters as determined from the daily coliform bacteria values for the last seven (7) days. To comply with the limit, the 7-day median MPN must not exceed 23 per 100 milliliters on any day during the week. However, only one violation is recorded for each calendar week, even if the 7-day median MPN value is greater than 23 for more than one day in the week.

d. pH:

The pH of the discharge shall be within 6.5 to 8.5 pH units at all times. Compliance with pH limits shall be determined as follows:

1) The total time during which the pH is outside the range of 6.5 - 8.5 pH units shall not exceed 7 hours and 26 minutes in any calendar month; and
2) No individual excursion from the range of 6.5 – 8.5 pH units shall exceed 60 minutes.

[5] Exclusive of discharges to surface waters from upstream publicly owned treatment works.

### 3. Toxicity Requirements

Compliance with toxicity requirements shall be measured at monitoring location M-001A (including Van Buren Bridge location) or M-002.

a. There shall be no acute or chronic toxicity in the plant effluent nor shall the plant effluent cause any acute or chronic toxicity in the receiving water. All waters shall be maintained free of toxic substances in concentrations which are toxic to, or which produce detrimental physiological responses in human, plant, animal, or indigenous aquatic life. This Order contains no numeric limitation for toxicity. However, the Discharger shall conduct chronic toxicity monitoring.

b. The Discharger shall implement the accelerated monitoring as specified in Attachment E when the result of any single chronic toxicity test of the effluent exceeds 1.0 TUc.

c. The Discharger shall implement the IITRE work plan required by Provision VI.C.2.c., below whenever the results of chronic toxicity tests of the effluent exceed:

1) A two month median value of 1.0 TUc for survival or reproduction endpoint or,
2) Any single test value of 1.7 TUc for survival endpoint.

## B. Recycled Water Specifications – Discharge Points 003 and 004

1. The Discharger shall comply with the following limitations for the production of recycled water.

a. Physical/Biological Limitations:

**Table 8 - Recycled Water Limitations**

| Parameter | Units | Recycled Water Limitations | |
|---|---|---|---|
| | | Average Monthly | Average Weekly |
| Biochemical Oxygen Demand 5-day @ 20°C | mg/L | 20 | 30 |
| Total Suspended Solids | mg/L | 20 | 30 |

b. Total Dissolved Solids:

1). The 12-month flow weighted running average total dissolved solids concentration of the discharge shall not exceed 650 mg/l unless the Discharger demonstrates to the satisfaction of the Regional Water Board's Executive Officer that:

a) Discharges in excess of the TDS limits are due to the quality of water supply sources utilized in the Discharger's service area, and that all reasonable steps, as agreed upon by the Executive Officer, have been taken to ensure that the best quality supplies are obtained and utilized in the Discharger's service area; and/or

b) Discharges in excess of the TDS limits are due solely to chemical additions in the treatment process needed to meet waste discharge requirements, and the Discharger has taken all reasonable steps to optimize chemical additions so as to minimize the increases; and

c) The Discharger implements an acceptable plan to offset discharges in excess of the TDS limit.

2). The 12-month flow weighted running average total dissolved solids concentration of the discharge shall not exceed the 12-month flow weighted running average total dissolved solids concentration in the water supply by more than 250 mg/L unless the Discharger demonstrates to the satisfaction of the Regional Water Board's Executive Officer that TDS discharges in excess of the 250 mg/L mineral increment are due solely to chemical additions in the treatment process needed to meet waste discharge requirements, and the Discharger has taken all reasonable steps to optimize chemical additions so as to minimize the TDS increases.

c. Tertiary Treated Recycled Water:

Recycled water, as described in Section 60307(a) of Division 4, Chapter 3, Title 22, California Code of Regulations, supplied for irrigation of food crops, parks and playground, school yards, residential landscaping and other irrigation uses not specified in Section 60304(a) of Division 4, Chapter 3, Title 22, California Code of Regulations or not prohibited in other Sections of the California Code of Regulations shall at all times be adequately oxidized, filtered, and disinfected tertiary treated wastewater and shall meet the following limitations:

1) Turbidity:

a) When filtration is through natural undisturbed soils or a bed of filter media, the turbidity of the filter effluent shall not exceed any of the following:

i. Average of 2 Nephelometric Turbidity Unit (NTU) within any 24-hour period;

ii. 5 NTU more than 5 percent of the time in any 24-hour period; and
iii. 10 NTU more than one minute.

b) When filtration is through microfiltration, the turbidity of the filter effluent shall not exceed any of the following:
   i. 0.2 Nephelometric Turbidity Unit (NTU) more than 5 % of the time within a 24-hour period; and
   ii. 1) 0.5 NTU at any time

2) Disinfection: The discharge shall meet the following:

a. When 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.

b. When a disinfection process combined with the filtration process is utilized, the combined process shall demonstrate inactivation and/or removal of 99.999 percent of the plaque-forming 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.

c. Where ultraviolet (UV) disinfection is solely used for disinfection, UV disinfection shall meet the requirements specified in the Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse, published by the National Water Research Institute, Second Edition, unless otherwise approved by the CDPH. The Discharger's UV disinfection system shall not be utilized as the sole source of disinfection except under emergency conditions unless it is re-commissioned under CDPH oversight.

3) <u>Coliform</u>:

The disinfected wastewater shall meet the following:

a) The average weekly[6] number of total coliform shall not exceed a Most Probable Number (MPN) of 2.2 total coliform bacteria per 100 milliliters (ml).

b) The number of total coliform organism shall not exceed an MPN of 23 total coliform bacteria per 100 ml in more than one sample in any 30-day period.

[6] To comply with the average weekly limit, the 7-day median MPN must not exceed 2.2 per 100 milliliters on any day during the week. However, only one violation is recorded for each calendar week, even if the 7-day median MPN value is greater than 2.2 for more than one day in the week.

c) No total coliform sample shall exceed an MPN of 240 total coliform bacteria per 100 ml.

d. Recycled water used for irrigation of food crops where the edible portion is produced above ground and not contacted by the recycled water shall at all times be adequately oxidized and disinfected so that the average weekly[6] concentration of total coliform bacteria in the disinfected effluent does not exceed a most probable number (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 during any 30-day period.

e. Recycled water supplied for the uses listed below shall be an oxidized and disinfected water so that:

1) The average weekly[6] total coliform bacteria in the disinfected effluent does not exceed a most probable number (MPN) of 23 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses have been completed. To comply with the limit, the 7-day median MPN must not exceed 23 per 100 milliliters on any day during the week. However, only one violation is recorded for each calendar week, even if the average weekly[6] MPN value is greater than 23 for more than one day in the week, and

2) The number of total coliform bacteria does not exceed an MPN of 240 per 100 milliliters in more than one sample in any 30 day period.

3) The uses are:

a) Industrial boiler feed, nonstructural fire-fighting, backfill consolidation around nonpotable piping, soil compaction, mixing concrete, dust control on roads and streets, cleaning roads, sidewalks and outdoor work areas and industrial process water that will not come into contact with workers.

b) Irrigation of cemeteries, freeway landscaping, restricted access golf courses, ornamental nursery stock and sod farms where access by the general public is restricted, pasture for animals producing milk for human consumption, and any nonedible vegetation where access is controlled so that irrigated area cannot be used as if it were part of a park, playground or school yard.

2. The Discharger shall be responsible for assuring that recycled water is delivered and utilized in conformance with this Order, the recycling criteria contained in Title 22, Division 4, Chapter 3, Sections 60301 through 60355, California Code of Regulations. The Discharger shall conduct periodic inspections of the facilities of the recycled water users to monitor compliance by the users with this Order.

3. The storage, delivery, or use of recycled water shall not individually or collectively, directly or indirectly, result in a pollution or nuisance, or adversely affect water quality, as defined in the California Water Code.

4. Prior to delivering recycled water to any new user, the Discharger shall submit to the Regional Water Board and CDPH a report containing the following information for review and approval:

   a. The specific boundaries of the proposed use site area, including a map showing the location of each facility, drinking water fountain, and impoundment to be used.

   b. The average number of persons estimated to be served at each use site area on a daily basis.

   c. The person or persons responsible for operation of the recycled water system at each use area.

   d. The specific use to be made of the recycled water at each use area.

   e. The methods to be used to assure that the installation and operation of the recycled system will not result in cross connections between the recycled water and potable water piping systems. This shall include a description of the pressure, dye or other test methods to be used to test the system.

   f. Plans and specifications which include following:
      1. Proposed piping system to be used.
      2. Pipe locations of both the recycled and potable systems.
      3. Type and location of the outlets and plumbing fixtures that will be accessible to the public.
      4. The methods and devices to be used to prevent backflow of recycled water into the potable water system.
      5. Plan notes relating to specific installation and use requirements.

5. The Discharger shall establish and enforce Rules and Regulations for Recycled Water users, governing the design and construction of recycled water use facilities and the use of recycled water in accordance with the uniform statewide recycling criteria established pursuant to the California Water Code Section 13521.

   a. Use of recycled water by the Discharger shall be consistent with its Rules and Regulations for Recycled Water Use.

   b. Any revisions made to the Rules and Regulations shall be subject to the review and approval of the Regional Water Board and CDPH. The revised Rules and Regulations or a letter certifying that the Discharger's Rules and Regulations are in conformance with this Order, shall be submitted to the Regional Water Board within 60 days of adoption of this Order.

6. The Discharger shall submit a Title 22 Engineering Report for review and approval by the Regional Water Board and CDPH before making any changes to treatment processes. The Engineering Report shall be in compliance with the California Code of Regulations, Title 22, Chapter 3.

## V. RECEIVING WATER LIMITATIONS AND SPECIFICATIONS

### A. Surface Water Limitations

1. Receiving water limitations are based upon water quality objectives contained in the Basin Plan. As such, they are a required part of this Order. The discharge shall not cause the following in the Santa Ana River, Reach 3, or in downstream Reaches of the Santa Ana River:

   a. Coloration of the receiving waters, which causes a nuisance or adversely affects beneficial uses.
   b. Deposition of oil, grease, wax or other materials in the receiving waters in concentrations which result in a visible film or in coating objects in the water, or which cause a nuisance or affect beneficial uses.
   c. An increase in the amounts of suspended or settleable solids in the receiving waters, which will cause a nuisance or adversely affect beneficial uses as a result of controllable water quality factors.
   d. Taste or odor-producing substances in the receiving waters at concentrations, which cause a nuisance or adversely affect beneficial uses.
   e. The presence of radioactive materials in the receiving waters in concentrations, which are deleterious to human, plant or animal life.
   f. The depletion of the dissolved oxygen concentration below 5.0 mg/l.
   g. The temperature of the receiving waters to be raised above 90°F (32°C) during the period of June through October, or above 78°F (26°C) during the rest of the year.
   h. The concentration of pollutants in the water column, sediments, or biota to adversely affect the beneficial uses of the receiving water. The discharge shall not result in the degradation of inland surface water communities and populations, including vertebrate, invertebrate, and plant species.

2. The discharge of wastes shall not cause a violation of any applicable water quality standards for receiving waters adopted by the Regional Water Board or State Water Board, as required by the Clean Water Act and regulations adopted thereunder.

3. Pollutants not specifically mentioned and limited in this Order shall not be discharged at levels that will bioaccumulate in aquatic resources to levels, which are harmful to human health.

4. The discharge shall not contain constituent concentrations of mercury that will result in the bioaccumulation of methylmercury in fish flesh tissue greater than 0.3 milligram methylmercury/kilogram. (See also Section VI.C.1.b. and VI.C.2., below).

### B. Groundwater Limitations

1. The discharge of waste or the use of recycled water shall not cause the underlying groundwater to be degraded, to exceed water quality objectives, unreasonably affect beneficial uses, or cause a condition of pollution or nuisance.

## VI. PROVISIONS

### A. Standard Provisions

1. The Discharger shall comply with all Standard Provisions included in Attachment D of this Order.

2. The Discharger shall comply with the following provisions:

   a. Neither the treatment nor the discharge of waste shall create, or threaten to create, a nuisance or pollution as defined by Section 13050 of the California Water Code.

   b. The Discharger shall maintain a copy of this Order at the site so that it is available to site operating personnel at all times. Key operating personnel shall be familiar with its content.

   c. The Discharger shall take all reasonable steps to minimize any adverse impact to receiving waters resulting from noncompliance with any requirements specified in this Order, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

   d. The Discharger shall optimize chemical additions needed in the treatment process to meet waste discharge requirements so as to minimize total dissolved solid increases in the treated wastewater.

   e. The provisions of this Order are severable, and if any provision of this Order, or the application of any provisions of this Order to any circumstance, is held invalid, the application of such provision to other circumstances, and the remainder of this Order shall not be affected thereby.

   f. Collected screenings, sludge, and other solids removed from liquid wastes shall be disposed of in a manner approved by the Regional Water Board's Executive Officer.

g. If the Discharger demonstrates a correlation between the biological oxygen demand ($BOD_5$) and total organic carbon (TOC) concentrations in the effluent to the satisfaction of the Executive Officer, compliance with the $BOD_5$ limits contained in this Order may be determined based on analyses of the TOC of the effluent.

h. In the event of any change in control or ownership of land or waste discharge facility presently owned or controlled by the Discharger, the Discharger shall notify the succeeding owner or operator of the existence of this Order by letter, a copy of which shall be forwarded to the Regional Water Board.

i. By September 1, 2017, the treatment facilities shall be designed, constructed, operated, and maintained to prevent inundation or washout due to floods with a 100-year return frequency. The Discharger shall provide progress reports regarding its efforts to achieve compliance with this provision in the monitoring reports that are due in January and July of each year until compliance is achieved.

j. Failure to comply with provisions or requirements of this Order, or violation of other applicable laws or regulations governing discharges from this facility, may subject the Discharger to administrative or civil liabilities, criminal penalties, and/or other enforcement remedies to ensure compliance. Additionally, certain violations may subject the Discharger to civil or criminal enforcement from appropriate local, state, or federal law enforcement entities.

k. In the event the Discharger does not comply or will be unable to comply for any reason, with any prohibition, maximum daily effluent limitation, or receiving water limitation of this Order, the Discharger shall notify the Regional Water Board. All noncompliance that may have an immediate impact on human health or the environment shall be reported by telephone (951) 782-4130 within 24 hours of having knowledge of such noncompliance, and shall confirm this notification in writing within five days, unless the Regional Water Board waives confirmation. The written notification shall state the nature, time, duration, and cause of noncompliance, and shall describe the measures being taken to remedy the current noncompliance and, prevent recurrence including, where applicable, a schedule of implementation. Other noncompliance requires written notification as above at the time of the normal monitoring report.

l. This Order may be modified, revoked and reissued, or terminated for cause including, but not limited to, the following.

1) Violation of any terms or conditions of this Order;
2) Obtaining this Order by misrepresentation or failure to disclose fully all relevant facts, or;
3) In addition to any other grounds specified herein, this Order may be modified or revoked at any time if, on the basis of any data, the Regional Water Board determines that continued discharges may cause unreasonable degradation of water quality.

m. If an effluent standard or discharge prohibition (including any schedule of compliance specified in such effluent standard or prohibition) is established under Section 307 (a) of the Clean Water Act for a toxic pollutant which is present in the discharge, and such standard or prohibition is more stringent than any limitation for that pollutant in this Order, this Order may be modified or revoked and reissued to conform to the effluent standard or discharge prohibition.

n. The Discharger shall file with the Regional Water Board a Report of Waste Discharge at least 180 days before making any material change in the character, location, or volume of the discharge. A material change includes, but is not limited to, the following:

1) Adding a major industrial waste discharge to a discharge of essentially domestic sewage, or adding a new process or product by an industrial facility resulting in a change in the character of the waste.
2) Significantly changing the disposal method or location, such as changing the disposal to another drainage area or water body.
3) Significantly changing the method of treatment.
4) Increasing the treatment plant design capacity beyond that specified in this Order.

## B. Monitoring and Reporting Program Requirements

The Discharger shall comply with the Monitoring and Reporting Program, and future revisions thereto, in Attachment E of this Order. This monitoring and reporting program may be modified by the Executive Officer at any time during the term of this Order, and may include an increase in the number of parameters to be monitored, the frequency of the monitoring or the number and size of samples to be collected. Any increase in the number of parameters to be monitored, the frequency of the monitoring or the number and size of samples to be collected may be reduced back to the levels specified in the original monitoring and reporting program at the discretion of the Executive Officer.

## C. Special Provisions

### 1. Reopener Provisions

a. This Order may be reopened to address any changes in State or federal plans, policies or regulations that would affect the quality requirements for the discharges.

b. This Order may be reopened to include effluent limitations for pollutants determined to be present in the discharge in concentrations that pose a reasonable potential to cause or contribute to violations of water quality objectives.

c. This Order may be reopened and modified in accordance with the requirements set forth at 40 CFR 122 and 124, to include the appropriate conditions or limits to address demonstrated effluent toxicity based on newly available information, or to implement any EPA-approved new State water quality standards applicable to effluent toxicity.

d. This Order may be reopened for modification, or revocation and reissuance, as a result of the detection of a reportable priority pollutant generated by special conditions included in this Order. These special conditions may be, but are not limited to, fish tissue sampling, whole effluent toxicity, monitoring requirements on internal waste stream(s), and monitoring for surrogate parameters. Additional requirements may be included in this Order as a result of the special condition monitoring data.

e. This Order may be reopened to include an appropriate bioaccumulation based effluent limit for mercury if test results (as required in Attachment E of this Order) show that the concentration levels of methylmercury in the fish tissue are at or above 0.3 milligrams per kilogram.

f. This Order may be reopened to incorporate appropriate biosolids requirements if the State Water Resources Control Board and the Regional Water Quality Control Board are given the authority to implement regulations contained in 40 CFR 503.

### 2. Special Studies, Technical Reports and Additional Monitoring Requirements

a. The Discharger is an active participant in the comprehensive mercury investigation program currently being conducted by a group of Santa Ana River system dischargers. If the Discharger discontinues its involvement with this comprehensive program, the Discharger shall, within 60 days of that date, submit for the approval of the Executive Officer its plan for the annual testing of mercury levels in fish flesh samples collected from the Santa Ana River, upstream of, at, and downstream of the point of the discharge point. Upon approval, the Discharger shall implement the plan.

b. Within 60-days of the effective date of this Order, the Discharger shall review and update, if necessary, its report that details the manner in which sampling, monitoring and reporting will be performed as required in the Order.

c. Toxicity Reduction Requirements.

1) Within 60-days of the effective date of this Order, the Discharger shall review and update, if necessary, its Initial Investigation Toxicity Reduction Evaluation (IITRE) work plan that describes the steps the Discharger intends to follow if required by subsection 2), below. The work plan shall include at a minimum:

a) A description of the investigation and evaluation techniques that will be used to identify potential causes/sources of the exceedance, effluent variability, and/or efficiency of the treatment system in removing toxic substances. This shall include a description of an accelerated chronic toxicity testing program.
b) A description of the methods to be used for investigating and maximizing in-house treatment efficiency and good housekeeping practices.
c) A description of the evaluation process to be used to determine if implementation of a more detailed TRE\TIE is necessary.

2) The Discharger shall implement the IITRE work plan whenever the results of chronic toxicity tests of the effluent exceed:

a) A two month median value of 1.0 TUc for survival or reproduction endpoint or,
b) Any single test value of 1.7 TUc for survival endpoint.

3) Within 60-days of the effective date of this Order, the Discharger shall review and update, if necessary, its detailed Toxicity Reduction Evaluation and Toxicity Identification Evaluation (TRE/TIE) work plan that describes the steps the Discharger intends to follow if the implemented IITRE fails to identify the cause of, or to rectify, the toxicity.

4) The Discharger shall use as guidance, at a minimum, EPA manuals EPA/600/2-88/070 (industrial), EPA/600/4-89-001A (municipal), EPA/600/6-91/005F (Phase I), EPA/600/R-92/080 (Phase II), and EPA-600/R-92/081 (Phase III) to identify the cause(s) of toxicity. If during the life of this Order the aforementioned EPA manuals are revised or updated, the revised/updated manuals may also be used as guidance. The detailed TRE/TIE work plan shall include:

a) Further actions to investigate and identify the cause of toxicity;

b) Actions the Discharger will take to mitigate the impact of the discharge and to prevent the recurrence of toxicity; and

c) A schedule for these actions.

5) The Discharger shall implement the TRE/TIE workplan if the IITRE fails to identify the cause of, or rectify, the toxicity, or if in the opinion of the Executive Officer the IITRE does not adequately address an identified toxicity problem.

6) The Discharger shall assure that adequate resources are available to implement the required TRE/TIE.

**3. Best Management Practices and Pollution Prevention**

a. Pollutant Minimization Program

1) The Discharger shall develop and conduct a Pollutant Minimization Program (PMP) as further described below when there is evidence (e.g., sample results reported as DNQ when the effluent limitation is less than the MDL, sample results from analytical methods more sensitive than those methods required by this Order, presence of whole effluent toxicity, health advisories for fish consumption, results of benthic or aquatic organism tissue sampling) that a priority pollutant is present in the effluent above an effluent limitation and either:

a) A sample result is reported as DNQ and the effluent limitation is less than the RL; or

b) A sample result is reported as ND and the effluent limitation is less than the MDL.

2) The PMP shall include, but not be limited to, the following actions and submittals acceptable to the Regional Water Board:

a) An annual review and semi-annual monitoring of potential sources of the reportable priority pollutant(s), which may include fish tissue monitoring and other bio-uptake sampling;

b) Quarterly monitoring for the reportable priority pollutant(s) in the influent to the wastewater treatment system;

c) Submittal of a control strategy designed to proceed toward the goal of maintaining concentrations of the reportable priority pollutant(s) in the effluent at or below the effluent limitation;

d) Implementation of appropriate cost-effective control measures for the reportable priority pollutant(s), consistent with the control strategy; and

e) An annual status report that shall be sent to the Regional Water Board including:

(1) All PMP monitoring results for the previous year;
(2) A list of potential sources of the reportable priority pollutant(s);
(3) A summary of all actions undertaken pursuant to the control strategy; and
(4) A description of actions to be taken in the following year.

**4. Construction, Operation and Maintenance Specifications**

a. The Discharger's wastewater treatment plant shall be supervised and operated by persons possessing certificates of appropriate grade pursuant to Title 23, Division 3, Chapter 14, California Code of Regulations.

b. The Discharger shall provide safeguards to assure that should there be reduction, loss, or failure of electric power, the Discharger will comply with the requirements of this Order.

c. The Discharger shall update as necessary, the "Operation and Maintenance Manual (O&M Manual)" which it has developed for the treatment facility to conform to latest plant changes and requirements. The O&M Manual shall be readily available to operating personnel onsite. The O&M Manual shall include the following:

1) Description of the treatment plant table of organization showing the number of employees, duties and qualifications and plant attendance schedules (daily, weekends and holidays, part-time, etc). The description should include documentation that the personnel are knowledgeable and qualified to operate the treatment facility so as to achieve the required level of treatment at all times.
2) Detailed description of safe and effective operation and maintenance of treatment processes, process control instrumentation and equipment.
3) Description of laboratory and quality assurance procedures.
4) Process and equipment inspection and maintenance schedules.
5) Description of safeguards to assure that, should there be reduction, loss, or failure of electric power, the Discharger will be able to comply with requirements of this Order.
6) Description of preventive (fail-safe) and contingency (response and cleanup) plans for controlling accidental discharges, and for minimizing the effect of such events. These plans shall identify the possible sources (such as loading and storage areas, power outage, waste treatment unit failure, process equipment failure, tank and piping failure) of accidental discharges, untreated or partially treated waste bypass, and polluted drainage.

## 5. Special Provisions for Municipal Facilities (POTWs Only)

a. Sewer Collection System Requirements: The Discharger's collection system is part of the system that is subject to this Order. As such, the Discharger must properly operate and maintain its collection system (40 CFR 122.41(e)). The Discharger must report any non-compliance (40 CFR 122.41(l)(6) and (7)) and mitigate any discharge from the collection system in violation of this Order (40 CFR 122.41(d)). See the Order at Standard Provision VI.A.2.b. and Attachment D, sections I.D, V.E, V.H, and I.C.

   Furthermore, the General Waste Discharge Requirements for Collection System Agencies (Order No. 2006-0003 DWQ) contains requirements for operation and maintenance of collection systems and for reporting and mitigating sanitary sewer overflows. While the Discharger must comply with both Order No. 2006-0003 DWQ and this Order, the General Collection System WDR more clearly and specifically stipulates requirements for operation and maintenance and for reporting and mitigating sanitary sewer overflows. The Discharger and other governmental agencies that are discharging wastewater into the facility are required to obtain enrollment for regulation under Order No. 2006-0003-DWQ.

b. Biosolids Disposal Requirements

   1) Collected screenings, sludge, and other solids removed from liquid wastes shall be disposed of in a manner that is consistent with State Water Resources Control Board and Integrated Waste Management Board's joint regulations (Title 27) of the California Code of Regulations and approved by the Water Board's Executive Officer.

   2) The use and disposal of biosolids shall comply with existing Federal and State laws and regulations, including permitting requirements and technical standards included in 40 CFR 503.

   3) Any proposed change in biosolids use or disposal practice from a previously approved practice should be reported to the Executive Officer and EPA Regional Administrator at least 90 days in advance of the change.

   4) The Discharger shall take all reasonable steps to minimize or prevent any discharge or biosolids use or disposal that has the potential of adversely affecting human health or the environment.

c. Pretreatment Program

1) The Discharger shall update as necessary and implement an acceptable pretreatment program.

2) The Discharger shall update as necessary the appropriate contractual agreements with all governmental agencies[7]. The contractual agreements shall give the Discharger the authority to implement and enforce the EPA approved pretreatment program within the sewer service areas of the treatment facility. The Discharger shall assure that any other steps necessary to provide this implementation and enforcement authority (e.g. adoption of ordinances, etc.) are taken by all governmental agencies. If a governmental agency has an EPA approved pretreatment program for any portion of the service area of the treatment facility, the Discharger's pretreatment program shall contain provisions ensuring that that governmental agency's program is implemented. In the event that any agency discharging to Discharger's facility fails to effectively implement its individual EPA approved pretreatment program, the Discharger shall implement and enforce its approved program within that agency's service area.

3) The Discharger shall ensure that the pretreatment programs for all contributory agencies discharging to the Discharger's treatment facility are implemented and enforced. The Discharger shall be responsible and liable for the performance of all Control Authority pretreatment requirements contained in 40 CFR 403, including any subsequent regulatory revisions to Part 403. Where Part 403 or subsequent revisions place mandatory actions upon the Discharger as Control Authority but does not specify a timetable for completion of the actions, the Discharger shall submit for approval of the Regional Water Board's Executive Officer, a schedule for implementation of the required actions and shall implement the approved schedule. The schedule for implementation shall be submitted within six months from the date that such mandatory actions are established. For violations of pretreatment requirements, the Discharger shall be subject to enforcement actions, penalties, fines and other remedies by the EPA, or other appropriate parties, as provided in the CWA, as amended (33 USC 1351 et seq.). The EPA or the Regional Water Board may also initiate enforcement action against an industrial user (IU) for non-compliance with applicable standards and requirements as provided in the CWA.

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7 *Member agencies and sewering agencies discharging wastewater into the facility.*

4) The Discharger shall perform the pretreatment functions as required in 40 CFR Part 403 including, but not limited to:

   a) Enforce the pretreatment requirements under 40 CFR 403.5 and 403.6;

   b) Implement the necessary legal authorities as provided in 40 CFR 403.8(f)(1);

   c) Implement the programmatic functions as provided in 40 CFR 403.8(f)(2);

   d) Publish a list of significant non-compliance as required by 40 CFR 403.8(f)(2)(vii); and

   e) Provide the requisite funding and personnel to implement the pretreatment program as provided in 40 CFR 403.8(f)(3).

5) The following wastes shall not be introduced into the treatment works:

   a) Wastes which create a fire or explosion hazard in the treatment works;

   b) Wastes which will cause corrosive structural damage to treatment works, but, in no case, wastes with a pH lower than 5.0 unless the works are designed to accommodate such wastes;

   c) Wastes at a flow rate and/or pollutant discharge rate which is excessive over relatively short time periods so that there is a treatment process upset and subsequent loss of treatment efficiency;

   d) Solid or viscous wastes in amounts that would cause obstruction to the flow in sewers or otherwise interfere with the proper operation of the treatment works.

6) The Discharger shall ensure compliance with any existing or future pretreatment standard promulgated by EPA under Section 307 of the CWA or amendments thereto for any discharge to the municipal system.

7) The Discharger shall comply with effluent standards or prohibitions established under Section 307(a) of the CWA for toxic pollutants within the time provided in the regulations that establish these standards or prohibitions, even if this Order has not yet been modified to incorporate the requirement.

8) The Discharger shall require each user not in compliance with any pretreatment standard to submit periodic notice (over intervals not to exceed nine months) of progress toward compliance with applicable toxic and pretreatment standards developed pursuant to the CWA or amendments thereto. The Discharger shall forward a copy of such notice to the Regional Water Board and to the EPA Regional Administrator.

## II. COMPLIANCE DETERMINATION

Compliance with the effluent limitations contained in Section IV of this Order will be determined as specified below:

**A. General.**
Compliance with effluent limitations for priority pollutants shall be determined using sample reporting protocols defined in the MRP and Appendix A of this Order. For purposes of reporting and administrative enforcement by the Regional and State Water Boards, Dischargers shall be deemed out of compliance with effluent limitations if the concentration of the priority pollutant in the monitoring sample is greater than the effluent limitation and greater than or equal to the minimum level (ML).

**B. Average Monthly Effluent Limitation (AMEL).**
If the average (or when applicable, the median determined by subsection B above for multiple sample data) of daily discharges over a calendar month exceeds the AMEL for a given parameter, this will represent a single violation, though the Discharger may be considered out of compliance for each day of that month for that parameter (e.g., resulting in 31 days of non-compliance in a 31-day month). If only a single sample is taken during the calendar month and the analytical result for that sample exceeds the AMEL, the Discharger may be considered out of compliance for that calendar month. The Discharger will only be considered out of compliance for days when the discharge occurs. For any one calendar month during which no sample (daily discharge) is taken, no compliance determination can be made for that calendar month.

**C. Average Weekly Effluent Limitation (AWEL).**
If the average (or when applicable, the median determined by subsection B above for multiple sample data) of daily discharges over a calendar week exceeds the AWEL for a given parameter, this will represent a single violation, though the Discharger may be considered out of compliance for each day of that week for that parameter, resulting in 7 days of non-compliance. If only a single sample is taken during the calendar week and the analytical result for that sample exceeds the AWEL, the Discharger may be considered out of compliance for that calendar week. The Discharger will only be considered out of compliance for days when the discharge occurs. For any one calendar week during which no sample (daily discharge) is taken, no compliance determination can be made for that calendar week.

**D. Maximum Daily Effluent Limitation (MDEL).**
If a daily discharge exceeds the MDEL for a given parameter, the Discharger will be considered out of compliance for that parameter for that 1 day only within the reporting period. For any 1 day during which no sample is taken, no compliance determination can be made for that day.

**E. Instantaneous Minimum Effluent Limitation.**
If the analytical result of a single grab sample is lower than the instantaneous minimum effluent limitation for a parameter, the Discharger will be considered out of compliance for that parameter for that single sample. Non-compliance for each sample will be considered separately (e.g., the results of two grab samples taken within a calendar day that both are lower than the instantaneous minimum effluent limitation would result in two instances of non-compliance with the instantaneous minimum effluent limitation).

**F. Instantaneous Maximum Effluent Limitation.**
If the analytical result of a single grab sample is higher than the instantaneous maximum effluent limitation for a parameter, the Discharger will be considered out of compliance for that parameter for that single sample. Non-compliance for each sample will be considered separately (e.g., the results of two grab samples taken within a calendar day that both exceed the instantaneous maximum effluent limitation would result in two instances of non-compliance with the instantaneous maximum effluent limitation).

**G. Multiple Sample Data.**
When determining compliance with an AMEL for priority pollutants and more than one sample result is available in a month, the Discharger shall compute the arithmetic mean unless the data set contains one or more reported determinations of "Detected, but Not Quantified" (DNQ) or "Not Detected" (ND). In those cases, the Discharger shall compute the median in place of the arithmetic mean in accordance with the following procedure:

1. The data set shall be ranked from low to high, ranking the reported ND determinations lowest, DNQ determinations next, followed by quantified values (if any). The order of the individual ND or DNQ determinations is unimportant.

2. The median value of the data set shall be determined. If the data set has an odd number of data points, then the median is the middle value. If the data set has an even number of data points, then the median is the average of the two values around the middle unless one or both of the points are ND or DNQ, in which case the median value shall be the lower of the two data points where DNQ is lower than a value and ND is lower than DNQ.

**H. 12-Month Running Average Effluent Limitation.**
Compliance with the 12-month running average limit under Effluent Limitations and Discharge Specification IV.A.1.c., IV.A.1.d., and IV.B.1.b. shall be determined by the arithmetic mean of the last twelve monthly averages.

**I. TDS Increment Limit.**
Compliance with Effluent Limitations and Discharge Specifications IV.A.1.c.2., and IV.B.1.b. may be determined by comparing the flow weighted TDS quality of the influent or secondary effluent to the flow weighted TDS water supply quality.

**J. Priority Pollutants.**

The Discharger shall be deemed out of compliance with an effluent limitation if the concentration of the priority pollutant in the monitoring sample is greater than the effluent limitation. Compliance determination shall be based on the reporting level selected from minimum level (ML)[8] specified in Attachment "I" of this Order, unless an alternative reporting level is approved by the Regional Water Board's Executive Officer. When there is more than one ML value for a given substance, the Discharger shall select the ML value that is below the calculated effluent limitation, and use its associated analytical method, listed in Attachment "I" of this Order. If no ML value is below the effluent limitation, then the Regional Water Board will select as the reporting level the lowest ML value and its associated analytical method.

**K. Non-Priority Pollutants.**

The discharge shall be considered to be in compliance with an effluent limitation that is less than or equal to the PQL specified in Attachment H of this Order if the arithmetic mean of all test results for the monitoring period is less than the constituent effluent limitation. Analytical results that are less than the specified PQL shall be assigned a value of zero.

**L. Compliance Determination**

Compliance determinations shall be based on available analyses for the time interval associated with the effluent limitation. Where only one sample analysis is available in a specified time interval (e. g., monthly or weekly average), that sample shall serve to characterize the discharge for the entire interval. If quarterly sample results show noncompliance with the average monthly limit and that sample result is used for compliance determinations for each month of the quarter, then three separate violations of the average monthly limit shall be deemed to have occurred.

Compliance with a single effluent limitation which applies to a group of chemicals (e.g., PCBs), based on a single sample shall be determined by considering the concentrations of individual members of the group to be zero if the analytical response for the individual chemical falls below the method detection limit (MDL or PQL) for that chemical.

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8 *Minimum level is the concentration at which the entire analytical system must give a recognizable signal and acceptable point. The ML is the concentration in a sample that is equivalent to the concentration of the lowest calibration standard analyzed by a specific analytical procedure, assuming that all the method specified sample weights, volumes, and processing steps have been followed.*

## ATTACHMENT A – DEFINITIONS

**Arithmetic Mean ($\mu$),** also called the average, is the sum of measured values divided by the number of samples. For ambient water concentrations, the arithmetic mean is calculated as follows:

Arithmetic mean = $\mu = \Sigma x / n$ where: $\Sigma x$ is the sum of the measured ambient water concentrations, and
$n$ is the number of samples.

**Average Monthly Effluent Limitation (AMEL):** the highest allowable average of daily discharges over a calendar month, calculated as the sum of all daily discharges measured during a calendar month divided by the number of daily discharges measured during that month.

**Average Weekly Effluent Limitation (AWEL):** the highest allowable average of daily discharges over a calendar week (Sunday through Saturday), calculated as the sum of all daily discharges measured during a calendar week divided by the number of daily discharges measured during that week.

**Best Management Practices** (BMPs) are methods, measures, or practices designed and selected to reduce or eliminate the discharge of pollutants to surface waters from point and nonpoint source discharges including storm water. BMPs include structural and non-structural controls, and operation and maintenance procedures, which can be applied before, during, and/or after pollution producing activities.

**Bioaccumulative** pollutants are those substances taken up by an organism from its surrounding medium through gill membranes, epithelial tissue, or from food and subsequently concentrated and retained in the body of the organism.

**Carcinogenic** pollutants are substances that are known to cause cancer in living organisms.

**Coefficient of Variation (*CV*)** is a measure of the data variability and is calculated as the estimated standard deviation divided by the arithmetic mean of the observed values.

**Criteria Continuous Concentration (CCC)** equals the highest concentration of a pollutant to which aquatic life can be exposed for an extended period of time (4 days) without deleterious effects.

**Criteria Maximum Concentration (CMC)** equals the highest concentration of a pollutant to which aquatic life can be exposed for a short period of time without deleterious effects.

**Daily Discharge:** Daily Discharge is defined as either: (1) the total mass of the constituent discharged over the calendar day (12:00 am through 11:59 pm) or any 24-hour period that reasonably represents a calendar day for purposes of sampling (as specified in the permit), for a constituent with limitations expressed in units of mass or; (2) the unweighted arithmetic mean

measurement of the constituent over the day for a constituent with limitations expressed in other units of measurement (e.g., concentration).

The daily discharge may be determined by the analytical results of a composite sample taken over the course of one day (a calendar day or other 24-hour period defined as a day) or by the arithmetic mean of analytical results from one or more grab samples taken over the course of the day.

**Detected, but Not Quantified (DNQ)** are those sample results less than the RL, but greater than or equal to the laboratory's MDL.

**Dilution Credit** is the amount of dilution granted to a discharge in the calculation of a water quality-based effluent limitation, based on the allowance of a specified mixing zone. It is calculated from the dilution ratio or determined through conducting a mixing zone study or modeling of the discharge and receiving water.

**Dilution Ratio** is the critical low flow of the upstream receiving water divided by the flow of the effluent discharged.

**Effluent Concentration Allowance (ECA)** is a value derived from the water quality criterion/objective, dilution credit, and ambient background concentration that is used, in conjunction with the coefficient of variation for the effluent monitoring data, to calculate a long-term average (LTA) discharge concentration. The ECA has the same meaning as waste load allocation (WLA) as used in U.S. EPA guidance (Technical Support Document For Water Quality-based Toxics Control, March 1991, second printing, EPA/505/2-90-001).

**Estimated Chemical Concentration** is the estimated chemical concentration that results from the confirmed detection of the substance by the analytical method below the ML value.

**Existing Discharger** means any discharger that is not a new discharger. An existing discharger includes an "increasing discharger" (i.e., an existing facility with treatment systems in place for its current discharge that is or will be expanding, upgrading, or modifying its existing permitted discharge after the effective date of this Policy).

**Infeasible** means not capable of being accomplished in a successful manner within a reasonable period of time, taking into account economic, environmental, legal, social, and technological factors.

**Inland Surface Waters** are all surface waters of the State that do not include the ocean, enclosed bays, or estuaries.

**Instantaneous Maximum Effluent Limitation:** the highest allowable value for any single grab sample or aliquot (i.e., each grab sample or aliquot is independently compared to the instantaneous maximum limitation).

**Instantaneous Minimum Effluent Limitation:** the lowest allowable value for any single grab sample or aliquot (i.e., each grab sample or aliquot is independently compared to the instantaneous minimum limitation).

**Load Allocation (LA)** is the portion of receiving water's total maximum daily load that is allocated to one of its nonpoint sources of pollution or to natural background sources.

**Maximum Daily Effluent Limitation (MDEL)** means the highest allowable daily discharge of a pollutant, over a calendar day (or 24-hour period). For pollutants with limitations expressed in units of mass, the daily discharge is calculated as the total mass of the pollutant discharged over the day. For pollutants with limitations expressed in other units of measurement, the daily discharge is calculated as the arithmetic mean measurement of the pollutant over the day.

**MEC**: Maximum Effluent Concentration.

**Median** is the middle measurement in a set of data. The median of a set of data is found by first arranging the measurements in order of magnitude (either increasing or decreasing order). If the number of measurements ($n$) is odd, then the median = $X_{(n+1)/2}$. If $n$ is even, then the median = $(X_{n/2} + X_{(n/2)+1})/2$ (i.e., the midpoint between the $n/2$ and $n/2+1$).

**Method Detection Limit (MDL)** is the minimum concentration of a substance that can be measured and reported with 99 percent confidence that the analyte concentration is greater than zero, as defined in 40 CFR 136, Appendix B, revised as of May 14, 1999.

**Minimum Level (ML)** is the concentration at which the entire analytical system must give a recognizable signal and acceptable calibration point. The ML is the concentration in a sample that is equivalent to the concentration of the lowest calibration standard analyzed by a specific analytical procedure, assuming that all the method specified sample weights, volumes, and processing steps have been followed.

**Mixing Zone** is a limited volume of receiving water that is allocated for mixing with a wastewater discharge where water quality criteria can be exceeded without causing adverse effects to the overall water body.

**New Discharger** includes any building, structure, facility, or installation from which there is, or may be, a discharge of pollutants, the construction of which commenced after the effective date of this Policy.

**Not Detected (ND)** are those sample results less than the laboratory's MDL.

**Objectionable Bottom Deposits** are an accumulation of materials or substances on or near the bottom of a water body, which creates conditions that adversely impact aquatic life, human health, beneficial uses, or aesthetics. These conditions include, but are not limited to, the accumulation of pollutants in the sediments and other conditions that result in harm to benthic organisms, production of food chain organisms, or fish egg development. The presence of such deposits shall be determined by RWQCB(s) on a case-by-case basis.

**Persistent** pollutants are substances for which degradation or decomposition in the environment is nonexistent or very slow.

**Pollutant Minimization Program (PMP)** means waste minimization and pollution prevention actions that include, but are not limited to, product substitution, waste stream recycling, alternative waste management methods, and education of the public and businesses. The goal of the PMP shall be to reduce all potential sources of a priority pollutant(s) through pollutant minimization (control) strategies, including pollution prevention measures as appropriate, to maintain the effluent concentration at or below the water quality-based effluent limitation. Pollution prevention measures may be particularly appropriate for persistent bioaccumulative priority pollutants where there is evidence that beneficial uses are being impacted. The Regional Water Board may consider cost-effectiveness when establishing the requirements of a PMP. The completion and implementation of a Pollution Prevention Plan, if required pursuant to CWC Section 13263.3(d), shall be considered to fulfill the PMP requirements.

**Pollution Prevention** means any action that causes a net reduction in the use or generation of a hazardous substance or other pollutant that is discharged into water and includes, but is not limited to, input change, operational improvement, production process change, and product reformulation (as defined in Water Code Section 13263.3). Pollution prevention does not include actions that merely shift a pollutant in wastewater from one environmental medium to another environmental medium, unless clear environmental benefits of such an approach are identified to the satisfaction of the SWRCB or RWQCB.

**Process Optimization** means minor changes to the existing facility and treatment plant operations that optimize the effectiveness of the existing treatment processes.

**Public Entity** includes the federal government or a state, county, city and county, city, district, public authority, or public agency.

**Reporting Level (RL)** is the ML corresponding to an approved analytical method for reporting a sample result that is selected either from Appendix 4 of the SIP by the Regional Water Board in accordance with Section 2.4.2 of the SIP or established in accordance with Section 2.4.3 of the SIP. The ML is based on the proper application of method-based analytical procedures for sample preparation and the absence of any matrix interferences. Other factors may be applied to the ML depending on the specific sample preparation steps employed. For example, the treatment typically applied in cases where there are matrix-effects is to dilute the sample or sample aliquot by a factor of ten. In such cases, this additional factor must be applied to the ML in the computation of the RL.

**Source of Drinking Water** is any water designated as municipal or domestic supply (MUN) in a RWQCB basin plan.

**Standard Deviation ($\sigma$)** is a measure of variability that is calculated as follows:

$$\sigma = (\Sigma[(x - \mu)^2]/(n - 1))^{0.5}$$

where:

x is the observed value;

$\mu$ is the arithmetic mean of the observed values; and

n is the number of samples.

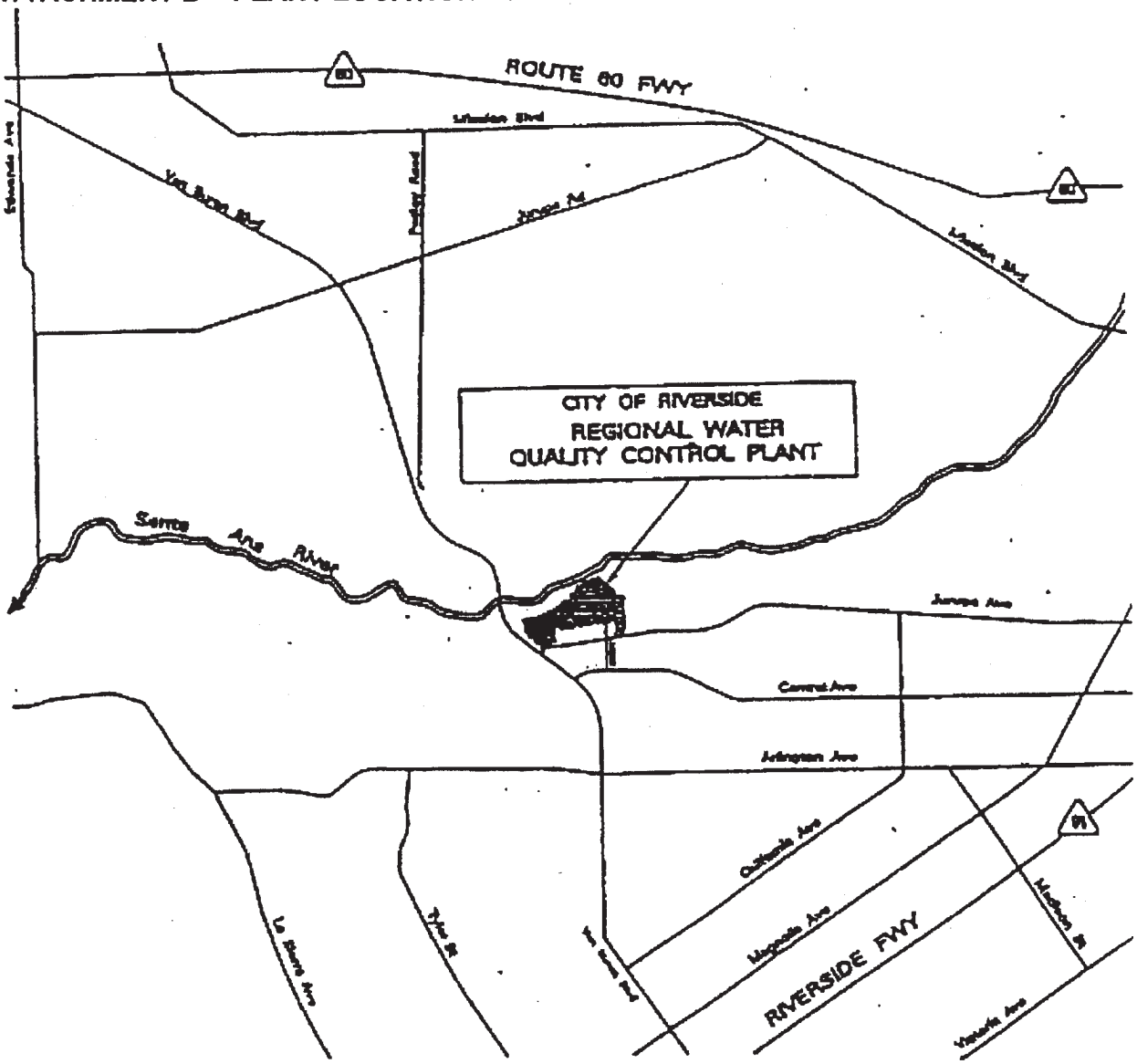
**Toxicity Reduction Evaluation (TRE)** is a study conducted in a step-wise process designed to identify the causative agents of effluent or ambient toxicity, isolate the sources of toxicity, evaluate the effectiveness of toxicity control options, and then confirm the reduction in toxicity. The first steps of the TRE consist of the collection of data relevant to the toxicity, including additional toxicity testing, and an evaluation of facility operations and maintenance practices, and best management practices. A Toxicity Identification Evaluation (TIE) may be required as part of the TRE, if appropriate. (A TIE is a set of procedures to identify the specific chemical(s) responsible for toxicity. These procedures are performed in three phases (characterization, identification, and confirmation) using aquatic organism toxicity tests.)

**Use Attainability Analysis** is a structured scientific assessment of the factors affecting the attainment of the use which may include physical, chemical, biological and economic factors as described in 40 CFR 131.10(g) (40 CFR 131.3, revised as of July 1, 1997).

**Water Effect Ratio (WER)** is an appropriate measure of the toxicity of a material obtained in a site water divided by the same measure of the toxicity of the same material obtained simultaneously in a laboratory dilution water.

**12-Month Running Average Effluent Limitation (12-MRAEL):** the highest allowable average of monthly discharges over last twelve months, calculated as the sum of all monthly discharges measured during last twelve months divided by the number of monthly discharges measured during that time period.

## ATTACHMENT B – PLANT LOCATION MAP

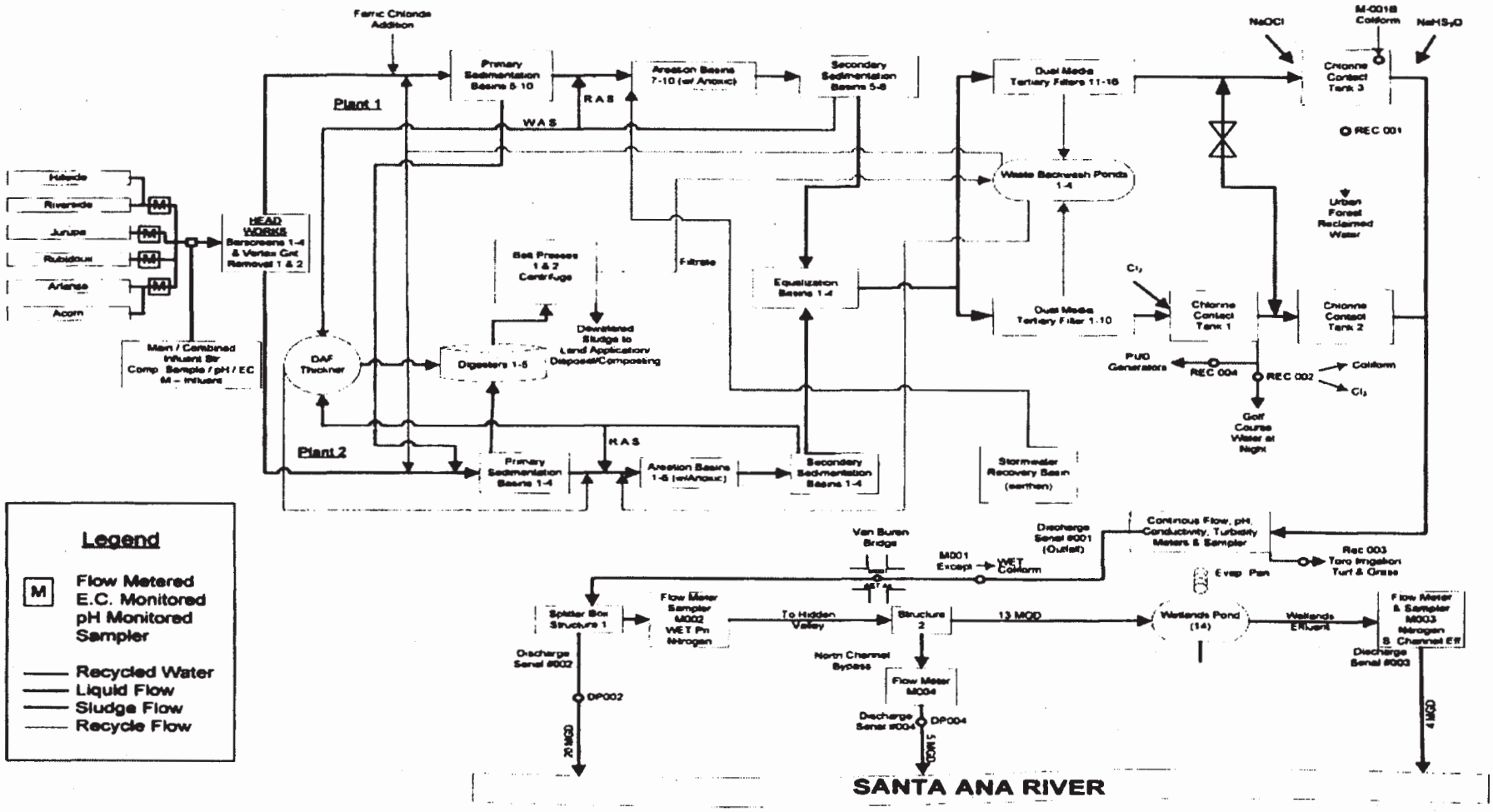



CITY OF RIVERSIDE

REGIONAL WATER
QUALITY CONTROL PLANT

VICINITY MAP

## ATTACHMENT C – FLOW SCHEMATIC



# ATTACHMENT D – STANDARD PROVISIONS

## I. STANDARD PROVISIONS – PERMIT COMPLIANCE

### A. Duty to Comply

1. The Discharger must comply with all of the conditions of this Order. Any noncompliance constitutes a violation of the Clean Water Act (CWA) and the California Water Code (CWC) and is grounds for enforcement action, for permit termination, revocation and reissuance, or modification; or denial of a permit renewal application [*40 CFR §122.41(a)*].

2. The Discharger shall comply with effluent standards or prohibitions established under Section 307(a) of the CWA for toxic pollutants and with standards for sewage sludge use or disposal established under Section 405(d) of the CWA within the time provided in the regulations that establish these standards or prohibitions, even if this Order has not yet been modified to incorporate the requirement [*40 CFR §122.41(a)(1)*].

### B. Need to Halt or Reduce Activity Not a Defense

It shall not be a defense for a Discharger in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this Order [*40 CFR §122.41(c)*].

### C. Duty to Mitigate

The Discharger shall take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this Order that has a reasonable likelihood of adversely affecting human health or the environment [*40 CFR §122.41(d)*].

### D. Proper Operation and Maintenance

The Discharger shall at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) which are installed or used by the Discharger to achieve compliance with the conditions of this Order. Proper operation and maintenance also includes adequate laboratory controls and appropriate quality assurance procedures. This provision requires the operation of backup or auxiliary facilities or similar systems that are installed by a Discharger only when necessary to achieve compliance with the conditions of this Order [*40 CFR §122.41(e)*].

## E. Property Rights

1. This Order does not convey any property rights of any sort or any exclusive privileges [*40 CFR §122.41(g)*].

2. The issuance of this Order does not authorize any injury to persons or property or invasion of other private rights, or any infringement of State or local law or regulations [*40 CFR §122.5(c)*].

## F. Inspection and Entry

The Discharger shall allow the Regional Water Quality Control Board (RWQCB), State Water Resources Control Board (SWRCB), United States Environmental Protection Agency (USEPA), and/or their authorized representatives (including an authorized contractor acting as their representative), upon the presentation of credentials and other documents, as may be required by law, to [*40 CFR §122.41(i)*] [*CWC 13383(c)*]:

1. Enter upon the Discharger's premises where a regulated facility or activity is located or conducted, or where records are kept under the conditions of this Order [*40 CFR §122.41(i)(1)*];

2. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this Order [*40 CFR §122.41(i)(2)*];

3. Inspect and photograph, at reasonable times, any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this Order [*40 CFR §122.41(i)(3)*];

4. Sample or monitor, at reasonable times, for the purposes of assuring Order compliance or as otherwise authorized by the CWA or the CWC, any substances or parameters at any location [*40 CFR §122.41(i)(4)*].

## G. Bypass

1. Definitions

   a. "Bypass" means the intentional diversion of waste streams from any portion of a treatment facility [*40 CFR §122.41(m)(1)(i)*].

   b. "Severe property damage" means substantial physical damage to property, damage to the treatment facilities, which causes them to become inoperable, or substantial and permanent loss of natural resources that can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production [*40 CFR §122.41(m)(1)(ii)*].

2. Bypass not exceeding limitations – The Discharger may allow any bypass to occur which does not cause exceedances of effluent limitations, but only if it is for essential maintenance to assure efficient operation. These bypasses are not subject to the provisions listed in Standard Provisions – Permit Compliance I.G.3, I.G.4, and I.G.5 below [*40 CFR §122.41(m)(2)*].

3. Prohibition of bypass – Bypass is prohibited, and the Regional Water Board may take enforcement action against a Discharger for bypass, unless [*40 CFR §122.41(m)(4)(i)*]:

   a. Bypass was unavoidable to prevent loss of life, personal injury, or severe property damage [*40 CFR §122.41(m)(4)(A)*];

   b. There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate back-up equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass that occurred during normal periods of equipment downtime or preventive maintenance [*40 CFR §122.41(m)(4)(B)*]; and

   c. The Discharger submitted notice to the Regional Water Board as required under Standard Provisions – Permit Compliance I.G.5 below [*40 CFR §122.41(m)(4)(C)*].

4. The Regional Water Board may approve an anticipated bypass, after considering its adverse effects, if the Regional Water Board determines that it will meet the three conditions listed in Standard Provisions – Permit Compliance I.G.3 above [*40 CFR §122.41(m)(4)(ii)*].

5. Notice

   a. Anticipated bypass. If the Discharger knows in advance of the need for a bypass, it shall submit a notice, if possible at least 10 days before the date of the bypass [*40 CFR §122.41(m)(3)(i)*].

   b. Unanticipated bypass. The Discharger shall submit notice of an unanticipated bypass as required in Standard Provisions - Reporting V.E below (24-hour notice) [*40 CFR Section 122.41(m)(3)(ii)*].

### H. Upset

Upset means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operational error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventive maintenance, or careless or improper operation [*40 CFR §122.41(n)(1)*].

1. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology based permit effluent limitations if the requirements of Standard Provisions – Permit Compliance I.H.2 below are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review [*40 CFR Section 122.41(n)(2)*].

2. Conditions necessary for a demonstration of upset. A Discharger who wishes to establish the affirmative defense of upset shall demonstrate, through properly signed, contemporaneous operating logs or other relevant evidence that [*40 CFR §122.41(n)(3)*]:

   a. An upset occurred and that the Discharger can identify the cause(s) of the upset [*40 CFR §122.41(n)(3)(i)*];

   b. The permitted facility was, at the time, being properly operated [*40 CFR §122.41(n)(3)(i)*];

   c. The Discharger submitted notice of the upset as required in Standard Provisions – Reporting V.E.2.b below (24-hour notice) [*40 CFR Section 122.41(n)(3)(iii)*]; and

   d. The Discharger complied with any remedial measures required under Standard Provisions – Permit Compliance I.C above [*40 CFR §122.41(n)(3)(iv)*].

3. Burden of proof. In any enforcement proceeding, the Discharger seeking to establish the occurrence of an upset has the burden of proof [*40 CFR §122.41(n)(4)*].

## II. STANDARD PROVISIONS – PERMIT ACTION

### A. General

This Order may be modified, revoked and reissued, or terminated for cause. The filing of a request by the Discharger for modification, revocation and reissuance, or termination, or a notification of planned changes or anticipated noncompliance does not stay any Order condition [*40 CFR §122.41(f)*].

**B. Duty to Reapply**

If the Discharger wishes to continue an activity regulated by this Order after the expiration date of this Order, the Discharger must apply for and obtain a new permit [*40 CFR §122.41(b)*].

**C. Transfers**

This Order is not transferable to any person except after notice to the Regional Water Board. The Regional Water Board may require modification or revocation and reissuance of the Order to change the name of the Discharger and incorporate such other requirements as may be necessary under the CWA and the CWC [*40 CFR §122.41(l)(3)*] [*40 CFR §122.61*].

## III. STANDARD PROVISIONS – MONITORING

**A.** Samples and measurements taken for the purpose of monitoring shall be representative of the monitored activity [*40 CFR §122.41(j)(1)*].

**B.** Monitoring results must be conducted according to test procedures under 40 CFR Part 136 or, in the case of sludge use or disposal, approved under 40 CFR Part 136 unless otherwise specified in 40 CFR Part 503 unless other test procedures have been specified in this Order [*40 CFR §122.41(j)(4)*] [*40 CFR §122.44(i)(1)(iv)*].

## IV. STANDARD PROVISIONS – RECORDS

**A.** Except for records of monitoring information required by this Order related to the Discharger's sewage sludge use and disposal activities, which shall be retained for a period of at least five years (or longer as required by 40 CFR Part 503), the Discharger shall retain records of all monitoring information, including all calibration and maintenance records and all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this Order, and records of all data used to complete the application for this Order, for a period of at least three (3) years from the date of the sample, measurement, report or application. This period may be extended by request of the Regional Water Board Executive Officer at any time [*40 CFR §122.41(j)(2)*].

**B. Records of monitoring information shall include:**

1. The date, exact place, and time of sampling or measurements [*40 CFR §122.41(j)(3)(i)*];

2. The individual(s) who performed the sampling or measurements [*40 CFR §122.41(j)(3)(ii)*];

3. The date(s) analyses were performed [*40 CFR §122.41(j)(3)(iii)*];

4. The individual(s) who performed the analyses [*40 CFR §122.41(j)(3)(iv)*];

5. The analytical techniques or methods used [*40 CFR §122.41(j)(3)(v)*]; and

6. The results of such analyses [*40 CFR §122.41(j)(3)(vi)*].

**C. Claims of confidentiality for the following information will be denied [*40 CFR §122.7(b)*]:**

1. The name and address of any permit applicant or Discharger [*40 CFR §122.7(b)(1)*]; and

2. Permit applications and attachments, permits and effluent data [*40 CFR §122.7(b)(2)*].

## V. STANDARD PROVISIONS – REPORTING

### A. Duty to Provide Information

The Discharger shall furnish to the Regional Water Board, SWRCB, or USEPA within a reasonable time, any information which the Regional Water Board, SWRCB, or USEPA may request to determine whether cause exists for modifying, revoking and reissuing, or terminating this Order or to determine compliance with this Order. Upon request, the Discharger shall also furnish to the Regional Water Board, SWRCB, or USEPA copies of records required to be kept by this Order [*40 CFR §122.41(h)*] [*CWC 13267*].

### B. Signatory and Certification Requirements

1. All applications, reports, or information submitted to the Regional Water Board, State Water Board, and/or USEPA shall be signed and certified in accordance with Standard Provisions – Reporting V.B.2, V.B.3, V.B.4, and V.B.5 below [*40 CFR Section 122.41(k)*].

2. All permit applications shall be signed by either a principal executive officer or ranking elected official. For purposes of this provision, a principal executive officer of a federal agency includes: (i) the chief executive officer of the agency, or (ii) a senior executive officer having responsibility for the overall operations of a principal geographic unit of the agency (e.g., Regional Administrators of USEPA) [40 CFR Section 122.22(a)(3)].

3. All reports required by this Order and other information requested by the Regional Water Board, State Water Board, or USEPA shall be signed by a person described in Standard Provisions – Reporting V.B.2 above, or by a duly authorized representative of that person. A person is a duly authorized representative only if:

   a. The authorization is made in writing by a person described in Standard Provisions – Reporting V.B.2 above [*40 CFR Section 122.22(b)(1)*];

   b. The authorization specified either an individual or a position having responsibility for the overall operation of the regulated facility or activity such as the position of plant manager, operator of a well or a well field, superintendent, position of equivalent responsibility, or an individual or position having overall responsibility for environmental matters for the company (a duly authorized representative may thus be either a named individual or any individual occupying a named position) [*40 CFR Section 122.22(b)(2)*]; and

   c. The written authorization is submitted to the Regional Water Board, State Water Board, or USEPA [*40 CFR Section 122.22(b)(3)*].

4. If an authorization under Standard Provisions – Reporting V.B.3 above is no longer accurate because a different individual or position has responsibility for the overall operation of the facility, a new authorization satisfying the requirements of Standard Provisions – Reporting V.B.3 above must be submitted to the Regional Water Board, State Water Board or USEPA prior to or together with any reports, information, or applications, to be signed by an authorized representative [40 CFR Section 122.22(c)].

5. Any person signing a document under Standard Provisions – Reporting V.B.2 or V.B.3 above shall make the following certification:

   "I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations" [*40 CFR Section 122.22(d)*].

## C. Monitoring Reports

1. Monitoring results shall be reported at the intervals specified in the Monitoring and Reporting Program in this Order [*40 CFR §122.41(l)(4)*].

2. Monitoring results must be reported on a Discharge Monitoring Report (DMR) form or forms provided or specified by the Regional Water Board or SWRCB for reporting results of monitoring of sludge use or disposal practices [*40 CFR §122.41(l)(4)(i)*].

3. If the Discharger monitors any pollutant more frequently than required by this Order using test procedures approved under 40 CFR Part 136 or, in the case of sludge use or disposal, approved under 40 CFR Part 136 unless otherwise specified in 40 CFR Part 503, or as specified in this Order, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the DMR or sludge reporting form specified by the Regional Water Board [*40 CFR §122.41(l)(4)(ii)*].

4. Calculations for all limitations, which require averaging of measurements, shall utilize an arithmetic mean unless otherwise specified in this Order [*40 CFR §122.41(l)(4)(iii)*].

## D. Compliance Schedules

Reports of compliance or noncompliance with, or any progress reports on, interim and final requirements contained in any compliance schedule of this Order, shall be submitted no later than 14 days following each schedule date [*40 CFR §122.41(l)(5)*].

## E. Twenty-Four Hour Reporting

1. The Discharger shall report any noncompliance that may endanger health or the environment. Any information shall be provided orally within 24 hours from the time the Discharger becomes aware of the circumstances. A written submission shall also be provided within five (5) days of the time the Discharger becomes aware of the circumstances. The written submission shall contain a description of the noncompliance and its cause; the period of noncompliance, including exact dates and times, and if the noncompliance has not been corrected, the anticipated time it is expected to continue; and steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance [*40 CFR §122.41(l)(6)(i)*].

2. The following shall be included as information that must be reported within 24 hours under this paragraph [*40 CFR §122.41(l)(6)(ii)*]:

   a. Any unanticipated bypass that exceeds any effluent limitation in this Order [*40 CFR §122.41(l)(6)(ii)(A)*].

   b. Any upset that exceeds any effluent limitation in this Order [*40 CFR §122.41(l)(6)(ii)(B)*].

3. The Regional Water Board may waive the above-required written report under this provision on a case-by-case basis if an oral report has been received within 24 hours [*40 CFR §122.41(l)(6)(iii)*].

## F. Planned Changes

The Discharger shall give notice to the Regional Water Board as soon as possible of any planned physical alterations or additions to the permitted facility. Notice is required under this provision only when [*40 CFR §122.41(l)(1)*]:

1. The alteration or addition to a permitted facility may meet one of the criteria for determining whether a facility is a new source in 40 CFR §122.29(b) [*40 CFR §122.41(l)(1)(i)*]; or

2. The alteration or addition could significantly change the nature or increase the quantity of pollutants discharged. This notification applies to pollutants which are subject neither to effluent limitations in this Order nor to notification requirements under 40 CFR Part 122.42(a)(1) (see Additional Provisions—Notification Levels VII.A.1) [*40 CFR §122.41(l)(1)(ii)*].

3. The alteration or addition results in a significant change in the Discharger's sludge use or disposal practices, and such alteration, addition, or change may justify the application of permit conditions that are different from or absent in the existing permit, including notification of additional use or disposal sites not reported during the permit application process or not reported pursuant to an approved land application plan [*40 CFR §122.41(l)(1)(iii)*].

## G. Anticipated Noncompliance

The Discharger shall give advance notice to the Regional Water Board or SWRCB of any planned changes in the permitted facility or activity that may result in noncompliance with General Order requirements [*40 CFR §122.41(l)(2)].*

## H. Other Noncompliance

The Discharger shall report all instances of noncompliance not reported under Standard Provisions – Reporting V.C, V.D, and V.E above at the time monitoring reports are submitted. The reports shall contain the information listed in Standard Provision – Reporting V.E above [*40 CFR Section 122.41(l)(7)*].

## I. Other Information

When the Discharger becomes aware that it failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application or in any report to the Regional Water Board, SWRCB, or USEPA, the Discharger shall promptly submit such facts or information [*40 CFR §122.41(l)(8)*].

## VI. STANDARD PROVISIONS – ENFORCEMENT

**A.** The Regional Water Board is authorized to enforce the terms of this permit under several provisions of the CWC, including, but not limited to, Sections 13385, 13386, and 13387.

## VII. ADDITIONAL PROVISIONS – NOTIFICATION LEVELS

### A. Publicly-Owned Treatment Works (POTWs)

All POTWs shall provide adequate notice to the Regional Water Board of the following [*40 CFR Section 122.42(b)*]:

1. Any new introduction of pollutants into the POTW from an indirect discharger that would be subject to Sections 301 or 306 of the CWA if it were directly discharging those pollutants [*40 CFR Section 122.42(b)(1)*]; and

2. Any substantial change in the volume or character of pollutants being introduced into that POTW by a source introducing pollutants into the POTW at the time of adoption of the Order [*40 CFR Section 122.42(b)(2)*].

3. Adequate notice shall include information on the quality and quantity of effluent introduced into the POTW as well as any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW [*40 CFR Section 122.42(b)(3)*].

## Attachment E – Monitoring and Reporting Program

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## ATTACHMENT E – MONITORING AND REPORTING PROGRAM (MRP)

California Water Code sections 13267 and 13383 authorize the Regional Water Board to require technical and monitoring reports. The Code of Federal Regulations (CFR) at 40 CFR 122.48 requires that all NPDES permits specify monitoring and reporting requirements. This Monitoring and Reporting Program establishes monitoring and reporting requirements that implement the state and federal regulations.

### I. GENERAL MONITORING PROVISIONS

1. All sampling and sample preservation shall be in accordance with the current edition of "*Standard Methods for the Examination of Water and Wastewater*" (American Public Health Association).

2. All laboratory analyses shall be performed in accordance with test procedures under 40 CFR 136 "Guidelines Establishing Test Procedures for the Analysis of Pollutants," promulgated by the United States Environmental Protection Agency (EPA), unless otherwise specified in this MRP. In addition, the Regional Water Board and/or EPA, at their discretion, may specify test methods that are more sensitive than those specified in 40 CFR 136. (See also I.6., below).

3. Chemical, bacteriological, and bioassay analyses shall be conducted at a laboratory certified for such analyses by the California Department of Health Services in accordance with the provision of Water Code Section 13176, and must include quality assurance/quality control data with their reports, or EPA or at laboratories approved by the Regional Water Board's Executive Officer.

4. Whenever the Discharger monitors any pollutant more frequently than is required by this Order, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the discharge monitoring report specified by the Executive Officer.

5. In conformance with federal regulations 40 CFR 122.45(c), analyses to determine compliance with the effluent limitations for metals shall be conducted using the total recoverable method. For Chromium (VI), the dissolved method in conformance with 40 CFR 136 may be used to measure compliance with the Chromium (VI) limitation.

6. For effluent monitoring:

   a. The Discharger shall require its testing laboratory to calibrate the analytical system down to the minimum level (ML)[1] specified in Attachment H for priority pollutants, unless an alternative minimum level is approved by the Regional Water Board's Executive Officer. When there is more than one ML value for a given substance, the discharger shall use the ML values, and their associated analytical methods listed in Attachment H that are below the effluent limitation. For analysis of priority pollutants without effluent limitations, the Discharger shall use an ML value that is below the trigger values listed in Attachment I. If no ML value is below the effluent limitation, or the trigger value listed in Attachment I, then the lowest ML and associated analytical method shall be used. Any internal quality control data associated with the sample must be reported when requested by the Executive Officer. The Regional Water Board will reject the quantified laboratory data if quality control data is unavailable or unacceptable.

   b. The discharger shall report the results of analytical determinations for the presence of chemical constituents in a sample using the following reporting protocols:

      1) Sample results greater than or equal to the reported ML shall be reported as measured by the laboratory (i.e., the measured chemical concentration in the sample).

      2) Sample results less than the reported ML, but greater than or equal to the laboratory's current Method Detection Limit (MDL)[2], shall be reported as "Detected, but Not Quantified," or "DNQ." The estimated chemical concentration of the sample shall also be reported.

      3) Sample results not detected above the laboratory's MDL shall be reported as "not detected" or "ND."

   c. The Discharger shall submit to the Regional Water Board reports necessary to determine compliance with effluent limitations in this Order and shall follow the chemical nomenclature and sequential order of priority pollutant constituents shown in Attachment G – Priority Pollutant List. The Discharger shall report with each sample result:

      1) The minimum level achieved by the testing laboratory; and

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1 *Minimum level is the concentration at which the entire analytical system must give a recognizable signal and acceptable point. The ML is the concentration in a sample that is equivalent to the concentration of the lowest calibration standard analyzed by a specific analytical procedure, assuming that all the method specified sample weights, volumes, and processing steps have been followed.*

2 *MDL is the minimum concentration of a substance that can be measured and reported with 99 percent confidence that the analytical concentration is greater than zero, as defined in 40 CFR 136, Appendix B.*

2) The laboratory's current MDL, as determined by the procedure found in 40 CFR 136.

d. For receiving water monitoring and for those priority pollutants without effluent limitations, the Discharger shall require its testing laboratory to quantify constituent concentrations to the lowest achievable MDL as determined by the procedure found in 40 CFR 136. In situations where the most stringent applicable receiving water objective (freshwater or human health (consumption of organisms only), as specified for that pollutant in 40 CFR 131.38 is below the minimum level value specified in Attachment H and the Discharger cannot achieve an MDL value for that pollutant below the ML value, the Discharger shall submit justification why a lower MDL value cannot be achieved. Justification shall be submitted together with monthly monitoring reports.

7. The Discharger shall have, and implement an acceptable written quality assurance (QA) plan for laboratory analyses. Duplicate chemical analyses must be conducted on a minimum of ten percent (10%) of the samples, or at least one sample per month, whichever is greater. A similar frequency shall be maintained for analyzing spiked samples. When requested by the Regional Water Board or EPA, the Discharger will participate in the NPDES discharge monitoring report QA performance study.

8. Monitoring and reporting shall be in accordance with the following:

   a. Samples and measurements taken for the purpose of monitoring shall be representative of the monitored activity.

   b. The monitoring and reporting of influent, effluent, and sludge shall be done more frequently as necessary to maintain compliance with this Order and or as specified in this Order.

   c. Whenever the Discharger monitors any pollutant more frequently than is required by this Order, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the discharge monitoring report specified by the Regional Water Board's Executive Officer.

   d. A "grab" sample is defined as any individual sample collected in less than 15 minutes.

   e. A composite sample is defined as a combination of no fewer than eight individual grab samples obtained over the specified sampling period. The volume of each individual grab sample shall be proportional to the discharge flow rate at the time of sampling. The compositing period shall equal the specific sampling period, or 24 hours, if no period is specified.

   f. Daily samples shall be collected on each day of the week.

g. Monthly samples shall be collected on any representative day of each month.

h. Quarterly samples: A representative grab sample shall be taken on any representative day of January, April, July, and October and test results shall be reported in micrograms/liter (ug/L) by the last day of the month following the month that the sample was taken.

i. Semi-annual samples shall be collected in January and July.

j. Annual samples shall be collected in accordance with the following schedule:

**Table 1. Annual Sampling Schedule**

| Year | Annual Samples |
|---|---|
| 2013 | July |
| 2014 | October |
| 2015 | January |
| 2016 | April |
| 2017 | July |
| 2018 | October |

9. The Discharger shall assure that records of all monitoring information are maintained and accessible for a period of at least five years (this retention period supersedes the retention period specified in Section IV.A. of Attachment D) from the date of the sample, report, or application. This period of retention shall be extended during the course of any unresolved litigation regarding this discharge or by the request of the Regional Water Board at any time. Records of monitoring information shall include:

   a. The information listed in Attachment D- IV Standard Provisions – Records, subparagraph B. of this Order;

   b. The laboratory which performed the analyses;

   c. The date(s) analyses were performed;

   d. The individual(s) who performed the analyses;

   e. The modification(s) to analytical techniques or methods used;

   f. All sampling and analytical results, including

      1) Units of measurement used;
      2) Minimum reporting level for the analysis (minimum level, practical quantitation level (PQL));
      3) Results less than the reporting level but above the method detection limit (MDL);
      4) Data qualifiers and a description of the qualifiers;
      5) Quality control test results (and a written copy of the laboratory quality assurance plan);
      6) Dilution factors, if used; and
      7) Sample matrix type.

g. All monitoring equipment calibration and maintenance records;

h. All original strip charts from continuous monitoring devices;

i. All data used to complete the application for this Order; and,

j. Copies of all reports required by this Order.

k. Electronic data and information generated by the Supervisory Control And Data Acquisition (SCADA) System.

10. The flow measurement system shall be calibrated at least once per year or more frequently, to ensure continued accuracy.

11. All monitoring instruments and devices used by the Discharger to fulfill the prescribed monitoring program shall be properly maintained and calibrated as necessary to ensure their continued accuracy. In the event that continuous monitoring equipment is out of service for greater than a 24-hour period, the Discharger shall obtain a representative grab sample each day the equipment is out of service. The Discharger shall correct the cause(s) of failure of the continuous monitoring equipment as soon as practicable. In its monitoring report, the Discharger shall specify the period(s) during which the equipment was out of service and if the problem has not been corrected, shall identify the steps which the Discharger is taking or proposes to take to bring the equipment back into service and the schedule for these actions.

## II. MONITORING LOCATIONS

The Discharger shall establish the following monitoring locations to demonstrate compliance with the effluent limitations, discharge specifications, and other requirements in this Order:

**Table 1. Monitoring Station Locations**

| Discharge Point Name | Monitoring Location Name | Monitoring Location Description | Latitude | Longitude |
|---|---|---|---|---|
| -- | M-INFA | Combined influent monitoring and sampling station | 33° 57' 52.33" N | 117° 27' 13.58" W |
| 001, 002 | M-001A | Effluent to Reach 3 of Santa Ana River, close to the end of effluent pipeline | 33° 57' 48.30" N | 117° 27' 40.95" W |
| 001,002 | M-001B | At the end of the chlorine contact tank 3. This station is used for coliform testing. | 33° 57' 42.24" N | 117° 27' 40.98" W |
| 003 | REC-001 | Recycled water for irrigation of Toro turf, at the end of the chlorine contact tank 3 | 33° 57' 42.15" N | 117° 27' 41.49" W |
| 004 | REC-002 | Recycled water for RPU Distribution system | 33° 57' 47.24" N | 117° 27' 42.18" W |
| -- | R-001U | Receiving surface water, upstream of Santa Ana River at the Metropolitan Water District pipeline crossing. | 33° 58' 4.53" N | 117° 26' 52.93" W |
| -- | R-001D | Santa Ana river, downstream of the most downstream point of discharge | 33° 56' 48.27" N | 117° 33' 56.15" W |
| -- | R-002 | Santa Ana River | -- | -- |
| -- | B-001 | Biosolids monitoring at end of dewatering | 33° 57' 44.24" N | 117° 27' 27.67" W |

## III. INFLUENT MONITORING REQUIREMENTS

1. Sampling stations shall be established for the points of inflow to the treatment plant. The sampling station(s) shall be located upstream of any in-plant return flows and where representative sample(s) of the influent of the treatment plant can be obtained.

2. The Discharger shall monitor the combined influent into the facility at M-INFas follows:

**Table 2. Influent Monitoring**

| Parameter | Units | Sample Type | Minimum Sampling Frequency (See Section I.13., above) | Required Analytical Test Method |
|---|---|---|---|---|
| Flow | mgd | Recorder/ Totalizer | Continuous | -- |
| Specific Conductance | µmhos/cm | Recorder | " | See Sections I.2. & I.3., above |
| pH | pH units | " | " | " |
| $BOD_5$ | mg/L | 24-hr Composite | Weekly | " |
| Suspended Solids | " | " | " | " |
| Ammonia-Nitrogen | " | Grab | " | " |
| Total Inorganic Nitrogen | " | 24-hr Composite | " | " |
| Total Dissolved Solids | " | " | " | " |
| Boron | " | " | Quarterly | " |
| Chloride | " | " | " | " |
| Fluoride | " | " | " | " |
| Sulfate | " | " | " | " |
| Total Hardness | " | " | " | " |
| Arsenic | µg/L | " | " | See Sections I.2., I.3. & I.6.., above |
| Cadmium | " | " | " | " |
| Chromium, VI or Total Chromium | "L | " | " | " |
| Copper | " | " | " | " |
| Cyanide (Free) | " | Grab | " | " |
| Lead | " | 24-hr Composite | " | " |
| Mercury | " | " | " | " |
| Nickel | " | " | " | " |
| Silver | " | " | " | " |
| Zinc | " | " | " | " |
| Volatile organic portion of EPA Priority Pollutants (See Attachment G) | " | Grab | Annually | " |
| Remaining EPA Priority Pollutants[8] (See Attachment G) | µg/L | 24-hr Composite | " | " |

[8] *Remaining EPA priority pollutants are those pollutants listed in Attachment G which are not volatile organics and pollutants not specifically listed in this monitoring program table.*

## IV. EFFLUENT MONITORING REQUIREMENTS – DP 001 AND 002

1. The Discharger shall monitor the discharge at monitoring location M-001A as follows:

**Table 3. Effluent Monitoring at M-001A**

| Parameter | Units | Sample Type | Minimum Sampling Frequency (See Section I.13., above) | Required Analytical Test Method and Reporting Level as noted herein |
|---|---|---|---|---|
| Flow | mgd | Recorder/ Totalizer | Continuous | --- |
| pH | pH units | Recorder | " | See Sections I.2. & I.3., above |
| Specific Conductance | µmhos/cm | " | " | " |
| Total Chlorine Residual | mg/L | " | " | " |
| Turbidity | NTU[10] | Recorder/Turbidimeter | Continuous[11] | " |
| Ammonia-Nitrogen | mg/L | 24-hr Composite | Daily | " |
| $BOD_5$ | " | " | " | " |
| Suspended Solids | " | " | " | " |
| Temperature | $^0C$ | Grab | Weekly | " |
| Total Dissolved Solids | mg/L | 24-hr Composite | Monthly | " |
| Total Inorganic Nitrogen | " | " | " | " |
| Nitrate Nitrogen | " | " | " | " |
| Total Hardness | " | " | " | " |
| Toxicity Monitoring | TU | 24-hr Composite (see Section V.A.3., below) | Monthly (see Section V.4., below) | " |
| Cyanide (Free) | µg/L | Grab | Monthly | See Sections I.2., I.3. & I.6., above |
| Chromium (VI)/ or Total Chromium | " | Grab/24-hr Composite | Quarterly | " |
| Mercury | " | 24-hr Composite | " | " |
| Cadmium | " | " | " | " |
| Copper | " | " | " | " |
| Lead | " | " | " | " |
| Selenium | " | " | " | " |
| Silver | " | " | " | " |
| Aluminum | mg/L | " | " | " |
| Boron | " | " | " | See Sections I.2. & I.3., above |

10 *NTU = Nephelometric Turbidity Units*

11 *Turbidity analysis shall be continuous, performed by a continuous recording turbidimeter. Compliance with the daily average operating filter effluent turbidity shall be determined by averaging the levels of recorded turbidity taken at a minimum of four-hour intervals over a 24-hour period. The results of the daily average turbidity determinations shall be reported monthly.*

5. The presence of chronic toxicity shall be estimated as specified in Short Term Methods for Estimating the Chronic Toxicity of Effluents and Receiving Waters to Freshwater Organisms. Fourth Edition. EPA-821-R-02-013.

6. Results for both survival and reproduction endpoints shall be reported in TUc, where TUc = 100/NOEC or 100/ICp or ECp (p is the percent effluent). The no observed effect concentration (NOEC) is the highest concentration of toxicant to which organisms are exposed in a chronic test, that causes no observable adverse effect on the tests organisms (e.g., the highest concentration of toxicant to which the values for the observed responses are not statistically significant different from the controls). The inhibition concentration (IC) is a point estimate of the toxicant concentration that causes a given percent reduction in a non-quantal biological measurement (e.g., reproduction or growth) calculated from a continuous model (the EPA Interpolation Method). The effective concentration (EC) is a point estimate of the toxicant concentration that would cause a given percent reduction in quantal biological measurement (e.g., larval development, survival) calculated from a continuous model (e.g., probit).

7. Additional Testing Requirements.

   a. A series of at least five dilutions and a control will be tested. Five dilutions of the series shall be within 60% to 100% effluent concentration.

   b. If organisms are not cultured in-house, concurrent testing with reference toxicants shall be conducted. Where organisms are cultured in-house, monthly reference toxicant testing is sufficient. Reference toxicants shall also be conducted using the same test conditions as the effluent toxicity test (e.g., same test duration, etc).

   c. If either of the reference toxicant test or the effluent tests do not meet all test acceptability criteria as specified in the manual[22], then the Discharger must re-sample and re-test within 14 days or as soon as the Discharger receives notification of failed tests.

   d. Control and dilution water should be receiving water or lab water. If the dilution water used is different from the culture water, a second control, using culture water shall also be used.

[22] *Refers to USEPA Manual "Short-Term Methods for Estimating the Chronic Toxicity of Effluents and Receiving Waters to Freshwater Organisms. Fourth Edition. EPA-821-R-02-013.*

8. Quality Assurance/Control:

   a. A quality assurance/quality control (QA/QC) program shall be instituted to verify the results of the effluent toxicity-monitoring program. The QA/QC program shall include but shall not be limited to the following: (1) Selection of an independent testing laboratory; (2) Approval by the Regional Board's Executive Officer or Executive Officer's designee of the independent testing laboratory; (3) Once during the year, the Discharger shall split samples with the independent laboratory for conducting chronic toxicity testing; (4) Results from the independent laboratory shall be submitted to the Regional Board and the Discharger for evaluation; (5) The Discharger shall review the test acceptability criteria in accordance with the EPA test protocols, EPA/600/4-91/002.

   b. Results from the independent laboratory of the annual QA/QC split samples are to be used for Quality Assurance/Quality Control (QA/QC) purposes only and not for purposes of determining compliance with other requirements of this Order.

9. The use of alternative methods for measuring chronic toxicity may be considered by the Executive Officer on a case–by-case basis. The use of a different test species, in lieu of conducting the required test species may be considered/approved by the Executive Officer on a case-by case basis upon submittal of the documentation supporting Discharger's determination that a different species is more sensitive and appropriate.

10. Reporting: Results of all toxicity testing conducted within the month following the reporting period shall be submitted monthly in accordance with "Short-term Methods for Estimating the Chronic Toxicity of Effluents and Receiving Waters to Freshwater Organisms", third edition, Environmental Monitoring Systems Laboratory, U.S. Environmental Protection Agency 2002, Cincinnati, Ohio (October 2002, EPA-821-R-02-013). The report shall include a determination of the median value of all chronic toxicity testing results conducted during the two previous months.

11. Whenever an Initial Investigation Reduction Evaluation is conducted, the results of the evaluation shall be submitted upon completion. In addition, monthly status reports shall be submitted as part of the Discharger's monitoring report.

## VI. LAND DISCHARGE MONITORING REQUIREMENTS – Not Applicable

## VII. RECYCLED WATER MONITORING REQUIREMENTS – DP 003 AND 004

The Discharger shall monitor recycled water at REC-001, and REC-002, as follows, unless samples collected at M-001A and M-001B are representative of the recycled water at DPs 003 and 004. Then, only flow and CT shall be required to be monitored at REC-001 and REC-002.

**Table 5. Recycled Water Monitoring at REC-001, REC-002**

| Parameter | Units | Sample Type | Minimum Sampling & Testing Frequency (See Section I.13., above) | Required Analytical Test Method |
|---|---|---|---|---|
| Flow | mgd | Recorder/Totalizer | Continuous | --- |
| pH | pH units | Recorder | " | " |
| CT | mg/L-min | " | " | " |
| Turbidity | NTU | " | " | " |
| $BOD_5$ | mg/L | Composite | Daily | See Section I.2., above |
| Total Suspended Solids | " | " | " | " |
| Coliform Organisms | MPN per 100 mL | Grab | " | " |
| Total Inorganic Nitrogen | " | " | Monthly | " |
| TDS | " | " | " | " |

## VIII. RECEIVING WATER MONITORING REQUIREMENTS – SURFACE WATER AND GROUNDWATER

1. The Discharger shall make measurement of the receiving water flow at R-001U in the Santa Ana River and determine whether a 20:1 dilution exists at the point of discharge before discharging secondary treated effluent. A dilution of 20:1 or more is required at the point of discharge. Flow measurements shall be made prior to any direct discharge to the river and shall continue on a daily basis until the discharged is terminated.

2. The Discharger shall monitor the receiving water at R-001U as follows:

**Table 6. Receiving Water Monitoring at R-001U**

| Parameter | Units | Sample Type | Minimum Sampling & Testing Frequency | Required Analytical Test Method |
|---|---|---|---|---|
| Dissolved Oxygen | mg/L | Grab | Weekly | See Sections I.2. & I.3., above |
| Temperature | $^0$C | " | " | " |
| pH | pH unit | Grab | " | " |
| EPA Priority Pollutants | µg/L | " | Annually | " |

3. The Discharger shall monitor the receiving water at R-001D as follows:

**Table 7. Receiving Water Monitoring at R-001D**

| Parameter | Units | Sample Type | Minimum Sampling & Testing Frequency | Required Analytical Test Method |
|---|---|---|---|---|
| Dissolved Oxygen | mg/L | Grab | Weekly | See Sections I.2. & I.3., above |
| Temperature | $^0$C | " | " | " |
| pH | pH unit | Grab | " | " |
| Color change, foam, deposition of material, odor | -- | observe | Weekly | " |

## C. Regional Monitoring for Fish Flesh Testing

Unless otherwise directed by the Regional Water Board Executive Officer, the Discharger shall implement the approved plan for the annual sampling and testing of mercury levels in fish flesh samples collected from the Santa Ana River. The frequency of monitoring and submission of reports shall be as stipulated in the approved plan.

## IX. OTHER MONITORING REQUIREMENTS

### A. Biosolids Monitoring at B-001

1. Biosolids monitoring shall be conducted as follows:

**Table 8. Biosolids Monitoring at B-001**

| Parameter | Units | Sample Type | Minimum Sampling & Testing Frequency |
|---|---|---|---|
| Priority Pollutants | mg/kg | A composite of six grab samples | Semi-annually |
| Moisture Content (% solid) | mg/kg | Grab | Quarterly |

2. The Discharger shall maintain a permanent log of solids hauled away from the treatment facilities for use/disposal elsewhere, including the date hauled, the volume or weight (in dry tons), type (screening, grit, raw sludge, biosolids), application (agricultural, composting, etc), and destination. This information shall be reported semi-annually.

### B. Water Supply Monitoring:

1. At least once per year, a sample of each source of the water supplied to the sewered area shall be obtained and analyzed for total dissolved solids.

2. Monthly reports shall be submitted stating the amount (in percentage or acre-feet) supplied to the sewered area from each source of water and the resulting flow-weighted water supply quality for total dissolved solids.

## C. Pretreatment Monitoring and Reporting:

1. The Discharger shall submit to the Regional Water Board and the EPA Region 9, a quarterly compliance status report. The quarterly compliance status reports shall cover the periods January 1 - March 31, April 1 - June 30, July 1 - September 30, and October 1 -December 31. Each report shall be submitted by the end of the month following the quarter, except that the report for October 1 - December 31 may be included in the annual report. This quarterly reporting requirement shall commence for the first full quarter following issuance of this Order. The reports shall identify:

   a. All significant industrial users (SIUs) which violated any standards or reporting requirements during that quarter;
   b. The violations committed (distinguish between categorical and local limits);
   c. The enforcement actions undertaken; and
   d. The status of active enforcement actions from previous periods, including closeouts (facilities under previous enforcement actions which attained compliance during the quarter).

2. Annually, the Discharger shall submit a report to the Regional Water Board, the State Water Resources Control Board and the EPA Region 9 describing the pretreatment activities within the service area during the previous year. In the event that any control authority within the service area is not in compliance with any conditions or requirements of this Order or their approved pretreatment program (such as due to industrial user discharges, interjurisdictional agency agreement implementation issues, or other causes,) then the Discharger shall also include the reasons for non-compliance and state how and when the Discharger and the control authority shall comply with such conditions and requirements. This annual report shall cover operations from January 1 through December 31 of each fiscal year and is due on April 1 of each year. The report shall contain, but not be limited to, the following information:

   a. A summary of analytical results from representative, flow-proportioned, 24-hour composite sampling of the POTW's influent and effluent wastewaters for those pollutants which are known or suspected to be discharged by industrial users (IUs) as identified by EPA under Section 307(a) of the CWA. The summary will include the result of annual full priority pollutant scan, with quarterly samples analyzed only for those pollutants[24] detected in the full scan. The Discharger shall also provide any influent or effluent monitoring data for non-priority pollutants which the Discharger believes may be causing or contributing to Interference, Pass Through or adversely impacting sludge quality. Sampling and analysis shall be performed in accordance with the techniques prescribed in 40 CFR 136 and amendments thereto.

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24 *The Discharger is not required to analyze for asbestos.*

b. A discussion of any upset, interference, or pass-through incidents at the treatment plant (if any), which the Discharger knows or suspects were caused by IUs of the POTW system. The discussion shall include the following:

   1) The reasons why the incidents occurred, the corrective actions taken, and, if known, the name and address of the IU(s) responsible.

   2) A review of the applicable pollutant limitations to determine whether any additional limitations, or changes to existing requirements, may be necessary to prevent pass through, interference or noncompliance with sludge disposal requirements.

c. A complete and updated list of the Discharger's significant industrial users (SIUs), including names, North American Industry Classification System (NAICS) and addresses, and a list of any SIU deletions and/or additions. The Discharger shall provide a brief explanation for each deletion. The SIU list shall identify the SIUs subject to Federal Categorical Standards by specifying which set(s) of standards are applicable to each SIU. The list shall also indicate which SIUs are subject to local limitations more stringent than Federal Categorical Standards and those, which are not subject to local limits.

d. A list or table characterizing the industrial compliance status of each SIU, including:

   1) SIU name;
   2) Industrial category;
   3) The type (processes) of wastewater treatment in place;
   4) Number of samples taken by the POTW during the year;
   5) Number of samples taken by the SIU during the year;
   6) Whether all needed certifications (if allowed) were provided by SIUs which have limits for total toxic organics;
   7) Federal and Regional Standards violated during the year, reported separately;
   8) Whether the SIU at any time in the year was in Significant Noncompliance (SNC)[25], as defined by 40 CFR 403.12 (f)(2)(vii); and
   9) A summary of enforcement actions against the SIU taken during the year, including the type of action, final compliance date, and amount of fines assessed/collected (if any). Proposed actions, if known, should be included.
   10) Number of inspections conducted at each SIU during the year.

e. A compliance summary table which includes:

   1) SIU's which were in SNC at any time during the year;
   2) The total number of SIUs which are in SNC with pretreatment compliance schedules during the year;

[25] *SNC is determined at the beginning of each quarter based on data of the previous six months.*

3) The total number of notices of violation and administrative orders issued against SIUs during the year;
4) The total number of civil and criminal judicial actions filed against SIUs during the year;
5) The number of SIUs which were published as being in SNC during the year; and
6) The number of IUs from which penalties were collected during the year.

f. A short description of any significant changes in operating the pretreatment program which differ from the previous year including, but not limited to changes concerning:

1) The program's administrative structure;
2) Local industrial discharge limitations;
3) Monitoring program or monitoring frequencies;
4) Legal authority or enforcement policy;
5) Funding mechanisms; and
6) Resource requirements and/or staffing levels.

g. A summary of the annual pretreatment budget, including the cost of pretreatment program functions and equipment purchases.

h. A summary of public participation activities to involve and inform the public.

i. A description of any changes in sludge disposal methods and a discussion of any concerns not described elsewhere in the report.

3. The cumulative number of industrial users that the Discharger has notified regarding Baseline Monitoring Reports and the cumulative number of industrial user responses.

4. The Discharger shall submit the quarterly compliance status reports and the annual pretreatment report to EPA Region 9, the State Board and the Regional Water Board.

**D. TDS Offset Program Monitoring and Reporting:**

If the discharger is offsetting exceedances of the TDS limit pursuant to Effluent Limitations IV.A.1.c of Order No. R8-2013-0016, the Discharger shall report the total salt removal accomplished pursuant to the offset program to demonstrate whether offset requirements are being met. The Discharger shall report quarterly the running balance of salt discharges compared to TDS removal. If offset in not occurring during the quarterly monitoring period, the quarterly report shall so state and identify when the offset will be achieved.

# X. REPORTING REQUIREMENTS

## A. General Monitoring and Reporting Requirements

1. The Discharger shall comply with all Standard Provisions (Attachment D) related to monitoring, reporting, and recordkeeping.

2. Any internal quality control data associated with the sample must be reported when requested by the Executive Officer. The Regional Water Board will reject the quantified laboratory data if quality control data is unavailable or unacceptable.

3. Discharge monitoring data shall be submitted in a format acceptable by the Regional Water Board. Specific reporting format may include preprinted forms and/or electronic media. The results of all monitoring required by this Order shall be reported to the Regional Water Board, and shall be submitted in such a format as to allow direct comparison with the limitations and requirements of this order.

4. The Discharger shall submit to the Regional Water Board reports necessary to determine compliance with effluent limitations in this Order and shall follow the chemical nomenclature and sequential order of priority pollutant constituents shown in Attachment G – Priority Pollutant Lists. The Discharger shall report with each sample result:

   a) The minimum level achieved by the testing laboratory; and
   b) The laboratory's current MDL, as determined by the procedure found in 40 CFR 136.
   c) For those priority pollutants without effluent limitations, the Discharger shall require its testing laboratory to quantify constituent concentrations to the lowest achievable MDL as determined by the procedure found in 40 CFR 136. In situations where the trigger value listed in Attachment I is below the minimum level value specified in Attachment H and the Discharger cannot achieve an MDL value for that pollutant below or equal to the ML value, the Discharger shall submit justification why a lower MDL value cannot be achieved. Justification shall be submitted together with monthly monitoring reports.

5. For every item of monitoring data where the requirements are not met, the monitoring report shall include a statement discussing the reasons for noncompliance, and of the actions undertaken or proposed which will bring the discharge into full compliance with requirements at the earliest time, and an estimate of the date when the Discharger will be in compliance. The Discharger shall notify the Regional Water Board by letter when compliance with the time schedule has been achieved.

6. The reports for June and December shall include a roster of plant personnel, including job titles, duties, and level of State certification for each individual.

7. The Discharger shall report monitoring results for specific parameters in accordance with the following table:

**Table 9. Reporting Requirements**

| Parameter | Measurement |
|---|---|
| Flow | Daily total flow |
| pH | Daily high and daily low |
| Total Chlorine Residual | Daily Maximum |
| Electrical Conductivity | Daily High |
| Turbidity | Daily maximum |

8. The Discharger shall file a written report with the Regional Board within ninety (90) days after the average dry-weather waste flow for any month equals or exceeds 75 percent of the design capacity of the waste treatment and/or disposal facilities. The Discharger's senior administrative officer shall sign a letter that which transmits that report and certifies that the policy making body is adequately informed about it. The report shall include:

   a. Average daily flow for the month, the date on which the instantaneous peak flow occurred, the rate of that peak flow, and the total flow for the day.

   b. The Discharger's best estimate of when the average daily dry-weather flow rate will equal or exceed the design capacity of the treatment facilities.

   c. The Discharger's intended schedule for studies, design, and other steps needed to provide additional capacity for the waste treatment and/or disposal facilities before the waste flow rate equals the capacity of present units.

## B. Self-Monitoring Reports (SMRs)

1. The Discharger shall submit Self-Monitoring Reports (SMRs) electronically using the State Water Board's California Integrated Water Quality System (CIWQS) Program Web site (http://www.waterboards.ca.gov/ciwqs/index.html). The CIWQS Web site will provide additional directions for SMR submittal in the event there will be service interruption for electronic submittal.

2. The Discharger shall report in the SMR the results for all monitoring specified in this MRP under sections III through IX. Additionally, the Discharger shall report in the SMR the results of any special studies, acute and chronic toxicity testing, TRE/TIE, PMP, and Pollution Prevention Plan required by Special Provisions – VI.C. list of this Order. The Discharger shall submit monthly, quarterly, and annual SMRs including the results of all required monitoring using USEPA-approved test methods or other test methods specified in this Order. If the Discharger monitors any pollutant more frequently than required by this Order, the results of this monitoring shall be included in the calculations and reporting of the data submitted in the SMR.

3. Monitoring periods and reporting for all required monitoring shall be completed according to the following schedule:

**Table 10. Monitoring and Reporting Schedule**

| Sampling Frequency | Monitoring Period Begins On | Monitoring Period | SMR Due Date[29] |
|---|---|---|---|
| Continuous | The effective date of this Order | All | Submit with monthly SMR |
| Daily | The effective date of this Order | (Midnight through 11:59 PM) or any 24-hour period that reasonably represents a calendar day for purposes of sampling. | Submit with monthly SMR |
| Weekly | The effective date of this Order | Sunday through Saturday | Submit with monthly SMR |
| Monthly | First day of calendar month following permit effective date or on permit date if that date is first day of the month | 1st day of calendar month through last day of calendar month | First day of the second month following the reporting period, submit as monthly SMR |
| Quarterly | Closest of January 1, April 1, July 1, or October 1 following permit effective date | January 1 through March 31, samples are collected in January; April 1 through June 30; samples are collected in April; July 1 through September 30; samples are collected in July; October 1 through December 31; samples are collected in October | First day of the second month following the reporting period, submit with monthly SMR |
| Semiannually | Closest of January 1 or July 1 following permit effective date | January 1 through June 30<br>July 1 through December 31 | First day of the second month following the reporting period, submit with monthly SMR |
| Annually | The effective day of this Order | 1st day of calendar month through last day of calendar month (See Section I.A.13.j, above) | First day of the second month following the reporting period, submit with monthly SMR |
| Pretreatment Annual Report | July 1, 2013 | July 1 through June 30 | September 1 |

4. Reporting Protocols. The Discharger shall report with each sample result the applicable Minimum Level (ML) and the current Method Detection Limit (MDL), as determined by the procedure in 40 CFR Part 136.

   The Discharger shall report the results of analytical determinations for the presence of chemical constituents in a sample using the following reporting protocols:

   a. Sample results greater than or equal to the ML shall be reported as measured by the laboratory (i.e., the measured chemical concentration in the sample).

   b. Sample results less than the ML, but greater than or equal to the laboratory's MDL, shall be reported as "Detected, but Not Quantified," or DNQ. The estimated chemical concentration[30] of the sample shall also be reported.

29 *Should the due date fall on a weekend or holiday, the due date shall be extended to the next work day.*

30 *See definition in Attachment A*

For the purposes of data collection, the laboratory shall write the estimated chemical concentration next to DNQ as well as the words "Estimated Concentration" (may be shortened to "Est. Conc."). The laboratory may, if such information is available, include numerical estimates of the data quality for the reported result. Numerical estimates of data quality may be percent accuracy (± a percentage of the reported value), numerical ranges (low to high), or any other means considered appropriate by the laboratory.

c. Sample results less than the laboratory's MDL shall be reported as "Not Detected," or ND.

d. Dischargers are to instruct laboratories to establish calibration standards so that the ML value (or its equivalent if there is differential treatment of samples relative to calibration standards) is the lowest calibration standard. At no time is the Discharger to use analytical data derived from extrapolation beyond the lowest point of the calibration curve.

5. The Discharger shall attach a cover letter to the SMR. The information contained in the cover letter shall clearly identify violations of the WDRs; discuss corrective actions taken or planned; and the proposed time schedule for corrective actions. Identified violations must include a description of the requirement that was violated and a description of the violation.

## C. Discharge Monitoring Reports (DMRs)

1. At any time during the term of this permit, the State or Regional Water Board may notify the Discharger to electronically submit SMRs that will satisfy federal requirements for submittal of Discharge Monitoring Reports (DMRs). Until such notification is given, the Discharger shall submit DMRs in accordance with the requirements described below.

2. DMRs must be signed and certified as required by the standard provisions (Attachment D). The Discharger shall submit the original DMR and one copy of the DMR to the address listed below:

| Standard Mail | FedEx/UPS/ Other Private Carriers |
|---|---|
| State Water Resources Control Board<br>Division of Water Quality<br>c/o DMR Processing Center<br>PO Box 100<br>Sacramento, CA 95812-1000 | State Water Resources Control Board<br>Division of Water Quality<br>c/o DMR Processing Center<br>1001 I Street, 15th Floor<br>Sacramento, CA 95814 |

3. All discharge-monitoring results must be reported on the official USEPA pre-printed DMR forms (EPA Form 3320-1). Forms that are self-generated or modified cannot be accepted.

## D. Other Reports

The Discharger shall report the results of any special studies, acute and chronic toxicity testing, TRE/TIE, PMP, and Pollution Prevention Plan required by Special Provisions – VI.C. of this Order. The Discharger shall submit reports with the first monthly SMR scheduled to be submitted on or immediately following the report due date in compliance with SMR reporting requirements described in subsection X.B.3 above.

**ATTACHMENT F – FACT SHEET**

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## ATTACHMENT F – FACT SHEET

As described in Section II of this Order, this Fact Sheet includes the legal requirements and technical rationale that serve as the basis for the requirements of this Order.

This Order has been prepared under a standardized format to accommodate a broad range of discharge requirements for dischargers in California. Only those sections or subsections of this Order that are specifically identified as "not applicable" have been determined not to apply to this Discharger. Sections or subsections of this Order not specifically identified as "not applicable" are fully applicable to this Discharger.

## I. DISCHARGER/FACILITY INFORMATION

The following table summarizes administrative information related to the facility.

Table 1. Discharger/Facility Information

| | |
|---|---|
| **WDID** | 8 330117003 |
| **Discharger** | City of Riverside, Department of Public Works |
| **Discharger Legally Responsible Officer** | Tom Boyd, Public Works Director (951)826-5575 |
| **Mailing Address** | 3900 Main St., Riverside, CA 92522 |
| **Name of Facility** | Riverside Regional Water Quality Control Plant, City of Riverside |
| **Facility Address** | 5950 Acorn Street, Riverside, CA 92504 |
| **Facility Contacts** | Gary Valladao, Wastewater Systems Manager, (951) 351-6140<br>Edward Filadelfia, Regulatory Affairs & Compliance, (951) 351- 6080<br>Kevin Street, Regulatory Programs and Compliance Manager, (951) 351- 6007 |
| **Billing Address** | Same as Facility Address |
| **Type of Facility** | Publicly Owned Treatment Works (POTW) |
| **Major or Minor Facility** | Major |
| **Threat to Water Quality** | 1 |
| **Complexity** | A |
| **Pretreatment Program** | Y |
| **Reclamation Requirements** | Y |
| **Facility Design Flow** | 46 mgd |
| **Facility Permitted Flow** | 46 mgd |

The City of Riverside, Department of Public Works (hereinafter Discharger) is the owner and operator of the Riverside Regional Water Quality Control Plant (hereinafter RRWQCP or Facility), a publicly owned wastewater treatment plant that produces tertiary treatment water.

The Facility discharges wastewater to Reach 3 of the Santa Ana River, a water of the United States, and is currently regulated by Order No. R8-2006-0009, which was adopted on April 21, 2006 and expired on April 21, 2011. The Discharger filed a report of waste discharge and submitted an application for renewal of its Waste Discharge Requirements (WDRs) and National Pollutant Discharge Elimination System (NPDES) permit on September 27, 2010. Since the Discharger submitted a timely application for renewal of the Order, the terms and conditions of Order R8-2006-0009 have been automatically continued and remain in effect until new Waste Discharge Requirements and NPDES permit are adopted pursuant to this Order.

## II. FACILITY DESCRIPTION

### A. Wastewater and Biosolids Treatment or Controls

The RRWQCP is a municipal wastewater treatment plant located on a 121-acre site at 5950 Acorn Street in the City of Riverside, south of the Santa Ana River near the intersection of Van Buren Boulevard. The RRWQCP discharges tertiary treated wastewater to Reach 3 of the Santa Ana River.

The RRWQCP treats wastewater from the City of Riverside and from Edgemont Community Services District, Jurupa Community Services District, and Rubidoux Community Services District. The RRWQCP serves a population of 303,871.

The RRWQCP is currently designed to tertiary treat 46 million gallons per day (mgd) of wastewater. The annual average daily flow discharged from this plant in 2012 was approximately 29 mgd.

The RRWQCP consists of two secondary treatment plants (Plants 1 and 2), one tertiary treatment plant that treats the flow from both Plants 1 and 2, and solids handling facilities that treat the biosolids produced from wastewater treatment. Preliminary treatment consists of fine bar screens and vortex grit removal. Secondary treatment consists of primary sedimentations basins, alum and/or polymer injection, aeration basins (11 basins), membrane bioreactors (8 MBRs will be installed by the end of 2015), and secondary sedimentation (4 clarifiers). Tertiary treatment consists of equalization basins, dual media filtration (16 filters), chlorine contact basins (3 basins) where chlorination by sodium hypochlorite, and dechlorination by sodium bisulfite occur. Solids handling includes dissolved air flotation (DAF) thickeners, anaerobic digestion (4 digesters), dewatering (2 belt presses and 2 centrifuges).

Tertiary treated wastewater is discharged from Discharge Points 001 and 002 to Reach 3 of the Santa Ana River, which is a water of the United States. When at least 20:1 dilution is provided by natural flows in the receiving water, the Discharger is authorized to discharge secondary treated wastewater to the Santa Ana River.

Currently, about 0.3 mgd of the tertiary treated and disinfected wastewater is recycled for irrigation by users near the RRWQCP. The City has determined that Riverside Public Utilities (RPU) is the appropriate agency for distribution of recycled water and has recently been issued a Master Reclamation Permit (Order No. R8-2013-0028) to that end. Existing Riverside Public Works customers will transfer to the RPU permit. Riverside Public Utilities is currently in the process of expanding its recycled water distribution system. As discussed in RPU's March 2011 Recycled Water Facility Plan, recycled water is expected to be used for irrigation and non-irrigation uses, such as for construction, commercial/industrial (including cooling), habitat development and maintenance, and recreational uses. RPU may convey recycled water to neighboring agencies, pending specific agreements and other permitting.

On-site storm water is collected and piped to the influent of Plant 1 aeration basins.

Attachment B provides a location map of the Facility. Attachment C provides a flow schematic of the Facility treatment systems.

## B. Discharge Points and Receiving Waters

### 1. Discharge Points

RRWQCP discharges tertiary treated wastewater to the Santa Ana River, Reach 3 at two discharge points designated Discharge Point (DP) 001 and 002. Wastewater from the Facility is discharged at DP 001 into an earthen channel that merges with the SAR. Within the next five years RRWQCP may commingle up to 2 MGD with Hole lake effluent at DP 002. Recycled water is delivered to a nearby user at DP 003. Recycled water is also discharged into the RPU main recycled water distribution system at DP 004, where the water will be delivered to various users under Order No. R8-2013-0028.

Table 2. Summary of Discharge Points

| Discharge Point | Latitude | Longitude | Description |
|---|---|---|---|
| 001 | 33° 57' 55"N | 117° 27' 28"W | Treated wastewater discharged directly to Reach 3 of the Santa Ana River |
| 002 | 33° 57' 44"N | 117° 29' 7"W | Treated wastewater commingled with Hole Lake effluent and then to Reach 3 of the Santa Ana River |
| 003 | 33° 57' 47"N | 117° 27' 41"W | Recycled water for irrigation of Toro turf. |
| 004 | 33° 57' 43"N | 117° 27' 42"W | Recycled water for Riverside Public Utilities distribution system |

### 2. Receiving Waters

a. Surface Waters: Treated wastewater from the Facility is discharged into Reach 3 of the Santa Ana River.

b. Groundwater: The discharger plans to distribute recycled water throughout the City of Riverside. The recycled water use areas overlie a number of Groundwater Management Zones.

### C. Compliance Summary

Based on a review of effluent monitoring data submitted by the Discharger, the wastewater discharged from the Facility was generally in compliance with the effluent limitations specified in the current waste discharge requirements.

## III. APPLICABLE PLANS, POLICIES, AND REGULATIONS

The requirements contained in the proposed Order are based on the requirements and authorities described in this section.

### A. Legal Authorities

This Order is issued pursuant to Chapter 5.5, Division 7 of the California Water Code (commencing with Section 13370) and section 402 of the federal Clean Water Act (CWA) and implementing regulations adopted by the U.S. Environmental Protection Agency (USEPA). This Order also serves as Waste Discharge Requirements (WDRs) pursuant to Article 4, Chapter 4, Division 7 of the Water Code (commencing with Section 13260). This Order shall also serve as an NPDES permit for point source discharges from this Facility to surface waters.

### B. California Environmental Quality Act (CEQA)

Under Water Code Section 13389, this action to adopt waste discharge requirements that serve as an NPDES permit is exempt from the provisions of CEQA, Public Resources Code section 21000 et seq. (County of Los Angeles v. California State Water Resources Control Board (2006) 143 Cal.App.4th 985, mod. (Nov. 6, 2006, B184034) 50 Cal.Rptr.3d 619, 632-636.)

## C. State and Federal Regulations, Policies, and Plans

1. **Water Quality Control Plans.** The Regional Water Board adopted a Water Quality Control Plan for the Santa Ana Basin (hereinafter Basin Plan) on January 24, 1995. The Basin Plan designates beneficial uses, establishes water quality objectives, and contains implementation programs and policies to achieve those objectives for all waters addressed through the plan. In addition, State Water Resources Control Board (State Water Board) Resolution No. 88-63 (Sources of Drinking Water Policy) requires that, with certain exceptions, the Regional Water Board assign the municipal and domestic water supply use to water bodies. Based on the exception criteria specified in Resolution No. 88-63, the Regional Board excepted Reach 3 of the Santa Ana River and downstream reaches from the municipal and domestic supply beneficial use.

   On January 22, 2004, the Regional Water Board adopted Resolution No. R8-2004-0001, amending the Basin Plan to incorporate revised boundaries for groundwater subbasins, now termed "management zones", new nitrate-nitrogen and TDS objectives for the new management zones, and new nitrogen and TDS management strategies applicable to both surface and groundwaters. The State Water Resources Control Board and Office of Administrative Law (OAL) approved the N/TDS Amendment on September 30, 2004 and December 23, 2004, respectively. The surface water standards components of the N/TDS Amendment were approved by the EPA on June 20, 2007. Effluent limitations for TDS and TIN in this Order are based on applicable surface water components of the N/TDS Amendment (i.e., N and TDS wasteload allocations) and are at least as stringent as the limits in the prior Order.

   As previously discussed, the Facility discharges into Reach 3 of the Santa Ana River. Also, recycled water from the Facility will be used over several groundwater management zones. The beneficial uses of these affected waterbodies are as follows:

**Table 3 - Basin Plan Beneficial Uses**

| Discharge Point | Receiving Water Name | Beneficial Use(s) |
|---|---|---|
| 001 and 002 | Reach 3 of Santa Ana River | Present or Potential: Agricultural supply, groundwater recharge, water contact recreation, non-contact water recreation, warm freshwater habitat, wildlife habitat, and rare, threatened or endangered species. |
| 001, 002, and 003 | Chino-South Groundwater Management Zone | Present or Potential: Municipal and domestic supply, agricultural supply, industrial service supply, and industrial process supply. |
| 004 | Chino-South; Arlington; Riverside-A, D, E, and F; and Temescal Groundwater Management Zones | Present or Potential: Municipal and domestic supply, agricultural supply, industrial service supply, and industrial process supply. |

The requirements of this Order implement the Basin Plan.

2. **National Toxics Rule (NTR) and California Toxics Rule (CTR).** USEPA adopted the NTR on December 22, 1992, and later amended it on May 4, 1995 and November 9, 1999. About forty criteria in the NTR applied in California. On May 18, 2000, USEPA adopted the CTR, which incorporated the NTR criteria that were applicable in California. The CTR was amended on February 13, 2001. The NTR and CTR contain water quality criteria for priority pollutants.

3. **State Implementation Policy.** On March 2, 2000, the State Water Board adopted the *Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays, and Estuaries of California* (State Implementation Policy or SIP). The SIP became effective on April 28, 2000 with respect to the priority pollutant criteria promulgated for California by the USEPA through the NTR and to the priority pollutant objectives established by the Regional Water Board in the Basin Plan. The SIP became effective on May 18, 2000 with respect to the priority pollutant criteria promulgated by the USEPA through the CTR. . The State Water Board adopted amendments to the SIP on February 24, 2005 that became effective on July 13, 2005. The SIP establishes implementation provisions for priority pollutant criteria and objectives and provisions for chronic toxicity control. Requirements of this Order implement the SIP.

4. **Alaska Rule.** On March 30, 2000, USEPA revised its regulation that specifies when new and revised State and Tribal water quality standards (WQS) become effective for CWA purposes (40 CFR 131.21, 65 FR 24641, April 27, 2000). Under the revised regulation (also known as the Alaska Rule), new and revised standards submitted to USEPA after May 30, 2000, must be approved by USEPA before being used for CWA purposes. The final rule also provides that standards already in effect and submitted to USEPA by May 30, 2000, may be used for CWA purposes, whether or not approved by USEPA.

5. **Antidegradation Policy.** Section 131.12 of 40 CFR requires that State water quality standards include an antidegradation policy consistent with the federal policy. The State Water Board established California's antidegradation policy in State Water Board Resolution 68-16, which incorporates the requirements of the federal antidegradation policy. Resolution 68-16 requires that existing water quality is maintained unless degradation is justified based on specific findings. As discussed in this Fact Sheet, the permitted discharge is consistent with the antidegradation provision of 40 CFR §131.12 and State Water Board Resolution 68-16.

   All effluent limitations in this Order are at least as stringent as those in prior waste discharge requirements for the Facility. Based on data currently available, discharges in compliance with the terms and conditions of this Order should not result in a lowering of water quality and are therefore consistent with antidegradation provisions.

6. **Anti-Backsliding Requirements.** Sections 402(o)(2) and 303(d)(4) of the CWA and 40 CFR §122.44(l) prohibit backsliding in NPDES permits. These anti-backsliding provisions require that effluent limitations in a reissued permit must be as stringent as those in the previous permit, with some exceptions in which limitations may be relaxed. All effluent limitations in the Order are at least as stringent as the effluent limitations in the previous Order.

7. **Monitoring and Reporting Requirements.** Section 122.48 of 40 CFR requires that all NPDES permits specify requirements for recording and reporting monitoring results. Sections 13267 and 13383 of the CWC authorizes the Regional Water Board to require technical and monitoring reports. The Monitoring and Reporting Program (MRP) establishes monitoring and reporting requirements to implement federal and State requirements. This MRP is provided in Attachment E.

8. **Pretreatment.** This Order contains requirements for the implementation of an effective pretreatment program pursuant to Section 307 of the Federal Clean Water Act; 40 CFR, Parts 35 and 403; and/or Section 2233, Title 23, California Code of Regulations. The Discharger has established an approved regional pretreatment program. The approved pretreatment program and its components, such as Ordinance No. 2330, local limits (adopted by the Discharger in April 18, 1985), and control mechanisms, among others, are an enforceable condition of this Order.

9. **Biosolids.** On February 19, 1993, the USEPA issued a final rule for the use and disposal of sewage sludge, 40 CFR, Part 503. This rule requires that producers of sewage sludge meet certain reporting, handling, and disposal requirements. The State of California has not been delegated the authority to implement this program, therefore, the USEPA is the implementing agency. However, this Order includes Biosolids monitoring requirements.

## D. Impaired Water Bodies on CWA 303(d) List

Reach 3 of the Santa Ana River is included in the USEPA approved 2010 CWA 303(d) list due to pathogen indicators, resulting principally from dairy operations inputs during storm events. This Order requires that the wastewater discharged from the Facility be essentially free of pathogens/pathogen indicators for surface water discharges.

Reach 3 of the Santa Ana River is also included in the USEPA approved 2010 CWA 303(d) list for copper and lead. As noted below, discharges of copper and lead from the Facility do not pose a reasonable potential to cause or contribute to an exceedance of water quality objectives for these constituents.

## IV. RATIONALE FOR EFFLUENT LIMITATIONS AND DISCHARGE SPECIFICATIONS FOR SURFACE WATER DISCHARGES

The CWA requires point source discharges to control the amount of conventional, non-conventional, and toxic pollutants that are discharged into the waters of the United States. The control of pollutants discharged is established through effluent limitations and other requirements in NPDES permits. There are two principal bases for effluent limitations; 40 CFR 122.44(a) requires that permits include applicable technology-based limitations and standards, and 40 CFR 122.44(d) requires that permits include water quality-based effluent limitations to attain and maintain applicable numeric and narrative water quality criteria to protect the beneficial uses of the receiving water.

### A. Discharge Prohibitions

The discharge prohibitions are based on the Federal Clean Water Act, Basin Plan, State Water Resources Control Board's plans and policies, U.S. Environmental Protection Agency guidance and regulations, and previous waste discharge requirements, Order No. R8-2006-0009 and are consistent with the requirements set for other discharges regulated by waste discharge requirements adopted by the Regional Water Board.

### B. Technology-Based Effluent Limitations

#### 1. Scope and Authority

Section 301(b) of the CWA and implementing USEPA permit regulations at 40 CFR 122.44 require that permits include conditions meeting applicable technology-based requirements at a minimum, and any more stringent effluent limitations necessary to meet applicable water quality standards. The discharge authorized by this Order must meet minimum federal technology-based requirements based on Secondary Treatment Standards at 40 CFR Part 133 and/or Best Professional Judgment (BPJ) in accordance with 40 CFR 125.3.

Regulations promulgated in 40 CFR §125.3(a)(1) require technology-based effluent limitations for municipal Dischargers to be placed in waste discharge requirements based on Secondary Treatment Standards or Equivalent to Secondary Treatment Standards.

The Federal Water Pollution Control Act Amendments of 1972 (PL 92-500) established the minimum performance requirements for POTWs [defined in Section 304(d)(1)]. Section 301(b)(1)(B) of that Act requires that such treatment works must, as a minimum, meet effluent limitations based on secondary treatment as defined by the USEPA Administrator. Based on this statutory requirement, USEPA developed secondary treatment regulations, which are specified in 40 CFR Part 133. These technology-based regulations apply to all municipal wastewater treatment plants and identify the minimum level of effluent quality attainable by secondary treatment in terms of biochemical oxygen demand ($BOD_5$), total suspended solids (TSS), and pH.

## 2. Applicable Technology-Based Effluent Limitations

This facility meets the technology-based regulations for the minimum level of effluent quality attainable by secondary treatment in terms of $BOD_5$, total suspended solids and removal rate as summarized in Table 4, below. These effluent limitations have been carried over from the previous Order for secondary treated wastewater discharge under conditions of 20:1 dilution provided by natural flow in the river.

Table 4. Summary of Technology-based Effluent Limitations under 20:1 Dilution

| Constituent | Average Weekly (mg/L) | Average Monthly (mg/L) | Average Monthly Removal Rate % |
|---|---|---|---|
| Biochemical Oxygen Demand, 5-day 20°C | 45 | 30 | 85 |
| Total Suspended Solids | 45 | 30 | 85 |

As noted in section V.C.2.c., below, tertiary treatment is required to protect beneficial uses of the Santa Ana River when 20:1 dilution conditions are not present. During these conditions, the technology-based limits, which are based on BPJ for levels achievable with tertiary treatment, are summarized in the Table below are applicable.

**Table 5. Summary of Technology-Based Effluent Limits for Tertiary Treatment**

| Constituent | Average Weekly (mg/L) | Average Monthly (mg/L) | Average Monthly Removal Rate % |
|---|---|---|---|
| Biochemical Oxygen Demand, 5-day 20°C | 30 | 20 | 85 |
| Total Suspended Solids | 30 | 20 | 85 |

## C. Water Quality-Based Effluent Limitations (WQBELs)

### 1. Scope and Authority

Section 301(b) of the CWA and 40 CFR 122.44(d) require that permits include limitations more stringent than applicable federal technology-based requirements where necessary to achieve applicable water quality standards.

40 CFR 122.44(d)(1)(i) mandates that permits include effluent limitations for all pollutants that are or may be discharged at levels that have a reasonable potential to cause or contribute to an exceedance of a water quality standard, including numeric and narrative objectives within a standard. Where reasonable potential has been established for a pollutant, but there is no numeric criterion or objective for the pollutant, water quality-based effluent limitations (WQBELs) must be established using: (1) USEPA criteria guidance under CWA section 304(a), supplemented where necessary by other relevant information; (2) an indicator parameter for the pollutant of concern; or (3) a calculated numeric water quality criterion, such as a proposed state criterion or policy interpreting the state's narrative criterion, supplemented with other relevant information, as provided in 40 CFR 122.44(d)(1)(vi).

The process for determining reasonable potential and calculating WQBELs when necessary is intended to protect the designated uses of the receiving water as specified in the Basin Plan, and achieve applicable water quality objectives and criteria that are contained in other state plans and policies, or any applicable water quality criteria contained in the CTR and NTR.

## 2. Applicable Beneficial Uses and Water Quality Criteria and Objectives

### a. The Basin Plan

Table 3, above, lists the beneficial uses of the Santa Ana River, Reach 3. The Basin Plan specifies narrative and numeric water quality objectives for all inland surface waters, including the Santa Ana River. Some of those that applicable to these receiving waters are listed in the following table.

Table 6 Examples of Basin Plan Water Quality Objectives

| Constituents | Basis for Limitations |
|---|---|
| Ammonia Nitrogen | Ammonia dissociates under certain conditions to the toxic un-ionized form. Thus ammonia discharges to the Santa Ana River pose a threat to aquatic life and instream beneficial uses, as well as to the beneficial uses of affected groundwater. The Basin Plan specifies un-ionized ammonia objectives and total ammonia nitrogen effluent limitations for discharges to the Santa Ana River. The total ammonia nitrogen effluent limit for discharges to the Santa Ana River, Reach 3, is 5.0 mg/L. |
| Hydrogen Ion (pH) | Hydrogen Ion (pH) is a measure of Hydrogen Ion concentration in the water. Extreme pH levels can have adverse effects on aquatic biota and can corrode pipes and concrete. The Basin Plan specifies that the pH in inland surface waters shall not be depressed below 6.5, nor raised above 8.5 as a result of controllable water quality factors. |
| Total Chlorine Residual | Chlorine and its reaction products are toxic to aquatic life. To protect aquatic life, the Basin Plan specifies that for wastewater discharged into inland surface waters, the chlorine residual should not exceed 0.1 mg/L |
| Total Dissolved Solids | TDS discharges to the Santa Ana River pose a threat to the beneficial uses of affected groundwater. The TDS limit for surface water discharges is based on the Basin Plan wasteload allocation of 650 mg/L at 46 mgd flow. |
| Total Inorganic Nitrogen | Nitrogen discharges to the Santa Ana River pose a threat to the beneficial uses of affected groundwater. The TIN limit for surface water discharges is based on the Basin Plan wasteload allocation of 10 mg/L at 46 mgd flow. |

This Order also includes a TDS limit based on the quality of the water supplied to the service area plus a reasonable use increment for TDS of 250 mg/L. This reasonable use increment is discussed and authorized in the Basin Plan. The more restrictive of the TDS wasteload allocation-based limit or the TDS limit based on water quality plus a reasonable use increment applies to discharges from the facility.

In accordance with 40 CFR Section 122.45(d), there may be instances in which the basis for a limit for a particular continuous discharge may be impracticable to be stated as a maximum daily, average weekly, or average monthly effluent limitation. The Regional Water Board has determined that it is not practicable to express TDS and TIN effluent limitations as average weekly and average monthly effluent limitations because the TDS and TIN objectives in the Basin Plan were established primarily to protect the underlying groundwater. Consequently, a 12-month average period is believed to be more appropriate.

**b. NTR, CTR and SIP**

The National Toxics Rule, California Toxics Rule (CTR) and State Implementation Policy specify numeric objectives for toxic substances and the procedures whereby these objectives are to be implemented. The procedures include those used to conduct reasonable potential analysis to determine the need for effluent limitations for priority and non-priority pollutants.

**c. Requirement to meet Title 22, Tertiary Treatment**

Article 3, Section 60305 of Title 22, Chapter 3, "Use of Recycled water for impoundments" of the California Code of Regulations specifies that recycled water used as a source of supply in a nonrestricted recreational impoundment shall be at all times an adequately disinfected, oxidized, coagulated, clarified, filtered wastewater (tertiary treated). The degree of treatment specified represents an approximately 5-log reduction in the virus content of the water. The California State Department of Health Services (CDHS) has determined that this degree of virus removal is necessary to protect the health of people using these impoundments for water contact recreation. The CDHS has developed wastewater disinfection guidelines ("Wastewater Disinfection for Health Protection", Department of Health Services, Sanitary Engineering Branch, February 1987) for discharges of wastewater to surface waters where water contact recreation (REC-1) is a beneficial use. The disinfection guidelines recommend the same treatment requirements for wastewater discharges to REC-1 waters as those stipulated in Title 22 for supply of recycled water to nonrestricted recreational impoundments, since the public health risks under both scenarios are analogous. The disinfection guidelines are based on sound science and are widely used as guidance to assure public health and beneficial use protection.

Santa Ana River, Reach 3 is not a "nonrestricted recreational impoundment," nor is "recycled water[1]" being used as a supply source for the River pursuant to the definitions in Title 22. However, except during major storms, most of the flow in the River is composed of treated municipal wastewater discharges. The River is used for water contact recreation and, accordingly, is designated REC-1 (water contact beneficial use). People recreating in the River face an exposure similar to those coming in contact with recycled water in an impoundment. Therefore, to protect the water contact recreation beneficial use and to prevent nuisance and health risk, it is necessary and appropriate to require the same degree of treatment for wastewater discharges to the River as would be required for the use of recycled water in a nonrestricted recreational impoundment. Thus, this Order specifies requirements based on tertiary or equivalent treatment.

The Regional Water Board has consulted with the CDPH regarding the applicability of the process design standards (specifically filter rates, CT, and modal contact) for discharges of waste to flowing streams. CDPH has determined that although compliance with these standards is necessary to protect public health when recycled water is used, compliance with these standards is not necessary to protect public health for discharges into waterbodies that provide dilution of the wastewater, provided the performance standards are consistently met. During periods when the receiving water can provide a 1:1 dilution of the wastewater discharge, the Order provides that the specified filter rates, CT, and modal contact time do not apply to wastewater discharges to surface water. The specified filter rates, CT, and modal contact time applies to recycled water use.

### 3. Determining the Need for WQBELs

In accordance with Section 1.3 of the SIP, the Regional Water Board conducted a reasonable potential analysis (RPA) for each priority pollutant with an applicable criterion or objective to determine if a WQBEL is required in the Order. The Regional Water Board analyzed effluent data to determine if a pollutant in a discharge has the reasonable potential to cause or contribute to an excursion above a state water quality standard. For all parameters that have the reasonable potential to cause or contribute to an excursion above a water quality standard, numeric WQBELs are required. The RPA considers criteria from the CTR, and when applicable, water quality objectives specified in the Basin Plan.

Sufficient data are needed to conduct a complete RPA. If data are not sufficient, the Discharger will be required to gather the appropriate data for the Regional Water Board to conduct the RPA. Upon review of the data, and if the Regional Water Board determines that WQBELs are needed to protect the beneficial uses, the permit will be reopened for appropriate modification.

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1 *As defined in the Reclamation Criteria, recycled water means water which, as a result of treatment of domestic wastewater, is suitable for a direct beneficial use or a controlled use that would not otherwise occur.*

The RPA was performed for the priority pollutants for which effluent data were available. Cyanide was determined to have a reasonable potential to exceed water quality objectives. Consequently, effluent limitations for that constituent are included in this Order. Table 7 is a summary of the RPA evaluation for cyanide using monitoring data submitted by the Discharger.

Table 7 RPA Evaluation

| Parameter | Unit | Effluent | CTR | | Is Effluent Limit Required? | |
|---|---|---|---|---|---|---|
| | | MEC | CMC | CCC | CMC | CCC |
| Cyanide | µg/L | 6.0 | 22 | 5.2 | No | Yes |

### 4. WQBEL Calculations

For priority pollutants, water quality based effluent limits are based on monitoring results and the calculation process outlined in Section 1.4 of the California Toxic Rule and the Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays and Estuaries of California. The calculation of the WQBEL for Cyanide is summarized in the following table:

Table 8 – Cyanide Effluent limit Calculation

| | | | | | CV = 0.6 | | | Aquatic | | Human | | Permit Limit | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Caltoxics | | | | Acute M | Chronic M | LTA | Objective/limits | | Health Limits | | Concentration Limit | |
| | Freshwater | | Human Health | | 0.321 | 0.527 | | 3.11 | 1.55 | 2.01 | | | |
| Constituent | CMC | CCC | H2O+Org | Org only | Acute LTA | Chronic LTA | | MDEL | AMEL | MDEL | AMEL | MDEL | AMEL |
| Cyanide | 22.00 | 5.20 | | | 7.06 | 2.74 | 2.74 | 8.52 | 4.25 | | | 8.5 | 4.2 |

### 5. Whole Effluent Toxicity (WET)

This Order does not specify WET limits but requires continued chronic toxicity monitoring. Effluent monitoring data indicated that the two month median value of 1.0 $TU_C$ for the survival or reproduction endpoint has never been exceeded.

## D. Summary of Effluent Limitations

### 1. Satisfaction of Anti-Backsliding Requirements

All effluent limitations in this Order are at least as stringent as the effluent limitations in the previous Order.

### 2. Satisfaction of Antidegradation Policy

Discharges in conformance with the requirements of this Order will not result in a lowering of water quality and therefore conform to antidegradation requirements specified in Resolution No. 68-16, which incorporates the federal antidegradation policy at 40 CFR 131.12.

### 3. Stringency of Requirements for Individual Pollutants

Water quality-based effluent limitations have been scientifically derived to implement water quality objectives that protect beneficial uses. Both the beneficial uses and the water quality objectives have been approved pursuant to federal law and are the applicable federal water quality standards. To the extent that toxic pollutant water quality-based effluent limitations were derived from the CTR, the CTR is the applicable standard pursuant to 40 CFR131.38. The scientific procedures for calculating the individual water quality-based effluent limitations for priority pollutants are based on the CTR-SIP, which was approved by USEPA on May 18, 2000. Apart from certain surface water standards changes resulting from the N/TDS Basin Plan amendment that do not materially affect the quality requirements for the discharges regulated by this Order, all beneficial uses and water quality objectives contained in the Basin Plan were approved under state law and submitted to and approved by USEPA prior to May 30, 2000. Any water quality objectives and beneficial uses submitted to USEPA prior to May 30, 2000, but not approved by USEPA before that date, are nonetheless "applicable water quality standards for purposes of the CWA" pursuant to 40 CFR 131.21(c)(1). Collectively, this Order's restrictions on individual pollutants are no more stringent than required to implement the requirements of the CWA.

### 4. Summary of Water Quality Based Effluent Limitations

Table 9 Summary of Water Quality-based Effluent Limitations

| Parameter | Units | Effluent Limitations | | | | | Basis |
|---|---|---|---|---|---|---|---|
| | | Average Monthly or as noted herein | Average Weekly | Max Daily | Instantaneous Minimum | Instantaneous Maximum | |
| Ammonia-Nitrogen | mg/L | 5.0 | | | | | BP |
| pH | Std. unit | -- | -- | -- | 6.5 | 8.5 | BP |
| TDS | mg/L | 650 (12-M avg) | | | | | BP |
| TIN | mg/L | 10 (12-M avg) | | | | | BP |
| Total Residual Chlorine | mg/L | -- | -- | | -- | 0.1 | BP |
| Cyanide | µg/L | 4.2 | | 8.5 | | | CTR, SIP |
| Coliform | MPN/ 100mL | | 2.2 | | | | Title 22 |

Notes: BP= Basin Plan, Title 22= Section 60305, Article 3, Chapter 3, Title 22 of the California Code of Regulations

## V. RATIONALE FOR RECYCLED WATER SPECIFICATIONS

1. Section 13523 of the California Water Code provides that a Regional Water Board, after consulting with and receiving the recommendations from the CDHS and any party who has requested in writing to be consulted, and after any necessary hearing, shall prescribe water reclamation requirements for water which is used or proposed to be used as recycled water, if, in the judgment of the Board, such requirements are necessary to protect the public health, safety, or welfare. Section 13523 further provides that such requirements shall include, or be in conformance with, the statewide uniform water recycling criteria established by the CDHS pursuant to California Water Code Section 13521.

2. Reclamation specifications in the Order are based on the recycling criteria contained in Title 22, Division 4, Chapter 3, Sections 60301 through 60355, California Code of Regulations and the California Water Code Section 13521.

3. Summary of Reclamation Effluent Limitations:

**Table 10 Summary of Recycled Water Limitations**

| Parameter | Units | Effluent Limitations | | | | | Basis |
|---|---|---|---|---|---|---|---|
| | | Average Monthly | Average Weekly | Maximum Daily | Instantaneous Minimum | Instantaneous Maximum | |
| $BOD_5$ | mg/L | 20 | 30 | -- | -- | -- | BPJ |
| Total Suspended Solids | mg/L | 20 | 30 | -- | -- | -- | BPJ |
| TDS | mg/L | 650 | | | | | BP |
| CT | milligram-minutes per liter | -- | -- | -- | 450 | -- | Title 22 |
| Coliform | MPN/100 mL | -- | 2.2 median in 7 days | -- | -- | -- | Title 22 |
| Turbidity | NTU | -- | -- | 2 avg. | | -- | Title 22 |

Notes: BP= Basin Plan, Title 22= Section 60305, Article 3, Chapter 3, Title 22 of the California Code of Regulations

## VI. RATIONALE FOR RECEIVING WATER LIMITATIONS

### A. Surface Water

The surface water receiving water limitations in the Order are based upon the water quality objectives contained in the Basin Plan.

### B. Groundwater

The groundwater water receiving water limitations in the Order are based upon the water quality objectives contained in the Basin Plan.

## VII. RATIONALE FOR MONITORING AND REPORTING REQUIREMENTS

Sections 13267 and 13383 of the CWC authorize the Regional Water Boards to require technical and monitoring reports. 40 CFR 122.48 also requires all NPDES permits to specify recording and reporting of monitoring results. The Monitoring and Reporting Program (MRP), Attachment E of this Order, establishes monitoring and reporting requirements to implement federal and State requirements. The following provides the rationale for the monitoring and reporting requirements contained in the MRP for this facility.

### A. Influent Monitoring

This Order carries forward the treatment plant influent monitoring requirements specified in the previous Order without change. Influent monitoring is required to help determine the effectiveness of the pretreatment program and assess treatment plant performance.

### B. Effluent Monitoring

The Discharger is required to conduct monitoring of the permitted discharges in order to evaluate compliance with permit conditions. Pollutants to be monitored include all pollutants for which effluent limitations are specified. Further, in accordance with Section 1.3 of the SIP, periodic monitoring is required for all priority pollutants defined by the CTR, for which criteria apply and for which no effluent limitations have been established, to evaluate reasonable potential to cause or contribute to an excursion above a water quality standard.

This Order requires the Discharger to conduct a priority pollutant scan once annually. This Order also requires the Discharger to conduct accelerated monitoring for those constituents that are detected in the annual priority pollutant scan.

### C. Whole Effluent Toxicity Testing Requirements

Whole effluent toxicity (WET) is an indicator of the toxic effects of the aggregate mixture of pollutants in the effluent on receiving waters. The WET approach allows for protection of the narrative "no toxics in toxic amounts" criterion while implementing numeric criteria for toxicity. There are two types of WET tests: acute and chronic. An acute toxicity test is conducted over shorter time period and measures mortality. A chronic toxicity test is conducted over a longer period of time and may measure mortality, reproduction, and growth.

The Basin Plan specifies a narrative objective for toxicity, requiring that all waters be maintained free of toxic substances in concentrations that are lethal to or produce other detrimental response on aquatic organisms. Detrimental response includes but is not limited to decreased growth rate, decreased reproductive success of resident or indicator species, and/or significant alterations in population, community ecology, or receiving water biota.

In addition to the Basin Plan requirements, Section 4 of the SIP states that a chronic toxicity effluent limitation is required in permits for all discharges that will cause, have the reasonable potential to cause, or contribute to chronic toxicity in receiving waters. Therefore, in accordance with the SIP, this Order requires the Discharger to conduct chronic toxicity testing. In addition, the Order establishes thresholds that when exceeded requires the Discharger to conduct accelerated toxicity testing and/or conduct toxicity identification evaluation (TIE) studies.

This Order requires the Discharger to conduct chronic toxicity testing of the effluent on a monthly basis. The Order also requires the Discharger to conduct an Initial Investigation Toxicity Reduction Evaluation (IITRE) program when either the two-month median of toxicity test results exceeds 1 TUc or any single test exceeds 1.7 TUc for survival endpoint. Based on the results of this investigation program and at the discretion of the Executive Officer, a more rigorous Toxicity Reduction Evaluation/Toxicity Identification Evaluation (TRE/TIE) may be required. A re-opener provision is included in the Order to incorporate a chronic toxicity effluent limitation if warranted by the toxicity test results.

## D. Receiving Water Monitoring

### 1. Surface Water

a. For discharges of secondary treated and disinfected effluent when 20:1 or more dilution is provided by the Santa Ana River at the point of discharge, the Order requires the Discharger to establish a sampling station(s) at a suitable location(s) where the flow[2] in the River at the point of discharge can be determined. The Order also requires that flow measurements in the river are made prior to any direct discharge to the river and shall continue on a daily basis until the discharge is terminated.

b. Receiving water monitoring is required to determine compliance with receiving water limitations and to characterize the water quality of the receiving water. Requirements are based on the Basin Plan.

### 2. Groundwater – Not Applicable

## E. Other Monitoring Requirements

1. **Water Supply Monitoring** - The Discharger will be required to collect a sample of each source of water supplied and analyze for total dissolved solids. The result of this monitoring will enable the discharger to show compliance with TDS incremental limitation in the Order.

2. **Biosolids Monitoring** - This Order continues the monitoring requirements specified in Order No. R8-2006-0009, with minor modification.

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2 *Exclusive of discharges to surface waters from upstream publicly owned treatment works.*

3. **Pretreatment Monitoring** - These monitoring and reporting requirements are established pursuant EPA 40 CFR 403 regulations.

## VIII. RATIONALE FOR PROVISIONS

### A. Standard Provisions

Standard Provisions, which apply to all NPDES permits in accordance with 40 CFR 122.41, and additional conditions applicable to specified categories of permits in accordance with 40 CFR 122.42, are provided in Attachment D.

Title 40 CFR Section 122.41(a)(1) and (b) through (n) establish conditions that apply to all state-issued NPDES permits. These conditions must be incorporated into the permits either expressly or by reference. If incorporated by reference, a specific citation to the regulations must be included in the Order. 40 CFR Section 123.25(a)(12) allows the State to omit or modify conditions to impose more stringent requirements. In accordance with Section 123.25, this Order omits federal conditions that address enforcement authority specified in 40 CFR Sections 122.41(j)(5) and (k)(2) because the enforcement authority under the CWC is more stringent. In lieu of these conditions, this Order incorporates by reference CWC section 13387(e).

This Order also contains Standard Provisions that the Regional Water Board incorporates into NPDES permits in addition to the federal Standard Provisions included in Attachment D. One of these is a requirement that the Facility be protected from inundation or washout due to floods with a 100-year return frequency. Past studies showed that the levee that separates the Facility from the Santa Ana River provided such protection. However, a recent reevaluation of the levee indicates that it does not provide the necessary protection. The Discharger proposes to construct a floodwall on top of the existing levee to provide sufficient hydraulic protection for a 100-year flood event. The Discharger proposes to complete construction by August 1, 2017 and achieve full compliance by September 1, 2017. This Order requires compliance with the subject provision by September 1, 2017 and requires the submittal of semiannual progress reports until compliance is achieved.

### B. Special Provisions

#### 1. Reopener Provisions

The reopener provision is based on 40 CFR Part 123. The Regional Water Board may reopen the permit to modify permit conditions and requirements. Causes for modifications include the promulgation of new regulations, modification in sludge use or disposal practices, or adoption of new regulations by the State Board or Regional Water Board, including revisions to the Basin Plan.

**2. Special Studies and Additional Monitoring Requirements**

a. This Order requires the Discharger to submit a report that details the manner in which sampling, monitoring and reporting will be performed as required in the Order. This is a standard requirement for all POTW dischargers within the Region.

b. This Order requires the Discharger to develop procedures to conduct Toxicity Identification and Reduction Evaluations. This provision is based on the SIP, Section 4, Toxicity Control Provisions.

**3. Best Management Practices and Pollution Prevention**

The requirements are based on the Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays, and Estuaries of California, Section 2.4.5.1. and are applicable to POTW facilities including the Discharger.

**4. Construction, Operation, and Maintenance Specifications**

The requirements are based on requirements that were specified in the prior Order and industry standards.

**5. Special Provisions for Municipal Facilities (POTWs Only)**

a. Sewer Collection System Requirements: The State Water Board issued General Waste Discharge Requirements for Sanitary Sewer Systems, Order No. 2006-0003-DWQ on May 2, 2006. The General Order requires public agencies that own or operate sanitary sewer systems with greater than one mile of pipes or sewer lines to enroll for coverage under the General Order. The General Order requires agencies to develop sanitary sewer management plans (SSMPs) and report all sanitary sewer overflows (SSOs), among other requirements and prohibitions.

   Furthermore, the General Order contains requirements for operation and maintenance of collection systems and for reporting and mitigating sanitary sewer overflows. Inasmuch that the Discharger's collection system is part of the system that is subject to this Order, certain standard provisions are applicable as specified in Provisions, section VI.C.5. For instance, the 24-hour reporting requirements in this Order are not included in the General Order. The Discharger must comply with both the General Order and this Order. The Discharger and public agencies that are discharging wastewater into the facility were required to obtain enrollment for regulation under the General Order.

b. Biosolids Disposal Requirements: On February 19, 1993, the USEPA issued a final rule for the use and disposal of sewage sludge, 40 CFR, Part 503. This rule requires that producers of sewage sludge meet certain reporting, handling, and disposal requirements. The State of California has not been delegated the authority to implement this program, therefore, the U.S. Environmental Protection Agency is the implementing agency.

c. Pretreatment Requirements: The treatment plant capacity is 46 mgd and there are significant industrial users within the service areas. Consequently, this Order contains requirements for the implementation of an effective pretreatment program pursuant to Section 307 of the Federal Clean Water Act; Parts 35 and 403 of Title 40, Code of Federal Regulations (40 CFR 35 and 40 CFR 403); and/or Section 2233, Title 23, California Code of Regulations.

## IX. PUBLIC PARTICIPATION

The California Regional Water Quality Control Board, Santa Ana Region (Regional Water Board) is considering the issuance of waste discharge requirements (WDRs) that will serve as a National Pollutant Discharge Elimination System (NPDES) permit for the Riverside Regional Water Quality Control Plant. As a step in the WDR adoption process, the Regional Water Board staff has developed tentative WDRs. The Regional Water Board encourages public participation in the WDR adoption process.

### A. Notification of Interested Parties

The Regional Water Board has notified the Discharger and interested agencies and persons of its intent to prescribe waste discharge requirements for the discharges and has provided them with an opportunity to submit their written comments and recommendations. Notification was provided through the posting of a Notice of Public Hearing in the area of the discharge, in the local newspaper, and at the Regional Water Board website.

### B. Written Comments

The staff determinations are tentative. Interested persons are invited to submit written comments concerning these tentative WDRs. Comments should be submitted either in person or by mail to the Executive Office at the Regional Water Board at the address above on the cover page of this Order.

To be fully responded to by staff and considered by the Regional Water Board, written comments should be received at the Regional Water Board offices by 5:00 p.m. on October 4, 2013.

### C. Public Hearing

The Regional Water Board will hold a public hearing on the tentative WDRs during its regular Board meeting on the following date and time and at the following location:

| | |
|---|---|
| Date: | November 1, 2013 |
| Time: | 9:00 A.M. |
| Location: | Irvine Ranch Water District<br>15600 Sand Canyon Avenue<br>Irvine, CA |

Interested persons are invited to attend. At the public hearing, the Regional Water Board will hear testimony, if any, pertinent to the discharge, WDRs, and permit. Oral testimony will be heard; however, for accuracy of the record, important testimony should be in writing.

Please be aware that dates and venues may change. Our web address http://www.waterboards.ca.gov/santaana where you can access the current agenda for changes in dates and locations.

**D. Waste Discharge Requirements Petitions**

Any aggrieved person may petition the State Water Resources Control Board to review the decision of the Regional Water Board regarding the final WDRs. The petition must be submitted within 30 days of the Regional Water Board's action to the following address:

State Water Resources Control Board
Office of Chief Counsel
P.O. Box 100, 1001 I Street
Sacramento, CA 95812-0100

**E. Information and Copying**

The Report of Waste Discharge (RWD), related documents, tentative effluent limitations and special provisions, comments received, and other information are on file and may be inspected at the address above at any time between 9:00 a.m. and 3:00 p.m. Monday through Friday. Copying of documents may be arranged through the Regional Water Board by calling (951) 782-4130.

**F. Register of Interested Persons**

Any person interested in being placed on the mailing list for information regarding the WDRs and NPDES permit should contact the Regional Water Board, reference this facility, and provide a name, address, and phone number.

**G. Additional Information**

Requests for additional information or questions regarding this Order should be directed to Najah N. Amin at (951) 320-6362

## ATTACHMENT G - EPA PRIORITY POLLUTANT LIST

| EPA PRIORITY POLLUTANT LIST | | | | | |
|---|---|---|---|---|---|
| **Metals** | | **Acid Extractibles** | | **Base/Neutral Extractibles (continuation)** | |
| 1. | Antimony | 45. | 2-Chlorophenol | 91. | Hexachloroethane |
| 2. | Arsenic | 46. | 2,4-Dichlorophenol | 92. | Indeno (1,2,3-cd) Pyrene |
| 3. | Beryllium | 47. | 2,4-Dimethylphenol | 93. | Isophorone |
| 4. | Cadmium | 48. | 2-Methyl-4,6-Dinitrophenol | 94. | Naphthalene |
| 5a. | Chromium (III) | 49. | 2,4-Dinitrophenol | 95. | Nitrobenzene |
| 5b. | Chromium (VI) | 50. | 2-Nitrophenol | 96. | N-Nitrosodimethylamine |
| 6. | Copper | 51. | 4-Nitrophenol | 97. | N-Nitrosodi-N-Propylamine |
| 7. | Lead | 52. | 3-Methyl-4-Chlorophenol | 98. | N-Nitrosodiphenylamine |
| 8. | Mercury | 53. | Pentachlorophenol | 99. | Phenanthrene |
| 9. | Nickel | 54. | Phenol | 100. | Pyrene |
| 10. | Selenium | 55. | 2, 4, 6 – Trichlorophenol | 101. | 1,2,4-Trichlorobenzene |
| 11. | Silver | | **Base/Neutral Extractibles** | | **Pesticides** |
| 12. | Thallium | 56. | Acenaphthene | 102. | Aldrin |
| 13. | Zinc | 57. | Acenaphthylene | 103. | Alpha BHC |
| | **Miscellaneous** | 58. | Anthracene | 104. | Beta BHC |
| 14. | Cyanide | 59. | Benzidine | 105. | Delta BHC |
| 15. | Asbestos (not required unless requested) | 60. | Benzo (a) Anthracene | 106. | Gamma BHC |
| 16. | 2,3,7,8-Tetrachlorodibenzo-P-Dioxin (TCDD) | 61. | Benzo (a) Pyrene | 107. | Chlordane |
| | **Volatile Organics** | 62. | Benzo (b) Fluoranthene | 108. | 4, 4' - DDT |
| 17. | Acrolein | 63. | Benzo (g,h,i) Perylene | 109. | 4, 4' - DDE |
| 18. | Acrylonitrile | 64. | Benzo (k) Fluoranthene | 110. | 4, 4' - DDD |
| 19. | Benzene | 65. | Bis (2-Chloroethoxy) Methane | 111. | Dieldrin |
| 20. | Bromoform | 66. | Bis (2-Chloroethyl) Ether | 112. | Alpha Endosulfan |
| 21. | Carbon Tetrachloride | 67. | Bis (2-Chloroisopropyl) Ether | 113. | Beta Endosulfan |
| 22. | Chlorobenzene | 68. | Bis (2-Ethylhexyl) Phthalate | 114. | Endosulfan Sulfate |
| 23. | Chlorodibromomethane | 69. | 4-Bromophenyl Phenyl Ether | 115. | Endrin |
| 24. | Chloroethane | 70. | Butylbenzyl Phthalate | 116. | Endrin Aldehyde |
| 25. | 2-Chloroethyl Vinyl Ether | 71. | 2-Chloronaphthalene | 117. | Heptachlor |
| 26. | Chloroform | 72. | 4-Chlorophenyl Phenyl Ether | 118. | Heptachlor Epoxide |
| 27. | Dichlorobromomethane | 73. | Chrysene | 119. | PCB 1016 |
| 28. | 1,1-Dichloroethane | 74. | Dibenzo (a,h) Anthracene | 120. | PCB 1221 |
| 29. | 1,2-Dichloroethane | 75. | 1,2-Dichlorobenzene | 121. | PCB 1232 |
| 30. | 1,1-Dichloroethylene | 76. | 1,3-Dichlorobenzene | 122. | PCB 1242 |
| 31. | 1,2-Dichloropropane | 77. | 1,4-Dichlorobenzene | 123. | PCB 1248 |
| 32. | 1,3-Dichloropropylene | 78. | 3,3'-Dichlorobenzidine | 124. | PCB 1254 |
| 33. | Ethylbenzene | 79. | Diethyl Phthalate | 125. | PCB 1260 |
| 34. | Methyl Bromide | 80. | Dimethyl Phthalate | 126. | Toxaphene |
| 35. | Methyl Chloride | 81. | Di-n-Butyl Phthalate | | |
| 36. | Methylene Chloride | 82. | 2,4-Dinitrotoluene | | |
| 37. | 1,1,2,2-Tetrachloroethane | 83. | 2-6-Dinitrotoluene | | |
| 38. | Tetrachloroethylene | 84. | Di-n-Octyl Phthalate | | |
| 39. | Toluene | 85. | 1,2-Dipenylhydrazine | | |
| 40. | 1,2-Trans-Dichloroethylene | 86. | Fluoranthene | | |
| 41. | 1,1,1-Trichloroethane | 87. | Fluorene | | |
| 42. | 1,1,2-Trichloroethane | 88. | Hexachlorobenzene | | |
| 43. | Trichloroethylene | 89. | Hexachlorobutadiene | | |
| 44. | Vinyl Chloride | 90. | Hexachlorocyclopentadiene | | |

**ATTACHMENT H – MINIMUM LEVELS**

**MINIMUM LEVELS IN PPB (µg/L)**

| Table 1- VOLATILE SUBSTANCES[1] | GC | GCMS |
|---|---|---|
| Acrolein | 2.0 | 5 |
| Acrylonitrile | 2.0 | 2 |
| Benzene | 0.5 | 2 |
| Bromoform | 0.5 | 2 |
| Carbon Tetrachloride | 0.5 | 2 |
| Chlorobenzene | 0.5 | 2 |
| Chlorodibromomethane | 0.5 | 2 |
| Chloroethane | 0.5 | 2 |
| Chloroform | 0.5 | 2 |
| Dichlorobromomethane | 0.5 | 2 |
| 1,1 Dichloroethane | 0.5 | 1 |
| 1,2 Dichloroethane | 0.5 | 2 |
| 1,1 Dichloroethylene | 0.5 | 2 |
| 1,2 Dichloropropane | 0.5 | 1 |
| 1,3 Dichloropropylene (volatile) | 0.5 | 2 |
| Ethylbenzene | 0.5 | 2 |
| Methyl Bromide (*Bromomethane* ) | 1.0 | 2 |
| Methyl Chloride (*Chloromethane* ) | 0.5 | 2 |
| Methylene Chloride (*Dichloromethane*) | 0.5 | 2 |
| 1,1,2,2 Tetrachloroethane | 0.5 | 1 |
| Tetrachloroethylene | 0.5 | 2 |
| Toluene | 0.5 | 2 |
| trans-1,2 Dichloroethylene | 0.5 | 1 |
| 1,1,1 Trichloroethane | 0.5 | 2 |
| 1,1,2 Trichloroethane | 0.5 | 2 |
| Trichloroethylene | 0.5 | 2 |
| Vinyl Chloride | 0.5 | 2 |
| 1,2 Dichlorobenzene (volatile) | 0.5 | 2 |
| 1,3 Dichlorobenzene (volatile) | 0.5 | 2 |
| 1,4 Dichlorobenzene (volatile) | 0.5 | 2 |

**Selection and Use of Appropriate ML Value:**

ML Selection: When there is more than one ML value for a given substance, the discharger may select any one of those ML values, and their associated analytical methods, listed in this Attachment that are below the calculated effluent limitation for compliance determination. If no ML value is below the effluent limitation, then the discharger shall select the lowest ML value, and its associated analytical method, listed in the PQL Table.

ML Usage: The ML value in this Attachment represents the lowest quantifiable concentration in a sample based on the proper application of all method-based analytical procedures and the absence of any matrix interferences. Assuming that all method-specific analytical steps are followed, the ML value will also represent, after the appropriate application of method-specific factors, the lowest standard in the calibration curve for that specific analytical technique. Common analytical practices sometimes require different treatment of the sample relative to calibration standards.
Note: chemical names in parenthesis and italicized is another name for the constituent.

1 *The normal method-specific factor for these substances is 1, therefore, the lowest standard concentration in the calibration curve is equal to the above ML value for each substance.*

**MINIMUM LEVELS IN PPB (µg/L)**

| Table 2 – Semi-Volatile Substances[2] | GC | GCMS | LC |
|---|---|---|---|
| 2-Chloroethyl vinyl ether | 1 | 1 | |
| 2 Chlorophenol | 2 | 5 | |
| 2,4 Dichlorophenol | 1 | 5 | |
| 2,4 Dimethylphenol | 1 | 2 | |
| 4,6 Dinitro-2-methylphenol | 10 | 5 | |
| 2,4 Dinitrophenol | 5 | 5 | |
| 2- Nitrophenol | | 10 | |
| 4- Nitrophenol | 5 | 10 | |
| 4 Chloro-3-methylphenol | 5 | 1 | |
| 2,4,6 Trichlorophenol | 10 | 10 | |
| Acenaphthene | 1 | 1 | 0.5 |
| Acenaphthylene | | 10 | 0.2 |
| Anthracene | | 10 | 2 |
| Benzidine | | 5 | |
| Benzo (a) Anthracene (*1,2 Benzanthracene*) | 10 | 5 | |
| Benzo(a) pyrene (*3,4 Benzopyrene*) | | 10 | 2 |
| Benzo (b) Flouranthene (*3,4 Benzofluoranthene*) | | 10 | 10 |
| Benzo(g,h,i)perylene | | 5 | 0.1 |
| Benzo(k)fluoranthene | | 10 | 2 |
| bis 2-(1-Chloroethoxyl) methane | | 5 | |
| bis(2-chloroethyl) ether | 10 | 1 | |
| bis(2-Chloroisopropyl) ether | 10 | 2 | |
| bis(2-Ethylhexyl) phthalate | 10 | 5 | |
| 4-Bromophenyl phenyl ether | 10 | 5 | |
| Butyl benzyl phthalate | 10 | 10 | |
| 2-Chloronaphthalene | | 10 | |
| 4-Chlorophenyl phenyl ether | | 5 | |
| Chrysene | | 10 | 5 |
| Dibenzo(a,h)-anthracene | | 10 | 0.1 |
| 1,2 Dichlorobenzene (semivolatile) | 2 | 2 | |
| 1,3 Dichlorobenzene (semivolatile) | 2 | 1 | |
| 1,4 Dichlorobenzene (semivolatile) | 2 | 1 | |
| 3,3' Dichlorobenzidine | | 5 | |
| Diethyl phthalate | 10 | 2 | |
| Dimethyl phthalate | 10 | 2 | |
| di-n-Butyl phthalate | | 10 | |
| 2,4 Dinitrotoluene | 10 | 5 | |
| 2,6 Dinitrotoluene | | 5 | |
| di-n-Octyl phthalate | | 10 | |
| 1,2 Diphenylhydrazine | | 1 | |
| Fluoranthene | 10 | 1 | 0.05 |
| Fluorene | | 10 | 0.1 |
| Hexachloro-cyclopentadiene | 5 | 5 | |
| 1,2,4 Trichlorobenzene | 1 | 5 | |

**MINIMUM LEVELS IN PPB (µg/L)**

| Table 2 - SEMI-VOLATILE SUBSTANCES[2] | GC | GCMS | LC | COLOR |
|---|---|---|---|---|
| Pentachlorophenol | 1 | 5 | | |
| Phenol [3] | 1 | 1 | | 50 |
| Hexachlorobenzene | 5 | 1 | | |
| Hexachlorobutadiene | 5 | 1 | | |
| Hexachloroethane | 5 | 1 | | |
| Indeno(1,2,3,cd)-pyrene | | 10 | 0.05 | |
| Isophorone | 10 | 1 | | |
| Naphthalene | 10 | 1 | 0.2 | |
| Nitrobenzene | 10 | 1 | | |
| N-Nitroso-dimethyl amine | 10 | 5 | | |
| N-Nitroso -di n-propyl amine | 10 | 5 | | |
| N-Nitroso diphenyl amine | 10 | 1 | | |
| Phenanthrene | | 5 | 0.05 | |
| Pyrene | | 10 | 0.05 | |

| Table 3– INORGANICS[4] | FAA | GFAA | ICP | ICPMS | SPGFAA | HYDRIDE | CVAA | COLOR | DCP |
|---|---|---|---|---|---|---|---|---|---|
| Antimony | 10 | 5 | 50 | 0.5 | 5 | 0.5 | | | 1000 |
| Arsenic | | 2 | 10 | 2 | 2 | 1 | | 20 | 1000 |
| Beryllium | 20 | 0.5 | 2 | 0.5 | 1 | | | | 1000 |
| Cadmium | 10 | 0.5 | 10 | 0.25 | 0.5 | | | | 1000 |
| Chromium (total) | 50 | 2 | 10 | 0.5 | 1 | | | | 1000 |
| Chromium VI | 5 | | | | | | | 10 | |
| Copper | 25 | 5 | 10 | 0.5 | 2 | | | | 1000 |
| Lead | 20 | 5 | 5 | 0.5 | 2 | | | | 10000 |
| Mercury | | | | 0.5 | | | 0.2 | | |
| Nickel | 50 | 5 | 20 | 1 | 5 | | | | 1000 |
| Selenium | | 5 | 10 | 2 | 5 | 1 | | | 1000 |
| Silver | 10 | 1 | 10 | 0.25 | 2 | | | | 1000 |
| Thallium | 10 | 2 | 10 | 1 | 5 | | | | 1000 |
| Zinc | 20 | | 20 | 1 | 10 | | | | 1000 |
| Cyanide | | | | | | | | 5 | |

2 *With the exception of phenol by colorimetric technique, the normal method-specific factor for these substances is 1000, therefore, the lowest standards concentration in the calibration curve is equal to the above ML value for each substance multiplied by 1000.*

3 *Phenol by colorimetric technique has a factor of 1.*

4 *The normal method-specific factor for these substances is 1, therefore, the lowest standard concentration in the calibration curve is equal to the above ML value for each substance.*

**MINIMUM LEVELS IN PPB (µg/L)**

| Table 4- PESTICIDES – PCBs[5] | GC |
|---|---|
| Aldrin | 0.005 |
| alpha–BHC (*a-Hexachloro-cyclohexane*) | 0.01 |
| beta–BHC (*b-Hexachloro-cyclohexane*) | 0.005 |
| Gamma–BHC (*Lindane; g-Hexachloro-cyclohexane*) | 0.02 |
| Delta-BHC (*d-Hexachloro-cyclohexane*) | 0.005 |
| Chlordane | 0.1 |
| 4,4'-DDT | 0.01 |
| 4,4'-DDE | 0.05 |
| 4,4'-DDD | 0.05 |
| Dieldrin | 0.01 |
| Alpha-Endosulfan | 0.02 |
| Beta-Endosulfan | 0.01 |
| Endosulfan Sulfate | 0.05 |
| Endrin | 0.01 |
| Endrin Aldehyde | 0.01 |
| Heptachlor | 0.01 |
| Heptachlor Epoxide | 0.01 |
| PCB 1016 | 0.5 |
| PCB 1221 | 0.5 |
| PCB 1232 | 0.5 |
| PCB 1242 | 0.5 |
| PCB 1248 | 0.5 |
| PCB 1254 | 0.5 |
| PCB 1260 | 0.5 |
| Toxaphene | 0.5 |

Techniques:
GC - Gas Chromatography
GCMS - Gas Chromatography/Mass Spectrometry
HRGCMS - High Resolution Gas Chromatography/Mass Spectrometry (i.e., EPA 1613, 1624, or 1625)
LC - High Pressure Liquid Chromatography
FAA - Flame Atomic Absorption
GFAA - Graphite Furnace Atomic Absorption
HYDRIDE - Gaseous Hydride Atomic Absorption
CVAA - Cold Vapor Atomic Absorption
ICP - Inductively Coupled Plasma
ICPMS - Inductively Coupled Plasma/Mass Spectrometry
SPGFAA - Stabilized Platform Graphite Furnace Atomic Absorption (i.e., EPA 200.9)
DCP - Direct Current Plasma
COLOR – Colorimetric

5 *The normal method-specific factor for these substances is 100, therefore, the lowest standard concentration in the calibration curve is equal to the above ML value for each substance multiplied by 100.*

## ATTACHMENT I – TRIGGERS FOR QUARTERLY MONITORING OF PRIORITY POLLUTANTS

The values shown in the table below are fifty percent of the most stringent applicable receiving water objectives (freshwater or human health (consumption of organisms only) as specified for that pollutant in 40 CFR 131.38[6]). For hardness dependent metals, the hardness value used is 272 mg/L and for pentachlorophenol, the pH value used is 7.5 standard units.

| | CONSTITUENT | µg/L |
|---|---|---|
| 1 | Antimony | 7 |
| 2 | Arsenic | 75 |
| 3 | Beryllium | -- |
| 4 | Cadmium | 3.7 |
| 5a | Chromium III | 120 |
| 5b | Chromium VI | 5.5 |
| 6 | Copper | 14.2 |
| 7 | Lead | 12.2 |
| 8 | Mercury | 0.026 |
| 9 | Nickel | 30 |
| 10 | Selenium | 2.5 |
| 11 | Silver | 11.2 |
| 12 | Thallium | 2.8 |
| 13 | Zinc | 70 |
| 14 | Cyanide | 2.6 |
| 15 | Asbestos | -- |
| 16 | 2,3,7,8-TCDD (Dioxin) | 0.000000007 |
| 17 | Acrolein | 160 |
| 18 | Acrylonitrile | 0.03 |
| 19 | Benzene | 0.6 |
| 20 | Bromoform | 2.2 |
| 21 | Carbon Tetrachloride | 0.13 |
| 22 | Chlorobenzene | 340 |
| 23 | Chlorodibromomethane | 0.22 |
| 24 | Chloroethane | -- |
| 25 | 2-Chloroethyl vinyl ether | -- |
| 26 | Chloroform | -- |
| 27 | Dichlorobromomethane | 0.28 |
| ***28*** | ***1,1-Dichloroethane*** | ***5*** |
| 29 | 1,2-Dichloroethane | 0.19 |
| 30 | 1,1-Dichloroethylene | 0.029 |
| 31 | 1,2-Dichloropropane | 0.26 |
| 32 | 1,3-Dichloropropylene | 5 |
| ***33*** | ***Ethylbenzene*** | ***0.3*** |
| 34 | Methyl Bromide | 24 |
| 35 | Methyl Chloride | -- |
| 36 | Methylene Chloride | 2.4 |
| 37 | 1,1,2,2-Tetratchloroethane | 0.085 |

| | CONSTITUENT | µg/L |
|---|---|---|
| 38 | Tetratchloroethylene | 0.4 |
| ***39*** | ***Toluene*** | ***0.15*** |
| ***40*** | ***1,2,-Trans-dichloroethylene*** | ***10*** |
| ***41*** | ***1,1,1-Trichloroethane*** | ***200*** |
| 42 | 1,1,2-Trichloroethane | 0.3 |
| 43 | Trichloroethylene | 1.35 |
| ***44*** | ***Vinyl Chloride*** | ***0.5*** |
| 45 | 2-Chlorophenol | 60 |
| 46 | 2,4-Dichlorophenol | 46.5 |
| 47 | 2,4-Dimethylphenol | 270 |
| 48 | 2-Methy-4,6-Dinitrophenol | 6.7 |
| 49 | 2,4-Dinitrophenol | 35 |
| 50 | 2-Nitrophenol | -- |
| 51 | 4-Nitrophenol | -- |
| 52 | 3-Methyl-4-Chlorophenol | -- |
| 53 | Pentachlorophenol | 0.14 |
| 54 | Phenol | 10500 |
| 55 | 2,4,6-Trichlorophenol | 1.05 |
| 56 | Acenapthene | 600 |
| 57 | Acenapthylene | -- |
| 58 | Anthracene | 4800 |
| 59 | Benzidine | 0.00006 |
| 60 | Benzo (a) anthracene | 0.0022 |
| 61 | Benzo (a) pyrene | 0.0022 |
| 62 | Benzo (b) fluoranthene | 0.0022 |
| 63 | Benzo (g,h,i) pyrylene | -- |
| 64 | Benzo (k) fluorantene | 0.0022 |
| 65 | Bis (2-Chloroethoxy) methane | -- |
| 66 | Bis (2-Chloroethyl) ether | 0.016 |
| 67 | Bis (2-Chloroisopropyl) ether | 700 |
| 68 | Bis (2-ethyhexyl) phthalate | 0.9 |
| 69 | 4-Bromophenyl phenyl ether | -- |
| 70 | Butyl benzyl phthalate | 1500 |
| 71 | 2- Chloronapthalene | 850 |
| 72 | 4-Chlrorphenyl phenyl ether | -- |
| 73 | Chrysene | 0.0022 |
| 74 | Dibenzo (a,h) anthracene | 0.0022 |
| ***75*** | ***1,2-Dichlorobenzene*** | ***0.6*** |

See notes below for italicized constituents.

6 *See Federal Register/ Vol. 65, No. 97 / Thursday, May 18, 2000 / Rules and Regulations*

**ATTACHMENT I. -Continued**

| | CONSTITUENT | µg/L |
|---|---|---|
| 76 | 1,3-Dichlorobenzene | 200 |
| ***77*** | ***1,4-Dichlorobenzene*** | ***5*** |
| 78 | 3,3-Dichlorobenzidine | 0.02 |
| 79 | Diethyl phthalate | 11,500 |
| 80 | Dimethyl phthalate | 156,500 |
| 81 | Di-N-butyl phthalate | 1,350 |
| 82 | 2,4-Dinitrotoluene | 0.055 |
| 83 | 2,6-Dinitrotoluene | -- |
| 84 | Di-N-octyl phthalate | -- |
| 85 | 1,2-Diphenylhydrazine | 0.02 |
| 86 | Fluoranthene | 150 |
| 87 | Fluorene | 650 |
| 88 | Hexachlorobenzene | 0.00038 |
| 89 | Hexachlorobutadiene | 0.22 |
| ***90*** | ***Hexachlorocyclopentadiene*** | ***50*** |
| 91 | Hexachloroethane | 0.95 |
| 92 | Indeno (1,2,3-cd) pyrene | 0.0022 |
| 93 | Isophorone | 4.2 |
| ***94*** | ***Naphthalene*** | ***17*** |
| 95 | Nitrobenzene | 8.5 |
| 96 | N-Nitrosodimethylamine | 0.00035 |
| 97 | N-Nitrosodi-N-propylamine | 0.0025 |
| 98 | N-Nitrosodiphenylamine | 2.5 |
| 99 | Phenantrene | -- |

| | CONSTITUENT | µg/L |
|---|---|---|
| 100 | Pyrene | 480 |
| ***101*** | ***1,2,4 -Trichlorobenzene*** | ***5*** |
| 102 | Aldrin | 0.00007 |
| 103 | BHC Alpha | 0.0020 |
| 104 | BHC Beta | 0.007 |
| 105 | BHC Gamma | 0.010 |
| 106 | BHC Delta | -- |
| 107 | Chlordane | 0.00029 |
| 108 | 4,4-DDT | 0.0003 |
| 109 | 4,4-DDE | 0.0003 |
| 110 | 4,4-DDD | 0.00042 |
| 111 | Dieldrin | 0.00007 |
| 112 | Endosulfan Alpha | 0.028 |
| 113 | Endosulfan Beta | 0.028 |
| 114 | Endosulfan Sulfate | 55 |
| 115 | Endrin | 0.018 |
| 116 | Endrin Aldehyde | 0.38 |
| 117 | Heptachlor | 0.00011 |
| 118 | Heptachlor Epoxide | 0.00005 |
| 119 | PCB 1016 | 0.000085 |
| 120 | PCB 1221 | 0.000085 |
| 125 | PCB 1260 | 0.000085 |
| 126 | Toxaphene | 0.00037 |
| | | |

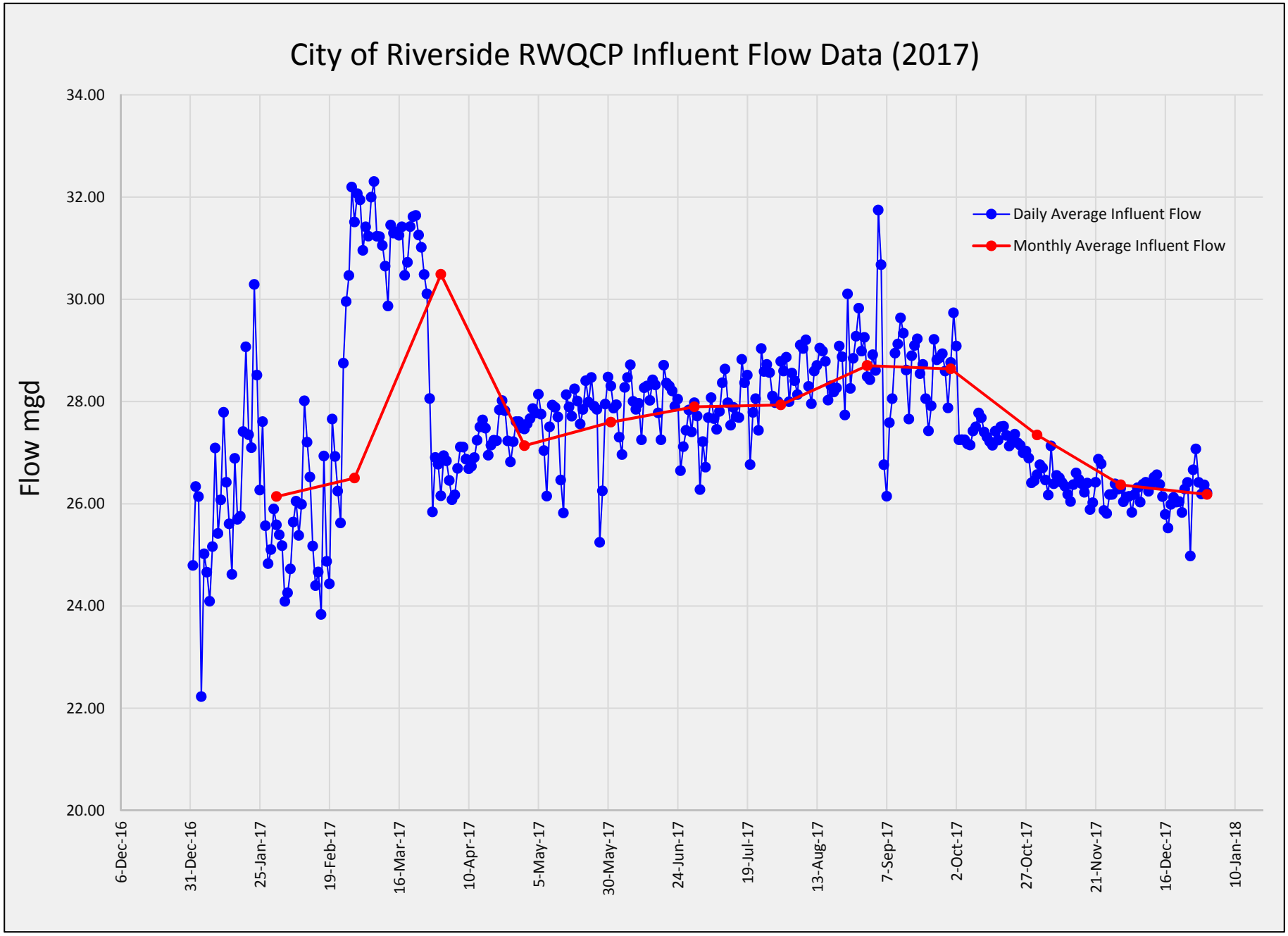
Notes:

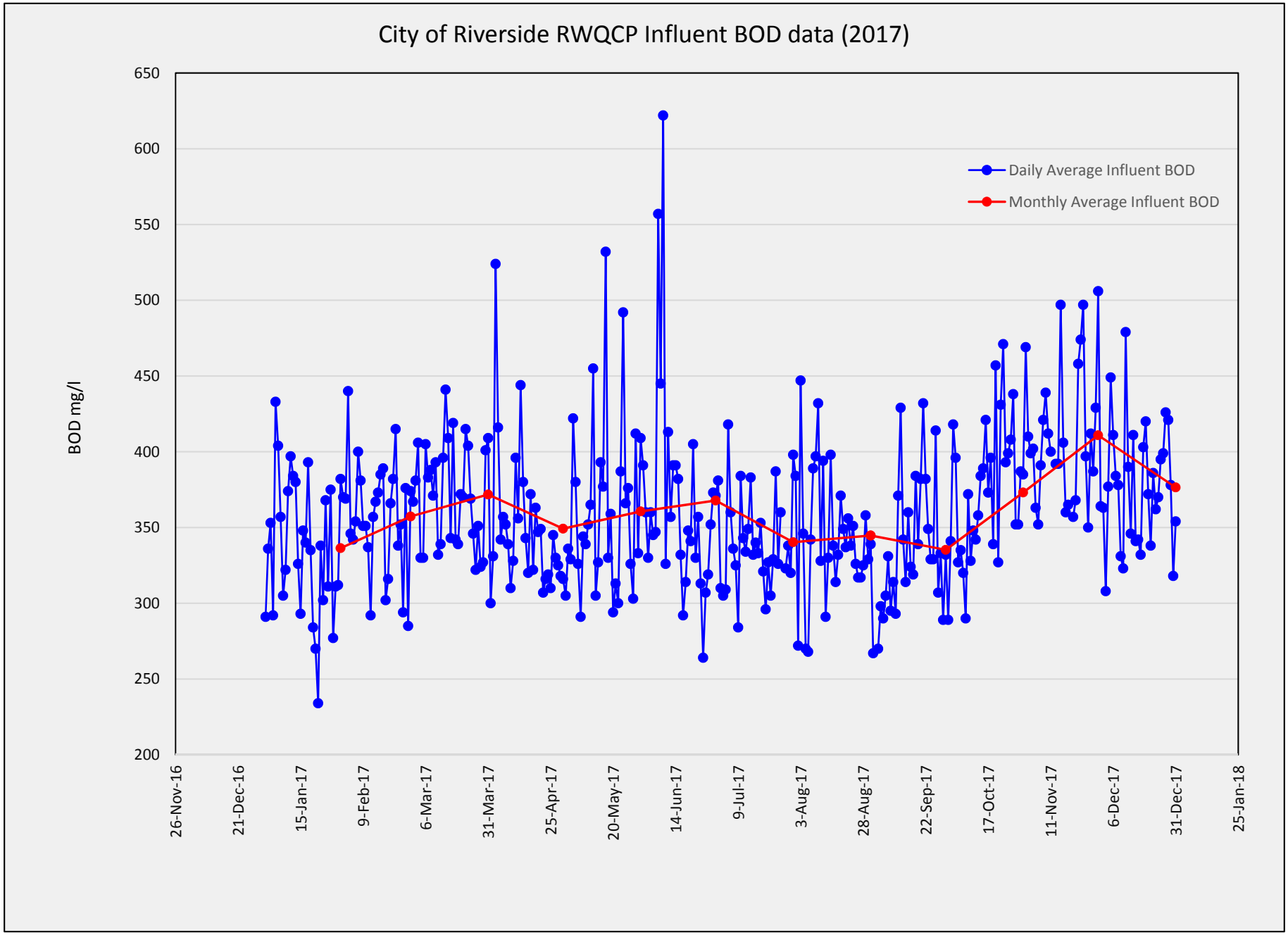
1. For constituents not shown italicized, the values shown in the Table are fifty percent of the most stringent applicable receiving water objectives (freshwater or human health (consumption of water and organisms) as specified for that pollutant in 40 CFR 131.38[7]).
2. For constituents shown bold and italicized, the values shown in the Table are based on the California Department of Health Services maximum contaminant levels (MCLs) or Notification Level. Notification Level based trigger is underlined.
3. For hardness dependent metals, the hardness value used is 120 mg/L and for pentachlorophenol, the pH value used is 7.5 standard units.

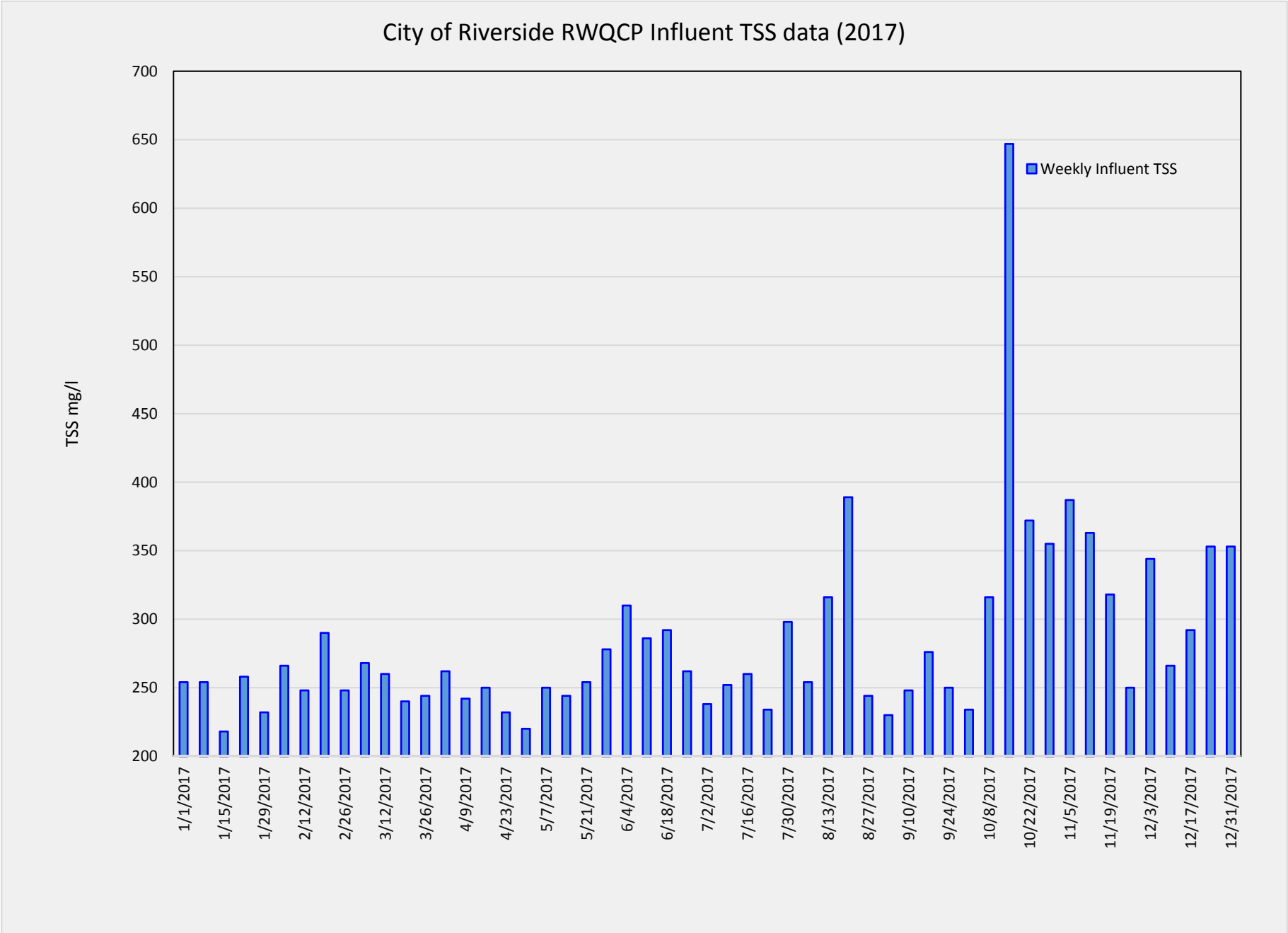
[7] *See Federal Register/ Vol. 65, No. 97 / Thursday, May 18, 2000 / Rules and Regulations.*

Appendix C

# INFLUENT CHARACTERISTICS


City of Riverside RWQCP Influent Flow Data (2017)
Flow mgd
34.00
32.00
30.00
28.00
26.00
24.00
22.00
20.00
6-Dec-16
31-Dec-16
25-Jan-17
19-Feb-17
16-Mar-17
10-Apr-17
5-May-17
30-May-17
24-Jun-17
19-Jul-17
13-Aug-17
7-Sep-17
2-Oct-17
27-Oct-17
21-Nov-17
16-Dec-17
10-Jan-18
Daily Average Influent Flow
Monthly Average Influent Flow


City of Riverside RWQCP Influent BOD data (2017)
BOD mg/l
650
600
550
500
450
400
350
300
250
200
26-Nov-16
21-Dec-16
15-Jan-17
9-Feb-17
6-Mar-17
31-Mar-17
25-Apr-17
20-May-17
14-Jun-17
9-Jul-17
3-Aug-17
28-Aug-17
22-Sep-17
17-Oct-17
11-Nov-17
6-Dec-17
31-Dec-17
25-Jan-18
Daily Average Influent BOD
Monthly Average Influent BOD


City of Riverside RWQCP Influent TSS data (2017)
TSS mg/l
700
650
600
550
500
450
400
350
300
250
200
Weekly Influent TSS
1/1/2017
1/15/2017
1/29/2017
2/12/2017
2/26/2017
3/12/2017
3/26/2017
4/9/2017
4/23/2017
5/7/2017
5/21/2017
6/4/2017
6/18/2017
7/2/2017
7/16/2017
7/30/2017
8/13/2017
8/27/2017
9/10/2017
9/24/2017
10/8/2017
10/22/2017
11/5/2017
11/19/2017
12/3/2017
12/17/2017
12/31/2017

Appendix D

# BIOWIN PROCESS MODEL

# BioWin user and configuration data

## Project details

Project name: Riverside RWQCP master plan update Project ref.: 10495A.00

Plant name: Riverside RWQCP User name: Will Mancell-Egala/Ron Appleton

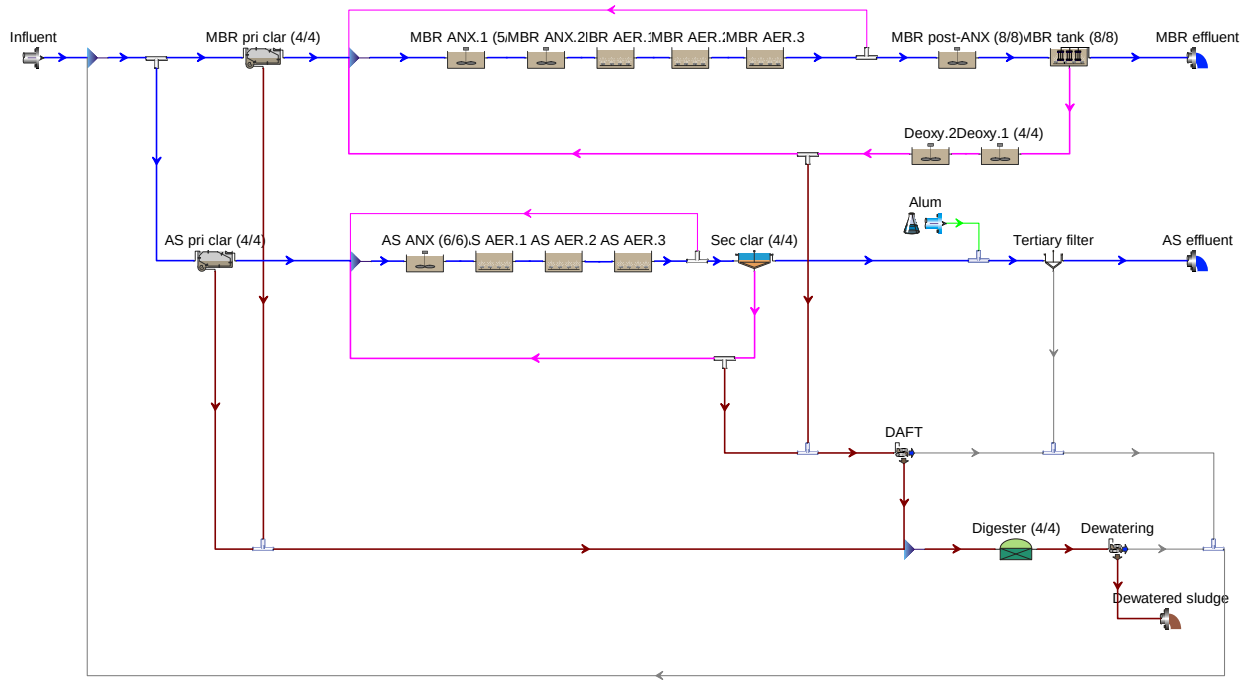
Created: 2/1/2017 Saved: 9/9/2017

## Steady state solution

Total MBR SRT: 10.60 days

Temperature: 20.0°C

## Flowsheet




# Configuration information for all Ideal primary settling tank units

## Physical data

| Element name | Volume [Mil. Gal] | Area [ft2] | Depth [ft] |
|---|---|---|---|
| MBR pri clar (4/4) | 4.3994 | 4.524E+4 | 13.000 |
| AS pri clar (4/4) | 1.9093 | 2.836E+4 | 9.000 |

## Operating data Average (flow/time weighted as required)

| Element name | Split method | Average Split specification |
|---|---|---|
| MBR pri clar (4/4) | Fraction | 0.01 |
| AS pri clar (4/4) | Fraction | 0.01 |

| Element name | Percent removal | Blanket fraction |
|---|---|---|
| MBR pri clar (4/4) | 60.00 | 0.10 |
| AS pri clar (4/4) | 60.00 | 0.10 |

# Configuration information for all Anaerobic Digester units

## Physical data

| Element name | Volume [Mil. Gal] | Area [ft2] | Depth [ft] | Head space volume |
|---|---|---|---|---|
| Digester (4/4) | 7.1460 | 2.547E+4 | 37.500 | 0.0 |

## Operating data Average (flow/time weighted as required)

| Element name | Pressure [psi] | pH |
|---|---|---|
| Digester (4/4) | 15.0 | - |

| Element name | Average Temperature |
|---|---|
| Digester (4/4) | 35.0 |

# Configuration information for all Bioreactor units

## Physical data

| Element name | Volume [Mil. Gal] | Area [ft2] | Depth [ft] | # of diffusers |
|---|---|---|---|---|
| AS AER.1 | 2.1000 | 1.599E+4 | 17.560 | 1429 |
| AS AER.2 | 2.1000 | 1.599E+4 | 17.560 | 648 |
| MBR AER.1 | 1.2905 | 1.017E+4 | 16.970 | 1047 |
| MBR AER.2 | 1.2905 | 1.017E+4 | 16.970 | 508 |
| MBR AER.3 | 1.2905 | 1.017E+4 | 16.970 | 408 |
| MBR post-ANX (8/8) | 0.7920 | 8252.1439 | 12.830 | Un-aerated |
| MBR ANX.1 (5/5) | 0.6030 | 4750.1108 | 16.970 | Un-aerated |
| AS ANX (6/6) | 1.5750 | 1.199E+4 | 17.560 | Un-aerated |
| Deoxy.1 (4/4) | 0.2920 | 3421.0977 | 11.410 | Un-aerated |
| AS AER.3 | 2.1000 | 1.599E+4 | 17.560 | 442 |
| MBR ANX.2 | 0.6030 | 4750.1108 | 16.970 | Un-aerated |
| Deoxy.2 | 0.2920 | 3421.0977 | 11.410 | Un-aerated |

# Operating data Average (flow/time weighted as required)

| Element name | Average DO Setpoint [mg/L] |
|---|---|
| AS AER.1 | 2.0 |
| AS AER.2 | 1.5 |
| MBR AER.1 | 2.0 |
| MBR AER.2 | 1.5 |
| MBR AER.3 | 1.0 |
| MBR post-ANX (8/8) | 0 |
| MBR ANX.1 (5/5) | 0 |
| AS ANX (6/6) | 0 |
| Deoxy.1 (4/4) | 0 |
| AS AER.3 | 1.0 |
| MBR ANX.2 | 0 |
| Deoxy.2 | 0 |

## Aeration equipment parameters

| Element name | k1 in C = k1(PC)^0.25 + k2 | k2 in C = k1(PC)^0.25 + k2 | Y in Kla = C Usg ^ Y - Usg in [m3/(m2 d)] | Area of one diffuser | Diffuser mounting height | Min. air flow rate per diffuser | Max. air flow rate per diffuser | 'A' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2 | 'B' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2 | 'C' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2 |
|---|---|---|---|---|---|---|---|---|---|---|
| AS AER.1 | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| AS AER.2 | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| MBR AER.1 | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| MBR AER.2 | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| MBR AER.3 | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| MBR post-ANX (8/8) | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| MBR ANX.1 (5/5) | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| AS ANX (6/6) | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| Deoxy.1 (4/4) | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| AS AER.3 | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| MBR ANX.2 | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |
| Deoxy.2 | 0.9229 | 1.4211 | 0.8898 | 2.6400 | 0.3048 | 122.3288 | 733.9727 | 2.9999 | 0 | 0 |

# Configuration information for all Membrane bioreactor units

# Physical data

| Element name | Volume [Mil. Gal] | Area [ft2] | Depth [ft] | # of diffusers | # of cassettes | Displaced volume / cassette [ft3/cassette] | Membrane area / cassette [ft2/cassette] | Total displaced volume [Mil. Gal] | Membrane surface area [ft2] |
|---|---|---|---|---|---|---|---|---|---|
| MBR tank (8/8) | 1.0080 | 1.125E+4 | 11.980 | 3135 | 112.00 | 368.270 | 17759.00 | 0.31 | 1989008.00 |

# Operating data Average (flow/time weighted as required)

| Element name | Average DO Setpoint [mg/L] |
|---|---|
| MBR tank (8/8) | 3.2 |

| Element name | Split method | Average Split specification |
|---|---|---|
| MBR tank (8/8) | Flowrate [Under] | 50.3 |

# Aeration equipment parameters

| Element name | k1 in C = k1(PC)^0.25 + k2 | k2 in C = k1(PC)^0.25 + k2 | Y in Kla = C Usg ^ Y - Usg in [m3/(m2 d)] | Area of one diffuser | Diffuser mounting height | Min. air flow rate per diffuser | Max. air flow rate per diffuser | 'A' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2 | 'B' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2 | 'C' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2 |
|---|---|---|---|---|---|---|---|---|---|---|
| MBR tank (8/8) | 0.0500 | 0.3800 | 1.0000 | 0.5382 | 0.2500 | 48.0000 | 1200.0000 | 1.0000 | 0 | 0 |

| Element name | Surface pressure [kPa] | Fractional effective saturation depth (Fed) [-] |
|---|---|---|
| MBR tank (8/8) | 101.3250 | 0.3000 |

| Element name | Supply gas CO2 content [vol. %] | Supply gas O2 [vol. %] | Off-gas CO2 [vol. %] | Off-gas O2 [vol. %] | Off-gas H2 [vol. %] | Off-gas NH3 [vol. %] | Off-gas CH4 [vol. %] | Off-gas N2O [vol. %] | Surface turbulence factor [-] |
|---|---|---|---|---|---|---|---|---|---|
| MBR tank (8/8) | 0.0350 | 20.9500 | 1.2000 | 19.9000 | 0 | 0 | 0 | 0 | 2.0000 |

# Configuration information for all Effluent units

# Configuration information for all Ideal clarifier units

## Physical data

| Element name | Volume[Mil. Gal] | Area[ft2] | Depth[ft] |
|---|---|---|---|
| Sec clar (4/4) | 4.1245 | 4.225E+4 | 13.050 |

## Operating data Average (flow/time weighted as required)

| Element name | Split method | Average Split specification |
|---|---|---|
| Sec clar (4/4) | Flowrate [Under] | 25.1 |

| Element name | Average Temperature | Reactive | Percent removal | Blanket fraction |
|---|---|---|---|---|
| Sec clar (4/4) | Uses global setting | Yes | 99.94 | 0.05 |

# Configuration information for all COD Influent units

## Operating data Average (flow/time weighted as required)

| Element name | Influent |
|---|---|
| Flow | 52.9 |
| Total COD mgCOD/L | 804.00 |
| Total Kjeldahl Nitrogen mgN/L | 49.80 |
| Total P mgP/L | 10.30 |
| Nitrate N mgN/L | 0.27 |
| pH | 7.30 |
| Alkalinity mmol/L | 6.10 |
| ISS Influent mgISS/L | 37.50 |
| Calcium mg/L | 160.00 |
| Magnesium mg/L | 20.00 |
| Dissolved O2 mg/L | 0 |

| Element name | Influent |
|---|---|
| Fbs - Readily biodegradable (including Acetate) [gCOD/g of total COD] | 0.1600 |
| Fac - Acetate [gCOD/g of readily biodegradable COD] | 0.3300 |
| Fxsp - Non-colloidal slowly biodegradable [gCOD/g of slowly degradable COD] | 0.7500 |
| Fus - Unbiodegradable soluble [gCOD/g of total COD] | 0.0250 |
| Fup - Unbiodegradable particulate [gCOD/g of total COD] | 0.1500 |
| Fna - Ammonia [gNH3-N/gTKN] | 0.6600 |
| Fnox - Particulate organic nitrogen [gN/g Organic N] | 0.5000 |
| Fnus - Soluble unbiodegradable TKN [gN/gTKN] | 0.0200 |
| FupN - N:COD ratio for unbiodegradable part. COD [gN/gCOD] | 0.0350 |
| Fpo4 - Phosphate [gPO4-P/gTP] | 0.5000 |
| FupP - P:COD ratio for unbiodegradable part. COD [gP/gCOD] | 0.0110 |

| | |
|---|---|
| FZbh - OHO COD fraction [gCOD/g of total COD] | 0.0200 |
| FZbm - Methylotroph COD fraction [gCOD/g of total COD] | 1.000E-4 |
| FZaob - AOB COD fraction [gCOD/g of total COD] | 1.000E-4 |
| FZnob - NOB COD fraction [gCOD/g of total COD] | 1.000E-4 |
| FZaao - AAO COD fraction [gCOD/g of total COD] | 1.000E-4 |
| FZbp - PAO COD fraction [gCOD/g of total COD] | 1.000E-4 |
| FZbpa - Propionic acetogens COD fraction [gCOD/g of total COD] | 1.000E-4 |
| FZbam - Acetoclastic methanogens COD fraction [gCOD/g of total COD] | 1.000E-4 |
| FZbhm - H2-utilizing methanogens COD fraction [gCOD/g of total COD] | 1.000E-4 |
| FZe - Endogenous products COD fraction [gCOD/g of total COD] | 0 |

# Configuration information for all Metal addition units

## Operating data Average (flow/time weighted as required)

| Element name | Alum |
|---|---|
| Ordinary heterotrophic organisms (OHO) mgCOD/L | 0 |
| Methylotrophs mgCOD/L | 0 |
| Ammonia oxidizing biomass (AOB) mgCOD/L | 0 |
| Nitrite oxidizing biomass (NOB) mgCOD/L | 0 |
| Anaerobic ammonia oxidizers (AAO) mgCOD/L | 0 |
| Polyphosphate accumulating organisms (PAO) mgCOD/L | 0 |
| Propionic acetogens mgCOD/L | 0 |
| Methanogens - acetoclastic mgCOD/L | 0 |
| Methanogens - hydrogenotrophic mgCOD/L | 0 |
| Endogenous products mgCOD/L | 0 |
| Slowly bio. COD (part.) mgCOD/L | 0 |
| Slowly bio. COD (colloid.) mgCOD/L | 0 |
| Part. inert. COD mgCOD/L | 0 |
| Part. bio. org. N mgN/L | 0 |

| | |
|---|---|
| Part. bio. org. P mgP/L | 0 |
| Part. inert N mgN/L | 0 |
| Part. inert P mgP/L | 0 |
| Stored PHA mgCOD/L | 0 |
| Releasable stored polyP mgP/L | 0 |
| Fixed stored polyP mgP/L | 0 |
| Readily bio. COD (complex) mgCOD/L | 0 |
| Acetate mgCOD/L | 0 |
| Propionate mgCOD/L | 0 |
| Methanol mgCOD/L | 0 |
| Dissolved H2 mgCOD/L | 0 |
| Dissolved CH4 mg/L | 0 |
| Ammonia N mgN/L | 0 |
| Sol. bio. org. N mgN/L | 0 |
| Nitrous Oxide N mgN/L | 0 |
| Nitrite N mgN/L | 0 |
| Nitrate N mgN/L | 0 |
| Dissolved N2 mgN/L | 0 |
| PO4-P (Sol. & Me Complexed) mgP/L | 0 |
| Sol. inert COD mgCOD/L | 0 |
| Sol. inert TKN mgN/L | 0 |
| ISS Influent mgISS/L | 0 |
| Struvite mgISS/L | 0 |
| Hydroxy-dicalcium-phosphate mgISS/L | 0 |
| Hydroxy-apatite mgISS/L | 0 |
| Magnesium mg/L | 0 |
| Calcium mg/L | 0 |
| Metal mg/L | 150000.00 |
| Other Cations (strong bases) meq/L | 5.00 |
| Other Anions (strong acids) meq/L | 16697.46 |
| Total CO2 mmol/L | 7.00 |
| User defined 1 mg/L | 0 |
| User defined 2 mg/L | 0 |
| User defined 3 mgVSS/L | 0 |
| User defined 4 mgISS/L | 0 |

| | |
|---|---|
| Dissolved O2 mg/L | 0 |
| Flow | 0 |

# Configuration information for all Sidestream Mixer units

## Physical data

| Element name | Volume[Mil. Gal] | Area[ft2] | Depth[ft] |
|---|---|---|---|
| Alum mixer | 0 | N/A | N/A |
| DAFT feed mixer | 0 | N/A | N/A |
| Recycle mixer 1 | 0 | N/A | N/A |
| PS mixer | 0 | N/A | N/A |
| Recycle mixer 2 | 0 | N/A | N/A |

# Configuration information for all General Mixer units

## Physical data

| Element name | Volume[Mil. Gal] | Area[ft2] | Depth[ft] |
|---|---|---|---|
| Influent mixer | 0 | N/A | N/A |
| Digester feed mixer | 0 | N/A | N/A |
| MBR Influent Mixer | 0 | N/A | N/A |
| AS Influent Mixer | 0 | N/A | N/A |

# Configuration information for all Dewatering unit units

## Physical data

| Element name | No Volume |
|---|---|
| DAFT | 0 |
| Dewatering | 0 |

## Operating data Average (flow/time weighted as required)

| Element name | Split method | Average Split specification |
|---|---|---|
| DAFT | Fraction | 0.15 |
| Dewatering | Fraction | 0.12 |

| Element name | Percent removal |
|---|---|
| DAFT | 95.00 |
| Dewatering | 95.00 |

# Configuration information for all Point clarifier units

## Physical data

| Element name | No Volume |
|---|---|

| | |
|---|---|
| Tertiary filter | 0 |

## Operating data Average (flow/time weighted as required)

| Element name | Split method | Average Split specification |
|---|---|---|
| Tertiary filter | Fraction | 0.03 |

| Element name | Percent removal |
|---|---|
| Tertiary filter | 80.00 |

# Configuration information for all Sludge units

# Configuration information for all Splitter units

## Physical data

| Element name | Volume[Mil. Gal] | Area[ft2] | Depth[ft] |
|---|---|---|---|
| Pri Inf splitter | 0 | N/A | N/A |
| AS WAS Splitter | 0 | N/A | N/A |
| MBR WAS Splitter | 0 | N/A | N/A |
| MBR MLR | 0 | N/A | N/A |
| AS MLR | 0 | N/A | N/A |

## Operating data Average (flow/time weighted as required)

| Element name | Split method | Average Split specification |
|---|---|---|
| Pri Inf splitter | Flowrate [Side] | 23 |
| AS WAS Splitter | Flowrate [Side] | 0.4308 |
| MBR WAS Splitter | Flowrate [Side] | 0.4016 |
| MBR MLR | Flowrate [Side] | 43.25 |
| AS MLR | Flowrate [Side] | 51.9 |

# BioWin Album

# Album page - Influent info

| Influent | | | |
|---|---|---|---|
| Parameters | Conc. (mg/L) | Mass rate (lb/d) | Notes |
| Volatile suspended solids | 329.93 | 145656.93 | |
| Total suspended solids | 368.48 | 162675.11 | |
| Particulate COD | 525.78 | 232115.24 | |
| Filtered COD | 278.22 | 122828.16 | |
| Total COD | 804.00 | 354943.40 | |
| Soluble PO4-P | 5.15 | 2273.58 | |
| Total P | 10.30 | 4547.16 | |
| Filtered TKN | 39.14 | 17277.53 | |
| Particulate TKN | 10.66 | 4707.77 | |
| Total Kjeldahl Nitrogen | 49.80 | 21985.30 | |
| Filtered Carbonaceous BOD | 183.55 | 81034.24 | |
| Total Carbonaceous BOD | 398.23 | 175806.05 | |
| Nitrite + Nitrate | 0.27 | 121.10 | |
| Total N | 50.07 | 22106.40 | |
| Total inorganic N | 33.14 | 14631.39 | |
| Alkalinity | 6.10 | 1221.51 | mmol/L and kmol/d |
| pH | 7.30 | | |
| Volatile fatty acids | 42.45 | 18741.01 | |
| ISS precipitate | 0 | 0 | |

| | | |
|---|---|---|
| ISS cellular | 1.05 | 462.98 |
| ISS Total | 38.55 | 17018.18 |
| Ammonia N | 32.87 | 14510.30 |
| Nitrate N | 0.27 | 121.10 |

| Parameters | Value | Units |
|---|---|---|
| pH | 7.30 | |
| Ionized ammonium | 2.33 | mmol/L |
| Unionized ammonia | 0.02 | mmol/L |
| Nitrous acid | 0 | mmol/L |
| Nitrite | 0 | mmol/L |
| Total dissolved CO2 | 0.58 | mmol/L |
| Bicarbonate | 5.49 | mmol/L |
| Carbonate | 0.01 | mmol/L |
| Unionized ortho-P | 0.00 | mmol/L |
| H2PO4- | 0.06 | mmol/L |
| HPO4-- | 0.11 | mmol/L |
| PO4--- | 0.00 | mmol/L |
| Metal phosphate (solid) | 0 | mmol/L |
| Metal hydroxide (solid) | 0 | mmol/L |
| Metal ion | 0 | mmol/L |
| MeH2PO4++ | 0 | mmol/L |
| MeHPO4+ | 0 | mmol/L |
| Acetic acid | 0.00 | mmol/L |
| Acetate | 0.66 | mmol/L |
| Propionic acid | 0 | mmol/L |
| Propionate | 0 | mmol/L |
| Ionic strength | 0.02 | |
| Monovalent act. coeff. | 0.88 | |
| Divalent act. coeff. | 0.60 | |
| Trivalent act. coeff. | 0.31 | |
| Flow | 52.90 | mgd |

# Album page - Influent info

| Influent | | | |
|---|---|---|---|
| State variable | Conc. (mg/L) | Mass rate (lb/d) | Notes |
| Ordinary heterotrophic organisms (OHO) | 16.08 | 7098.87 | |
| Methylotrophs | 0.08 | 35.49 | |
| Ammonia oxidizing biomass (AOB) | 0.08 | 35.49 | |
| Nitrite oxidizing biomass (NOB) | 0.08 | 35.49 | |
| Anaerobic ammonia oxidizers (AAO) | 0.08 | 35.49 | |
| Polyphosphate accumulating organisms (PAO) | 0.08 | 35.49 | |
| Propionic acetogens | 0.08 | 35.49 | |
| Methanogens - acetoclastic | 0.08 | 35.49 | |
| Methanogens - hydrogenotrophic | 0.08 | 35.49 | |
| Endogenous products | 0 | 0 | |
| Slowly bio. COD (part.) | 388.45 | 171490.90 | |
| Slowly bio. COD (colloid.) | 129.48 | 57163.63 | |
| Part. inert. COD | 120.60 | 53241.51 | |
| Part. bio. org. N | 5.27 | 2327.52 | |
| Part. bio. org. P | 3.45 | 1521.95 | |
| Part. inert N | 4.22 | 1863.45 | |
| Part. inert P | 1.33 | 585.66 | |
| Stored PHA | 0 | 0 | |
| Releasable stored polyP | 0.01 | 3.55 | |
| Fixed stored polyP | 0 | 0 | |
| Readily bio. COD (complex) | 86.19 | 38049.93 | |
| Acetate | 42.45 | 18741.01 | |
| Propionate | 0 | 0 | |
| Methanol | 0 | 0 | |
| Dissolved H2 | 0 | 0 | |
| Dissolved CH4 | 0 | 0 | |
| Ammonia N | 32.87 | 14510.30 | |
| Sol. bio. org. N | 5.27 | 2327.52 | |
| Nitrous Oxide N | 0 | 0 | |
| Nitrite N | 0 | 0 | |
| Nitrate N | 0.27 | 121.10 | |

| | | | |
|---|---|---|---|
| Dissolved N2 | 16.00 | 7063.55 | |
| PO4-P (Sol. & Me Complexed) | 5.15 | 2273.58 | |
| Sol. inert COD | 20.10 | 8873.59 | |
| Sol. inert TKN | 1.00 | 439.71 | |
| ISS Influent | 37.50 | 16555.20 | |
| Struvite | 0 | 0 | |
| Hydroxy-dicalcium-phosphate | 0 | 0 | |
| Hydroxy-apatite | 0 | 0 | |
| Magnesium | 20.00 | 8829.44 | |
| Calcium | 160.00 | 70635.50 | |
| Metal | 0 | 0 | |
| Other Cations (strong bases) | 5.00 | 1001.24 | meq/L and keq/d |
| Other Anions (strong acids) | 10.50 | 2103.05 | meq/L and keq/d |
| Total CO2 | 6.08 | 1217.79 | mmol/L and kmol/d |
| User defined 1 | 0 | 0 | |
| User defined 2 | 0 | 0 | |
| User defined 3 | 0 | 0 | |
| User defined 4 | 0 | 0 | |
| Dissolved O2 | 0 | 0 | |
| | | | |
| Parameters | Value | Units | |

# Album page - Tables (1)

| Elements | Flow [mgd] | Temperature [deg. C] | Total suspended solids [mg/L] | Volatile suspended solids [mg/L] | Total COD [mg/L] | Filtered COD [mg/L] | Volatile fatty acids [mg/L] | Total Carbonaceous BOD [mg/L] | Filtered Carbonaceous BOD [mg/L] | Total N [mgN/L] | Total inorganic N [mgN/L] | Total Kjeldahl Nitrogen [mgN/L] | Filtered TKN [mgN/L] | Ammonia N [mgN/L] | Nitrite N [mgN/L] | Nitrate N [mgN/L] | Total P [mgP/L] | Soluble PO4-P [mgP/L] | pH [] | Alkalinity [mmol/L] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|

| | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Influent | 52.90 | 20.00 | 368.48 | 329.93 | 804.00 | 278.22 | 42.45 | 398.23 | 183.55 | 50.07 | 33.14 | 49.80 | 39.14 | 32.87 | 0 | 0.27 | 10.30 | 5.15 | 7.30 | 6.10 |
| Influent mixer | 54.66 | 20.00 | 374.69 | 333.62 | 801.18 | 270.96 | 42.01 | 390.04 | 178.34 | 54.87 | 37.27 | 54.44 | 42.95 | 36.83 | 0.00 | 0.43 | 13.39 | 7.99 | 7.23 | 6.29 |
| Pri Inf splitter | 31.66 | 20.00 | 374.69 | 333.62 | 801.18 | 270.96 | 42.01 | 390.04 | 178.34 | 54.87 | 37.27 | 54.44 | 42.95 | 36.83 | 0.00 | 0.43 | 13.39 | 7.99 | 7.23 | 6.29 |
| MBR pri clar (4/4) | 31.48 | 20.00 | 150.72 | 134.20 | 484.24 | 270.96 | 42.01 | 263.50 | 178.34 | 48.00 | 37.27 | 47.57 | 42.95 | 36.83 | 0.00 | 0.43 | 10.16 | 7.99 | 7.23 | 6.29 |
| MBR AER.3 | 124.63 | 20.00 | 6929.75 | 5659.62 | 8357.10 | 22.06 | 0.00 | 2229.91 | 1.00 | 502.42 | 8.53 | 494.02 | 1.69 | 0.13 | 0.11 | 8.29 | 213.10 | 6.23 | 6.78 | 3.33 |
| MBR post-ANX (8/8) | 81.38 | 20.00 | 6926.44 | 5656.22 | 8351.23 | 21.48 | 0.01 | 2226.05 | 0.51 | 500.67 | 7.08 | 493.99 | 1.72 | 0.40 | 0.13 | 6.55 | 213.10 | 6.20 | 6.79 | 3.47 |
| MBR tank (8/8) | 31.08 | 20.00 | 0 | 0 | 22.08 | 22.08 | 0.00 | 0.98 | 0.98 | 8.52 | 6.98 | 1.59 | 1.59 | 0.05 | 0.01 | 6.92 | 5.88 | 5.88 | 7.06 | 3.42 |
| | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| Pri Inf splitter (U) | 23.00 | 20.00 | 374.69 | 333.62 | 801.18 | 270.96 | 42.01 | 390.04 | 178.34 | 54.87 | 37.27 | 54.44 | 42.95 | 36.83 | 0.00 | 0.43 | 13.39 | 7.99 | 7.23 | 6.29 |
| AS pri clar (4/4) | 22.87 | 20.00 | 150.72 | 134.20 | 484.24 | 270.96 | 42.01 | 263.50 | 178.34 | 48.00 | 37.27 | 47.57 | 42.95 | 36.83 | 0.00 | 0.43 | 10.16 | 7.99 | 7.23 | 6.29 |
| AS AER.3 | 99.44 | 20.00 | 3902.16 | 3268.34 | 4829.37 | 21.55 | 0.00 | 1356.51 | 0.94 | 295.68 | 7.98 | 287.77 | 1.63 | 0.07 | 0.02 | 7.89 | 97.78 | 6.81 | 6.74 | 3.35 |

| | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sec clar (4/4) | 22.44 | 20.00 | 4.72 | 3.95 | 27.27 | 21.46 | 0.00 | 2.49 | 0.88 | 9.88 | 7.98 | 1.97 | 1.62 | 0.07 | 0.02 | 7.89 | 6.92 | 6.81 | 6.74 | 3.35 |
| | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| MBR effluent | 31.08 | 20.00 | 0 | 0 | 22.08 | 22.08 | 0.00 | 0.98 | 0.98 | 8.52 | 6.98 | 1.59 | 1.59 | 0.05 | 0.01 | 6.92 | 5.88 | 5.88 | 7.06 | 3.42 |
| AS effluent | 21.77 | 20.00 | 0.97 | 0.81 | 22.66 | 21.46 | 0.00 | 1.21 | 0.88 | 9.61 | 7.98 | 1.70 | 1.62 | 0.07 | 0.02 | 7.89 | 6.83 | 6.81 | 6.74 | 3.35 |
| | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| Dewatering | 0.38 | 20.00 | 1378.93 | 1048.16 | 1757.13 | 166.99 | 133.67 | 255.64 | 98.32 | 810.56 | 729.01 | 810.56 | 730.52 | 728.44 | 0.00 | 0.00 | 439.01 | 412.18 | 6.78 | 43.41 |
| Recycle mixer 2 | 1.76 | 20.00 | 561.14 | 444.35 | 716.56 | 52.49 | 28.60 | 143.82 | 21.54 | 199.11 | 161.30 | 193.95 | 157.57 | 156.02 | 0.04 | 5.12 | 106.14 | 93.34 | 6.81 | 11.95 |

# Album page - Tables (1)

| Elements | Total suspended solids [lb /d] | Volatile suspended solids [lb /d] | Total Kjeldahl Nitrogen [lb N/d] | Total P [lb P/d] | Struvite [lb ISS/d] |
|---|---|---|---|---|---|
| Influent | 162675.11 | 145656.93 | 21985.30 | 4547.16 | 0 |
| | - | - | - | - | - |
| AS pri clar (4/4) (U) | 43151.27 | 38421.65 | 1369.61 | 630.05 | 7.94 |
| MBR pri clar (4/4) (U) | 59398.01 | 52887.64 | 1885.27 | 867.27 | 10.93 |
| PS mixer | 102549.28 | 91309.29 | 3254.88 | 1497.32 | 18.87 |
| | - | - | - | - | - |
| AS WAS Splitter (U) | 26544.43 | 22231.32 | 1953.09 | 643.59 | 0.00 |
| MBR WAS Splitter (U) | 37486.46 | 30590.76 | 2673.44 | 1143.35 | 0.00 |
| DAFT feed mixer | 64030.89 | 52822.08 | 4626.53 | 1786.94 | 0.00 |

| | | | | | |
|---|---|---|---|---|---|
| | - | - | - | - | - |
| DAFT (U) | 60829.34 | 50180.97 | 4385.68 | 1662.05 | 0.00 |
| | - | - | - | - | - |
| Digester feed mixer | 163378.62 | 141490.26 | 7640.56 | 3159.37 | 18.87 |
| Digester (4/4) | 86629.88 | 65849.41 | 7640.56 | 3159.37 | 628.95 |
| Dewatered sludge | 82298.38 | 62556.94 | 5094.42 | 1780.36 | 597.50 |

# Album page - Tables (1)

| Elements | Total suspended solids [lb /d] | Volatile suspended solids [lb /d] | Total Kjeldahl Nitrogen [lb N/d] | Ammonia N [lb N/d] | Total P [lb P/d] | Soluble PO4-P [lb P/d] |
|---|---|---|---|---|---|---|
| Influent | 162675.11 | 145656.93 | 21985.30 | 14510.30 | 4547.16 | 2273.58 |
| | - | - | - | - | - | - |
| DAFT | 3201.54 | 2641.10 | 240.85 | 2.55 | 124.88 | 37.75 |
| Tertiary filter (U) | 707.32 | 591.64 | 61.14 | 0.39 | 54.77 | 38.25 |
| | - | - | - | - | - | - |
| Recycle mixer 2 | 8240.36 | 6525.22 | 2848.12 | 2291.13 | 1558.67 | 1370.73 |
| | - | - | - | - | - | - |
| Influent mixer | 170915.47 | 152182.15 | 24833.42 | 16801.42 | 6105.83 | 3644.31 |

# Album page - Tables (2)

| Elements | Temperature [deg. C] | Element HRT [hours] | Air flow rate [ft3/min (20 C, 1 atm)] | # of diffusers [] | Air flow rate / diffuser [ft3/min (20 C, 1 atm)] | Actual DO sat. conc. [mg/L] | Alpha [[]] | Beta [[]] | OUR - Total [mg O/L/hr] | OTR [lb/hr] | OTE [%] | SOTR [lb/hr] | SOTE [%] | Off gas flow rate (dry) [ft3/min (field)] | Off gas Oxygen [%] | Off gas Carbon dioxide [%] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ASAER.1 | 20.00 | 0.51 | 15497.04 | 1429.00 | 10.84 | 9.30 | 0.36 | 0.95 | 78.68 | 1448.01 | 8.95 | 5940.01 | 36.72 | 15555.26 | 19.00 | 1.90 |
| ASAER.2 | 20.00 | 0.51 | 7399.52 | 648.00 | 11.42 | 9.11 | 0.45 | 0.95 | 50.84 | 873.78 | 11.31 | 2751.94 | 35.63 | 7388.29 | 18.61 | 2.15 |
| ASAER.3 | 20.00 | 0.51 | 3675.70 | 442.00 | 8.32 | 8.83 | 0.54 | 0.95 | 32.39 | 550.44 | 14.35 | 1403.88 | 36.59 | 3656.65 | 18.04 | 2.47 |
| | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| Total | 60.00 | 1.52 | 26572.26 | 2519.00 | 30.58 | 27.24 | 1.35 | 2.85 | 161.92 | 2872.23 | 34.61 | 10095.83 | 108.94 | 26600.20 | 55.65 | 6.52 |

# Album page - Tables (2)

| Elements | Temperature [deg. C] | Element HRT [hours] | Air flow rate [ft3/min (20 C, 1 atm)] | # of diffusers [] | Air flow rate / diffuser [ft3/min (20 C, 1 atm)] | Actual DO sat. conc. [mg/L] | Alpha [[]] | Beta [[]] | OUR - Total [mgO/L/hr] | OTR [lb/hr] | OTE [%] | SOTR [lb/hr] | SOTE [%] | Off gas flow rate (dry) [ft3/min (field)] | Off gas Oxygen [%] | Off gas Carbon dioxide [%] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| MBRAER.1 | 20.00 | 0.25 | 18121.49 | 1047.00 | 17.31 | 9.33 | 0.36 | 0.95 | 137.64 | 1569.06 | 8.29 | 6378.38 | 33.72 | 18171.06 | 19.16 | 1.57 |
| MBRAER.2 | 20.00 | 0.25 | 10426.93 | 508.00 | 20.53 | 9.16 | 0.45 | 0.95 | 106.92 | 1129.85 | 10.38 | 3515.31 | 32.30 | 10405.30 | 18.81 | 1.88 |
| MBRAER.3 | 20.00 | 0.25 | 4832.32 | 408.00 | 11.84 | 8.88 | 0.54 | 0.95 | 65.37 | 682.35 | 13.53 | 1720.91 | 34.12 | 4802.43 | 18.23 | 2.20 |
| | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| Total | 60.00 | 0.75 | 33380.74 | 1963.00 | 49.68 | 27.37 | 1.35 | 2.85 | 309.94 | 3381.27 | 32.20 | 11614.60 | 100.13 | 33378.79 | 56.20 | 5.65 |
| | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |

| | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| MBR tank (8/8) | 20.00 | 0.30 | 50893.29 | 3135.00 | 16.23 | 9.59 | 0.40 | 0.95 | 85.63 | 590.24 | 1.11 | 2359.98 | 4.44 | 50994.40 | 20.68 | 0.44 |

# Album page - Tables (2)

| Elements | Flow [mgd] | Temperature [deg. C] | Element HRT [hours] | VSS destruction [%] | Off gas flow rate (dry) [ft3/min (field)] | Off gas Methane [%] | Off gas Carbon dioxide [%] | Off gas Hydrogen [%] | Liquid volume [Mil. Gal] | pH [] | Alkalinity [mmol/L] |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Digester (4/4) | 0.43 | 35.00 | 400.28 | 53.46 | 815.25 | 62.60 | 36.88 | 0.44 | 7.15 | 6.71 | 43.48 |

# Album page - Tables (3)

| Elements | Dissolved O2 [mg/L] | Filtered TKN [mgN/L] | Ammonia N [mgN/L] | Nitrite N [mgN/L] | Nitrate N [mgN/L] | Soluble PO4-P [mgP/L] | pH [] | Alkalinity [mmol/L] |
|---|---|---|---|---|---|---|---|---|
| AS Influent Mixer | 0.53 | 11.15 | 8.60 | 0.03 | 5.78 | 7.09 | 6.85 | 4.06 |
| AS ANX (6/6) | 0.00 | 10.24 | 8.75 | 0.02 | 0.23 | 6.84 | 6.92 | 4.51 |
| AS AER.1 | 2.00 | 4.24 | 2.71 | 1.04 | 4.60 | 6.74 | 6.85 | 3.71 |
| AS AER.2 | 1.50 | 1.90 | 0.35 | 0.20 | 7.48 | 6.74 | 6.78 | 3.39 |
| AS AER.3 | 1.00 | 1.63 | 0.07 | 0.02 | 7.89 | 6.81 | 6.74 | 3.35 |
| Sec clar (4/4) | 0.85 | 1.62 | 0.07 | 0.02 | 7.89 | 6.81 | 6.74 | 3.35 |
| | - | - | - | - | - | - | - | - |
| MBR Influent Mixer | 0.35 | 12.13 | 9.54 | 0.07 | 4.61 | 6.53 | 6.99 | 4.20 |
| MBR ANX.1 (5/5) | 0.00 | 11.33 | 9.59 | 0.02 | 0.24 | 6.90 | 7.04 | 4.56 |
| MBR post-ANX (8/8) | 0.01 | 1.72 | 0.40 | 0.13 | 6.55 | 6.20 | 6.79 | 3.47 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| MBR AER.1 | 2.00 | 5.92 | 4.43 | 1.22 | 3.57 | 7.06 | 6.94 | 3.88 |
| MBR AER.2 | 1.50 | 2.52 | 0.98 | 1.00 | 6.82 | 6.60 | 6.83 | 3.43 |
| MBR AER.3 | 1.00 | 1.69 | 0.13 | 0.11 | 8.29 | 6.23 | 6.78 | 3.33 |
| MBR tank (8/8) | 3.20 | 1.59 | 0.05 | 0.01 | 6.92 | 5.88 | 7.06 | 3.42 |
| | - | - | - | - | - | - | - | - |
| Deoxy.1 (4/4) | 0.03 | 1.61 | 0.22 | 0.07 | 5.51 | 5.83 | 7.01 | 3.53 |

# Album page - Tables (3)

| Elements | Flow [mgd] | Slowly bio. COD (part.) [mgCOD/L] | Part. inert. COD [mgCOD/L] | Ordinary heterotrophic organisms (OHO) [mgCOD/L] | Methylotrophs [mgCOD/L] | Polyphosphate accumulating organisms (PAO) [mgCOD/L] | Ammonia oxidizing biomass (AOB) [mgCOD/L] | Nitrite oxidizing biomass (NOB) [mgCOD/L] | Anaerobic ammonia oxidizers (AAO) [mgCOD/L] | Propionic acetogens [mgCOD/L] | Methanogens - acetoclastic [mgCOD/L] | Methanogens - hydrogenotrophic [mgCOD/L] | Endogenous products [mgCOD/L] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| AS WAS Splitter (U) | 0.43 | 202.11 | 2601.19 | 4189.30 | 1.19 | 5.03 | 120.86 | 73.84 | 1.46 | 0.49 | 0.89 | 0.15 | 1899.52 |
| MBR WAS Splitter (U) | 0.40 | 271.17 | 3969.64 | 5642.30 | 1.70 | 215.04 | 170.68 | 103.23 | 2.12 | 0.65 | 1.15 | 0.20 | 3055.05 |
| | - | - | - | - | - | - | - | - | - | - | - | - | - |
| Digester feed mixer | 0.43 | 29231.71 | 15655.27 | 10671.61 | 8.86 | 212.34 | 286.58 | 176.46 | 9.56 | 11.75 | 23.84 | 16.24 | 4929.06 |
| Digester (4/4) | 0.43 | 165.31 | 15655.27 | 3409.52 | 5.01 | 127.38 | 94.00 | 57.88 | 6.43 | 156.79 | 531.28 | 331.81 | 7326.32 |

# Album page - Tables (3)

| Elements | pH [] | Ionic strength [] | Monovalent act. coeff. [] | Divalent act. coeff. [] | Trivalent act. coeff. [] | Magnesium [mg/L] | Ionized ammonium [mmol/L] | H2PO4- [mmol/L] | HPO4-- [mmol/L] | PO4--- [mmol/L] | Struvite [mgISS/L] | Alkalinity [mmol/L] | Bicarbonate [mmol/L] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Digester (4/4) | 6.71 | 0.08 | 0.79 | 0.39 | 0.12 | 19.73 | 51.77 | 7.89 | 5.41 | 0.00 | 175.90 | 43.48 | 35.77 |

# Album page - Tables (4)

| Elements | Flow [mgd] |
|---|---|
| Influent | 52.90 |
| Influent mixer | 54.66 |
| | - |
| Pri Inf splitter | 31.66 |
| MBR tank (8/8) (U) | 50.30 |
| MBR MLR (U) | 43.25 |
| MBR Influent Mixer | 124.63 |
| MBR effluent | 31.08 |
| | - |
| Pri Inf splitter (U) | 23.00 |
| Sec clar (4/4) (U) | 25.10 |
| AS MLR (U) | 51.90 |
| AS Influent Mixer | 99.44 |
| AS effluent | 21.77 |

# Album page - Tables (4)

| Elements | Flow [mgd] |
|---|---|
| AS pri clar (4/4) (U) | 0.13 |
| MBR pri clar (4/4) (U) | 0.18 |
| PS mixer | 0.31 |

| | |
|---|---|
| | - |
| MBR WAS Splitter (U) | 0.40 |
| AS WAS Splitter (U) | 0.43 |
| DAFT feed mixer | 0.83 |
| | - |
| DAFT (U) | 0.12 |
| | - |
| Digester feed mixer | 0.43 |
| Digester (4/4) | 0.43 |
| Dewatered sludge | 0.05 |
| | - |
| DAFT | 0.71 |
| Tertiary filter (U) | 0.67 |
| | - |
| Recycle mixer 2 | 1.76 |

# Album page - Tables (4)

| Elements | Liquid volume [Mil. Gal] | # of cassettes [] | Mixed liquor flow [mgd] | Membrane flux [gal/ft2/d (gfd)] | Air flow rate / cassette [ft3/min (20C, 1 atm)] |
|---|---|---|---|---|---|
| MBR tank (8/8) | 0.70 | 112.00 | 50.30 | 15.63 | 454.40 |

# Album page - Tables (5)

| Elements | Surface overflow rate [gal/(ft2 d)] | Solids loading rate [lb/(ft2 d)] | Percent TSS removal [%] | Percent BOD removal [%] |
|---|---|---|---|---|
| AS pri clar (4/4) | 806.46 | 2.54 | 60.00 | 32.82 |
| MBR pri clar (4/4) | 695.90 | 2.19 | 60.00 | 32.82 |
| Sec clar (4/4) | 531.13 | 36.64 | 99.94 | 99.91 |

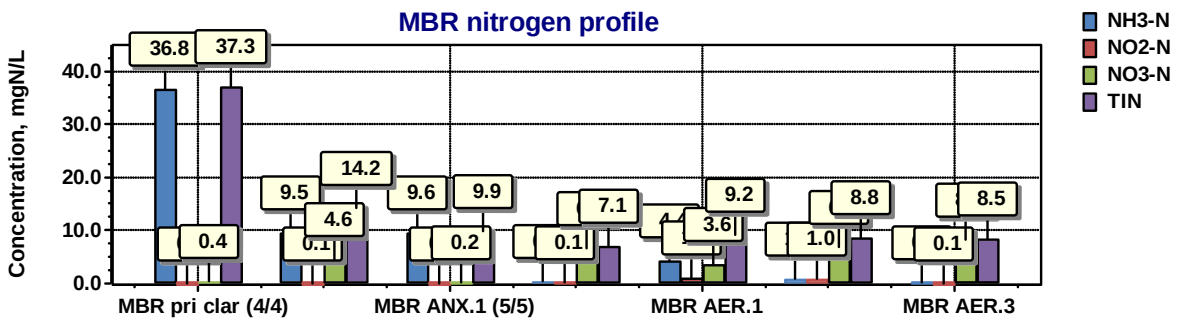
# Album page - Tables (5)

| Elements | Liquid volume [Mil. Gal] |
|---|---|
| MBR ANX.1 (5/5) | 0.60 |
| MBR ANX.2 | 0.60 |
| MBR AER.1 | 1.29 |
| MBR AER.2 | 1.29 |
| MBR AER.3 | 1.29 |
| | - |
| Total | 5.08 |
| | - |
| MBR post-ANX (8/8) | 0.79 |
| MBR tank (8/8) | 0.70 |
| Deoxy.1 (4/4) | 0.29 |
| Deoxy.2 | 0.29 |
| | - |
| Total | 7.15 |

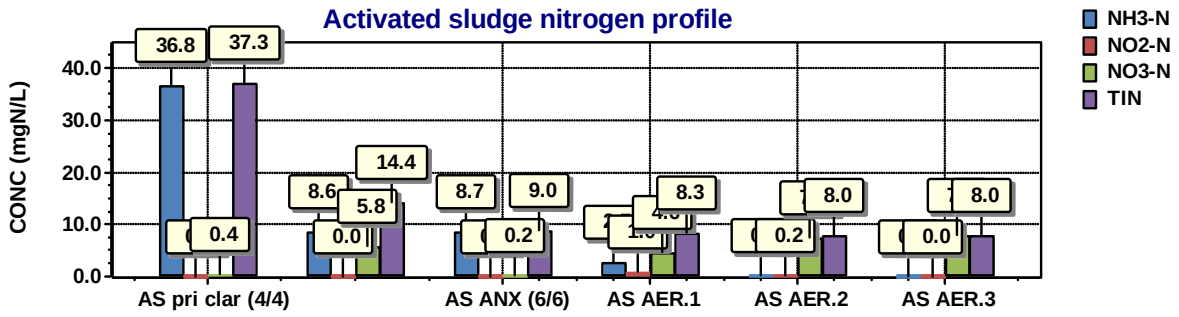
# Album page - Tables (5)

| Elements | Liquid volume [Mil. Gal] |
|---|---|
| AS ANX (6/6) | 1.57 |
| AS AER.1 | 2.10 |
| AS AER.2 | 2.10 |
| AS AER.3 | 2.10 |
| | - |
| Total | 7.88 |

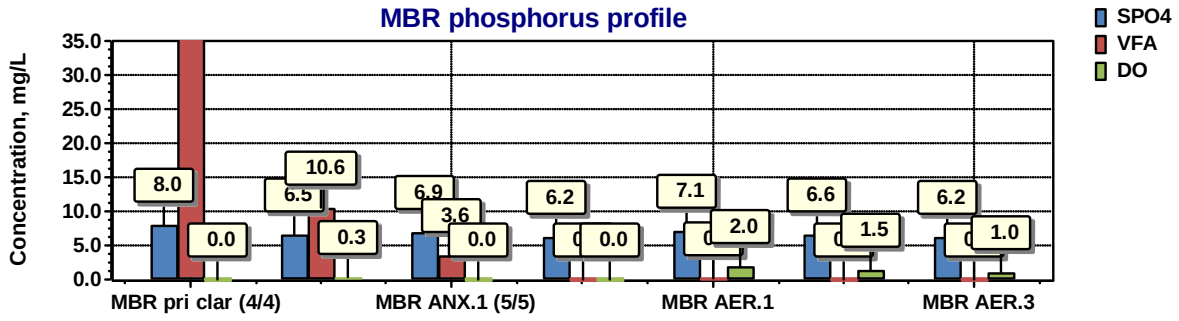
# Album page - N profile



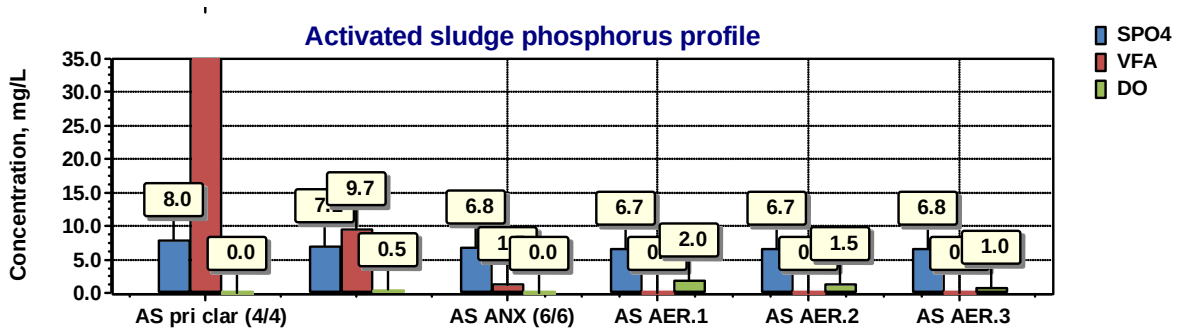

Album page - N profile




Album page - P profile




Album page - P profile




# Album page - Page 9

| Elements | Ordinary heterotrophic organisms (OHO) [mgCOD/L] | Methylotrophs [mgCOD/L] | Polyphosphate accumulating organisms (PAO) [mgCOD/L] | Ammonia oxidizing biomass (AOB) [mgCOD/L] | Nitrite oxidizing biomass (NOB) [mgCOD/L] | Anaerobic ammonia oxidizers (AAO) [mgCOD/L] | Propionic acetogens [mgCOD/L] | Methanogens - acetoclastic [mgCOD/L] | Methanogens - hydrogenotrophic [mgCOD/L] | Endogenous products [mgCOD/L] | Volatile suspended solids [mg/L] |
|---|---|---|---|---|---|---|---|---|---|---|---|
| MBR AER.3 | 3492.95 | 1.05 | 132.76 | 105.82 | 64.00 | 1.31 | 0.41 | 0.73 | 0.13 | 1883.65 | 5659.62 |
| AS AER.3 | 2214.55 | 0.63 | 2.66 | 63.89 | 39.03 | 0.77 | 0.26 | 0.47 | 0.08 | 1002.97 | 3268.34 |

# Album page - Page 9

# Album page - Page 9

# Global Parameters

# Common

| Name | Default | Value |
|---|---|---|

| | | | |
|---|---|---|---|
| Hydrolysis rate [1/d] | 2.1000 | 2.1000 | 1.0290 |
| Hydrolysis half sat. [-] | 0.0600 | 0.0600 | 1.0000 |
| Anoxic hydrolysis factor [-] | 0.2800 | 0.2800 | 1.0000 |
| Anaerobic hydrolysis factor (AS) [-] | 0.0400 | 0.0400 | 1.0000 |
| Anaerobic hydrolysis factor (AD) [-] | 0.5000 | 0.5000 | 1.0000 |
| Adsorption rate of colloids [L/(mgCOD d)] | 0.1500 | 0.1500 | 1.0290 |
| Ammonification rate [L/(mgCOD d)] | 0.0800 | 0.0800 | 1.0290 |
| Assimilative nitrate/nitrite reduction rate [1/d] | 0.5000 | 0.5000 | 1.0000 |
| Endogenous products decay rate [1/d] | 0 | 0 | 1.0000 |

# AOB

| Name | Default | Value | |
|---|---|---|---|
| Max. spec. growth rate [1/d] | 0.9000 | 0.9000 | 1.0720 |
| Substrate (NH4) half sat. [mgN/L] | 0.7000 | 0.7000 | 1.0000 |
| Byproduct NH4 logistic slope [-] | 50.0000 | 50.0000 | 1.0000 |
| Byproduct NH4 inflection point [mgN/L] | 1.4000 | 1.4000 | 1.0000 |
| AOB denite DO half sat. [mg/L] | 0.1000 | 0.1000 | 1.0000 |
| AOB denite HNO2 half sat. [mgN/L] | 5.000E-6 | 5.000E-6 | 1.0000 |
| Aerobic decay rate [1/d] | 0.1700 | 0.1700 | 1.0290 |
| Anoxic/anaerobic decay rate [1/d] | 0.0800 | 0.0800 | 1.0290 |
| KiHNO2 [mmol/L] | 0.0050 | 0.0050 | 1.0000 |

# NOB

| Name | Default | Value | |
|---|---|---|---|
| Max. spec. growth rate [1/d] | 0.7000 | 0.7000 | 1.0600 |
| Substrate (NO2) half sat. [mgN/L] | 0.1000 | 0.1000 | 1.0000 |
| Aerobic decay rate [1/d] | 0.1700 | 0.1700 | 1.0290 |
| Anoxic/anaerobic decay rate [1/d] | 0.0800 | 0.0800 | 1.0290 |

| | | | |
|---|---|---|---|
| KiNH3 [mmol/L] | 0.0750 | 0.0750 | 1.0000 |

# AAO

| Name | Default | Value | |
|---|---|---|---|
| Max. spec. growth rate [1/d] | 0.2000 | 0.2000 | 1.1000 |
| Substrate (NH4) half sat. [mgN/L] | 2.0000 | 2.0000 | 1.0000 |
| Substrate (NO2) half sat. [mgN/L] | 1.0000 | 1.0000 | 1.0000 |
| Aerobic decay rate [1/d] | 0.0190 | 0.0190 | 1.0290 |
| Anoxic/anaerobic decay rate [1/d] | 0.0095 | 0.0095 | 1.0290 |
| Ki Nitrite [mgN/L] | 1000.0000 | 1000.0000 | 1.0000 |
| Nitrite sensitivity constant [L / (d mgN) ] | 0.0160 | 0.0160 | 1.0000 |

# OHO

| Name | Default | Value | |
|---|---|---|---|
| Max. spec. growth rate [1/d] | 3.2000 | 3.2000 | 1.0290 |
| Substrate half sat. [mgCOD/L] | 5.0000 | 5.0000 | 1.0000 |
| Anoxic growth factor [-] | 0.5000 | 0.5000 | 1.0000 |
| Denite N2 producers (NO3 or NO2) [-] | 0.5000 | 0.5000 | 1.0000 |
| Aerobic decay rate [1/d] | 0.6200 | 0.6200 | 1.0290 |
| Anoxic decay rate [1/d] | 0.2330 | 0.2330 | 1.0290 |
| Anaerobic decay rate [1/d] | 0.1310 | 0.1310 | 1.0290 |
| Fermentation rate [1/d] | 1.6000 | 1.6000 | 1.0290 |
| Fermentation half sat. [mgCOD/L] | 5.0000 | 5.0000 | 1.0000 |
| Fermentation growth factor (AS) [-] | 0.2500 | 0.2500 | 1.0000 |
| Free nitrous acid inhibition [mol/L] | 1.000E-7 | 1.000E-7 | 1.0000 |

# Methylotrophs

| Name | Default | Value | |
|---|---|---|---|
| Max. spec. growth rate [1/d] | 1.3000 | 1.3000 | 1.0720 |
| Methanol half sat. [mgCOD/L] | 0.5000 | 0.5000 | 1.0000 |
| Denite N2 producers (NO3 or NO2) [-] | 0.5000 | 0.5000 | 1.0000 |
| Aerobic decay rate [1/d] | 0.0400 | 0.0400 | 1.0290 |
| Anoxic/anaerobic decay rate [1/d] | 0.0300 | 0.0300 | 1.0290 |
| Free nitrous acid inhibition [mmol/L] | 1.000E-7 | 1.000E-7 | 1.0000 |

# PAO

| Name | Default | Value | |
|---|---|---|---|
| Max. spec. growth rate [1/d] | 0.9500 | 0.9500 | 1.0000 |
| Max. spec. growth rate, P-limited [1/d] | 0.4200 | 0.4200 | 1.0000 |
| Substrate half sat. [mgCOD(PHB)/mgCOD(Zbp)] | 0.1000 | 0.1000 | 1.0000 |
| Substrate half sat., P-limited [mgCOD(PHB)/mgCOD(Zbp)] | 0.0500 | 0.0500 | 1.0000 |
| Magnesium half sat. [mgMg/L] | 0.1000 | 0.1000 | 1.0000 |
| Cation half sat. [mmol/L] | 0.1000 | 0.1000 | 1.0000 |
| Calcium half sat. [mgCa/L] | 0.1000 | 0.1000 | 1.0000 |
| Aerobic/anoxic decay rate [1/d] | 0.1000 | 0.1000 | 1.0000 |
| Aerobic/anoxic maintenance rate [1/d] | 0 | 0 | 1.0000 |
| Anaerobic decay rate [1/d] | 0.0400 | 0.0400 | 1.0000 |
| Anaerobic maintenance rate [1/d] | 0 | 0 | 1.0000 |
| Sequestration rate [1/d] | 4.5000 | 4.5000 | 1.0000 |
| Anoxic growth factor [-] | 0.3300 | 0.3300 | 1.0000 |

# Acetogens

| Name | Default | Value | |
|---|---|---|---|
| Max. spec. growth rate [1/d] | 0.2500 | 0.2500 | 1.0290 |

| | | | |
|---|---|---|---|
| Substrate half sat. [mgCOD/L] | 10.0000 | 10.0000 | 1.0000 |
| Acetate inhibition [mgCOD/L] | 10000.0000 | 10000.0000 | 1.0000 |
| Anaerobic decay rate [1/d] | 0.0500 | 0.0500 | 1.0290 |
| Aerobic/anoxic decay rate [1/d] | 0.5200 | 0.5200 | 1.0290 |

# Methanogens

| Name | Default | Value | |
|---|---|---|---|
| Acetoclastic max. spec. growth rate [1/d] | 0.3000 | 0.3000 | 1.0290 |
| H2-utilizing max. spec. growth rate [1/d] | 1.4000 | 1.4000 | 1.0290 |
| Acetoclastic substrate half sat. [mgCOD/L] | 100.0000 | 100.0000 | 1.0000 |
| Acetoclastic methanol half sat. [mgCOD/L] | 0.5000 | 0.5000 | 1.0000 |
| H2-utilizing CO2 half sat. [mmol/L] | 0.1000 | 0.1000 | 1.0000 |
| H2-utilizing substrate half sat. [mgCOD/L] | 1.0000 | 1.0000 | 1.0000 |
| H2-utilizing methanol half sat. [mgCOD/L] | 0.5000 | 0.5000 | 1.0000 |
| Acetoclastic propionic inhibition [mgCOD/L] | 10000.0000 | 10000.0000 | 1.0000 |
| Acetoclastic anaerobic decay rate [1/d] | 0.1300 | 0.1300 | 1.0290 |
| Acetoclastic aerobic/anoxic decay rate [1/d] | 0.6000 | 0.6000 | 1.0290 |
| H2-utilizing anaerobic decay rate [1/d] | 0.1300 | 0.1300 | 1.0290 |
| H2-utilizing aerobic/anoxic decay rate [1/d] | 2.8000 | 2.8000 | 1.0290 |

# pH

| Name | Default | Value |
|---|---|---|
| OHO low pH limit [-] | 4.0000 | 4.0000 |
| OHO high pH limit [-] | 10.0000 | 10.0000 |
| Methylotrophs low pH limit [-] | 4.0000 | 4.0000 |
| Methylotrophs high pH limit [-] | 10.0000 | 10.0000 |
| Autotrophs low pH limit [-] | 5.5000 | 5.5000 |
| Autotrophs high pH limit [-] | 9.5000 | 9.5000 |

| | | |
|---|---|---|
| PAO low pH limit [-] | 4.0000 | 4.0000 |
| PAO high pH limit [-] | 10.0000 | 10.0000 |
| OHO low pH limit (anaerobic) [-] | 5.5000 | 5.5000 |
| OHO high pH limit (anaerobic) [-] | 8.5000 | 8.5000 |
| Propionic acetogens low pH limit [-] | 4.0000 | 4.0000 |
| Propionic acetogens high pH limit [-] | 10.0000 | 10.0000 |
| Acetoclastic methanogens low pH limit [-] | 5.0000 | 5.0000 |
| Acetoclastic methanogens high pH limit [-] | 9.0000 | 9.0000 |
| H2-utilizing methanogens low pH limit [-] | 5.0000 | 5.0000 |
| H2-utilizing methanogens high pH limit [-] | 9.0000 | 9.0000 |

# Switches

| Name | Default | Value |
|---|---|---|
| OHO DO half sat. [mgO2/L] | 0.0500 | 0.0500 |
| PAO DO half sat. [mgO2/L] | 0.0500 | 0.0500 |
| Anoxic/anaerobic NOx half sat. [mgN/L] | 0.1500 | 0.1500 |
| AOB DO half sat. [mgO2/L] | 0.2500 | 0.2500 |
| NOB DO half sat. [mgO2/L] | 0.5000 | 0.5000 |
| AAO DO half sat. [mgO2/L] | 0.0100 | 0.0100 |
| Anoxic NO3(->NO2) half sat. [mgN/L] | 0.1000 | 0.1000 |
| Anoxic NO3(->N2) half sat. [mgN/L] | 0.0500 | 0.0500 |
| Anoxic NO2(->N2) half sat. (mgN/L) | 0.0100 | 0.0100 |
| NH3 nutrient half sat. [mgN/L] | 0.0050 | 0.0050 |
| PolyP half sat. [mgP/mgCOD] | 0.0100 | 0.0100 |
| VFA sequestration half sat. [mgCOD/L] | 5.0000 | 5.0000 |
| P uptake half sat. [mgP/L] | 0.1500 | 0.1500 |
| P nutrient half sat. [mgP/L] | 0.0010 | 0.0010 |
| Autotroph CO2 half sat. [mmol/L] | 0.1000 | 0.1000 |
| H2 low/high half sat. [mgCOD/L] | 1.0000 | 1.0000 |
| Propionic acetogens H2 inhibition [mgCOD/L] | 5.0000 | 5.0000 |
| Synthesis anion/cation half sat. [meq/L] | 0.0100 | 0.0100 |

# Common

| Name | Default | Value |
|---|---|---|
| Biomass volatile fraction (VSS/TSS) | 0.9200 | 0.9200 |
| Endogenous residue volatile fraction (VSS/TSS) | 0.9200 | 0.9200 |
| N in endogenous residue [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in endogenous residue [mgP/mgCOD] | 0.0220 | 0.0220 |
| Endogenous residue COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |
| Particulate substrate COD:VSS ratio [mgCOD/mgVSS] | 1.6000 | 1.6000 |
| Particulate inert COD:VSS ratio [mgCOD/mgVSS] | 1.6000 | 1.6000 |
| Molecular weight of other anions [mg/mmol] | 35.5000 | 35.5000 |
| Molecular weight of other cations [mg/mmol] | 39.1000 | 39.1000 |

# AOB

| Name | Default | Value |
|---|---|---|
| Yield [mgCOD/mgN] | 0.1500 | 0.1500 |
| AOB denite NO2 fraction as TEA [-] | 0.5000 | 0.5000 |
| Byproduct NH4 fraction to N2O [-] | 0.0025 | 0.0025 |
| N in biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| Fraction to endogenous residue [-] | 0.0800 | 0.0800 |
| COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |

# NOB

| Name | Default | Value |
|---|---|---|
| Yield [mgCOD/mgN] | 0.0900 | 0.0900 |

| | | |
|---|---|---|
| N in biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| Fraction to endogenous residue [-] | 0.0800 | 0.0800 |
| COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |

# AAO

| Name | Default | Value |
|---|---|---|
| Yield [mgCOD/mgN] | 0.1140 | 0.1140 |
| Nitrate production [mgN/mgBiomassCOD] | 2.2800 | 2.2800 |
| N in biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| Fraction to endogenous residue [-] | 0.0800 | 0.0800 |
| COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |

# OHO

| Name | Default | Value |
|---|---|---|
| Yield (aerobic) [-] | 0.6660 | 0.6660 |
| Yield (fermentation, low H2) [-] | 0.1000 | 0.1000 |
| Yield (fermentation, high H2) [-] | 0.1000 | 0.1000 |
| H2 yield (fermentation low H2) [-] | 0.3500 | 0.3500 |
| H2 yield (fermentation high H2) [-] | 0 | 0 |
| Propionate yield (fermentation, low H2) [-] | 0 | 0 |
| Propionate yield (fermentation, high H2) [-] | 0.7000 | 0.7000 |
| CO2 yield (fermentation, low H2) [-] | 0.7000 | 0.7000 |
| CO2 yield (fermentation, high H2) [-] | 0 | 0 |
| N in biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| Endogenous fraction - aerobic [-] | 0.0800 | 0.0800 |

| | | |
|---|---|---|
| Endogenous fraction - anoxic [-] | 0.1030 | 0.1030 |
| Endogenous fraction - anaerobic [-] | 0.1840 | 0.1840 |
| COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |
| Yield (anoxic) [-] | 0.5400 | 0.5400 |
| Yield propionic (aerobic) [-] | 0.6400 | 0.6400 |
| Yield propionic (anoxic) [-] | 0.4600 | 0.4600 |
| Yield acetic (aerobic) [-] | 0.6000 | 0.6000 |
| Yield acetic (anoxic) [-] | 0.4300 | 0.4300 |
| Yield methanol (aerobic) [-] | 0.5000 | 0.5000 |
| Adsorp. max. [-] | 1.0000 | 1.0000 |
| Max fraction to N2O at high FNA over nitrate [-] | 0.0500 | 0.0500 |
| Max fraction to N2O at high FNA over nitrite [-] | 0.1000 | 0.1000 |

# Methylotrophs

| Name | Default | Value |
|---|---|---|
| Yield (anoxic) [-] | 0.4000 | 0.4000 |
| N in biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| Fraction to endogenous residue [-] | 0.0800 | 0.0800 |
| COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |
| Max fraction to N2O at high FNA over nitrate [-] | 0.1000 | 0.1000 |
| Max fraction to N2O at high FNA over nitrite [-] | 0.1500 | 0.1500 |

# PAO

| Name | Default | Value |
|---|---|---|
| Yield (aerobic) [-] | 0.6390 | 0.6390 |
| Yield (anoxic) [-] | 0.5200 | 0.5200 |
| Aerobic P/PHA uptake [mgP/mgCOD] | 0.9300 | 0.9300 |

| | | |
|---|---|---|
| Anoxic P/PHA uptake [mgP/mgCOD] | 0.3500 | 0.3500 |
| Yield of PHA on sequestration [-] | 0.8890 | 0.8890 |
| N in biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| N in sol. inert [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| Fraction to endogenous part. [-] | 0.2500 | 0.2500 |
| Inert fraction of endogenous sol. [-] | 0.2000 | 0.2000 |
| P/Ac release ratio [mgP/mgCOD] | 0.5100 | 0.5100 |
| COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |
| Yield of low PP [-] | 0.9400 | 0.9400 |
| Mg to P mole ratio in polyphosphate [mmolMg/mmolP] | 0.3000 | 0.3000 |
| Cation to P mole ratio in polyphosphate [meq/mmolP] | 0.1500 | 0.1500 |
| Ca to P mole ratio in polyphosphate [mmolCa/mmolP] | 0.0500 | 0.0500 |
| Cation to P mole ratio in organic phosphate [meq/mmolP] | 0.0100 | 0.0100 |

# Acetogens

| Name | Default | Value |
|---|---|---|
| Yield [-] | 0.1000 | 0.1000 |
| H2 yield [-] | 0.4000 | 0.4000 |
| CO2 yield [-] | 1.0000 | 1.0000 |
| N in biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| Fraction to endogenous residue [-] | 0.0800 | 0.0800 |
| COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |

# Methanogens

| Name | Default | Value |
|---|---|---|
| Acetoclastic yield [-] | 0.1000 | 0.1000 |

| | | |
|---|---|---|
| Methanol acetoclastic yield [-] | 0.1000 | 0.1000 |
| H2-utilizing yield [-] | 0.1000 | 0.1000 |
| Methanol H2-utilizing yield [-] | 0.1000 | 0.1000 |
| N in acetoclastic biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| N in H2-utilizing biomass [mgN/mgCOD] | 0.0700 | 0.0700 |
| P in acetoclastic biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| P in H2-utilizing biomass [mgP/mgCOD] | 0.0220 | 0.0220 |
| Acetoclastic fraction to endog. residue [-] | 0.0800 | 0.0800 |
| H2-utilizing fraction to endog. residue [-] | 0.0800 | 0.0800 |
| Acetoclastic COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |
| H2-utilizing COD:VSS ratio [mgCOD/mgVSS] | 1.4200 | 1.4200 |

# General

| Name | Default | Value |
|---|---|---|
| Tank head loss per metre of length (from flow) [m/m] | 0.0025 | 0.0025 |

# Heating fuel/Chemical Costs

| Name | Default | Value |
|---|---|---|
| Methanol [$/gal] | 1.6656 | 1.6656 |
| Ferric [$/gal] | 0.3785 | 0.3785 |
| Aluminium [$/gal] | 0.3028 | 0.3028 |
| Natural gas [$/MMBTU] | 3.1652 | 3.1652 |
| Heating oil [$/gal] | 1.8927 | 1.8927 |
| Diesel [$/gal] | 2.6498 | 2.6498 |
| Custom fuel [$/gal] | 3.7854 | 3.7854 |
| Biogas sale price [$/MMBTU] | 2.1101 | 2.1101 |

# Anaerobic digester

| Name | Default | Value |
|---|---|---|
| Bubble rise velocity (anaerobic digester) [cm/s] | 23.9000 | 23.9000 |
| Bubble Sauter mean diameter (anaerobic digester) [cm] | 0.3500 | 0.3500 |
| Anaerobic digester gas hold-up factor [] | 1.0000 | 1.0000 |

# Combined Heat and Power (CHP) engine

| Name | Default | Value |
|---|---|---|
| Methane heat of combustion [kJ/mole] | 800.0000 | 800.0000 |
| Hydrogen heat of combustion [kJ/mole] | 240.0000 | 240.0000 |
| CHP engine heat price [$/kWh] | 0 | 0 |
| CHP engine power price [$/kWh] | 0.1500 | 0.1500 |

# Calorific values of heating fuels

| Name | Default | Value |
|---|---|---|
| Calorific value of natural gas [BTU/lb] | 20636 | 20636 |
| Calorific value of heating fuel oil [BTU/lb] | 18057 | 18057 |
| Calorific value of diesel [BTU/lb] | 19776 | 19776 |
| Calorific value of custom fuel [BTU/lb] | 13758 | 13758 |

# Density of liquid heating fuels

| Name | Default | Value |
|---|---|---|
| Density of heating fuel oil [lb/ft3] | 56 | 56 |

| | | |
|---|---|---|
| Density of diesel [lb/ft3] | 55 | 55 |
| Density of custom fuel [lb/ft3] | 49 | 49 |

# Mass transfer

| Name | Default | Value | |
|---|---|---|---|
| Kl for H2 [m/d] | 17.0000 | 17.0000 | 1.0240 |
| Kl for CO2 [m/d] | 10.0000 | 10.0000 | 1.0240 |
| Kl for NH3 [m/d] | 1.0000 | 1.0000 | 1.0240 |
| Kl for CH4 [m/d] | 8.0000 | 8.0000 | 1.0240 |
| Kl for N2 [m/d] | 15.0000 | 15.0000 | 1.0240 |
| Kl for N2O [m/d] | 8.0000 | 8.0000 | 1.0240 |
| Kl for O2 [m/d] | 13.0000 | 13.0000 | 1.0240 |

# Henry's law constants

| Name | Default | Value | |
|---|---|---|---|
| CO2 [M/atm] | 3.4000E-2 | 3.4000E-2 | 2400.0000 |
| O2 [M/atm] | 1.3000E-3 | 1.3000E-3 | 1500.0000 |
| N2 [M/atm] | 6.5000E-4 | 6.5000E-4 | 1300.0000 |
| N2O [M/atm] | 2.5000E-2 | 2.5000E-2 | 2600.0000 |
| NH3 [M/atm] | 5.8000E+1 | 5.8000E+1 | 4100.0000 |
| CH4 [M/atm] | 1.4000E-3 | 1.4000E-3 | 1600.0000 |
| H2 [M/atm] | 7.8000E-4 | 7.8000E-4 | 500.0000 |

# Properties constants

| Name | Default | Value |
|---|---|---|

| | | |
|---|---|---|
| K in Viscosity = K e ^(Ea/RT) [Pa s] | 6.849E-7 | 6.849E-7 |
| Ea in Viscosity = K e ^(Ea/RT) [j/mol] | 1.780E+4 | 1.780E+4 |
| Y in ML Viscosity = H2O viscosity * (1+A*MLSS^Y) [-] | 1.0000 | 1.0000 |
| A in ML Viscosity = H2O viscosity * (1+A*MLSS^Y) [m3/g] | 1.000E-7 | 1.000E-7 |
| A in ML Density = H2O density + A*MLSS [(kg/m3)/(g/m3)] | 3.248E-4 | 3.248E-4 |
| A in Antoine equn. [T in K, P in Bar {NIST}] | 5.2039 | 5.2039 |
| B in Antoine equn. [T in K, P in Bar {NIST}] | 1733.9260 | 1733.9260 |
| C in Antoine equn. [T in K, P in Bar {NIST}] | -39.5 | -39.5 |

# Chemical precipitation rates

| Name | Default | Value | |
|---|---|---|---|
| Struvite precipitation rate [1/d] | 3.000E+10 | 3.000E+10 | 1.0240 |
| Struvite redissolution rate [1/d] | 3.000E+11 | 3.000E+11 | 1.0240 |
| Struvite half sat. [mgTSS/L] | 1.0000 | 1.0000 | 1.0000 |
| HDP precipitation rate [L/(molP d)] | 1.000E+8 | 1.000E+8 | 1.0000 |
| HDP redissolution rate [L/(mol P d)] | 1.000E+8 | 1.000E+8 | 1.0000 |
| HAP precipitation rate [molHDP/(L d)] | 5.000E-4 | 5.000E-4 | 1.0000 |

# Chemical precipitation constants

| Name | Default | Value |
|---|---|---|
| Struvite solubility constant [mol/L] | 6.918E-14 | 6.918E-14 |
| HDP solubility product [mol/L] | 2.750E-22 | 2.750E-22 |
| HDP half sat. [mgTSS/L] | 1.0000 | 1.0000 |
| Equilibrium soluble PO4 with Al dosing at pH 7 [mgP/L] | 0.0100 | 0.0100 |
| Al to P ratio [molAl/molP] | 0.8000 | 0.8000 |
| Al(OH)3 solubility product [mol/L] | 1.259E+9 | 1.259E+9 |
| AlHPO4+ dissociation constant [mol/L] | 7.943E-13 | 7.943E-13 |
| Equilibrium soluble PO4 with Fe dosing at pH 7 [mgP/L] | 0.0100 | 0.0100 |

| | | |
|---|---|---|
| Fe to P ratio [molFe/molP] | 1.6000 | 1.6000 |
| Fe(OH)3 solubility product [mol/L] | 0.0500 | 0.0500 |
| FeH2PO4++ dissociation constant [mol/L] | 5.012E-22 | 5.012E-22 |

# Pipe and pump parameters

| Name | Default | Value |
|---|---|---|
| Static head [ft] | 0.8202 | 0.8202 |
| Pipe length (headloss calc.s) [ft] | 164.0420 | 164.0420 |
| Pipe inside diameter [in] | 19.68504 | 19.68504 |
| K(fittings) - Total minor losses K | 5.0000 | 5.0000 |
| Pipe roughness [in] | 0.00787 | 0.00787 |
| 'A' in overall pump efficiency = A + B*Q + C*(Q^2)[ - ] | 0.8500 | 0.8500 |
| 'B' in overall pump efficiency = A + B*Q + C*(Q^2)[ [ - ]/(mgd) ] | 0 | 0 |
| 'C' in overall pump efficiency = A + B*Q + C*(Q^2)[ [ - ]/(mgd)^2 ] | 0 | 0 |

# Fittings and loss coefficients ('K' values)

| Name | Default | Value |
|---|---|---|
| Pipe entrance (bellmouth) | 0.0500 | 1.0000 |
| 90° bend | 0.7500 | 5.0000 |
| 45° bend | 0.3000 | 2.0000 |
| Butterfly value (open) | 0.3000 | 1.0000 |
| Non-return value | 1.0000 | 0 |
| Outlet (bellmouth) | 0.2000 | 1.0000 |

# Aeration

| Name | Default | Value |
|---|---|---|
| Surface pressure [kPa] | 101.3250 | 101.3250 |
| Fractional effective saturation depth (Fed) [-] | 0.3250 | 0.3250 |
| Supply gas CO2 content [vol. %] | 0.0350 | 0.0350 |
| Supply gas O2 [vol. %] | 20.9500 | 20.9500 |
| Off-gas CO2 [vol. %] | 2.0000 | 2.0000 |
| Off-gas O2 [vol. %] | 18.8000 | 18.8000 |
| Off-gas H2 [vol. %] | 0 | 0 |
| Off-gas NH3 [vol. %] | 0 | 0 |
| Off-gas CH4 [vol. %] | 0 | 0 |
| Off-gas N2O [vol. %] | 0 | 0 |
| Surface turbulence factor [-] | 2.0000 | 2.0000 |
| Set point controller gain [] | 1.0000 | 1.0000 |

# Blower

| Name | Default | Value |
|---|---|---|
| Intake filter pressure drop [psi] | 0.5076 | 0.5076 |
| Pressure drop through distribution system (piping/valves) [psi] | 0.4351 | 0.4351 |
| Adiabatic/polytropic compression exponent (1.4 for adiabatic) | 1.4000 | 1.4000 |
| 'A' in blower efficiency = A + B*Qa + C*(Qa^2)[ - ] | 0.7500 | 0.7500 |
| 'B' in blower efficiency = A + B*Qa + C*(Qa^2)[ [ - ]/(ft3/min (20C, 1 atm)) ] | 0 | 0 |
| 'C' in blower efficiency = A + B*Qa + C*(Qa^2)[ [ - ]/(ft3/min (20C, 1 atm))^2 ] | 0 | 0 |

# Diffuser

| Name | Default | Value |
|---|---|---|
| k1 in C = k1(PC)^0.25 + k2 | 1.2400 | 0.9229 |
| k2 in C = k1(PC)^0.25 + k2 | 0.8960 | 1.4211 |
| Y in Kla = C Usg ^ Y - Usg in [m3/(m2 d)] | 0.8880 | 0.8898 |

| | | |
|---|---|---|
| Area of one diffuser [ft2] | 0.4413 | 2.6400 |
| Diffuser mounting height [ft] | 0.8202 | 1.0000 |
| Min. air flow rate per diffuser ft3/min (20C, 1 atm) | 0.2943 | 3.0000 |
| Max. air flow rate per diffuser ft3/min (20C, 1 atm) | 5.8858 | 18.0000 |
| 'A' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2 [psi] | 0.4351 | 0.4351 |
| 'B' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2[psi/(ft3/min (20C, 1 atm)) ] | 0 | 0 |
| 'C' in diffuser pressure drop = A + B*(Qa/Diff) + C*(Qa/Diff)^2[psi/(ft3/min (20C, 1 atm))^2] | 0 | 0 |

# Surface aerators

| Name | Default | Value |
|---|---|---|
| Surface aerator Std. oxygen transfer rate [lb O /(hp hr)] | 2.46697 | 2.46697 |
| Maximum power per rotor [hp] | 26.80965 | 26.80965 |

# Modified Vesilind

| Name | Default | Value |
|---|---|---|
| Maximum Vesilind settling velocity (Vo) [ft/min] | 0.387 | 0.387 |
| Vesilind hindered zone settling parameter (K) [L/g] | 0.370 | 0.370 |
| Clarification switching function [mg/L] | 100.000 | 100.000 |
| Specified TSS conc.for height calc. [mg/L] | 2500.000 | 2500.000 |
| Maximum compactability constant [mg/L] | 15000.000 | 15000.000 |

# Double exponential

| Name | Default | Value |
|---|---|---|
| Maximum Vesilind settling velocity (Vo) [ft/min] | 0.934 | 0.934 |
| Maximum (practical) settling velocity (Vo') [ft/min] | 0.615 | 0.615 |

| | | |
|---|---|---|
| Hindered zone settling parameter (Kh) [L/g] | 0.400 | 0.400 |
| Flocculent zone settling parameter (Kf) [L/g] | 2.500 | 2.500 |
| Maximum non-settleable TSS [mg/L] | 20.0000 | 20.0000 |
| Non-settleable fraction [-] | 0.0010 | 0.0010 |
| Specified TSS conc. for height calc. [mg/L] | 2500.0000 | 2500.0000 |

# Emission factors

| Name | Default | Value |
|---|---|---|
| Carbon dioxide equivalence of nitrous oxide | 296.0000 | 296.0000 |
| Carbon dioxide equivalence of methane | 23.0000 | 23.0000 |

# Biofilm general

| Name | Default | Value | |
|---|---|---|---|
| Attachment rate [ g / (m2 d) ] | 80.0000 | 80.0000 | 1.0000 |
| Attachment TSS half sat. [mg/L] | 100.0000 | 100.0000 | 1.0000 |
| Detachment rate [g/(m3 d)] | 8.000E+4 | 8.000E+4 | 1.0000 |
| Solids movement factor [] | 10.0000 | 10.0000 | 1.0000 |
| Diffusion neta [] | 0.8000 | 0.8000 | 1.0000 |
| Thin film limit [mm] | 0.5000 | 0.5000 | 1.0000 |
| Thick film limit [mm] | 3.0000 | 3.0000 | 1.0000 |
| Assumed Film thickness for tank volume correction (temp independent) [mm] | 0.7500 | 0.7500 | 1.0000 |
| Film surface area to media area ratio - Max.[ ] | 1.0000 | 1.0000 | 1.0000 |
| Minimum biofilm conc. for streamer formation [gTSS/m2] | 4.0000 | 4.0000 | 1.0000 |

# Maximum biofilm concentrations [mg/L]

| Name | Default | Value | |
|---|---|---|---|
| Ordinary heterotrophic organisms (OHO) | 5.000E+4 | 5.000E+4 | 1.0000 |
| Methylotrophs | 5.000E+4 | 5.000E+4 | 1.0000 |
| Ammonia oxidizing biomass (AOB) | 1.000E+5 | 1.000E+5 | 1.0000 |
| Nitrite oxidizing biomass (NOB) | 1.000E+5 | 1.000E+5 | 1.0000 |
| Anaerobic ammonia oxidizers (AAO) | 5.000E+4 | 5.000E+4 | 1.0000 |
| Polyphosphate accumulating organisms (PAO) | 5.000E+4 | 5.000E+4 | 1.0000 |
| Propionic acetogens | 5.000E+4 | 5.000E+4 | 1.0000 |
| Methanogens - acetoclastic | 5.000E+4 | 5.000E+4 | 1.0000 |
| Methanogens - hydrogenotrophic | 5.000E+4 | 5.000E+4 | 1.0000 |
| Endogenous products | 3.000E+4 | 3.000E+4 | 1.0000 |
| Slowly bio. COD (part.) | 5000.0000 | 5000.0000 | 1.0000 |
| Slowly bio. COD (colloid.) | 4000.0000 | 4000.0000 | 1.0000 |
| Part. inert. COD | 5000.0000 | 5000.0000 | 1.0000 |
| Part. bio. org. N | 0 | 0 | 1.0000 |
| Part. bio. org. P | 0 | 0 | 1.0000 |
| Part. inert N | 0 | 0 | 1.0000 |
| Part. inert P | 0 | 0 | 1.0000 |
| Stored PHA | 5000.0000 | 5000.0000 | 1.0000 |
| Releasable stored polyP | 1.150E+6 | 1.150E+6 | 1.0000 |
| Fixed stored polyP | 1.150E+6 | 1.150E+6 | 1.0000 |
| Readily bio. COD (complex) | 0 | 0 | 1.0000 |
| Acetate | 0 | 0 | 1.0000 |
| Propionate | 0 | 0 | 1.0000 |
| Methanol | 0 | 0 | 1.0000 |
| Dissolved H2 | 0 | 0 | 1.0000 |
| Dissolved CH4 | 0 | 0 | 1.0000 |
| Ammonia N | 0 | 0 | 1.0000 |
| Sol. bio. org. N | 0 | 0 | 1.0000 |
| Nitrous Oxide N | 0 | 0 | 1.0000 |
| Nitrite N | 0 | 0 | 1.0000 |
| Nitrate N | 0 | 0 | 1.0000 |
| Dissolved N2 | 0 | 0 | 1.0000 |
| PO4-P (Sol. & Me Complexed) | 1.000E+10 | 1.000E+10 | 1.0000 |
| Sol. inert COD | 0 | 0 | 1.0000 |

| | | | |
|---|---|---|---|
| Sol. inert TKN | 0 | 0 | 1.0000 |
| ISS Influent | 1.300E+6 | 1.300E+6 | 1.0000 |
| Struvite | 8.500E+5 | 8.500E+5 | 1.0000 |
| Hydroxy-dicalcium-phosphate | 1.150E+6 | 1.150E+6 | 1.0000 |
| Hydroxy-apatite | 1.600E+6 | 1.600E+6 | 1.0000 |
| Magnesium | 0 | 0 | 1.0000 |
| Calcium | 0 | 0 | 1.0000 |
| Metal | 1.000E+10 | 1.000E+10 | 1.0000 |
| Other Cations (strong bases) | 0 | 0 | 1.0000 |
| Other Anions (strong acids) | 0 | 0 | 1.0000 |
| Total CO2 | 0 | 0 | 1.0000 |
| User defined 1 | 0 | 0 | 1.0000 |
| User defined 2 | 0 | 0 | 1.0000 |
| User defined 3 | 5.000E+4 | 5.000E+4 | 1.0000 |
| User defined 4 | 5.000E+4 | 5.000E+4 | 1.0000 |
| Dissolved O2 | 0 | 0 | 1.0000 |

# Effective diffusivities [m2/s]

| Name | Default | Value | |
|---|---|---|---|
| Ordinary heterotrophic organisms (OHO) | 5.000E-14 | 5.000E-14 | 1.0290 |
| Methylotrophs | 5.000E-14 | 5.000E-14 | 1.0290 |
| Ammonia oxidizing biomass (AOB) | 5.000E-14 | 5.000E-14 | 1.0290 |
| Nitrite oxidizing biomass (NOB) | 5.000E-14 | 5.000E-14 | 1.0290 |
| Anaerobic ammonia oxidizers (AAO) | 5.000E-14 | 5.000E-14 | 1.0290 |
| Polyphosphate accumulating organisms (PAO) | 5.000E-14 | 5.000E-14 | 1.0290 |
| Propionic acetogens | 5.000E-14 | 5.000E-14 | 1.0290 |
| Methanogens - acetoclastic | 5.000E-14 | 5.000E-14 | 1.0290 |
| Methanogens - hydrogenotrophic | 5.000E-14 | 5.000E-14 | 1.0290 |
| Endogenous products | 5.000E-14 | 5.000E-14 | 1.0290 |
| Slowly bio. COD (part.) | 5.000E-14 | 5.000E-14 | 1.0290 |
| Slowly bio. COD (colloid.) | 5.000E-10 | 5.000E-12 | 1.0290 |
| Part. inert. COD | 5.000E-14 | 5.000E-14 | 1.0290 |

| | | | |
|---|---|---|---|
| Part. bio. org. N | 5.000E-14 | 5.000E-14 | 1.0290 |
| Part. bio. org. P | 5.000E-14 | 5.000E-14 | 1.0290 |
| Part. inert N | 5.000E-14 | 5.000E-14 | 1.0290 |
| Part. inert P | 5.000E-14 | 5.000E-14 | 1.0290 |
| Stored PHA | 5.000E-14 | 5.000E-14 | 1.0290 |
| Releasable stored polyP | 5.000E-14 | 5.000E-14 | 1.0290 |
| Fixed stored polyP | 5.000E-14 | 5.000E-14 | 1.0290 |
| Readily bio. COD (complex) | 6.900E-10 | 6.900E-10 | 1.0290 |
| Acetate | 1.240E-9 | 1.240E-9 | 1.0290 |
| Propionate | 8.300E-10 | 8.300E-10 | 1.0290 |
| Methanol | 1.600E-9 | 1.600E-9 | 1.0290 |
| Dissolved H2 | 5.850E-9 | 5.850E-9 | 1.0290 |
| Dissolved CH4 | 1.963E-9 | 1.963E-9 | 1.0290 |
| Ammonia N | 2.000E-9 | 2.000E-9 | 1.0290 |
| Sol. bio. org. N | 1.370E-9 | 1.370E-9 | 1.0290 |
| Nitrous Oxide N | 1.607E-9 | 1.607E-9 | 1.0290 |
| Nitrite N | 2.980E-9 | 2.980E-9 | 1.0290 |
| Nitrate N | 2.980E-9 | 2.980E-9 | 1.0290 |
| Dissolved N2 | 1.900E-9 | 1.900E-9 | 1.0290 |
| PO4-P (Sol. & Me Complexed) | 2.000E-9 | 2.000E-9 | 1.0290 |
| Sol. inert COD | 6.900E-10 | 6.900E-10 | 1.0290 |
| Sol. inert TKN | 6.850E-10 | 6.850E-10 | 1.0290 |
| ISS Influent | 5.000E-14 | 5.000E-14 | 1.0290 |
| Struvite | 5.000E-14 | 5.000E-14 | 1.0290 |
| Hydroxy-dicalcium-phosphate | 5.000E-14 | 5.000E-14 | 1.0290 |
| Hydroxy-apatite | 5.000E-14 | 5.000E-14 | 1.0290 |
| Magnesium | 7.200E-10 | 7.200E-10 | 1.0290 |
| Calcium | 7.200E-10 | 7.200E-10 | 1.0290 |
| Metal | 4.800E-10 | 4.800E-10 | 1.0290 |
| Other Cations (strong bases) | 1.440E-9 | 1.440E-9 | 1.0290 |
| Other Anions (strong acids) | 1.440E-9 | 1.440E-9 | 1.0290 |
| Total CO2 | 1.960E-9 | 1.960E-9 | 1.0290 |
| User defined 1 | 6.900E-10 | 6.900E-10 | 1.0290 |
| User defined 2 | 6.900E-10 | 6.900E-10 | 1.0290 |
| User defined 3 | 5.000E-14 | 5.000E-14 | 1.0290 |

| | | | |
|---|---|---|---|
| User defined 4 | 5.000E-14 | 5.000E-14 | 1.0290 |
| Dissolved O2 | 2.500E-9 | 2.500E-9 | 1.0290 |

# EPS Strength coefficients [ ]

| Name | Default | Value | |
|---|---|---|---|
| Ordinary heterotrophic organisms (OHO) | 1.0000 | 1.0000 | 1.0000 |
| Methylotrophs | 1.0000 | 1.0000 | 1.0000 |
| Ammonia oxidizing biomass (AOB) | 5.0000 | 5.0000 | 1.0000 |
| Nitrite oxidizing biomass (NOB) | 25.0000 | 25.0000 | 1.0000 |
| Anaerobic ammonia oxidizers (AAO) | 10.0000 | 10.0000 | 1.0000 |
| Polyphosphate accumulating organisms (PAO) | 1.0000 | 1.0000 | 1.0000 |
| Propionic acetogens | 1.0000 | 1.0000 | 1.0000 |
| Methanogens - acetoclastic | 1.0000 | 1.0000 | 1.0000 |
| Methanogens - hydrogenotrophic | 1.0000 | 1.0000 | 1.0000 |
| Endogenous products | 1.0000 | 1.0000 | 1.0000 |
| Slowly bio. COD (part.) | 1.0000 | 1.0000 | 1.0000 |
| Slowly bio. COD (colloid.) | 1.0000 | 1.0000 | 1.0000 |
| Part. inert. COD | 1.0000 | 1.0000 | 1.0000 |
| Part. bio. org. N | 1.0000 | 1.0000 | 1.0000 |
| Part. bio. org. P | 1.0000 | 1.0000 | 1.0000 |
| Part. inert N | 1.0000 | 1.0000 | 1.0000 |
| Part. inert P | 1.0000 | 1.0000 | 1.0000 |
| Stored PHA | 1.0000 | 1.0000 | 1.0000 |
| Releasable stored polyP | 1.0000 | 1.0000 | 1.0000 |
| Fixed stored polyP | 1.0000 | 1.0000 | 1.0000 |
| Readily bio. COD (complex) | 0 | 0 | 1.0000 |
| Acetate | 0 | 0 | 1.0000 |
| Propionate | 0 | 0 | 1.0000 |
| Methanol | 0 | 0 | 1.0000 |
| Dissolved H2 | 0 | 0 | 1.0000 |
| Dissolved CH4 | 0 | 0 | 1.0000 |
| Ammonia N | 0 | 0 | 1.0000 |

| | | | |
|---|---|---|---|
| Sol. bio. org. N | 0 | 0 | 1.0000 |
| Nitrous Oxide N | 0 | 0 | 1.0000 |
| Nitrite N | 0 | 0 | 1.0000 |
| Nitrate N | 0 | 0 | 1.0000 |
| Dissolved N2 | 0 | 0 | 1.0000 |
| PO4-P (Sol. & Me Complexed) | 1.0000 | 1.0000 | 1.0000 |
| Sol. inert COD | 0 | 0 | 1.0000 |
| Sol. inert TKN | 0 | 0 | 1.0000 |
| ISS Influent | 0.3300 | 0.3300 | 1.0000 |
| Struvite | 1.0000 | 1.0000 | 1.0000 |
| Hydroxy-dicalcium-phosphate | 1.0000 | 1.0000 | 1.0000 |
| Hydroxy-apatite | 1.0000 | 1.0000 | 1.0000 |
| Magnesium | 0 | 0 | 1.0000 |
| Calcium | 0 | 0 | 1.0000 |
| Metal | 1.0000 | 1.0000 | 1.0000 |
| Other Cations (strong bases) | 0 | 0 | 1.0000 |
| Other Anions (strong acids) | 0 | 0 | 1.0000 |
| Total CO2 | 0 | 0 | 1.0000 |
| User defined 1 | 0 | 0 | 1.0000 |
| User defined 2 | 0 | 0 | 1.0000 |
| User defined 3 | 1.0000 | 1.0000 | 1.0000 |
| User defined 4 | 1.0000 | 1.0000 | 1.0000 |
| Dissolved O2 | 0 | 0 | 1.0000 |

# Riverside Regional Water Quality Control Plant

## References

Calibration Data -PD
Plant provided data
Duration: 3 months
Period: 5/1/2016

Design Criteria: -DC
Plant provided - 2011

Drawings:

| | |
|---|---|
| MBR Expansion PDR | - Exp. |
| 1967 WWTP Additions - Plant 2 AB No.1 | -1967 |
| 1987 Plant 1 Upgrade | -1987 |

Model.

| | |
|---|---|
| 2016 Carollo BioTran Model | -2016 |
| 2010 Black and Veatch Biowin Model | -2010 |

## Influent Characteristics

Combined Plant

| | | |
|---|---|---|
| Flow: | 26.939 MGD | (PD) |
| COD: | 777.42 mg/L | (PD) |
| TSS: | 265.29 mg/L | (PD) |
| Ammonia: | 32.29 mg/L | (PD) |

## Primary Treatment

### Plant 1: MBR Train

| | | |
|---|---|---|
| Flow: | 12.55 MGD | (PD) |

MBR Primary Clarifiers

| | | |
|---|---|---|
| Number of Units: | 4 | (DC) |
| Units In Operation: | 2 | (Personal communication) |
| Diameter: | 120 ft | (DC, 1987) |
| Side water depth: | 18 ft | (DC,- Hydraulic Profile) |
| Volume (each): | 1.015 MG | (DC, 1987) |
| Volume (Total): | 4.06 MG | (Calculation) |
| Volume (In Service): | 2.03 MG | (Calculation) |
| Target Sludge conc: | 4.1% | (2010) |
| Effluent | | |
| TSS: | | |
| COD: | | |

### Plant 2: Activated Treatment Train

| | | |
|---|---|---|
| Flow: | 14.38 MGD | (Calculation) |

AS Primary Clarifiers

| | | |
|---|---|---|
| Number of Units: | 4 | (DC) |
| Units in Operation: | 4 | (Personal communication) |
| Diameter: | 95 ft | (DC, 1967) |
| Side water depth: | 9 ft | (DC,- Hydraulic Profile) |
| Volume (each): | 0.48 MG | (DC, 1967) |
| Volume (Total): | 1.91 MG | (Calculation) |
| Volume (In Service): | 1.91 MG | (Calculation) |
| Target Sludge conc: | 4.1% | (2010) |

Effluent
TSS:
COD:

## Secondary Treatment

### Plant 1: MBR Train

Aeration Basins 1-5

| | | |
|---|---|---|
| Number of Units: | 5 | (DC, 1987, Exp) |
| Units in Service: | 5 | (Assumption) |
| Length: | 200 ft | (DC, 1987, Exp) |
| Width | 40 ft | (DC, 1987, Exp) |
| Side Water Depth | 17 ft | (Hydraulic Profile) |
| Volume (each): | 1.01 MG | (DC, 1987, Exp) |
| Volume (Total): | 5.04 MG | (Calculation) |
| Volume (in Service): | 5.04 MG | (Calculation) |
| Anoxic Fraction: | 25% | (DC) |
| Volume (Anoxic): | 1.26 MG | (DC) |
| Volume (Oxic): | 3.78 MG | (DC) |

MBR Tank

| | | |
|---|---|---|
| Number of Units: | 8 | (DC, 1987, Exp) |
| Units in Service: | 8 | (Assumption) |
| Length: | 238.5 ft | (DC, 1987, Exp) |
| Width | 19.8 ft | (DC, 1987, Exp) |

MBR Tank (2nd Stage Anoxic Zones)

| | | |
|---|---|---|
| Number of Units: | 8 | (DC, 1987, Exp) |
| Units in Service: | 8 | (Assumption) |
| Length: | - | - |
| Width | 19.8ft | (DC, 1987, Exp) |
| Side Water Depth: | 11.66ft | (DC,- Hydraulic Profile) |
| Volume (each): | 0.116 MG | (DC, 2010) |
| Volume (Total): | 0.925 MG | (Calculation) |
| Volume (in Service): | 0.925 MG | (Calculation) |

MBR Tank (Membrane Basins)

| | | |
|---|---|---|
| Number of Units: | 8 | (DC, 1987, Exp) |
| Units in Service: | 8 | (Assumption) |
| Length: | 80 ft | (DC, 1987, Exp) |
| Width | 19.8ft | (DC, 1987, Exp) |
| Side Water Depth: | 11.66ft | (DC,- Hydraulic Profile) |
| Volume (each): | | |

Membrane System Characteristics

| | | |
|---|---|---|
| Membrane Identification: | ZW-500d | (2016) |
| Membrane Area (per module): | 370 sf | (2016) |
| Modules per Cassette: | 48 | (2016) |
| Total Membrane Area | 1,989,120 scf | (DC, 2016) |

Cassette Characteristics

| | | | |
|---|---|---|---|
| Cassesette per Unit | | 20 | (DC, 2016) |
| Cassesette used | | 14 | (DC, 2016) |
| Cassesette spare | | 6 | (DC, 2016) |
| Cassete dimensions | | | |
| Length | 7.1 ft | | (2016) |
| Width | | 5.7 ft | (2016) |
| Depth | | 8.3 ft | (2016) |
| Cassette submergence | 9.1 ft | | (2016) |
| Cassette volume per zone | | 5156 cf | (2016) |
| Total Cassette volume | 0.31 MG | | (2016) |

Biowin MBR inputs

| | | | |
|---|---|---|---|
| Packing density | 13.34 sf/cf | | (Calculation) |
| Displaced volume | | 38569.56112 cf/cassette | (Calculation) |
| memb. SA per cas. | | 1760 sf/cassette | (Calculation) |

RAS Tank (Deoxyic Zone)

| | | | |
|---|---|---|---|
| Number of units: | | 4 | (1987) |
| Units in Service: | | 4 | (Personal Communication) |
| Length (Total): | 166.5 ft | (1987) | |
| Width: | 41ft | | (1987) |
| Side Water Depth: | | 11ft | (Hydraulic Profile) |
| Volume: | | 0.56 MG | (Calculation) |

Process zones: DO Profile

| | | | |
|---|---|---|---|
| 1. 1st Anoxic (Aeration Basin 1 - 5) | 0 mg/L | (2016) | |
| 2. Oxic (Aeration Basin 1-5) | 2 mg/L | (2016) | |
| 3. 2nd Anoxicn(MBR Tank) | 0 mg/L | (2016) | |
| 4. MBR (MBR Tank) | 3.2 mg/L | | (2016) |
| 5. DeOxic (RAS Tank) | 0 mg/L | (2010) | |

Operational conditions

| | | | |
|---|---|---|---|
| Reactor MLSS: | 3430 mg TSS/L | (PD) | |
| Permeate Flow: | 5.3 MG | (PD) | |
| WAS flow: | | 0.411 MG | (PD) |
| WAS TSS: | | 5911.78 | (PD) |

**Plant 2: Activated Treatment Train**

Aeration Basins 1-6

| | | | |
|---|---|---|---|
| Number of Units: | | 6 | (1967) |
| Units in Service: | | 6 | (Assumption) |
| Length: | 260 ft | | (1967) |
| Width | | 40 ft | (DC, 1967, Exp) |
| Side Water Depth | | 17 ft | (Hydraulic Profile) |

| | | |
|---|---|---|
| Volume (each): | 1.308 MG | (DC, 1967, Exp) |
| Volume (Total): | 7.85 MG | (Calculation) |
| Volume (in Service): | 7.85 MG | (Calculation) |
| Anoxic Fraction: | 25% | (DC) |
| Volume (Anoxic): | 1.96 MG | (DC) |
| Volume (Oxic): | 5.89 MG | (DC) |

Secondary Clarifiers 1-2

| | | |
|---|---|---|
| Diameter: | 130 ft | (DC, 1967) |
| Sidewater depth: | 14 ft | (DC, Hydraulic profile) |
| Surface Area (each) | 13273 sf | (Caculation) |
| Surface Area (total) | 26546 sf | (Calculation) |

Secondary Clarifiers 3-3

| | | |
|---|---|---|
| Diameter: | 100 ft | (DC, 1967) |
| Sidewater depth: | 10 ft | (DC, Hydraulic profile) |
| Surface Area (each) | 7854 sf | (Caculation) |
| Surface Area (total) | 15708 sf | (Calculation) |

Secondary Clarifiers (1-4)

| | | |
|---|---|---|
| Surface Area (total) | 42254 sf | (Calculation) |
| Depth (wt. average) | 12.5ft | (Calculation) |

Operational conditions

| | | |
|---|---|---|
| Reactor MLSS: | 2811 mg TSS/L (PD) | |
| Reactor VSS: | 2387 mg VSS/L (PD) | |
| RAS Flow: | 9.111 MG | (PD) |
| WAS Flow: | 0.451 MG | (PD) |
| Underflow: | 9.562 MG | (Calculation) |
| Effluent COD: | 24.17 mg/L | (PD) |

## Solids Handling

Appendix E

# TORO COMPANY AGREEMENT

# AGREEMENT

THIS AGREEMENT is made and entered into this 3rd day of December 1985, by and between the CITY OF RIVERSIDE (hereinafter referred to as "City"), The TORO COMPANY, a Delaware corporation (hereinafter referred to as "Toro"), and the COUNTY OF RIVERSIDE (hereinafter referred to as "County").

W I T N E S S E T H

WHEREAS, County is the owner of certain real property commonly known as the Martha McLean-Anza Narrows Park (hereinafter referred to as "Park"); and

WHEREAS, the Park is adjacent to the corporate limits of the City near the City's wastewater treatment plant; and

WHEREAS, the Park is utilized by residents of the City as well as residents of the unincorporated areas of the County, and the proximity of the Park and its continued use and maintenance is a municipal concern; and

WHEREAS, City and Toro are parties to a ground lease dated November 15, 1982, by which Toro leases from the City approximately 20.34 acres owned by City on which Toro establishes plots of grass for testing of mowing equipment manufactured by Toro; and

WHEREAS, the parties hereto desire to join in the development and use of a pipeline for the transporation of tertiary-treated sewage effluent for irrigation use from the wastewater treatment plant of City to said Park and that certain property leased by Toro from City for the purpose of using low cost, reclaimed water, which pipeline and related facilities is hereinafter generally referred to as the "Project";

NOW, THEREFORE, in consideration of the premises, and of the mutual covenants and considerations hereinafter set forth, the parties hereto do hereby covenant and agree as follows:

//

//

1. The parties hereto shall jointly participate in the construction of said Project as generally shown on the plan attached hereto as Exhibit A and incorporated herein by this reference as though fully set forth herein and hereat and in accordance with all of the other terms and conditions hereof. The specific plans and specifications for the Project shall be approved by County's Parks Director, the City's Public Works Director, and Toro's Test Site Supervisor; and a copy of such plans and specifications as approved shall be filed with each party hereto.

2. The Project shall be funded by a contribution not to exceed $15,000 each from City, County and Toro for a total Project cost of $45,000; it being specifically understood and agreed that nothing herein shall either require or prohibit any party from contributing funds in excess of said $15,000, although Toro's maximum contibution shall not exceed $20,000. Said funds shall only be used for: acquisition of any necessary right-of-way from other than a party to this agreement; for the fees for any permits required for the construction of the Project; and for the cost of the equipment, material and installation of the Project.

3. County shall act as lead agency for the Project and shall perform the following functions as such without compensation:

a. Prepare such detailed plans and specifications as may be necessary for the construction of the Project subject to the approval of Toro and City as heretofore provided.

b. Obtain all rights-of-ways and permits required for construction of the Project.

c. Be responsible for complying with any terms or conditions imposed for the approval of any permit including the fulfillment of the requirements of the Santa Ana Region of the California Regional Water Quality Control Board for the use of reclaimed water.

Appendix E

d. Collect funds to be provided pursuant to paragraph 2 of this agreement by each of the parties and make disbursements for all work accomplished hereunder, including reimbursements to Toro, in accordance with approved billing statements detailing such expenditures.

e. Render an accounting to the City and Toro following the completion of the construction of the Project by submitting a written statement of expenditures within thirty (30) days after such completion. In the event the total expenditures do not exceed $45,000, the County shall return any unobligated funds on a pro-rata basis.

4. City shall provide such necessary rights-of-way for pipeline construction within its property along the proposed route as shown on Exhibit A, and shall allow placement of the pump and ingress and egress for pump service and maintenance at its sewage treatment plant as shown on said Exhibit A. If requested by County, City shall issue to the County at no cost a license in a form satisfactory to City Attorney of City granting use of said property for the pipeline and pump and for ingress and egress for maintenance and service.

5. Toro shall provide such necessary rights-of-way for the pipeline construction within its property along the proposed route as shown on Exhibit A. Toro shall also purchase or otherwise acquire the pump, pipe, connectors and any and all other materials necessary to construct and install said Project in accordance with the approved construction plans; provided, however, County shall without compensation assist Toro with pipeline installation by supplying day labor for opening and backfilling trenches and connecting pipes.

6. Following the completion of the installation of the Project, the County and Toro shall jointly maintain the entire Project, including the pipeline and pump systems, and shall equally share the maintenance cost thereof; and further, the electrical cost for pump operations shall be proportioned between County and Toro based upon the amount of water used by each such party,

Appendix E

which water use shall be individually metered and calculated based upon cubic feet of water transported, multiplied by the electrical kilowatt hour unit cost per cubic foot.

7. The City shall, upon the completion of the construction of the Project, furnish reclaimed water from its sewage treatment plant at such rates as may be set from time to time by the City Council of City for such reclaimed water, provided that such rates shall be substantially less than that charged industrial users for domestic, potable water. The charges shall be paid by the County and Toro based upon the amount of water used by each such party. City shall notify County and Toro within four hours of the occurrence of any treatment plant shutdown or failure, or upon the discharge of water not meeting Water Quality Control Board standards for tertiary-treated water; provided, however, nothing herein shall require the City to provide water from any other source, nor shall any liability be imposed on the City for such shutdown or failure.

8. County shall maintain the Park to the highest standards possible in consideration of available resources and annual budget limitations; and savings realized by County by the use of reclaimed water, less the cost of electrical pumping, shall be committed to the Park for Park maintenance.

9. Each party hereto shall indemnify, defend, save and hold harmless the other parties, and each of them, and their respective agents, servants and employees, of and from any and all liabilities, claims, demands, debts, suits, actions and causes of action arising out of or in any manner connected with any act or omission of such indemnifying party hereunder, or its agents, servants or employees, done or performed pursuant to the terms and provisions hereof.

10. In the event the Project is abondoned, any unobligated funds shall be returned to the parties on a proportionate basis. In the event the Project is abandoned within ten (10) years of the date of this agreement, the salvage of any of the pipes and equipment shall be returned to the parties on the basis

of one-third of the value of such salvage, and the rights-of-way herein granted shall automatically terminate.

11. If that ground lease dated November 15, 1982, between City and Toro, or any extensions, renewals, or replacement thereof terminates, Toro has the right to discontinue its obligations to use said water and all of its obligations under Paragraph 6 and 7 above by giving thirty (30) days written notice to City and County. Even if a notice of termination is provided by Toro under this paragraph, if Toro can arrange to use said water within two (2) years from the termination of said ground lease on an alternate site comprising other property then owned by it, or on other property to be subsequently leased by Toro, Toro can reinstate its right to use said water and its obligations under Paragraph 6 and 7 effective as of the date that any new connections needed for transporting said water to said alternate site shall have been completed by Toro.

12. The parties hereto agree that the construction of the Project shall commence within one (1) year following the execution of this Agreement.

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IN WITNESS WHEREOF, the parties hereto have executed this document the day and year first above written.

Facsimile Signature affixed by Clerk per Sec. 25103 Gov. Code

COUNTY OF RIVERSIDE

By Patricia A. Larson
Chairman, Board of Supervisors

ATTEST: DEC 3 1985
GERALD A. MALONEY, Clerk

By Joan Mooney
DEPUTY

Attest
County Clerk of the Board

CITY OF RIVERSIDE

By Al Brown
Mayor

Attest Alice A. Hare
City Clerk

TORO COMPANY

By Kenneth D. Larson
Executive Vice President

APPROVED AS TO FORM:

Deputy County Counsel

APPROVED AS TO FORM:

Assistant City Attorney

APPROVED AS TO FORM:

Toro

Appendix 9B

# CONDITIONAL ACCEPTANCE OF FREE CHLORINE DISINFECTION

CALIFORNIA
Water Boards

GAVIN NEWSOM
GOVERNOR

JARED BLUMENFELD
SECRETARY FOR
ENVIRONMENTAL PROTECTION

**State Water Resources Control Board**
Division of Drinking Water

May 6, 2019

Edward Filadelfia
Technical and Compliance Manager
City of Riverside Public Works Department
3900 Main Street
Riverside, CA 92522

Subject: Conditional Acceptance of Free Chlorine Disinfection at City of Riverside to comply with California Water Recycling Criteria (3390031-729)

Dear Mr. Filadelfia,

The Division of Drinking Water's (Division) Recycled Water Unit has reviewed a request, dated January 9, 2019, from Carollo Engineering to consider free chlorine as an alternative disinfection technology for compliance with the California Water Recycling Criteria (Title 22), Section 60320.5. Accompanying the request was a report entitled, "City of Riverside, Chlorine CT and Modal Contact Time Study, Contact Time Testing and CT Demonstration Results", dated November 2018. The report outlines findings from a study conducted at the Riverside Regional Water Quality Control Plant (Riverside) located in the City of Riverside, CA using Riverside's full-scale chlorine contact basin #2. The purpose of the study is to show Riverside can use free chlorine disinfection to meet Title 22 requirements, thus allowing a shorter contact time in their chlorine contact basin #2. This will allow Riverside to reduce disinfection byproducts and improve overall operational efficiency.

Demonstration studies conducted using free chlorine at Riverside have satisfactorily shown an equal degree of treatment and reliability as those technologies listed in Title 22. Based on a review of the materials submitted, the Division grants conditional acceptance of free chlorine as an alternative treatment technology for recycled water disinfection at Riverside's chlorine contact basin #2 only, subject to the following conditions:

1. Riverside must deliver a minimum free chlorine residual contact time (FCRCT) of 30 mg-min/L at all times to meet both the virus and total coliform reduction requirements of Title 22, section 60301.230.

2. Riverside must meet a minimum free chlorine residual (FCR) of 1.0 mg/L at all times at the chlorine contact basin #2 sample point #2.

3. Riverside must maintain a minimum free chlorine modal contact time at the chlorine contact basin #2 sample point #2 of 29-minutes at all times, based on a peak flow of 46-mgd.

4. Riverside must monitor for ammonia prior to the free chlorine addition and optimize the secondary treatment system to prevent ammonia breakthrough.

5. The free chlorine disinfection process must meet Title 22, Section 60301.230(b) requirements for total coliform.

6. Riverside must be operated with a built-in automatic reliability feature that must be triggered when the minimum FCRCT or FCR are below the targets. The reliability features shall be approved in a new or updated Title 22 Engineering Report for Riverside.

7. Ammonia analyzers, free chlorine analyzers, and flow meters must be installed and properly calibrated to ensure proper disinfection at all times.

8. Ammonia and free chlorine analyzers must be inspected and checked against a reference bench-top unit routinely to determine accuracy. If an on-line analyzer reading varies from the bench-top reading, the on-line analyzer must be recalibrated by a procedure recommended by the manufacturer.

9. Flow meters must be inspected and checked against other flow determination methods routinely to determine accuracy.

10. Riverside should be operated in accordance with an approved operations plan, which specifies clearly the operational limits and responses required for critical alarms. The operations plan should be submitted and approved prior to issuance of the permit. A copy of the approved operations plan should be maintained at the treatment plant and be readily available to operations personnel and regulatory agencies. A quick reference plant operations data sheet should be posted at the treatment plant and include the following information:
    a. The alarm set points for tertiary turbidity, high flow, low FCR, low contact time, and low FCRCT.
    b. The values of high tertiary turbidity, high flow, low FCR, and low FCRCT when recycled water flow must be diverted.
    c. The required frequency of calibration for critical online instruments; such as online turbidity meters, ammonia analyzers, and chlorine analyzers.

11. Riverside should maintain records of compliance with these conditions, made available to DDW or the Regional Water Quality Control Board (RWQCB) upon request. In addition, Riverside should report a summary of compliance with an approved operations plant as part of its annual reporting to the RWQCB.

12. These conditions shall be incorporated into the final permit for Riverside.

Conditional acceptance is specific to the free chlorine disinfection process at Riverside's chlorine contact basin #2 only, as detailed in the report dated November 2018 described above. The Division must review in advance any future proposed changes made in the design or

operation of this treatment technology to determine whether the modifications will require additional testing to ensure Title 22 equivalency.

Review and approval of free chlorine disinfection at Riverside is provided through the RWQCB's Water Reclamation permitting process. Prior to Riverside being permitted by the RWQCB, an approved Title 22 Engineering Report for the project is required. The engineering report shall include how Riverside meets the requirements of Title 22 and this letter. The Division's local district offices will also review the free chlorine disinfection process at Riverside to confirm full compliance with all applicable treatment and reliability features required by Title 22.

If you have any questions regarding this letter, please contact Randy Barnard at (619) 525-4022 or via email at Randy.Barnard@waterboards.ca.gov.

Sincerely,

Randy Barnard, P.E.
Recycled Water Unit Chief
Recycled Water Unit
Division of Drinking Water
State Water Resources Control Board
1350 Front St., Rm. 2050
San Diego, CA 92101

cc:

1. Hope Smythe, Executive Officer, RWQCB 8 (via email)
2. Steve Williams, P.E., District Engineer, DDW-District 20 (via email)
3. Robert Eland, City of Riverside, RWQCP, 5950 Acorn Street, Riverside, CA 92504

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 10

## Wastewater Treatment System
## Environmental Review

carollo

Engineers...Working Wonders With Water®

City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 10: ENVIRONMENTAL REVIEW

FINAL | June 2019

City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 10: ENVIRONMENTAL REVIEW

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.08.05 15:28:19-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 10

# ENVIRONMENTAL REVIEW

## 10.1 Purpose

The purpose of this chapter is to list the treatment plant processes and facilities recommended for construction in this update of the Master Plan, to list the projects recommended for construction in the 2008 Master Plan noting which facilities from that master plan were ultimately constructed as part the recent Phase I Expansion Project, to compare and summarize the findings from the two lists, and to summarize what documentation might be required under CEQA for each of the new or upgraded facilities. The listings and any related discussions for the most part cover only facilities on the treatment plant site. Improvements to the collection system are not covered in this chapter. The chapter includes a comparison of the facilities recommended in the update of the Master Plan with those recommended in the 2008 Master Plan.

The chapter is meant to serve as an aid to the City in determining the scope of work under the CEQA that must be undertaken prior to construction of the recommended additions and upgrades to the plant during the planning period for the update to the Master Plan. It is recommended that the City's Environmental Permitting Department and/or the City's CEQA consultant review this Chapter prior to the start of design work.

## 10.2 Background

The 2008 Master Plan prepared by Carollo recommended various new, modified, and upgraded facilities needed primarily to increase treatment capacity of the plant, upgrade the level of treatment to meet new regulatory requirements, or improve or restore the condition of existing facilities. Some changes to the planned facilities and their locations were made during subsequent design and construction projects referred to collectively as the MBR treatment train (Plant 1) Phase I Expansion.

A PEIR for the project was prepared for and certified by the City as the lead agency in 2010. This document was based on the 2008 Master Plan.

## 10.3 2008 Master Plan and MBR Treatment Train (Plant 1) Phase I Expansion Project

Major treatment facilities recommended in the 2008 Master Plan are as listed in Table 10.1 and their proposed locations are shown on Figure 10.1. For convenience, the Item number in Table 10.1 corresponds to the proposed locations marked on Figure 10.1.

Table 10.1 Treatment Facilities Recommended in the 2008 Master Plan and Constructed

| Item | Facility | Constructed | Comments |
|---|---|---|---|
| 1 | New Headworks | No | Subsequent analysis showed that a new headworks was not needed in this phase. |
| 2 | Biofilters (odor control) | Yes | Separate odor control systems were built for primary clarifiers and the headworks. |

Table 10.1 Treatment Facilities Recommended in the 2008 Master Plan and Constructed (continued)

| Item | Facility | Constructed | Comments |
|---|---|---|---|
| 3 | Plant 1 Primary Clarifiers (4) | Yes | |
| 4 | Primary Sludge Pump Station | Yes | For MBR treatment train (Plant 1) primary clarifiers |
| 5 | Primary Effluent EQ Basins | Yes | |
| 6 | MBR treatment train (Plant 1) Aeration Basin Additions (1 new pass) | Yes | |
| 7 | MBR treatment train (Plant 1) MBR Facility | Yes | Included an MBR fine screening facility |
| 8 | New CCB | Yes | Modified and extended existing Basin 2. The proposed new Basin 4 adjacent to Basin 3 was not constructed. |
| 9 | Primary Sludge Thickeners | No | Benefit of incorporating the process was reconsidered. |
| 10 | New WAS Thickeners | No | |
| 11 | New Digesters (2) | Yes | Included an adjacent gas storage facility. |
| 12 | New Maintenance Building | Yes | |
| 13 | Parking Lot Expansion | No | |

Note that not all the originally recommended facilities were constructed as part of the Phase I project. Some of the planned facilities were listed as Phase 2 facilities in the 2008 Master Plan or at the time of Phase I design and their construction was deferred. For others, the need for or benefits of their construction were reconsidered. Figure 10.2 shows the locations of the MBR treatment train (Plant 1) Phase I facilities that were constructed.

## 10.4 Update of the Master Plan

Various chapters of this update of the Master Plan have identified projects that are recommended for construction within the near term (at various times within the next 10 years), and through 2037. The projects are needed to rectify the deteriorated conditions of some facilities, to address O&M issues, to install equipment employing current technology, and to meet new, more stringent regulatory requirements. Construction of the major and most costly projects, including those involving modifications and upgrades to the MBR treatment train (Plant 1) and modifications to the ACT treatment train (Plant 2) and tertiary filters, are driven by regulatory requirements mandating a greater degree of nutrient removal, and construction of an AWT plant for removal of dissolved salts in the effluent.

Construction of most of the smaller projects, particularly those involving rehabilitation and upgrading of existing facilities and equipment, will be carried out within the confines of existing structures and process areas.

In this update of the Master Plan, increases in treatment capacity for the plant and its major processes are not major factors. The increases in plant inflow predicted in earlier studies have not been realized, and new evaluations based on current data show smaller and slower increases. The

primary reasons for the smaller flow increases appear to be water conservation throughout the City's system and a slowdown in housing construction.

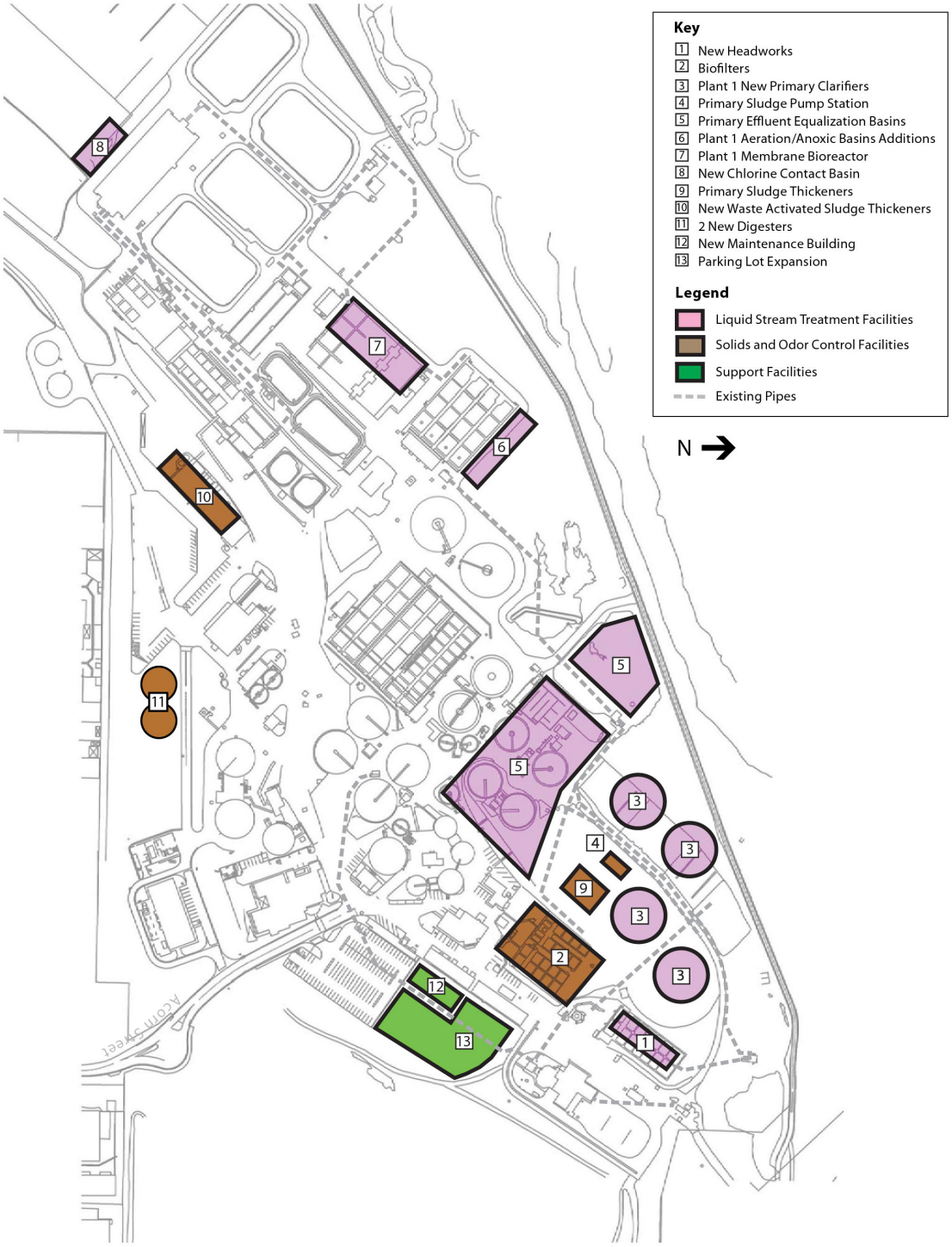


Figure 10.1 Site Layout from the 2008 Master Plan

**KEY**

1 Stormwater Retention Pond No. 2
2 Expanded Chlorine Contact Basin No. 2
3 Chemical Storage and Feed Area
4 MBR and Aeration Standby Generators No. 1 and No. 2
5 Scour Air/Blower Building
6 MBR RAS Splitter Box
7 MBR Aeration Basin No. 5
8 Activated Secondary Clarifiers
9 MBR Fine Screen Facility
10 Stormwater Pond No. 1
11 Existing Stormwater Pump Station No. 1
12 Material Storage Bed No. 3 & 4
13 MBR Primary Clarifiers
14 Primary Effluent EQ Basins Pump Station
15 MBR Primaries & Education Electrical Building
16 Stormwater Pumping Station No. 2
17 Chemical Storage and Feed Area
18 MBR Facility
19 MBR Aeration Basin Splitter Box
20 Blower Building No. 2
21 Digester No. 1
22 TWAS Disintegration Facility
23 Digester Flares 1 & 2
24 Existing Gas Flare
25 Maintenance Storage Facility
26 Digester No. 3
27 Digester Control Building
28 Digester No. 4
29 Solids Blending Facility
30 Digester Gas Storage Facility
31 Fog Receiving and Processing Station
32 Fog Receiving Station
33 Existing Digester No. 2
34 Future Food Waste Digester No. 5
35 Activated Primary Clarifiers with Odor Control Covers
36 Digested Sludge Holding Tank
37 Primary Clarifiers Odor Control Facility
38 Digested Sludge Pump Station
39 Primary Clarifier Splitter Box
40 Expanded Maintenance Scheduling and Collection Administration Building
41 Existing Sewerline Maintenance Building
42 Existing Plant 1 Primary Clarifiers
43 Inlet Channel To Be Demolished
44 MBR Influent Diversion Structure
45 Existing Headworks
46 Headworks Odor Control Facility
47 Primary Effluent EQ Basin
48 MBR Influent Flow Meter
49 Existing Material Storage Bed No. 1 & 2
50 Existing Plant 1 Primary Clarifiers
51 Ferric Chloride Dosing Station
52 Power Generation Standby Generators 1 & 2

**LEGEND**

New Facilities

Existing Facilities To Be Modified

Existing Facilities To Be Demolished
(See demolition sheets for more information)

Figure 10.2 Existing Site Plan

The near-term projects identified in this update of the Master Plan are given in Table 10.2. The table also gives the primary driver for each project and the Volume and Chapter where it is discussed in this update of the Master Plan. Figure 10.3 shows the current site layout with facilities color-coded by process area and plant identity (i.e., MBR treatment train (Plant 1) and ACT treatment train (Plant 2)). Figure 10.4 shows the locations of the projects included in this update to the Master Plan.

Table 10.2 Recommended Project List from Update of the Master Plan

| Project No. | Project Name | Project Driver | Master Plan Source |
|---|---|---|---|
| FI-01 | Influent Flow Metering Project | O&M Initiative | Vol. 4 Ch. 2/ Vol. 4 Ch. 4 |
| FI-02 | Headworks Rehabilitation | Condition | Vol. 6 Ch. 1 |
| FI-03 | Headworks Screening Conveyor Replacement | Condition | Vol. 6 Ch. 1/ Vol. 4 Ch. 4 |
| FI-04 | Headworks Bypass | O&M Initiative | Vol. 4 Ch. 4 |
| FI-05 | Headworks Grit Classifiers and Pumps | Condition | Vol. 6 Ch. 1 |
| FI-06 | MBR treatment train (Plant 2) Phase II Expansion | O&M Initiative | Vol. 4 Ch. 6 |
| FI-07 | MBR treatment train (Plant 2) Enhanced Nutrient Removal | Regulatory | Vol. 4 Ch. 6 |
| FI-08 | ACT treatment train (Plant 2) Rehabilitation | Condition | Vol. 6 Ch. 1 |
| FI-09 | ACT treatment train (Plant 2) Mixers Rehabilitation | Condition | Vol. 6 Ch. 1 |
| FI-10 | ACT treatment train (Plant 2) RAS/WAS Pump Station Rehabilitation | Condition | Vol. 6 Ch. 1 |
| FI-11 | ACT treatment train (Plant 2) RAS/WAS Pumps Replacement | Condition | Vol. 6 Ch. 1 |
| FI-12 | ACT treatment train (Plant 2) Enhanced Nutrient Removal | Regulatory | Vol. 4 Ch. 6 |
| FI-13 | Tertiary Filter Replacement and CCB Retrofits | Condition | Vol. 6 Ch. 1/ Vol. 4 Ch. 7 |
| FI-14 | CCB Rehabilitation | Condition | Vol. 6 Ch. 1 |
| FI-15 | First Primary Sludge Pumping Rehabilitation | Condition | Vol. 6 Ch. 1 |
| FI-16 | Second Primary Sludge Pumping Rehabilitation | Condition | Vol. 6 Ch. 1 |
| FI-17 | WAS Thickening Project | Capacity | Vol. 5 Ch. 4 |
| FI-18 | AWT (Desalting) Project | Regulatory | Vol. 4 Ch. 8 |
| FI-19 | Levee Rehabilitation Phase II | Site Protection | Vol. 4 Ch. 11 |

| KEY | |
|---|---|
| 1. | Influent Flow Monitoring |
| 2. | Final Effluent Metering Flow Control Outflow |
| 3. | Headworks Facility |
| 4. | Headworks Odor Control Facility |
| 5. | Primary Clarifier |
| 6. | Aeration Basin |
| 7. | Secondary Clarifier |
| 8. | Tertiary Filter |
| 9. | Chlorine Contact Basin |
| 10. | Primary Clarifiers Odor Control Facility |
| 11. | Primary Sludge and Scum Pump Station |
| 12. | RAS/WAS Pump Station |
| 13. | RAS Pump Station |
| 14. | Diurnal Flow Equalization Basin |
| 15. | Wet Weather Equalization Basin |
| 16. | MBR Fine Screen Facility |
| 17. | MBR Filtration Units |
| 18. | MBR Train Influent Flow Meter |
| 19. | MBR Air Scour/Blower Building |
| 20. | Blower Building |
| 21. | Chemical Storage and Feed Area |
| 22. | Dechlorination Building |
| 23. | Digester |
| 24. | Digester Control Building |
| 25. | Digested Sludge Pump Station |
| 26. | Digested Sludge Storage Tank |
| 27. | Digester Gas Storage Facility |
| 28. | Digester Flares |
| 29. | Dissolved Air Flotation Thickeners |
| 30. | Solids Metering Facility |
| 31. | Bio-Solids Blending Facility |
| 32. | Bio-Solids Dewatering Facility |
| 33. | Bio-Solids Silos/Loading Facility |
| 34. | TWAS Disintegration Facility |
| 35. | FOG Receiving Station |
| 36. | Secondary Effluent Equalization Pond |
| 37. | Stormwater Retention Pond |
| 38. | Waste Backwash Lagoon |
| 39. | Backwash Storage Tank |
| 40. | RWQCP Plant Effluent Outfall |
| 41. | Flow Monitoring or Flow Control Structure |
| 42. | Splitter Box or Diversion Structure |
| 43. | Pump Station/Switchgear Room |
| 44. | Sampling Station |
| 45. | Electrical or MCC Structure |
| 46. | Power Generation or Standby Generators |
| 47. | Fuel Cell |
| 48. | Cogeneration |
| 49. | Administration Building |
| 50. | Cooling Tower |
| 51. | Maintenance, Storage, or Misc. |
| 52. | Material Storage Bed |
| 53. | Native American Rock Reserve Area |
| 54. | Recycled Water Pump Station |

Figure 10.3 Existing Facilities by Process Area and Plant Designations

## 10.5 Summary of the Recommended Projects from the 2008 Master Plan and the Update to the Master Plan

This section summarizes and compares the recommended projects from the 2008 Master Plan and the update of the Master Plan on a process area by process area basis and notes, in most cases, whether or not work will be done within existing structures or dedicated areas. Not all of the small rehabilitation work is defined or included as a specific item.

1. Headworks: The new headworks recommended in the 2008 Master Plan was not constructed. Due primarily to lower than predicted inflow to the plant, the update of the Master Plan recommends modifications and upgrades (mostly replacement of older equipment with modern equipment) to the existing headworks. This work will take place primarily within the existing headworks facility.
2. Odor Control Biofilters: The odor control facilities for the headworks and new Plant 1 primary clarifiers were constructed in Phase 1. Additional work is not anticipated in the Phase 2 construction.
3. Plant 1 Primary Clarifiers and Sludge Pumping: These were constructed under the Phase 1 projects and no new construction is anticipated.
4. MBR treatment train (Plant 1) Upgrades: Upgrades to the previously constructed aeration basins and MBR facility are recommended to meet new regulatory requirements, to facilitate O&M, and take advantage of current, more efficient technology. These upgrades would be constructed within the existing facilities.
5. Plant 1 Aeration Basins: Under the Phase 1 projects, one new basin (Basin 5) was added to the existing Plant 1 aeration basins. The update to the Master Plan recommends construction of two additional basins (Basins 6 and 7) to the existing. These basins are needed to meet new regulatory requirements for nutrient removal. They include adding anaerobic zones within each basin which reduces the capacity of the basins. Construction of Basin 7 requires construction of a new MBR influent screening facility.
6. ACT treatment train (Plant 2) Rehabilitation: Major work on Plant 2 was not done in the Phase 1 project. The update of the Master Plan includes upgrades to several parts of the plant's facilities including installing new mixers and upgrading several pumping stations. This work would take place within existing facilities.
7. Plant 2 Enhanced Nutrient Removal: This work was not considered in the 2008 Master Plan, or the Phase 1 construction. As noted in Item 5 above, modifications to the aeration basins due to more stringent regulatory requirement for nutrient removal will result in the loss of some capacity of the aeration basins. Due to the lack of space around the basins, the basins cannot be enlarged to regain the capacity. Therefore, some of the required nutrient removal must take place elsewhere on the plant site or in another process. The method recommended in the update of the Master Plan is to convert the tertiary filters to dual purpose denitrifying filters. With the exception of providing storage for chemicals supplying supplemental carbon to the filters, all the new facilities for this process will be within existing processes and structures.
8. CCB: The 2008 Master Plan recommended construction of new CCB 4 adjacent to CCB 3. Instead, CCB 2 was extended and modified in the Phase 1 project to provide the needed increase in disinfection capacity. The update of the Master Plan recommends rehabilitation work on all three existing basins. The rehabilitation would take place within the limits of the basins.

9. Primary Sludge Thickeners: This process was included in the 2008 Master Plan, but was not constructed. It is not included in the update of the Master Plan and is no longer considered beneficial.
10. WAS Thickeners: Adding WAS thickeners was recommended in the 2008 Master Plan, but they were not constructed. Facilities to increase WAS thickening capacity and provide some redundancy are recommended in the update of the Master Plan. The update recommends installing new thickening equipment, such as RDTs, within the existing thickening building. The existing DAFT facilities will be used as back up to the new thickening process.
11. Digestion: The Phase 1 project included construction of two new digesters and modifications to other solids treatment facilities. The digestion system now provides adequate solids treatment capacity. No new digestion capacity is recommended in the update of the Master Plan.
12. AWT Projects: The AWT Projects were not considered in the 2008 Master Plan. More stringent regulatory requirements covering dissolved solids concentrations in the plant effluent may make future desalting of a portion of the plant effluent a necessity. Construction of a desalting facility will require dedicating space on-site.

## 10.6 Anticipated CEQA Requirements for Projects Recommended in the Update of the Master Plan

It is probable that some environmental documentation under CEQA, either new documentation or additions and/or amendments to existing documentation, will be required to cover construction of some of the new facilities and modification to existing facilities recommended in the update of the Master Plan. Figure 10.4 highlights those facilities.

The type of facility to be constructed, when the facility will be constructed in relation to other new facilities, and what was covered in the previously noted PEIR will determine what new documentation will be required. Carollo's expectation of what new documentation will be needed is presented in Table 10.3. It should be noted that judging what the various jurisdictional agencies and the public might require is difficult to predict accurately. Therefore, it is recommended that the City's environmental staff and CEQA consultant review Carollo's conclusions and recommend an environmental strategy to pursue for future projects.

The required CEQA documentation could range from preparing a new EIR, to amending the PEIR, to preparing negative or mitigated negative declarations, to not requiring any new documentation (Exemption). It should be kept in mind that all proposed facilities, except those involving disposal of waste from the future AWT desalting facility, will be constructed on a site already dedicated to wastewater reclamation and that all the projects are beneficial to environmental protection. It is expected that most of the smaller projects, particularly those where construction will take place within existing structures or process areas, could be viewed as maintenance upgrades or upgrades with no significant impacts and not requiring CEQA documentation other than possibly a formal exemption or a negative declaration, particularly if the projects are undertaken as small, individual projects. The majority of the projects are expected to have no significant environmental impacts.

For facilities involving more major expansion of existing facilities or structures, and where there are only minor process modifications, Carollo expects that, again, only a negative declaration would be needed.

Figure 10.4 Locations of the Projects Recommended in the Update of the Master Plan

At some point during the planning period of the update to the Master Plan, it is anticipated that dissolved salts would need to be removed from the effluent stream to meet discharge requirements. As such, construction of an AWT (desalting) facility would be needed. This would be a new process involving new structures, and depending on the method of brine disposal selected, it could involve an offsite brine transmission pipeline or considerable offsite trucking. In this case, a new EIR or amendment of the PEIR probably would be required.

Recent analyses of the water surface elevations in the Santa Ana River during the 100-year flood condition placed the flood elevation above portions of the levee protecting the plant site from the river. (See Volume 4, Chapter 11, Capital Project Studies, and its appendices). To provide adequate protection, portions of the levee were raised to an elevation above the flood elevation as Phase 1 of an overall levee enhancement project. Phase 2 of the project involves strengthening the existing armor on the levee face by grouting the riprap along the river side of the levee from near its top to its base several feet below the existing toe of slope. Because of the endangered and threatened species associated with the river, preparation of a mitigated negative declaration at the very least and probably, due to the potential for challenges by other agencies or the public, an EIR will be required.

Table 10.3 Anticipated CEQA Requirements

| Project No. | Project Name | Anticipated Start of Construction | Anticipated CEQA Requirement(1) | Comments |
|---|---|---|---|---|
| FI-01 | Influent Flow Metering Project | 0-5 | CE | The work involves installing flow meters in existing on-site pipelines. |
| FI-02(2) | Headworks Rehabilitation | 0-5 | CE | The work is primarily replacing existing equipment and appurtenances within an existing process. There are no external impacts. |
| FI-03(2) | Headworks Screening Conveyor Replacement | 0-5 | CE | See FI-02 |
| FI-04(2) | Headworks Bypass | 0-5 | CE | The work involves minor piping and conveyance upgrades. |
| FI-05 | Headworks Grit Classifiers and Pumps | 10-20 | CE | See FI-02 |
| FI-06 | MBR treatment train (Plant 2) Phase II Expansion | 0-5 | Neg Dec | This work involves construction of two new MBR basins and an MBR screening facility |
| FI-07 | MBR treatment train (Plant 2) Enhanced Nutrient Removal | 10-20 | CE or Neg Dec | See FI-02 |

Table 10.3 Anticipated CEQA Requirements (continued)

| Project No. | Project Name | Anticipated Start of Construction | Anticipated CEQA Requirement(1) | Comments |
|---|---|---|---|---|
| FI-08 | ACT treatment train (Plant 2) Rehabilitation | 0-5 | CE | See FI-02 |
| FI-09 | ACT treatment train (Plant 2) Mixers Rehabilitation | 10-20 | CE | See FI-02 |
| FI-10 | ACT treatment train (Plant 2) RAS/WAS Pump Station Rehabilitation | 0-5 | CE | See FI-02 |
| FI-11 | ACT treatment train (Plant 2) RAS/WAS Pumps Replacement | 5-10 | CE | See FI-02 |
| FI-12 | ACT treatment train (Plant 2) Enhanced Nutrient Removal | 10-20 | CE | Documentation will depend on what ENR process is selected for construction. |
| FI-13 | Tertiary Filter Replacement and CCB Retrofits | 0-5 | EC | See FI-02 |
| FI-14 | CCB Rehabilitation | 5-10 | CE | See FI-02 |
| FI-15 | First Primary Sludge Pumping Rehabilitation Project | 5-10 | CE | See FI-02 |
| FI-16 | Second Primary Sludge Pumping Rehabilitation Project | 10-20 | CE | See FI-02 |
| FI-17 | WAS Thickening Project | 10-20 | CE | See FI-02 |
| FI-18 | AWT (Desalting) Project | 10-20 | EIR | This includes construction of a new, complete desalting process. |
| FI-19 | Levee Rehabilitation Phase II | 10-20 | Neg Dec or EIR | This project will affect areas inhabited by threatened species. |

Notes:
(1) Anticipated CEQA Requirement:
CE Exempt – Categorical Exemption (1530ID).
Neg Dec – Prepare a negative declaration or mitigated negative declaration.
EIR – Prepare an EIR or an amendment to the PEIR.
(2) Projects F1-02, F1-03, and F1-04 should be continued in one Categorical Exemption.

## 10.7 Summary

This Chapter compared projects identified in the 2008 Master Plan with facilities that were subsequently built during the MBR treatment train (Plant 1) Phase I Expansion and with projects identified in this Update of the Master Plan. The locations of new, future facilities were identified and opinions were given as to the level of CEQA documentation needed if the planned projects were constructed. Table 10.3 identifies the anticipated CEQA documents required if the improvements are constructed as individual projects over the next 20 years. Other options for coverage under CEQA under long-term project implementations and planning could include preparing an addendum to the 2008 Master Plan Final PEIR and evaluating the probable minor changes of implementing the projects identified in this update.

When an AWT facility (Desalting facility) is needed in the future, an addendum to the current PEIR would not be applicable due to the impacts outside of the RWQCP for the new concentrate disposal pipelines. An option would be to prepare a new PEIR to evaluate the projects identified in this update, including the AWT facility. Preparing a new PEIR would give an opportunity to update the analyses to be consistent with current regulations (air quality/greenhouse emissions, etc.) and updated CEQA thresholds that have changed since the preparation of the last EIR.

It is recommended that the opinions expressed in this chapter be reviewed by the City's Environmental Permitting Department and the City's CEQA consultant to set an overall strategy for CEQA compliance.

City of Riverside Public Works Department

UPDATE OF THE INTEGRATED

# MASTER PLAN FOR THE WASTEWATER COLLECTION AND TREATMENT FACILITIES

VOL. 4

CHAPTER 11

## Wastewater Treatment System

## Capital Project Studies

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Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 11: CAPITAL PROJECT STUDIES

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Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

# CHAPTER 11: CAPITAL PROJECT STUDIES

Carollo Project No. 10495A00

Digitally signed by Graham J.G. Juby
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.17 16:32:31-07'00'

REGISTERED PROFESSIONAL ENGINEER
GRAHAM J. G. JUBY
No. 67217
CIVIL
STATE OF CALIFORNIA

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Chapter 11

# CAPITAL PROJECT STUDIES

## 11.1 Purpose

The purpose of this chapter is to discuss the miscellaneous plant facilities, structures, or projects recommended by this update of the Master Plan. This chapter will serve as an overview for the following studies. More comprehensive reports are attached as appendices.

1. Digester 5 Evaluation.
2. Biomethane Utilization Project.
3. Levee Phase 2 Project.

## 11.2 Digester 5 Evaluation

Recent and forecasted regulatory changes in California have required the diversion of organic waste from landfills. In an effort to comply with these regulations, the City contracted Carollo to evaluate the structural integrity of Digester No. 5 for future use as a food waste digester. Findings and recommendations for digester upgrades and repairs are discussed in detail in a report submitted to the City separately and included in Appendix 11A for reference. A summary of the recommendations are presented in the following:

1. Digester No. 5 is generally in good condition and could be rehabilitated to be converted to a standalone food waste digester.
2. Rehabilitation of Digester No. 5 will involve both structural and mechanical work. Structural work includes both internal coating of the digester as well as wall strengthening to bring the structure up to seismic code. Rehabilitation work is recommended to include:
   a. Epoxy or chemical grout injection to seal approximately 200 linear ft of concrete cracks in the digester wall.
   b. Blasting and recoating the interior steel dome surfaces inside of the digester. Exterior rehabilitation of the steel may be limited to localized recoating and resealing of damaged seals at the dome perimeter.
   c. Repair of steel members that have experienced section loss with lap-welded plates as required. It is estimated that the occurrence of these repairs would be infrequent.
   d. Application of a surface-applied waterproofing mortar, such as Xypex Concentrate, that is treated with an anti-microbial agent to provide both concrete protection and waterproofing for the digester wall. This should be applied to the existing wall surface above the new concrete wall addition that is required to bring the digester up to code, and allow it to operate at a maximum level of 29 ft.
   e. Provide new aluminum guardrail and kick-plate at the top stair landing.
   f. Mechanical work includes a new standby pump mixer, some piping modifications, and additional instrumentation.
3. A maximum level of 29 ft could be considered with additional concrete reinforcement and this would result in 858,750 gal of operating volume, which is sufficient to treat the

bioslurry and some ADM max 30 percent of total food waste at an organic loading rate of 0.2 lbs VSS/cu ft day.

4. Digester No. 5 should be able to treat approximately 100 tons per day of food waste. Initial estimates indicated that approximately 252,000 cfd of additional biogas could be generated from food waste digestion in Digester No. 5. Understanding that different food waste sources generate different amounts of biogas, it is recommended that laboratory testing of various food waste sources be carried out to confirm the potential gas production from the mixture of food waste expected at the RWQCP.
5. It appears that the existing gas flaring capacity will be sufficient to handle the additional biogas anticipated from Digester No. 5, once the mechanical issues associated with the existing flaring system are addressed.
6. An important assumption in the study was that all food waste would be pre-processed off-site by the contracted hauling companies to a specification provided by the City. Therefore, no further pre-processing, such as on-site screening, would be provided upstream of Digester No. 5. A food waste bioslurry receiving station design was proposed upstream of Digester No. 5. This system is expected to have a project cost of approximately $1.3 million.
7. The estimated cost to rehabilitate Digester No. 5 is approximately $1.9 million.

## 11.3 Biomethane Utilization Project

In March 2019, the City contracted Carollo to evaluate historic and projected biogas production in an effort to maximize biogas utilization. This project will:

- Evaluate 3 years of historical data as a basis for estimating biogas production anticipated at RWQCP.
- Evaluate the existing digester complex for improvements to maximize biogas production, including food waste addition and other sources.
- Develop a preliminary report on the injection and marketing of the biogas as RNG in collaboration with SoCal Gas.
- Create a spreadsheet model of the solids/gas system to provide a solids balance and gas balance.
- Evaluate the current and necessary capacity of the flare as it pertains to the increased biogas generation from food waste bioslurry.
- Produce a financial analysis model tool for the City's use to evaluate the feasibility and costs of the biomethane utilization project in the future. This tool will include capital costs, O&M costs, and potential revenue streams.

The Biomethane Utilization Project was still underway at the point at which this update to the Master Plan was finalized. Therefore, no costs, summaries, or recommendations resulting from this project are included in this update of the Master Plan.

## 11.4 Levee Phase 2 Project

The RWQCP property line along the north edge of the plant is bounded by a roughly 4,800 ft levee built to protect the plant from the Santa Ana River. In 2016, Tetra Tech submitted a 100 percent design report describing the rehabilitations that would be required to protect the RWQCP from a 100-year storm event. A copy of this 100 percent design report is included in Appendix 11B. As a result of the 100 percent, the levee rehabilitation work was divided into two phases. Phase 1

improvements consist of floodwall protection improvements to protect from the 100-year flood water surface elevation, and these improvements have been completed. Phase 2 improvements are to remedy the scour protection deficiency along the levee for the 100-year flood event. The Phase 2 improvements are to be designed and completed as a future project due to environmental permitting and funding needs.

Tetra Tech's initial study evaluated alternative approaches to the levee rehabilitation project. The alternatives focused on construction of a floodwall along the top of the levee to provide the hydraulic protection needed. Two floodwall sections (for various heights) were to be placed along the south side of the SART, so that the public can view the river, or along the north side so that the public will view the RWQCP. Tetra Tech then expanded the review to include a partial levee raise of 6 feet with a smaller floodwall, up to 4 feet, placed along the south side of the SART. The raised levee with the floodwall on the plant side was selected as the design basis to move forward with the project. Figure 11.1 shows a map extracted from Tetra Tech's 100 percent design report showing the existing location of the levee.

Figure 11.1 RWQCP Existing Levee

### 11.4.1 Project Description

This section describes both phases of the levee rehabilitation project.

#### 11.4.1.1 Phase 1

Phase 1 of the levee rehabilitation project is complete and it raised the existing levee by 6 feet and installed a floodwall on top of the plant side of the newly raised levee. The floodwall height varies from 0 to 4 feet high with the top of footing flush with the top of new levee elevations. The new slopes required slope paving to be placed as slope protection. A bike trail and chain link fence were

also installed. Phase 1 included installation of a 5-foot-high retaining wall in two sections within the plant area to increase access road widths and offsets from proposed buildings as part of the plant expansion.

#### 11.4.1.2 Phase 2

Phase 2 construction work will consist of grouting the existing rip rap embankment from approximately station 27+00 to upstream of the project limits. This will require the grouting of the toe scour protection below the channel invert. Additional details are included in Appendix 11B.

### 11.4.2 Potential Permitting Requirements

As discussed in Appendix 11B, Tetra Tech identified two avenues for potential permit requirements. Assuming project construction can avoid the least Bell's vireo breeding season (March 1 – September 15), the project is not expected to require consultation or permits from the USFWS or CDFW. If the vireo breeding season cannot be avoided, implementation of noise attenuation measures (e.g., sound walls, blankets, bafflers, etc.) and consultation and permitting with the USFWS and CDFW could be required pursuant to the ESA and CESA. Similarly, as currently designed and assuming project construction would avoid jurisdictional wetland resources, the project is not expected to require permits from the USACE, RWQCB, or CDFW.

If the design changes or temporary encroachment into the Santa Ana River side of the existing levee is necessary during construction, then agency consultation and permitting would be required. Encroachment would require notification and permitting with the USFWS, USACE, RWQCB, and/or CDFW, depending on the extent of encroachment. If encroachment is limited to the existing riprap streambank above the toe-of-slope, then notification and permitting would only be required with CDFW. Under this scenario, a standard 5-year Streambed Alteration Agreement would be issued by CDFW for the activities. If encroachment is required down at the toe-of-slope and north into the active floodplain, then notification and permitting would be required with USACE, RWQCB, and CDFW. Under this scenario, a CWA Section 404 permit [e.g., Nationwide Permit 3 (Maintenance), 13 (Bank Stabilization), or 31 (Maintenance of Existing Flood Control Facilities)], CWA Section 401 Water Quality Certification, and standard 5-year Streambed Alteration Agreement would be issued by the USACE, RWQCB, and CDFW, respectively.

Further, if the encroachment would impact perennial stream or riparian habitat, impacts to the Santa Ana River and least Bell's vireo could occur, including designated critical habitat. Under this scenario, potential effects on sucker and vireo (and critical habitat) would require formal consultation between the USACE and USFWS pursuant to Section 7 of the ESA. The USFWS would issue a Biological Opinion and Incidental Take Statement for the project that would become attached to the USACE's CWA Section 404 permit. A consistency analysis would then likely be issued for impacts to vireo by CDFW pursuant to Section 2080.1 of the CESA.

Figure 11.2 summarizes Tetra Tech's interpretation of the potential permit impacts. As shown in Figure 11.2, an EIR may be required depending on the scope and schedule of the Levee Phase II project. Additionally, the permit requirements could vary from not needing a permit to needing several through a variety of agencies. Additional discussion on the potential permit requirements for this project can be found in Volume 4, Chapter 10, Environmental Review. For this reason, it is highly recommended that the City have this update to the Master Plan reviewed by the City's environmental permitting consultant of choice early in the planning period for the Levee Phase II project.

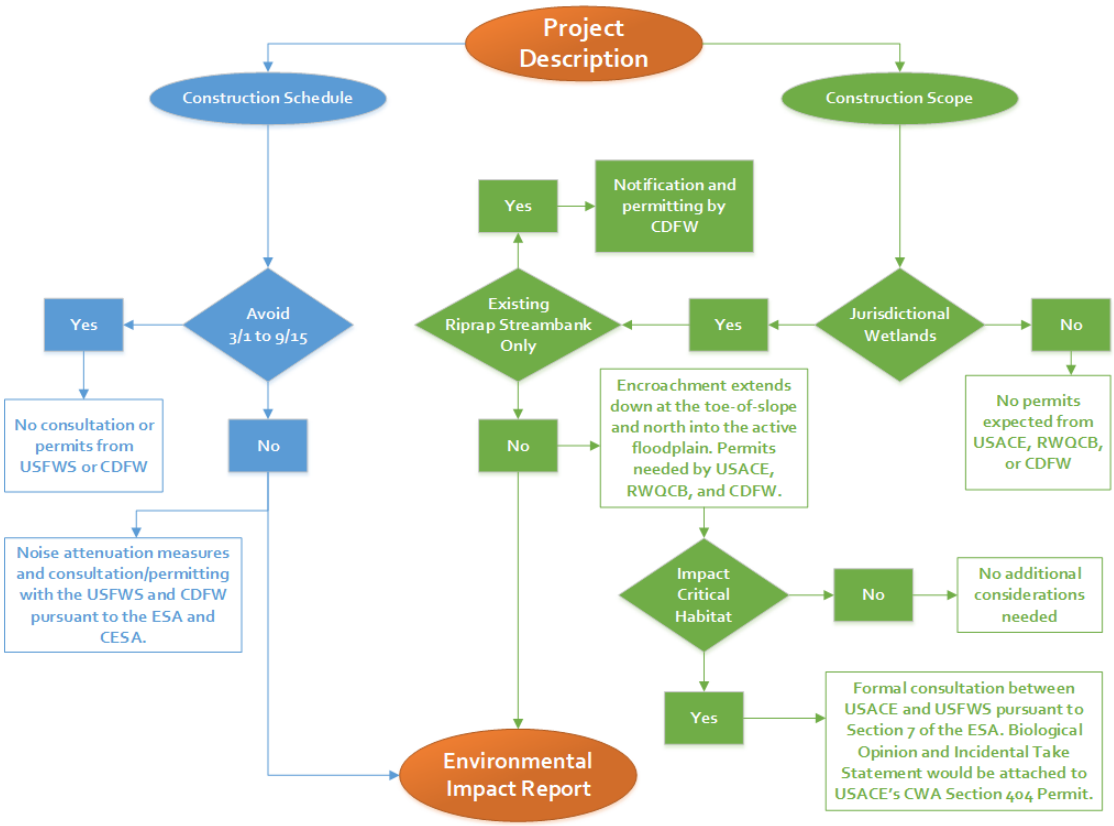


Figure 11.2 Levee Phase II Permitting Flowchart

### 11.4.3 Capital Costs

The cost estimate presented in this section was developed in accordance with the design information referenced in this document. Reference Volume 2 Chapter 4, Basis of Cost Estimates, for any information pertaining to the approach used to estimate the project cost. The detailed cost estimates and quantity take-offs are presented in Appendix 11C. For planning purposes, a $1 million dollar placeholder was included for potential permitting costs. The project cost was estimated at $9.1 million dollars and includes a $250,000 preliminary design study specific to the Levee Phase 2 Rehabilitation scope of work.

Appendix 11A

# DIGESTER NO. 5 EVALUATION REPORT

City of Riverside Public Works Department

Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

# VOLUME 4: WASTEWATER TREATMENT SYSTEMS

## CHAPTER 11: CAPITAL PROJECT STUDIES

# APPENDIX 11A: DIGESTER NO. 5 – FOOD WASTE EVALUATION

FINAL | June 2019

City of Riverside Public Works Department
Update of the Integrated Master Plan for the Wastewater Collection and Treatment Facilities

VOLUME 4: WASTEWATER TREATMENT SYSTEMS

CHAPTER 11: CAPITAL PROJECT STUDIES

# APPENDIX 11A: DIGESTER NO. 5 – FOOD WASTE EVALUATION

Carollo Project No. 10495A00

Digitally signed by Christian Tasser
Contact Info: Carollo Engineers, Inc.
Date: 2019.07.16 14:21:38-07'00'

REGISTERED PROFESSIONAL ENGINEER
CHRISTIAN TASSER
No. 77156
CIVIL
STATE OF CALIFORNIA

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Section 1

# PROJECT OBJECTIVE

This project is part of the City's effort to comply with California's organic waste diversion regulations. The purpose of this report is to evaluate the structural integrity of the Digester No. 5 for future use as a food waste digester. Findings and recommendations for digester upgrades, repairs and additional controls for digester feeding and operation are included in the evaluation. A preliminary layout for a food waste receiving station is provided. The detailed layout and specific equipment type and sizing will be determined during the design.

Section 2

# BACKGROUND

The City of Riverside Public Works Department operates a comprehensive wastewater treatment and disposal system that serves most of the City, as well as the CSDs of Jurupa, Rubidoux, and Edgemont, and the community of Highgrove. Treatment of the wastewater occurs at the 28 mgd RWQCP, where influent flows undergo preliminary, primary, secondary, and tertiary treatment, followed by disinfection using sodium hypochlorite, and dechlorination using sodium bisulfite. A limited volume of the final effluent is reclaimed for non-potable reuse and the remainder is discharged to Reach 3 of the Santa Ana River. The RWQCP is currently treating about 28 mgd.

The RWQCP completed a major expansion in 2017. The expansion was to include the demolition of Digester No. 5 (formerly Digester No. 3). However, the City decided to halt the demolition and instead study the feasibility of using this digester for anaerobic digestion of food waste substrates, also known as Anaerobic Digestible Materials. The findings of that evaluation are the focus of this report.

The following areas were included in the evaluation of Digester No. 5:

- Its structural integrity, and the estimated cost of any needed repairs or upgrades to make it suitable for operation as a food waste digester.
- Determination of whether additional digester mixing equipment is needed, and or process controls and instrumentation.
- Determination of a potential location for a food waste receiving station.
- The impact of additional digested solids production on the solids dewatering system.
- The impact of additional biogas production on the existing gas fare system.

An independent study by Global Green undertaken for the City in October 2018 has determined that approximately 100 tons per day of food waste will need to be re-directed from City landfills to the RWQCP for treatment. This information was used as a basis for determining the required

capacity of the food waste receiving station, as well as the potential impacts on the solids dewatering equipment and the potential biogas production.

Section 3

# EXISTING FACILITIES

## 3.1 FOG Receiving and Rendering (Co-Digestion) Facility

As part of the recent expansion project, a new FOG receiving station was constructed. It is equipped with pumps and a basket strainer, and the details are included here for completeness. This system would introduce FOG to the plant biosolids (thickened primary solids and WAS) as a way to enhance biogas production. This system is totally independent to the proposed food waste processing in Digester No. 5.

The existing FOG Receiving and Rendering Station (FOG facility) is capable of receiving, processing and transferring 60,000 gal of grease per day to the Sludge Blending Facility. The FOG facility is designed to grind and remove larger solids. The FOG is discharged to below-grade hoppers for subsequent pumping to the SBTs. The co-digestion facility includes a FOG (septage) receiving station, a pumped recirculation mixing system, heat exchangers, and a FOG transfer pump.

### 3.1.1 FOG Receiving Station

The receiving station is a packaged system that includes camlock connections for the trucks that deliver the FOG to the RWQCP, rock traps, in-line solids grinders, receiving tanks, screening and washing systems, inclined augers, and compactors. It has two bays so FOG can be received from two trucks at the same time. Table 1 summarizes the design criteria for the FOG receiving station.

Table 1 FOG Receiving Station Design Criteria[1]

| Description | Value |
|---|---|
| **FOG Receiving System** | |
| Overall System Capacity | 60,000 gpd |
| **FOG Receiving Bays** | |
| Number | 2 |
| Number of FOG Deliveries[2] | 8 for 5,500 gal Trucks<br>10 for 3,500 Trucks |
| Time to Empty Truck | 60 min for 5,500 Trucks<br>53 for 3,500 gal Trucks |
| Quick Coupling Diameter | 4 inches |
| Number of Rock Traps | 2 |

Table 1 FOG Receiving Station Design Criteria[1] (continued)

| Description | Value |
|---|---|
| **Inline Solids Grinder** | |
| Number | 2 |
| Line Size | 4 inch |
| Motor Horsepower | 5 hp |
| **FOG Screens** | |
| Number | 2 |
| Screening Capacity[3], each | 50 gpm |
| Auger Motor Horsepower | 2 hp |
| Size of the screen openings | 1/4 inch |
| Discharge Port Size | 12 inch |
| **Lateral Rock Transfer Conveyors** | |
| Number | 2 |
| Motor Horsepower | 2 hp |
| **Inclined Main Rock Screw Conveyor** | |
| Number | 1 |
| Motor Horsepower | 2 hp |

Notes:
(1) Source: RWQCP Phase 1 Expansion Design Criteria.
(2) Per 8-hour day.
(3) Based on clean water.

### 3.1.2 FOG Receiving Hoppers

Each bay of the FOG receiving station discharges screened FOG to below-grade hoppers. The hoppers have a combined capacity to store a half-day worth of FOG, about 30,000 gal. Each hopper includes a pumped recirculation system and heat exchanger to maintain the FOG in a homogenous and flowable state. Table 2 summarizes the design criteria for the FOG hoppers.

Table 2 FOG Hoppers Design Criteria[1]

| Description | Value |
|---|---|
| Number | 2 |
| Capacity, each | 15,000 gal |
| Length | 17 ft |
| Width | 10 ft |
| Side Water Depth | 8 ft |

Notes:
(1) Source: RWQCP Phase 1 Expansion Design Criteria.

### 3.1.3 FOG Pumped Recirculation System

The pumped recirculation system for the FOG hoppers is designed to turn over the hopper contents in one hour. Table 3 summarizes the design criteria for the FOG pumped recirculation system mixing pumps.

Table 3 Existing FOG Pumped Recirculation System Mixing Pump(1)

| Description | Value |
|---|---|
| Number | 2 (1 per Heat Exchanger) |
| Type | Constant Speed, Chopper |
| Capacity, each | 125 gpm |
| Total Dynamic Head | 77 ft |
| Motor Horsepower | 15 hp |

Notes:
(1) Source: RWQCP Phase 1 Expansion Design Criteria.

### 3.1.4 Heat Exchangers

The FOG is heated to maintain flow and facilitate pumping. Each FOG hopper includes a heat exchanger to maintain the temperature at 100 degrees F to 110 degrees F. Table 4 summarizes the design criteria for the FOG heat exchangers.

Table 4 Existing FOG Heat Exchanger(1)

| Description | Value |
|---|---|
| **Heat Exchangers** | |
| Number | 2 Duty (1 per hopper) |
| Type | Spiral Plate Counter-Current lows |
| Design Temperature Rise | 30 degrees F |
| Heat Exchange, Total | 1.375 MMBtu/hr |
| Volumetric Flow Rate – Hot Water | 150 gpm |
| Volumetric Flow Rate – FOG | 125 gpm |
| **Hot Water Secondary Loop Pump** | |
| Number | 2 Duty (1 per Heat Exchanger) |
| Type | Constant Speed, Centrifugal |
| Capacity | 150 gpm |
| Total Discharge Head | 45 ft |
| Motor Horsepower | 5 hp |

Notes:
(1) Source: RWQCP Phase 1 Expansion Design Criteria.

### 3.1.5 FOG Transfer Pumps

The FOG transfer pumps feed FOG from the hopper to the SBTs, where it is combined with incoming thickened primary and WAS. Table 5 summarizes the design criteria for the FOG transfer pumps.

Table 5 FOG Transfer Pump Design Criteria(1)

| Description | Value |
|---|---|
| Number | 2 (1 per hopper) |
| Type | Constant Speed, Progressive Cavity |
| Capacity, each | 167 gpm |
| Discharge Pressure | 33 psi |
| Motor Horsepower | 30 hp |

Notes:
Abbreviations: psi – pounds per square inch.
(1) Source: RWQCP Phase 1 Expansion Design Criteria.

## 3.2 Anaerobic Digestion System

In the digestion system, the blended solids from primary treatment, the dissolved air flotation thickeners, and FOG receiving station are processed in the absence of air. This reduces the solids volume, stabilizes the sludge, and produces methane gas that can be used in the existing fuel cells for the generation of electrical power and heat, or in boilers for digester heating. The biosolids and FOG digestion system consists of anaerobic Digester Nos. 1, 2, 3 and 4 (Digesters 3 and 4 are new), linear motion mixers, standby pumped recirculation systems, digester heating recirculation pumps, heat exchangers, foam spray pumps, and sludge transfer pumps.

### 3.2.1 Anaerobic Digester No. 5

Digester No. 5 (formerly Digester No. 3) is a spare digester that is not needed for biosolids digestion. It was constructed in 1982 and as mentioned it was originally to be demolished as part of the recent expansion project. It is a 75-ft diameter digester with a capacity of 1.1 MG. Table 6 summarizes the design criteria for the anaerobic digester. The City is evaluating whether Digester No. 5 can be used as a standalone food waste digester. This would require a new food waste receiving station, which would be piped to feed substrate directly to Digester No. 5.

Table 6 Anaerobic Digester No. 5(1)

| Description | Value |
|---|---|
| Digester No. 5 (Standby) | |
| Number | 1 |
| Cover Type | Gas Cover Dome |
| Diameter, each | 75 ft |
| Side Water Depth (Total) | 32 ft(2) |
| Bottom Cone Slope, Horizontal to Vertical | 4 |
| Bottom Cone Depth | 11.25 ft |
| Cover Dome Height | 10 ft |
| Total Volume | 1.150 MG |

Notes:
(1) Source: RWQCP Phase 1 Expansion Design Criteria.
(2) See comments later following seismic evaluation.

### 3.2.2 Digester No. 5 Mixing System

The goal of digester mixing systems is to provide high turnover of the digester contents, to minimize deposition of grit, and to break up scum. Effective digester mixing is widely acknowledged to be a critical factor in achieving good anaerobic digestion, both in terms of volatiles reduction and process stability. Linear motion mixers were provided in the new Digesters Nos. 3 and 4 and retrofitted to digesters Nos. 1 and 2. The linear motion mixers utilize oscillating ring-shaped impellers to provide near isotropic mixing while minimizing turbulence intensity and vorticity associated with rotary type mechanical mixers. Digester No. 5 is mixed using a pumped recirculation system using one chopper pump as shown on Figure 1. The information on this system is included in Table 7.

Table 7 Digester No. 5 Mixing System Design Criteria

| Description | Value |
|---|---|
| **Digester No. 5 Mixing System** | |
| Number of Pumps | 1 |
| Type | Constant Speed, Chopper |
| Capacity, each | 6,670 gpm |
| Total Dynamic Head | 41 ft |
| Pump Speed | 750 rpm |
| Motor Horsepower | 50 hp |
| Digester Tank Turnover Time | 2.6 hr |
| Maximum solids content for pump mixing | 15 percent |

Figure 1 Existing Hydraulic Mixing System for Digester No. 5

Section 4

# NEW FOOD WASTE RECEIVING STATION

If Digester No. 5 is converted to a standalone food waste digester then a new food waste receiving station will be required. In this section the Food Waste receiving station options and overall layout are analyzed and presented. The operating principle is discussed along with the advantages and disadvantage of the corresponding screening technology. In addition, the design criteria necessary to meet the need to accept 15-20 truckloads per day is presented, together with a potential site location and capital cost estimate for each technology. Detailed cost estimates are located in Attachment C.

Food Waste slurry will be processed off-site in hammer mills or grinders which are equipped with screens of 5-10 mm. In order to remove the remaining contamination such as plastics, metal and glass, screening of the waste material during unloading of the trucks is recommended. The typical screen size for the receiving station could be up to 5 mm, but preferably 2 mm. The City decided to have a third party provide the pre-screening services off-site so that they are required to deliver the AMD and bioslurry material based on specific quality requirements.

The following sections describe the conceptual layout for the receiving tanks and equipment.

## 4.1 Receiving Facility Equipment and Storage Tanks

The necessary equipment for this facility includes slurry receiving tanks, tank mixing pumps, slurry metering pumps, and odor control. With an anticipated slurry flow rate of 40,000 gpd, it is recommended to install three 30,000 gal receiving tanks, equating to 2 days of storage above ground and additional equalization is available when using the SBTs. This will provide flexibility when offloading the trucks and dosing the digesters over weekends. Glass lined ductile iron and plug valves are recommended for slurry piping and valving.

The receiving tanks will be installed above grade, assuming a slurry total solids concentration of 10-15 percent. A complete equipment list with recommended manufacturers, sizing, and power requirements is summarized in Table 8.

Table 8 Slurry Receiving Facility Equipment List

| Equipment | Equipment/ Material Type | Recommended Manufacturer(s) | Size | Quantity | Power, each, hp |
|---|---|---|---|---|---|
| Rock Trap/Grinder | Muffin Monster | JWC Environmental | 275 gpm | 3 | 3 |
| Equalization Tank(1)(2) | FRP | Xerxes, Augusta, Enduro | 30,000 gal | 3 | - |
| Tank Mixing Pump | Chopper Pump | Flygt(3), Vaughan(4) | 200 gpm | 3 | 15-45 |
| Slurry Metering Pump | Rotary Lobe | Böerger(5) | 30 gpm | 3 | 10(6) |

Table 8 Slurry Receiving Facility Equipment List (continued)

| Equipment | Equipment/ Material Type | Recommended Manufacturer(s) | Size | Quantity | Power, each, hp |
|---|---|---|---|---|---|
| Heat Exchanger | Tube-in-tube | Walker Process Equipment | Sized to maintain temperature of 80 degrees. F | 3 | - |
| Odor Control | Carbon Canister | Evoqua, Environmental Solutions, Calgon Carbon | 50 cfm | 2 | - |

Notes:
(1) Sized to provide 2 days of storage at design throughput.
(2) If above ground, tanks should be insulated.
(3) For below grade tank mixing.
(4) For above grade tank mixing.
(5) Standardizing on Böerger rotary lobe pumps for similar applications.
(6) VFDs are recommended for the slurry metering pumps to provide consistent slurry feed to the digester.

Due to the significant odors anticipated from the food waste slurry, odor control is vital to treat the tank headspace. For this application, a carbon canister with coconut shell-based material was shown to adequately treat the odors to an $H_2S$ concentration of less than 1 ppm.

Biofilters are located near the Dewatering Building and Headworks Building to treat existing odors produced onsite. It is likely these biofilters do not have the capacity to treat the food waste odors without pretreatment. A carbon canister, followed by one of the existing biofilters as a polishing step is recommended for this application. Carbon canisters and biofilters are used at food waste processing facilities around the country including Los Angeles Sanitation District, Johnson County's Middle Basin Plant, the City of Gresham Wastewater Treatment Plant, IEUA, and East Bay Municipal Utility District.

A process flow diagram of the slurry receiving facility is shown on Figure 2.

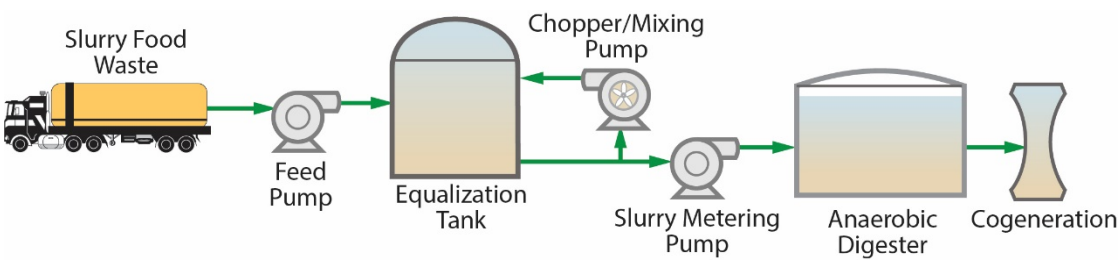



Figure 2 Food Waste Slurry Receiving Facility

## 4.2 Conceptual Site Layout

A conceptual layout of the food waste slurry receiving facility is presented on Figure 3. The layout includes rock trap/grinders to remove impurities in the slurry prior to the two receiving tanks. There are two mixing pumps used to mix each tank and two transfer pumps to dose the slurry to the digesters. Two heat exchangers will provide heat and the carbon canister will service the headspace from both tanks. The pumps and heat exchangers are recommended to be located indoors to serve as protection from the elements and for ease of operations and maintenance. The rock trap/grinders, tanks, and carbon canister can be located outdoors.

The new facility should be provided with heating, ventilation, and air conditioning systems meeting 2016 NFPA 820 standards for wastewater facilities and 2015 Mechanical Code requirements. While the slurry receiving facility is not intended to convey sewage, the food waste slurry will generate off gases. Per NFPA 820, the receiving tanks will be considered classified which requires a three ft distance between the tanks and any non-classified equipment. The building is unclassified unless it is placed directly above a below grade storage tank.

### 4.2.1 Site Location and Tank Sizing

The footprint of a new standalone facility was determined based on the need to received and hold 100 tons per day of bioslurry. The bioslurry will be pre-processed at a Material Recovery Facility, where the food waste solids will be processed, decontaminated, and diluted to 10 percent solids concentration to generate a pumpable bioslurry. The trucked volume will be around 40,000 gpd depending on water content. The facility will need to be 52 ft by 35 ft in order to accommodate three 30,000 gal bioslurry storage tanks, the mixing pumps, the screening system, and utilities. Figure 3 shows the dimensions, potential location south of the Digester No. 5 and east of the FOG receiving station.

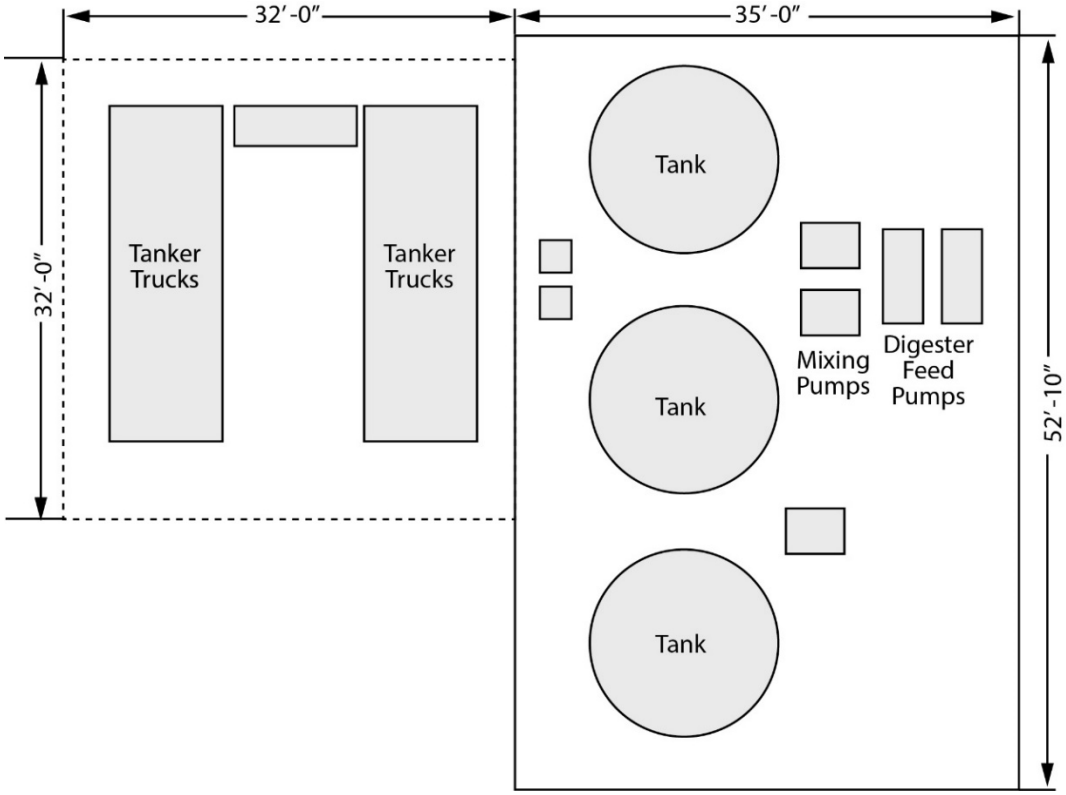



Figure 3 Food Waste Receiving Facility Conceptual Site Layout and Rendering

Several locations for the food waste slurry receiving facility were evaluated, including a modification to the existing FOG station. The first location using the existing FOG receiving station is shown on Figure 4 and is located south of the Digester No. 5. This option would utilize two existing parallel receiving bays which have 1/4-inch screening systems and two 15,000 gallon hoppers. This option would require the addition of two 30,000 gallon holding tanks while using one of the existing below ground sludge blending tanks for additional storage and to feed the

sludge digesters. An additional above ground pipeline would be added to feed the food waste to Digester No. 5.

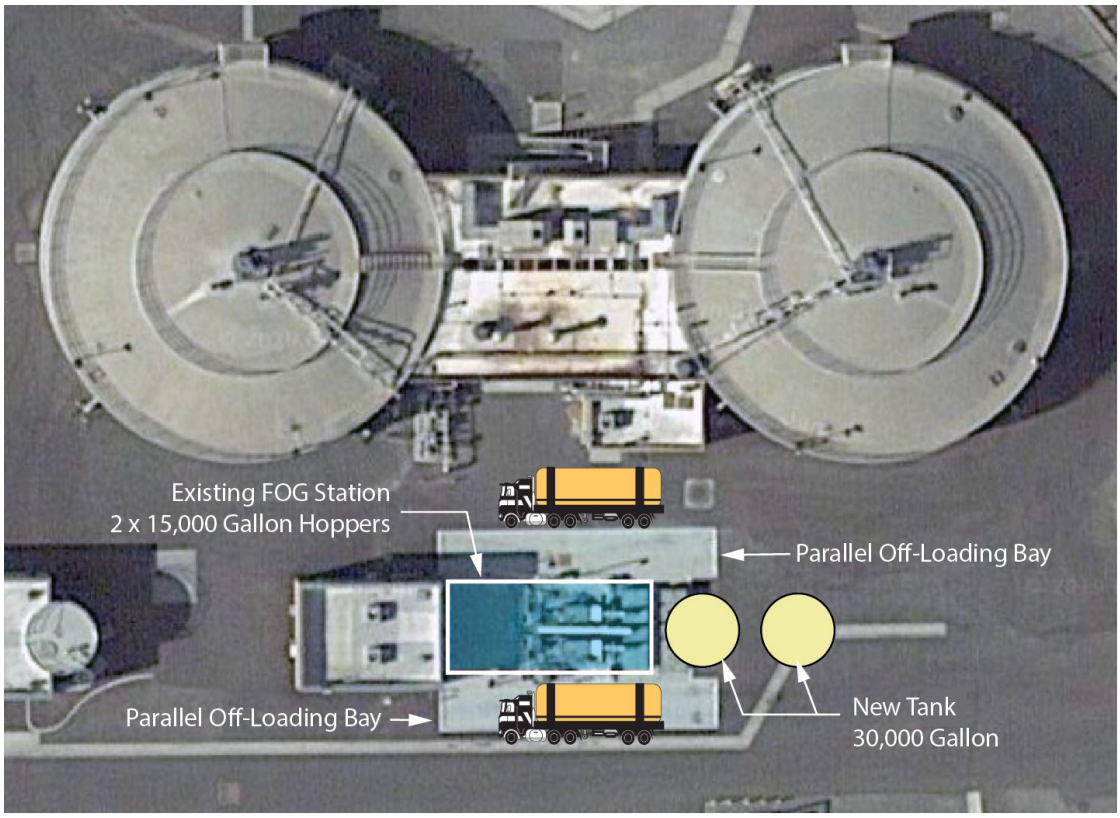



Figure 4 Food Waste Receiving Using the Modified FOG Station Option No. 1

Another option that was considered, would be to retrofit the old cogeneration building, which provides an enclosed area of 79 ft by 67 ft, which would be beneficial for odor containment. Figure 5 and Figure 6 show this location and layout of four receiving tanks and pumping equipment including an odor control system. These two potential layout alternatives can be evaluated in more detail, along with other options, during the preliminary design phase of the project.

Figure 5 Future Food Waste Receiving Station Option No. 2

Table 9 presents a summary of the preliminary design criteria for a new food waste receiving station including storage tanks and bioslurry pumping, similar to that depicted on Figure 6. The preliminary capital cost estimate for such a system is $1.3 M and a detailed cost breakdown is provided in Attachment C.

Table 9 Design Criteria for Receiving Station and Screening and Estimated Costs

| Element | Receiving Station Tanks, and Mixing Pumps | Bioslurry Screening |
|---|---|---|
| Footprint (sq ft) | 3,000 | 100 |
| Operational Reliability (-) | High | High |
| Solids Concentration (%) | 10-15 | 10-15 |
| Horsepower (hp) | 20 | 10 |
| Maintenance Requirement (-) | Low | Low |
| Operator Attention Requirements (-) | Low | Low |
| Equipment Cost Estimate ($) | $521,800 | - |
| Total Equipment Costs ($) | | $521,800 |
| Total Estimated Project Costs ($) | | $1,324,623 |

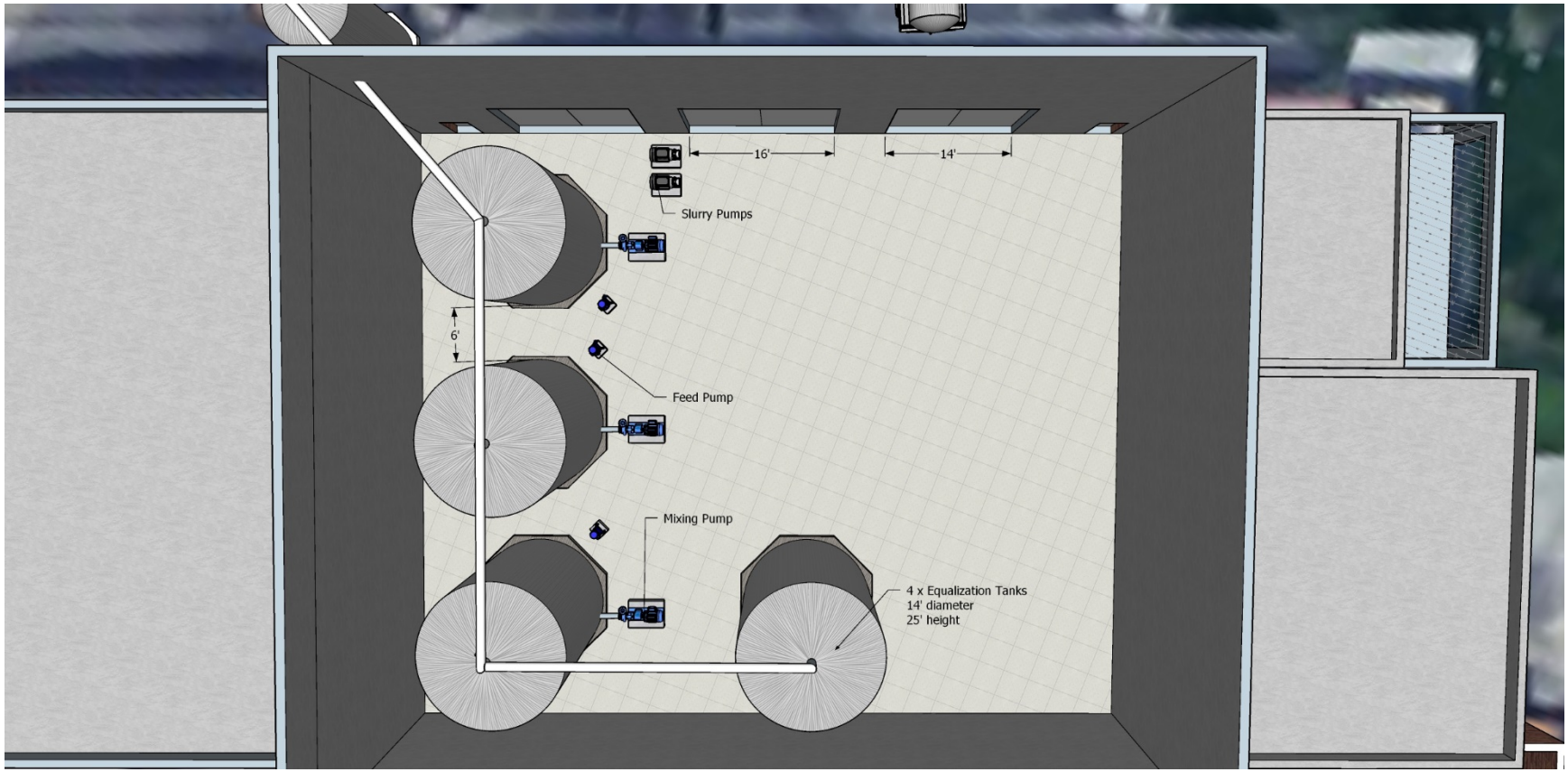



Figure 6 Rendering for Future Food Waste Receiving Station Option 2

Section 5

# ASSESSMENT OF DIGESTER NO. 5

This section discusses the structural and process/mechanical assessment of Digester No. 5 and its potential to be used as a food waste digester, together with the costs associated with any upgrades/modifications/refurbishment needed.

## 5.1 Structural Assessments

### 5.1.1 External Visual Assessment of Digester No. 5

This section summarizes the findings of a site visit to Digester No. 5 and external inspection made on Thursday, March 8, 2018. The site visit began around 9:00 a.m. and ended at 11:00 a.m. Pacific Standard Time. The air temperature was about 65 degrees F and the air was calm. The exterior of the digester is shown on Figure 7 (side view) and Figure 8 (top view).

The digester was filled with about 32 ft of water and the mixers were turned on. The air space below the dome was isolated and the gas pressure monitor varied from about 6.9 to 7.0 inches of water column and was relatively steady throughout the inspection.

In general, the exterior of the concrete walls appeared to be in fair condition with several active weeping leaks. The concrete was discolored throughout and evidence of previous crack repairs (see Figure 9) was observed, especially at the exterior wall ledge where loose surface mortar repair material was prevalent. The length of the leaking cracks was estimated and noted for future use (see Figure 10) in preparing rehabilitation drawings to form the basis for concrete crack injection of the wall. In total the length of leaking cracks was estimated to be about 90 ft. When repairing these cracks, the length of repair would be expected to double as sealing efforts often result in moving the leak to other cracks or away from the sealed length in repaired cracks.

Figure 7 Exterior Concrete Condition of Digester No. 5

Figure 8 Roof View of Digester No. 5

### 5.1.2 Gas Leak Detection of Digester No. 5

At the same site visit for the exterior visual assessment of Digester No. 5, the existing steel dome was also observed from the exterior and tested for gas leakage. The inspection began at the perimeter of the dome just south of the stair landing and continued around the dome in a counter-clockwise pattern, spiraling towards the center. A soapy water solution was applied with a plastic spray bottle to the surface of the dome at areas with surface defects, rust stains, discoloration, and at the perimeter that is covered with sealant. No major gas leaks were detected using this method. A minute gas leak was detected with a short-lived, yet consistent, gas bubble formation for about 30 seconds at the east side of the dome at a small corroded spot about 8 ft in from the perimeter. The perimeter edge of the dome (interface between the concrete wall and the dome edge) was coated with a foam sealant material. Some cracks, tears, and a patch were observed at a few locations (Figure 9), but no gas leakage was detected with the soapy water solution test.

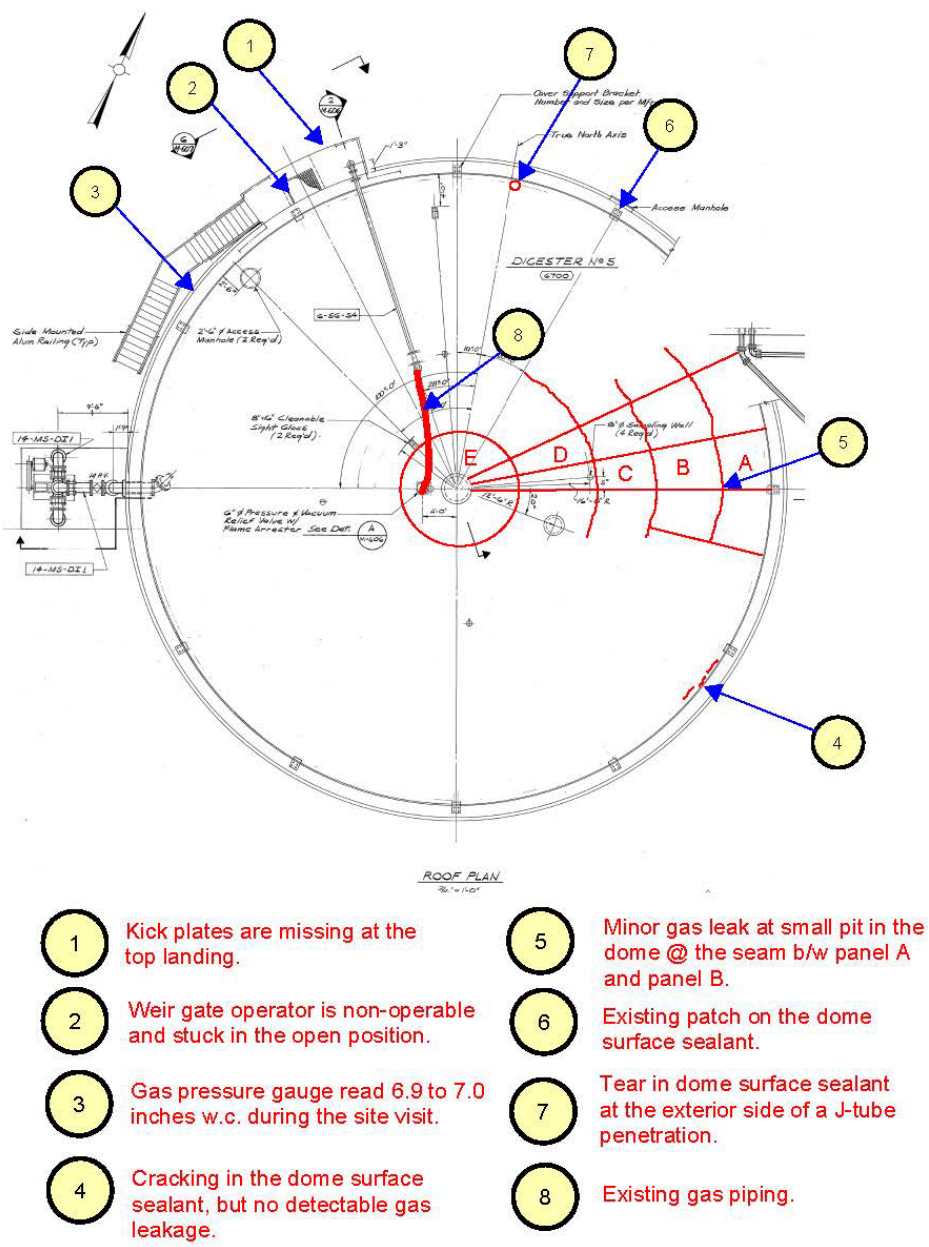
Figure 9 Concrete Crack Repairs at Digester No. 5

The external surface of the steel dome appeared to be in good condition with localized minor surface corrosion with associated coating failure along the circumferential and radial panel joints (Figure 10 and Figure 11). Most of the observed steel corrosion was limited to the outer three panels. The dome had numerous penetrations of various sizes, but these appeared to be flanged and gasketed. A few locations, where rusting of the flange bolts was observed, were tested for gas leakage using the soapy water solution test, but no detectable leaks were identified.

Other observations included the following:

- The operator for the effluent gate at the overflow box was inoperable and the gate was stuck in the open position.
- A condensate pipe at the overflow box was severely corroded.
- The guardrail at the top landing of the stair and overflow box is missing a metal kick plate. Small boards were wire tied to the guardrail posts to serve as a temporary kick plate.

## STEEL DOME SITE OBSERVATIONS




1. Kick plates are missing at the top landing.
2. Weir gate operator is non-operable and stuck in the open position.
3. Gas pressure gauge read 6.9 to 7.0 inches w.c. during the site visit.
4. Cracking in the dome surface sealant, but no detectable gas leakage.
5. Minor gas leak at small pit in the dome @ the seam b/w panel A and panel B.
6. Existing patch on the dome surface sealant.
7. Tear in dome surface sealant at the exterior side of a J-tube penetration.
8. Existing gas piping.

Figure 10 Steel Dome Site Observations

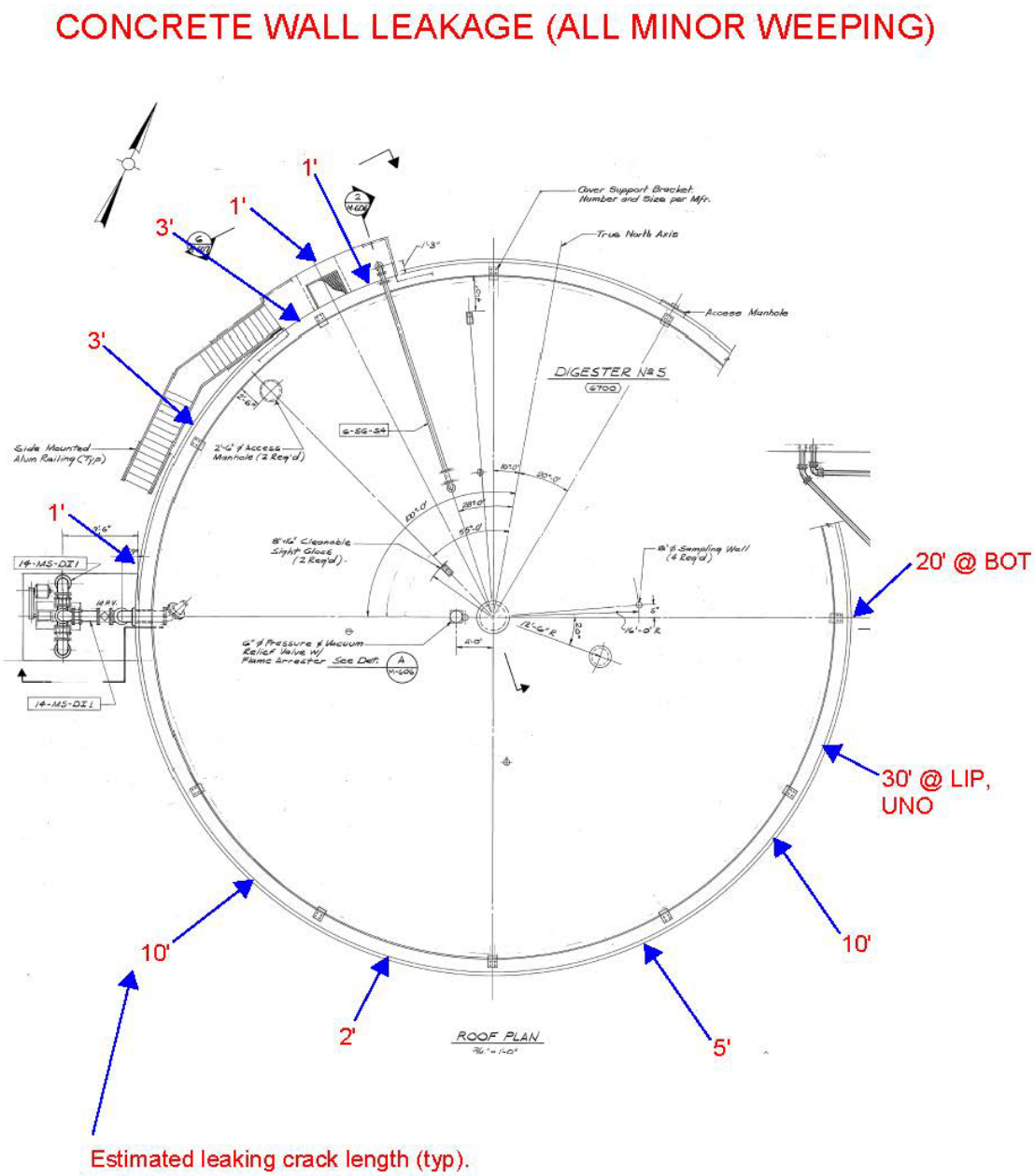



Figure 11 Concrete Wall Leakage (All Minor Weeping)

### 5.1.3 Internal Visual Assessment of Digester No. 5

The interior of Digester No. 5 was drained for an interior visual assessment; however, the digester could not be fully drained and the water level was stabilized to the invert of the access manway. Also, grit build-up in the bottom of the digester is considered to be substantial with several ft of depth estimated as the level of grit build-up at the southwest side of the digester was visibly higher than the water level. Therefore, entry into the digester was not possible. To provide a visual assessment of the digester interior, a drone inspection was made on November 13, 2018. The drone was successful at capturing high-definition video of the interior concrete wall, dome skirt, dome framing, and the dome plate membrane. Approximately 42 minutes of video footage was

captured by the drone, which was professionally operated by Nick Harwood of the Mistras Group. Refer to Figure 12, Figure 13, and Figure 14 for interior views of the digester wall, dome skirt, and dome framing, respectively. The drone is protected by a fiberglass cage, which is visible in Figures 12 through 14.

It appears that the outer sections of the dome have been submerged during the previous digester operation based on the stains that are visible on the dome beams.

Based on a review of the drone video footage, the following observations of the digester interior were made:

- Overall, the coating on the steel members appears to be fair to poor condition at many locations in the outer half of the dome where rust staining occurs frequently. The existing coating appears to be in good to fair condition towards the center half of the dome interior. Discoloration of the coating is prevalent throughout the interior surfaces of the dome.
- It appears that the outer sections of the dome have been submerged during the previous operation of the digester based on the visible stains on the webs of the dome beams and the collection of dried sludge on the bottom flange of the dome beams. Refer to Attachment A – Photos 1 and 2.
- The connection of the dome beams to the dome skirt columns are heavily soiled with dried sludge. It is difficult to ascertain the condition of the steel members and connections, but no obvious signs of delaminating steel or section loss were seen. It is assumed that the members and connections have sustained moderate corrosion, but are likely in a repairable condition. Refer to Attachment A – Photos 3 and 4.
- The dome skirt appears to be in good condition overall, but heavily soiled with sludge and discolored. The bottom seal of the dome skirt is comprised of a stainless steel closure angle that is anchored to the digester wall with stainless steel anchors. The top of the closure angle is caked with dried sludge. About half the length of the perimeter skirt appears to project beyond the stainless steel angle leg towards the inside of the digester and there appears to be moderate to severe corrosion at the exposed ends of the dome skirt. The dome is assumed to be hung from above, so repairs/coating can be implemented to prevent further damage, but the deterioration does not appear to impact structural support. Refer to Attachment A – Photos 5 and 6.
- Overall the concrete appears to be in good condition, but heavily soiled with dried sludge and discolored. There were no obvious signs of concrete deterioration that warrant structural repairs. Refer to Attachment A – Photos 7 and 8.

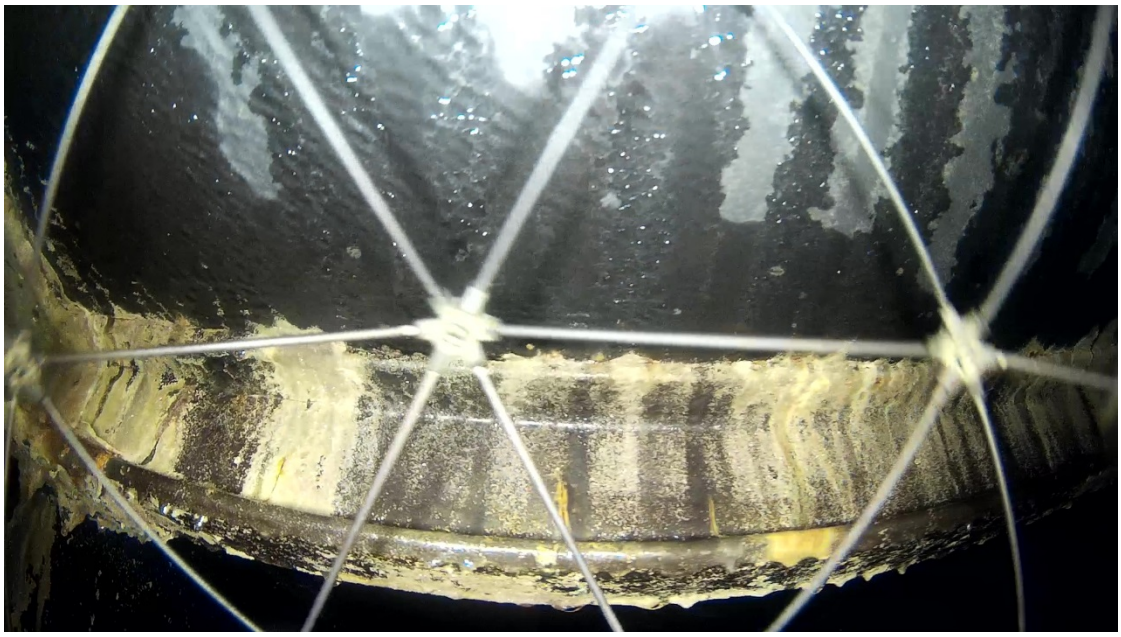


Figure 12 Interior Views of Digester Wall

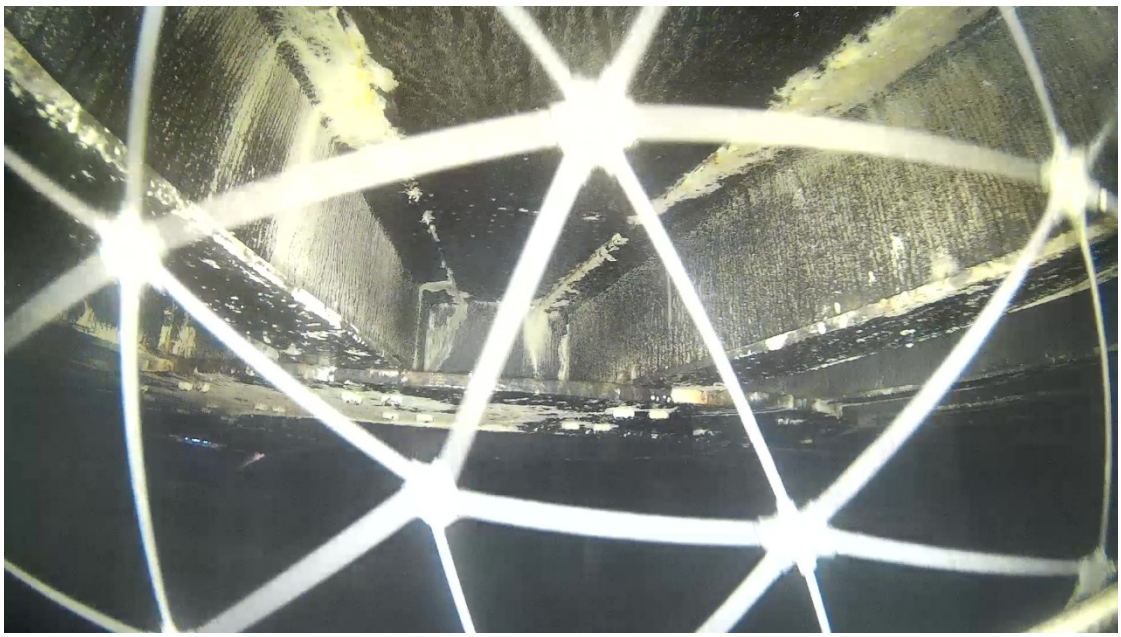


Figure 13 Interior Views of Dome Skirt

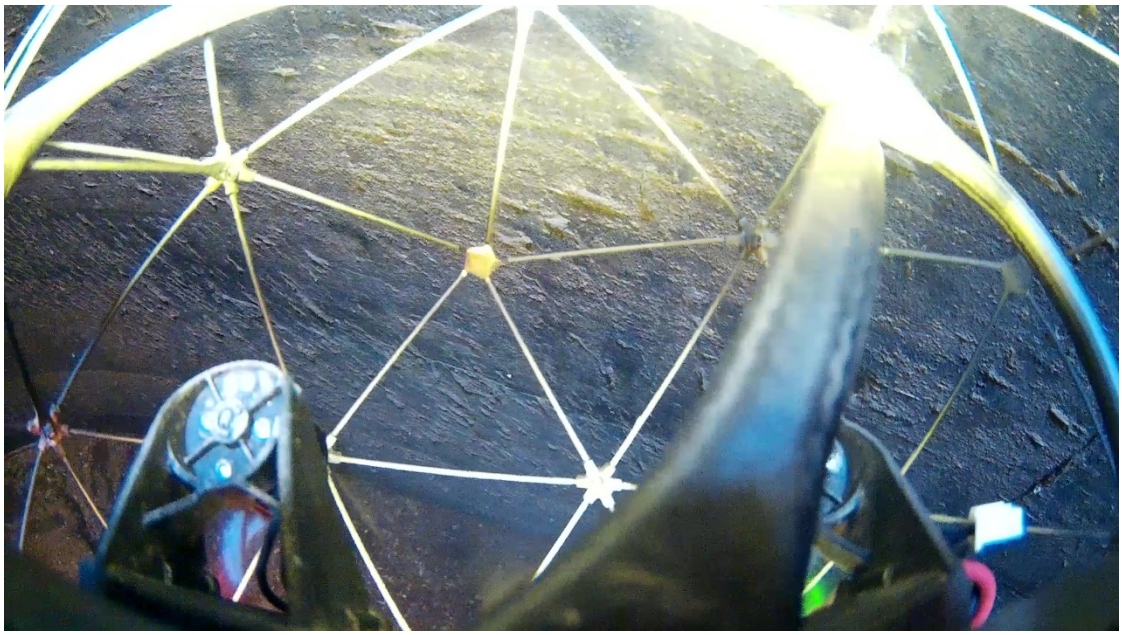


Figure 14 Interior Views of Dome Framing

### 5.1.4 Seismic Evaluation

Digester No. 5 was part of a primary and secondary replacement project, which was completed around 1982. The digester is a concrete circular tank with an inside diameter of 75 ft and a side wall height of 34.5 ft, steel dome cover, and cone shaped foundation slab sloping down towards the center. The operational high and low water heights are approximately 32 ft and 26 ft, respectively, from the high point elevation of the foundation slab at the side wall. The digester is partially buried on the southwest side to an approximate depth of 15 ft and slopes down to about a few ft above the foundation slab at the opposite side. The overall condition of the digester showed concrete cracks and signs of past leakage. Additional inspection photos of the digester are provided in Attachment A of this report.

The available original design and construction documents were reviewed to perform a seismic evaluation of the digester structure. The goal of this evaluation is to identify potential structural vulnerabilities by estimating the seismic demands and the associated material stresses of the structural members when subjected to those seismic loads, as well as loads due to self-weight, soils, and water. Initially, the evaluation is comprised of data gathering, establishment of a seismic evaluation and acceptance criteria, assumptions regarding material properties, and mathematical analyses of the structural systems and members. The results of this evaluation may serve as a basis for developing mitigation strategies.

#### 5.1.4.1 Data Collection and Review

To obtain data and information necessary for use in the evaluation of the digester, the following documents were reviewed:

- Construction drawings of "Primary-Secondary Replacement Capacity Project for Jurupa and Rubidoux", prepared by CDM, dated December 1979.
- "Technical Memorandum No. 9 - Integrated Digestion System", prepared by CDM, dated March 2010.

#### 5.1.4.2 Evaluation Criteria

*Codes and Standards*

The seismic evaluation of the digester was performed using ASCE 7-10, ACI 350.3-06, and ACI 350-06. The seismic forces (hydrodynamic forces) were calculated using ASCE 7-10, Chapter 15, in conjunction with ACI 350.3-06. The seismic design spectral accelerations were estimated for both ASCE 7-10 and ASCE 41-13, BSE-1E seismic level, assuming soil site class D. The values that are used in conjunction with ASCE 7-10 and ASCE 41-13 are based on the 2008 seismological data established by the USGS.

The structural material properties could not be found on the available construction documents and were estimated following ASCE 41-13 guidelines. No field-testing was performed to determine the structural properties of any of the existing members.

*Gravity Load Evaluation Parameters*

Gravity loads acting on the digester are limited to the self-weight of the structure (dead load), potential live loads applied to the roof dome, hydrostatic pressures, and at-rest earth pressure applied to the walls. Table 10 indicates the gravity load parameters used in this evaluation. The value assumed for the roof live load is considered to be the minimum required live load for roof structures.

Table 10 Gravity Load Parameters

| Parameter | Value |
|---|---|
| Roof Live Load | 20 psf |
| Concrete Unit Weight | 150 lbs/ft$^3$ |
| Water Unit Weight | 64.27 lbs/ft$^3$ |
| At-Rest Earth Pressure | 60 psf/ft |
| Operating High Water Height<br>Operating Low Water Height | 32 ft<br>26 ft |

Groundwater in the soil was assumed to be below the structure and not included in the structural analyses.

*Seismic Load Evaluation Parameters*

Table 11 indicates the seismic evaluation parameters used to estimate the seismic loads. The proposed use of the digester will be to process food waste that is delivered to the plant. Digester No. 5 is not anticipated to be a critical facility in the treatment plant. While gas production within the digester may be considered as a hazardous substance, the loads and actions that occur in the upper portion of the tank where methane and $H_2S$ gas reside, are anticipated to be relatively small compared to those loads and actions that occur in the lower portions of the structure. Therefore, the Risk Category for Digester No. 5 has been assumed to be II. This approach is considered to be consistent with the proposed use of the digester and its relative importance to other treatment

facilities at the plant. If another use of the digester is to be considered, the findings in this evaluation may need to be revisited.

Table 11 Seismic Parameters

| Parameter | Value |
|---|---|
| Soil Site Class | D |
| SDS (ASCE 7-10) | 1.000 g |
| SD1 (ASCE 7-10) | 0.600 g |
| SXS (ASCE 41-13, BSE-1E)<br>SX1 (ASCE 41-13, BSE-1E) | 0.918 g<br>0.531 g |
| TL | 8 seconds |
| Risk Category<br>Importance Factor, I | II<br>1.00 |
| Seismic Design Category<br>Response Modification Factor, Impulsive | D<br>2 |
| Response Modification Factor, Convective<br>Soil Dynamic Pressure | 1.5<br>25 H |

*Seismic Base Shear and Sloshing Wave Height*

The hydrodynamic base shear is the combination of impulsive ($V_i$) and convective ($V_c$) components. Impulsive forces are those inertial forces associated with the fundamental response to the ground acceleration. Convective forces are those forces that are generated by the longer period sloshing response to earthquake motion. These two components are typically out-of-phase from one another, but both contribute significantly to the total forces that a water-bearing structure might be subjected to. For this evaluation, these two component values were determined as follows:

$V_i = 0.50W_i$ (ASCE 7-10); $V_i = 0.46W_i$ (ASCE 41-13, BSE-1E).

$V_c = 0.11W_c$ (ASCE 7-10); $V_c = 0.09W_c$ (ASCE 41-13, BSE-1E).

Where:

Wi = Equivalent weight of impulsive component of the stored liquid.

Wc = Equivalent weight of the convective component of the stored liquid.

Additionally, vertical acceleration due to seismic ground motion will increase the internal hydrostatic lateral pressure on the structure. The vertical acceleration was estimated to be 0.20 gram.

The sloshing action of the water within the structure during an earthquake can generate a maximum wave height inside of the structure. When insufficient freeboard is provided, the water can slosh and surcharge the bottom side of the roof at or near the perimeter of the structure. The surcharge force will be directly proportional to the amount of freeboard deficit. The associated loading to the underside of a roof structure can be substantial and can cause significant damage or collapse, especially if the roof is framed with a lightweight material.

The wave height was determined using the equations and procedures set forth in ASCE 7-10. The sloshing wave height was estimated to be 4.2 ft for the high operating water level.

#### 5.1.4.3 Material Properties

The material properties of the existing concrete and reinforcing steel were assumed per ASCE 41-13, Chapter 10, based on the year of the design and construction. Table 12 summarizes the expected material strength properties used in this evaluation.

Table 12 Material Properties (ASCE 41-13, Chapter 10)

| Material | Default Lower-Bound | Factors for Expected Strength | Expected Strength | Reference Tables |
|---|---|---|---|---|
| Concrete Wall & Slab | $f_c' = 3{,}000\ psi$ | 1.5 | $f_c' = 4{,}500\ psi$ | 10-1 & 10-2 |
| Reinforcing Steel | $f_y = 40\ ksi$ | 1.25 | $f_y = 50\ ksi$ | 10-1 & 10-3 |

#### 5.1.4.4 Analysis Procedure

In general, the load combinations used in the evaluation of the structural members is based on those set forth in ACI 350-06 and ASCE 7-10. The governing load combinations used in this evaluation are presented in Table 13.

Table 13 Governing Load Combinations for Seismic Evaluation(1)

| Load Combinations | Type |
|---|---|
| D+F+H+E | Un-factored seismic load case for foundation bearing pressure. |
| 1.2(D+F)+1.6H+1.4E | Factored seismic load case for structural material strength evaluation. |

Notes:
D = Dead Load, L = Live Load, F = Hydrostatic Load, H = Soil Pressure, E = Seismic Load.
(1) The unbalanced soil pressure was assumed to be present under seismic load conditions and additive to the hydrodynamic pressures acting in the same direction.

Seismic loads were determined using Carollo's proprietary in-house structural analysis programs tailored for analysis of water-bearing structures. Additionally, hand calculations and a finite element software program (STAADPro v8i) were used as analysis tools. The following assumptions were applied to the finite element modeling and analysis:

1. Concrete cracked properties using 0.5EI for wall and foundation slab, where E is the elastic modulus for concrete and I is the moment of inertia for the member.
2. Foundation slab subgrade modulus is based on soil bearing pressure of 2,000 psf assuming a 1/2-inch settlement.
3. The analyses were performed for both full fixity and pinned condition between wall and foundation slab. Pinned condition was achieved by releasing rotational stiffness of the wall bottom nodes at the foundation slab in the finite element model.

### 5.1.5 Findings

The digester concrete wall and foundation slab were evaluated and the corresponding findings are presented herein. The metric used in this evaluation to quantify the degrees of distress of an existing structural component is referred to as the DCR.

$$\text{DCR} = \frac{\text{Load Demand}}{\text{Available Capacity}}$$

DCR values that exceed 1.0 are typically considered to be overstressed. Values that exceed 1.5 are significantly overstressed and may be treated with greater priority in a seismic retrofit. A summary of the DCR values for the subject structure for various liquid levels are set forth in Table 14 and

Table 15 for load demands calculated in accordance with ASCE 7-10 and ASCE 41-13, respectively. The DCR values indicated in the Tables are the maximum values determined from the evaluation. Where a DCR was determined to be less than 1.0, only that condition is reported in the Table.

Table 14 Demand-Capacity Ratio (DCR) (ASCE 7-10 Seismic Level)

| Component | Liq Ht=32ft | Liq Ht=26ft | Liq Ht=22ft | Liq Ht=16.5ft |
|---|---|---|---|---|
| Wall Hoop Tension | < 1.0 | < 1.0 | < 1.0 | < 1.0 |
| Wall Shear, Out-of-Plane | < 1.0 | < 1.0 | < 1.0 | < 1.0 |
| Wall Shear, In-Plane | < 1.0 | < 1.0 | < 1.0 | < 1.0 |
| Wall Bending Moment at Vertical Dowel | 1.72 | 1.25 | 1.04 | 0.93 |
| Foundation Slab, Ring Tension | 2.85 | 2.05 | 1.80 | 1.54 |
| Foundation Slab, Bending Moment for Bottom Radial Bars | 1.74 | 1.39 | 1.30 | 1.21 |
| Foundation Slab, Bending Moment for Top Radial Bars | < 1.0 | < 1.0 | < 1.0 | < 1.0 |

Table 15 Demand-Capacity Ratio (DCR) (ASCE 41-13, BSE-1E Seismic Level)

| Component | Liq Ht=32ft | Liq Ht=26ft | Liq Ht=22ft | Liq Ht=16.5ft |
|---|---|---|---|---|
| Wall Hoop Tension | < 1.0 | < 1.0 | < 1.0 | < 1.0 |
| Wall Shear, Out-of-Plane | < 1.0 | < 1.0 | < 1.0 | < 1.0 |
| Wall Shear, In-Plane | < 1.0 | < 1.0 | < 1.0 | < 1.0 |
| Wall Bending Moment at Vertical Dowel | 1.61 | 1.19 | 1.00 | 0.92 |
| Foundation Slab, Ring Tension | 2.65 | 1.97 | 1.73 | 1.50 |
| Foundation Slab, Bending Moment for Bottom Radial Bars | 1.63 | 1.35 | 1.26 | 1.19 |
| Foundation Slab, Bending Moment for Top Radial Bars | < 1.0 | < 1.0 | < 1.0 | < 1.0 |

The findings indicate significant overstress of the wall dowels and foundation slab reinforcing bars near the wall. Mitigation options to address these potential vulnerabilities are presented in next.

### 5.1.6 Wall Dowels

The digester wall is subject to bending moments at the bottom of the wall due to the applied loads. The DCR for the wall dowels is estimated to be from 1.72 to 1.19, depending on the seismic level, for the current operating water height between 32 ft and 26 ft. This level of overstress is considered to be moderate to severe, depending on the liquid height in the digester. The wall can potentially experience extensive cracking during a major earthquake at these levels of stress.

It is recommended that the operating water height be limited to 22 ft, at which point the DCR is within acceptable limits. Alternatively, the wall can be strengthened by means of an interior shotcrete addition as shown on Figure 15.

### 5.1.7 Foundation Slab Reinforcing Bars

The foundation slab is subject to ring tension and bending moments from the wall reactions and soil bearing pressure. The DCRs for the slab ring tension (circumferential bars) and bending moment (radial bottom bars) are estimated to be from 2.85 to 1.50 and 1.74 to 1.19, respectively, depending on the seismic level, for the water heights between 32 ft and 16.5 ft. This level of overstress is considered to be moderate to excessive depending on the liquid height in the digester. Extensive cracking is anticipated during a major earthquake at these levels of stress at operating water heights of 32 ft and 26 ft.

It is recommended that the existing slab be thickened and that additional reinforcing bars be provided, which will give additional strength to the slab bending moment capacity and ring tension capacity. Refer to Figure 15.

For the operation at a higher liquid height of 32-ft (high liquid level) the following conditions would apply:

1. Seismic soil bearing pressure of 5,000 psf is expected. This should be verified with the soil engineer for the acceptability.
2. Liquid slushing height of 5.5-ft is expected during seismic which will impart uplift pressure to the lower outer (about 10-ft) ring of the roof dome. Low elevation of the dome is about 2.5-ft above the liquid level:
   a. Dome anchor connection could not be evaluated due to lack of information in the available drawings.
   b. Limit the liquid height to 29-ft or raise the wall height to provide adequate freeboard, or;
   c. Replace or strengthen existing dome to be able to take additional stresses.
3. Thicken the wall and slab by at least 18-in with additional reinforcing bars per attached.

### 5.1.8 Foundation Slab Soil Bearing Pressure

A summary of the foundation slab soil bearing pressures is presented in Table 16. The slab soil bearing pressure is estimated to vary from 5,026 psf to 3,571 psf, depending on the seismic level, for an operating water height between 32 ft and 26 ft. This level of bearing pressure is considered to be moderate to excessive and may result in additional settlement that can impart additional stresses to the concrete structure. Allowable soil bearing pressure is estimated to be 3,325 psf for the seismic load case, obtained by applying a one-third increase to an assumed allowable soil bearing pressure of 2,500 psf.

It is recommended that the operating water height be limited to no more than 26 ft, at which point the soil bearing pressure is estimated to be slightly higher than the estimated allowable bearing pressure at which the soil would still not be anticipated to have a bearing failure.

Table 16 Foundation Slab Soil Bearing Pressure (Service Level)

| Component | Liquid Height=32 ft | Liquid Height=26 ft | Liquid Height=22 ft | Liquid Height=16.5 ft |
|---|---|---|---|---|
| **Fixed Base** | | | | |
| ASCE 7-10 | 4,666 psf | 3,643 psf | 3,125 psf | 2,534 psf |
| ASCE 41-13, BSE-1E | 4,522 psf | 3,571 psf | 3,067 psf | 2,506 psf |
| **Pinned Base** | | | | |
| ASCE 7-10 | 5,026 psf | 4,018 psf | 3,528 psf | 3,010 psf |
| ASCE 41-13, BSE-1E | 4,882 psf | 3,946 psf | 3,470 psf | 2,981 psf |

### 5.1.9 Sloshing Wave Height

The sloshing wave height was estimated to be 4.2 ft. Based on the available construction documents, the current freeboard available from high water elevation is approximately 2.5 ft which can result in a net upward force applied to the steel dome. This complication can be resolved by limiting the operating water height as recommended in Sections 3.1 and 3.3. Otherwise, the dome may experience damage to the steel members, dome seal, and dome anchorage at the top of the wall.

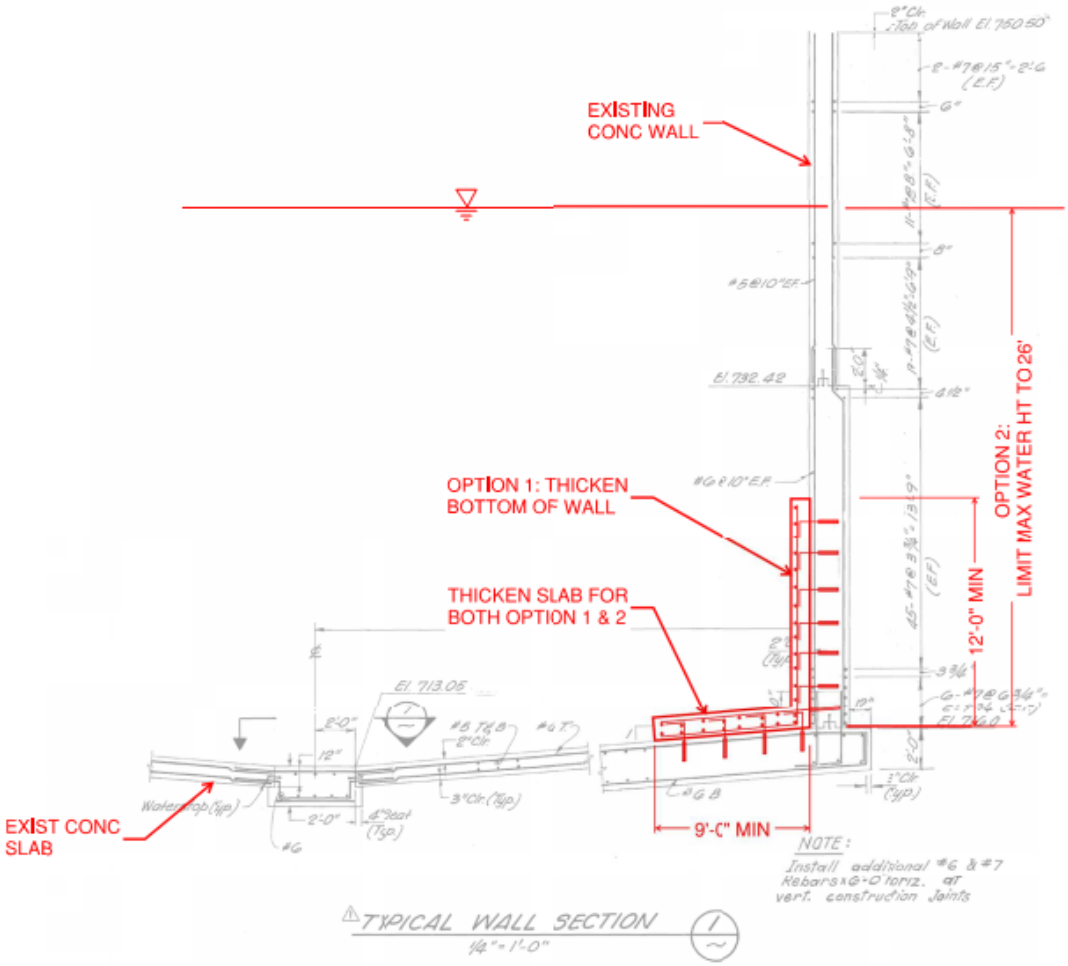



Figure 15 Conceptual Details

## 5.2 Process/Mechanical Assessment

### 5.2.1 Digester Mixing

As described in Section 3, only one Vaughn chopper pump is currently available for mixing of Digester No. 5. Since having an installed standby will require major modifications to the piping, it is recommended that an uninstalled spare pump be stored in the warehouse to allow maintenance to switch pumps in a short period in case of failure.

Currently, for pump mixing, sludge is withdrawn from the top of the digester and discharged at the bottom via nozzle. There is a top discharge nozzle to be used for breaking up scum. However,

since the digester will be operated at varying levels, changes to the piping inlet configuration should be implemented during tank repairs to move the suction to the bottom and center of the digester and the discharge to the bottom and top along the perimeter of the digester.

The online DG pressure indicator in the DG piping is in good working condition and allows the monitoring and control of the DG pressure (by adjusting the organic loading rate) which will be critical in reducing the risk of DG venting occurrences.

### 5.2.2 Estimated Solids Production

#### 5.2.2.1 Projected Municipal Solids Production

In order to evaluate the impact of additional food waste biosolids on the overall solids handling system, an evaluation was carried out. Table 17 summarizes the anticipated solids projections (Thickened WAS and primary solids) for the 39.0-mgd ADF condition, which is projected for 2037. These values were developed from the calibrated BioWin model that was calibrated based on data that was gathered when the plant was receiving an ADF of 25 mgd.

Table 17 Solids Projections at 39.0 mgd ADF

| Element | Unit | Value |
|---|---|---|
| Thickened WAS Production (ADF)(1) | | |
| • Thickened WAS Flow Rate | gpd | 83,624 |
| • Solids | ppd | 43,938 |
| • Solids Concentration | % | 6.3 |
| • VSS | % | 85 |
| Thickened WAS Production (Max Month)(2) | | |
| • WAS Flow Rate | gpd | 99,511 |
| • Solids | ppd | 52,285 |
| • Solids Concentration | % | 6.3 |
| • VSS | % | 85 |
| Primary Sludge Production | | |
| • PS Flow Rate | gpd | 214,000 (2.5%), 119,000 (4.5%) |
| • Solids | ppd | 44,600 |

Notes:
(1) Results from calibrated BioWin prediction.
(2) Estimated from Mathematical Models.

The Thickened WAS projection for the 39.0-mgd ADF condition was based on the projected WAS production and historical dissolved air flotation thickening performance. More detailed information is located in Volume 4, Chapter 3: Process Design and Reliability Criteria and Volume 5, Chapter 4: Sludge Production and Thickening of the Master Plan. The primary sludge projection information is obtained from Volume 4 Chapter 5: Primary Treatment of the Master Plan.

Based on Table 17, the anticipated maximum monthly solids flowrate for 39-mgd ADF is 313,511 gpd. The required dewatering capacity for 24-hour operation is 218 gpm.

#### 5.2.2.2 Digester No. 5 Capacity and Food Waste Solids Production

The expected additional organics from food waste and the impact on the digester loading rates, and solids generation are described in Table 18. The Solids generation rate will be compared with the existing dewatering capacity for digestate after addition of the organics.

Table 18 Digester Loading Rates and Solids Generation

| Parameter | Values |
|---|---|
| Digester volume (Digester No. 5) | 132,469 cu ft |
| Maximum | 990,866 gal |
| Recommended maximum operating volume | 858,750 gal |
| Diameter | 75 ft |
| Tank height | 30 ft |
| Recommended liquid level | 29 ft |
| Digester No. 5 organic loading rate design criteria | 0.20 lbs/cfd |
| Design solids loading | 22,961 ppd |
| | 10 dry tons per day |
| | 100 WTPD (at 10% solids) |
| Truck loads per day | ~6 per day (5,000 gal) |
| | 26,843 gpd |
| VSR | 75 percent |
| | 16,790 ppd |
| Solids for disposal | 5,597 dry ppd |
| Estimated cake solids concentration | 25% |
| Solids for disposal | 13 WTPD |
| Estimated sludge flowrate to dewatering | 26,843 gpd |
| Biogas generation | 251,856 cfd at 15 cu ft per lbs VSR(1) |
| | 175 cfm |

Notes:
(1) Food Waste ranged from 11 to 35 cu ft per ton of VS processed. A biogas production rate of 15 cu ft/lbs VSR was used for this estimate.

### 5.2.3 Dewatering Capacity

The sludge dewatering process capacity was determined based on a screw press peak solids loading rate of 700 lbs per hour per unit and a peak hydraulic loading rate of 55 gpm per unit with six screw presses in service, operating 24 hours per day, 7 days per week. Under these conditions, the available dewatering capacity is 330 gpm, assuming hydraulic loading rate is limiting. With the addition of food waste bioslurry at 26,843 gpd, an average 18.64 gpm of digested sludge would be added to the dewatering units. Based on the excess capacity of the existing system, the additional digested bioslurry is not expected to cause any dewatering capacity issues. However, adjustments to the dewatering polymer dose might be needed.

### 5.2.4 Estimated Gas Production and Flare

The expected DG production from food waste is presented in Table 19. The DG flow rates will be compared with the existing flaring capacity.

Table 19 Sludge Dewatering System Design Criteria(1)

| Dewatering Screw Presses | Value |
|---|---|
| Number | 6 |
| Dewatered Cake Solids Concentration | 19 percent |
| Solids Capture | 95 percent |
| Average Feed Rate | 40 gpm |

Notes:
(1) Source: 2008 RWQCP Wastewater Collection and Treatment Facilities Integrated Master Plan, and Field Inspection.

#### 5.2.4.1 Digester Gas Generation and Existing Flaring Capacity

Digestion of food waste in Digester No. 5 at a loading rate of 0.20 lbs/ft$^3$ will allow the treatment of 100 WTPD of food waste based on a 10 percent solids content and assuming that 70 percent of the food waste will be from bioslurry and only 30 percent from ADM. The digester tank volume at an operating level of 29 ft and the lower solids concentration of the Burrtec bioslurry with 6-7 percent solids and a limit of the other food waste material (ADM) of 30 percent due to the higher solids concentration of 20 percent TS and potential foaming concerns. Assuming a 75 percent VSR the calculated DG generation will be approximately 252,000 cfd. This will be verified by the City during a future DG study using COD concentrations and biogas yield test data.

The additional DG generation from food waste digestion will be beneficially used in fuel cells and future biomethane projects. If these systems are out of service or undergoing maintenance, the existing flaring capacity will need to process all DG generated. The existing flaring capacity is designed for 1,445,000 cfd (1003.5 scfm), for a maximum heat release of 52 MMBtu/hr (max). Two flares are installed, Digester Gas Flare No. 1 (Tag # FDG-892-1) and Digester Gas Flare No. 2 (Tag # FDG-892-2).

The current DG flow rate to the existing fuel cell is 300 cfm and the total generation is 400 cfm. The average DG generated in the food waste Digester No. 5 would be approximately 135 scfm, but could be up to about 200 scfm. Thus, the total DG flow to the flare system would be a maximum of about 600 scfm. The projected DG flow is therefore within the existing flare design capacity.

The operation of the new flares is currently unreliable due to mechanical issues related to low feed pressure shut-downs caused by the DG booster blowers. Based on current AQMD permit conditions the flaring of DG over 560 cfm would result in exceedance of the daily $SO_x$ limits, which is 5 ppd, until such time as the City is able to overcome the existing mechanical issues. The DG would have to be scrubbed for $H_2S$ prior to flaring.

#### 5.2.4.2 Digestate Dewatering and Solids Disposal

The upgrade of the dewatering facility with screw presses was discussed above. The additional solids loading from food waste digestion at a digester loading rate of 0.20 lbs/cfd and 75 percent VSR will be 5,597 dry ppd. It is estimated that 13 wt of sludge will be generated from food waste alone and will need to be hauled off-site.

#### 5.2.4.3 Bioslurry Screening

Food Waste slurry will be processed off-site in hammer mills or grinders which are typically equipped with screens of 5 to 10 mm. The contamination of bioslurry can include fibers, plastics, glass, grit, and metal and can account for around 10 percent of the total mass, as shown on Figure 16 shows an example of bioslurry contamination. In order to remove such remaining contamination, screening of the waste material during unloading of the trucks is recommended. The screening onsite is still recommended as a precaution against poor pretreatment by the third party. Unremoved inert materials would accumulate in the digester over time and reduce the available digester volume, and increase the cleaning frequency and associated costs.

Figure 16 Typical Reject from Food Waste Screening: Plastic, Glass, Fiber and Metals

The typical screen size for a food waste receiving station is 6 mm, which is based on pilot tests at Napa Sanitary District with a Huber StrainPress on a FOG solid-liquid separation system, and food waste screening at IEUA RP-5 Solids Handling Facility using a Vincent Screw press.

Based on recent experience at the RWQCP, bucket strainers are not recommended as they get clogged frequently causing a halt in the truck unloading and material handling process.

### 5.2.5 Instrumentation

Instrumentation to monitor and control the operation of a digester are important components of the system. Digester No. 5 has some existing instrumentation, but some additional equipment is recommended.

#### 5.2.5.1 Instrumentation for Monitoring Digester Operations

Only mechanical drawings for DG piping showing flame arrestors and valves were available for review (CDM drawing set from 1979). Based on the site assessment there are two DG flow meters

connected to SCADA (as shown in the process and instrumentation diagram for the new digesters):

- One DG flow meter for the vented gas, which is installed downstream of the pressure relieve valve to monitor any DG going to the carbon canisters in order to monitor venting events for AQMD compliance
- One DG flow meter on the DG collection header and this line is also equipped with a pressure indicator.

#### 5.2.5.2 Digester Feeding and Control of Flow Rate and Organic Loading of the Digester

A consistent organic loading rate to the digester is important to avoid operational challenges and to produce a constant DG flow to the fuel cells or other future beneficial DG uses. The organic loading rate can be expressed as COD or VSS concentrations and flow rates per volume of digester available. In-line COD analyzers based on UVAS technology have been developed and tested in the field for industrial digesters. As part of the recommended instrumentation, HACH UVAS 254 or similar is recommended. A flow meter to monitor instantaneous influent flow rate and totalized flow should be included. This instrument should be connected to the plant's SCADA system.

#### 5.2.5.3 pH Neutralization Needs for Acidic Waste Streams

The organic acids which will be generated in the food waste due to hydrolysis will result in a lower pH of the substrate. The volatile acids will be readily transformed into shorter chain volatile organic compounds for methane formers and will be converted to methane and carbon dioxide. The only substrates which will need pH adjustment are industrial organic liquids such as spent vinegar. Therefore a pH injection and control system for this type of substrate would be recommended. Figure 17 shows a schematic of the proposed control system and instrumentation arrangement for Digester No. 5. Some of the instrumentation is existing and some will be new.

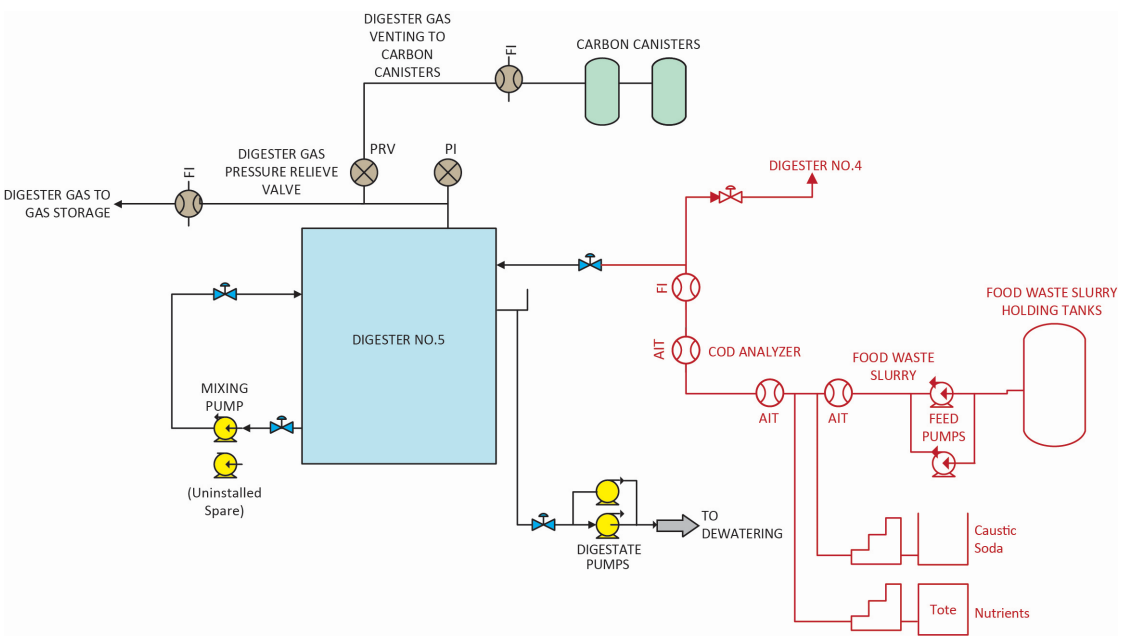



Figure 17 Schematic of Proposed Digester Control System

#### 5.2.5.4 Other Chemical Injection Options (Magox, Micronutrients)

Micronutrients are mostly needed for industrial organic substrates. This will be added on as needed basis and can be set up in a tote with chemical feed pumps set up in the receiving station.

## 5.3 Digester No. 5 Rehabilitation Cost Estimate

The goal of the evaluation of Digester No. 5 was to identify specific structural and mechanical vulnerabilities for the purpose of improving the overall reliability of the digester for its proposed use as a dedicated food waste digester. The findings presented in this report identify several seismic vulnerabilities that warrant retrofit/rehabilitation of the structure. Mitigation strategies involve limiting operational water height and thickening the foundation slab with additional reinforcing bars. The cost for implementing this alternative will be limited to the amount of effort required to control the water level in the digester and the construction of new slab/wall over the existing members.

Table 20 presents a summary of the cost estimate to rehabilitation Digester No. 5 for use as a food waste digester. The digesters effective holding volume with the liquid height limited to 29 ft will be 858,750 gal.

Table 20 Estimated Cost to Rehabilitate Digester No. 5

| Description | Estimated Cost |
|---|---|
| Structural Modifications | |
| Internal Walls rehabilitation | $199,862 |
| Seismic Retrofits | $254,609 |
| Mechanical | |
| Uninstalled Standby Mixing Pump | $95,000 |
| Instrumentation and Valves | $50,000 |
| **Sub-total** | **$784,471** |
| Electrical | $85,000 |
| Mechanical, pipe, supports, etc. | $100,000 |
| **Total Direct Cost** | **$784,471** |
| Contingency - 30% | $235,341 |
| General Conditions, Contractor Overhead, Profit and Risk - 25% | $254,953 |
| Escalation to Mid-Point (3.5 % per year for 3 years, 10.5%) | $133,850 |
| Sales tax | $123,254 |
| **Total Estimated Construction Cost** | **$1,531,870** |
| Engineering 10% | $459,561 |
| **Estimated Project Cost** | **$1,991,430** |

More detailed estimates of the cost are located in Attachment C.

Section 6

# SUMMARY AND RECOMMENDATIONS

Based on the evaluation of the existing Digester No. 5 presented in this report, the following conclusions and recommendations are made.

1. Digester No. 5 is generally in good condition and could be rehabilitated to be converted to a standalone food waste digester.
2. Rehabilitation of Digester No. 5 will involve both structural and mechanical work. Structural work includes both internal coating of the digester as well as wall strengthening to bring the structure up to seismic code. The rehabilitation work is recommended to include:
   a. Epoxy or chemical grout injection to seal approximately 200 linear ft of concrete cracks in the digester wall.
   b. Blast and recoat the interior steel dome surfaces inside of the digester. Exterior rehabilitation of the steel may be limited to localized recoating and resealing of damaged seals at the dome perimeter.
   c. Repair of steel members that have experienced section loss with lap-welded plates as required. It is estimated that the occurrence of these repairs would be infrequent.
   d. Application of a surface-applied waterproofing mortar, such as Xypex Concentrate, that is treated with an anti-microbial agent to provide both concrete protection and waterproofing for the digester wall. This should be applied to the existing wall surface above the new concrete wall addition.
   e. Provide new aluminum guardrail and kick-plate at the top stair landing.
   f. Mechanical work includes a new standby pump mixer, some piping modifications, and additional instrumentation.
3. Due to seismic considerations, the operating level in Digester No. 5 is limited to 29-ft, which results in a working volume of 858,750 gal.
4. A maximum level of 29 ft could be considered with additional concrete reinforcement and this would result in 858,750 gal of operating volume, which is sufficient to treat the bioslurry and some ADM max 30 percent of total food waste at an organic loading rate of 0.2 lbs VSS/cu ft day.
5. Digester No. 5 should be able to treat approximately 100 tons per day of food waste.
6. Initial estimates indicated that approximately 252,000 cfd of additional biogas could be generated from food waste digestion in Digester No. 5. Understanding that different food waste sources generate different amounts of biogas, it is recommended that laboratory testing of various food waste sources be carried out to confirm the potential gas production from the mixture of food waste expected at the RWQCP.
7. It appears that the existing gas flaring capacity will be sufficient to handle the additional biogas anticipated from Digester No. 5, once the mechanical issues for the existing flaring system are addressed.
8. A food waste bioslurry receiving station design was proposed upstream of Digester No. 5. This system is expected to have a project cost of approximately $1.3 million.
9. The estimated cost to rehabilitate Digester No. 5 is approximately $2 million.

Attachment A

# EXISTING DIGESTER INSPECTION PHOTOS

*Figure A1* Photo 1 and 2, Outer Section of the Dome

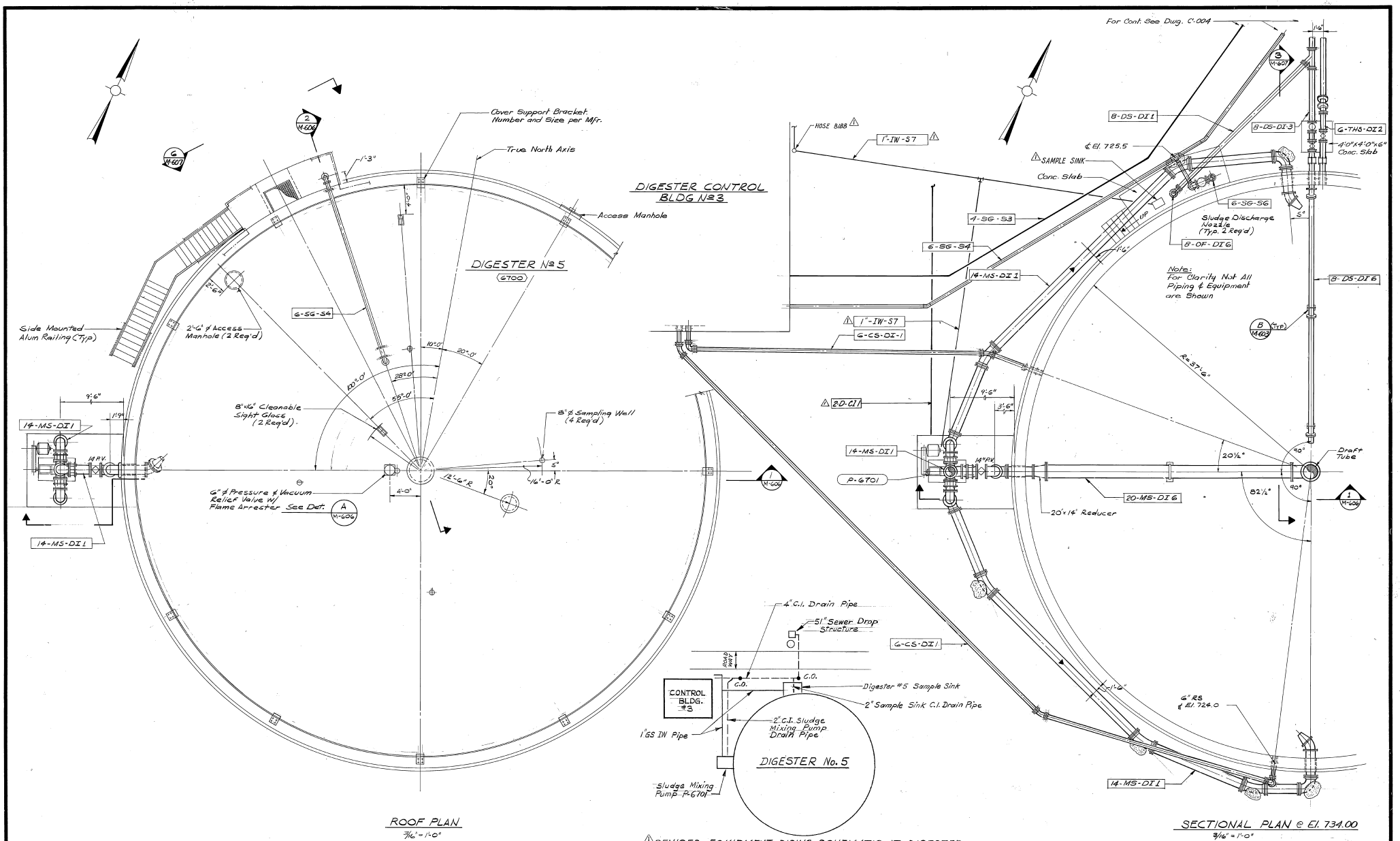
*Figure A2* Photo 3 and 4, Connection of the Dome Beams to the Dome Skirt Columns

*Figure A3* Photo 5 and 6, Dome Skirt

*Figure A4* Photo 7 and 8, Concrete

Attachment B

# SEISMIC CALCULATIONS




REVISED EQUIPMENT PIPING SCHEMATIC AT DIGESTER CONTROL BUILDING No. 3 & DIGESTER No. 5
N.T.S.

**RECORD DRAWING**

AS BUILT 10-31-82 LP

INDEXED LP

**CAMP DRESSER & McKEE INC.**
Southwestern Regional Office
283 South Lake Avenue
Pasadena California 91101

**CDM**
environmental engineers, scientists, planners & management consultants

| Approved by: | Date | Name |
|---|---|---|
| Structural | | |
| Architectural | | |
| Electrical | | |
| Instrumentation | | |
| HVAC | | |
| Project Engineer | 12/79 | ASR |
| Project Manager | 12/79 | DMS |

| MARK | REVISIONS | APPR. | DATE |
|---|---|---|---|
| 1 | RECORD DRAWING | | 12-14-82 |

DESIGNED BY LG DRAWN BY CC2 CHECKED BY EL

# CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT

APPROVED BY
PRINCIPAL ENGINEER
PARK DEPARTMENT
TRAFFIC DIVISION
CHIEF F. M. ENGINEER

APPROVED BY
PUBLIC WORKS DIRECTOR
DATE 12-20-79

PRIMARY - SECONDARY REPLACEMENT CAPACITY PROJECT

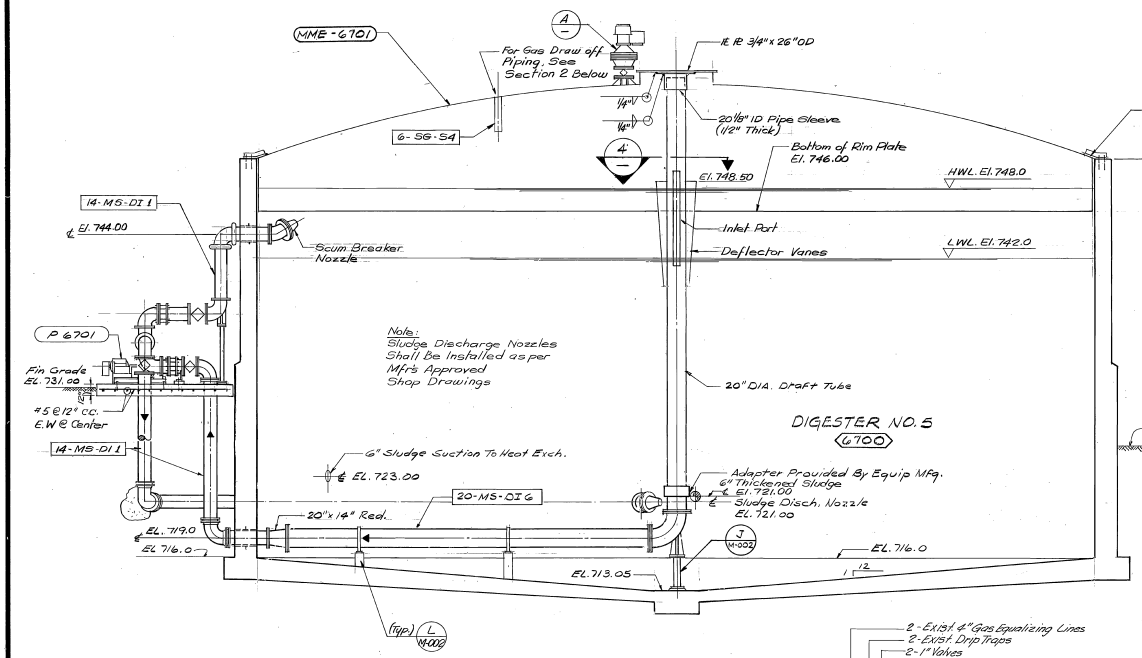
DIGESTER NO.5
PLAN

HORIZ. SCALE: 3/16" = 1'-0" VERT. SCALE: 1" -

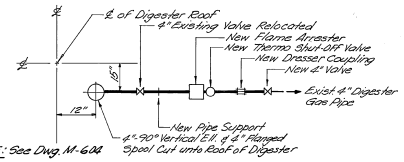
ACCOUNT NO. T-14

SHEET M-605 OF 55

FILE NO. 2012-7-087

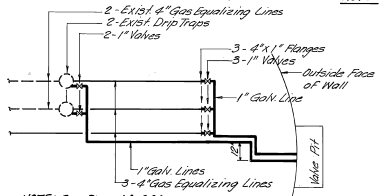



SECTION 1
3/16 = 1'-0"
M-605



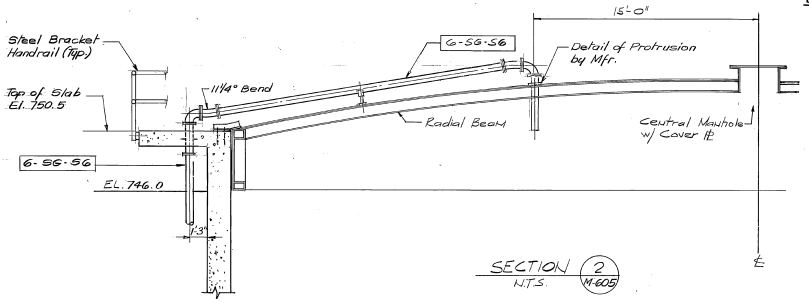

NOTE: See Dwg. M-604

DIGESTER No. 4 RELOCATED GAS PIPING SCHEMATIC
N.T.S.



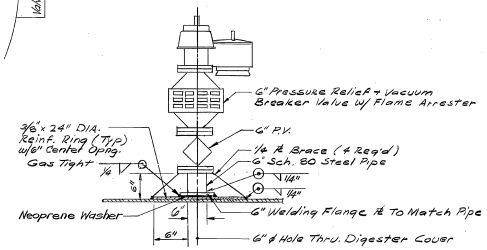

NOTE: See Dwg. M-604

DIGESTER No. 4 DRIP TRAP DRAIN SCHEMATIC
N.T.S.

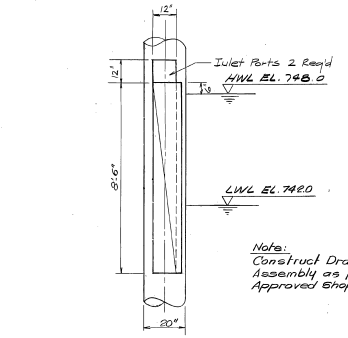



SECTION 2
N.T.S.
M-605

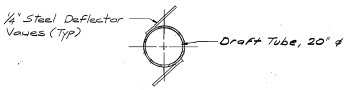
Note: Digester Cover Shall be Installed as per Mfr's Approved Drawings.



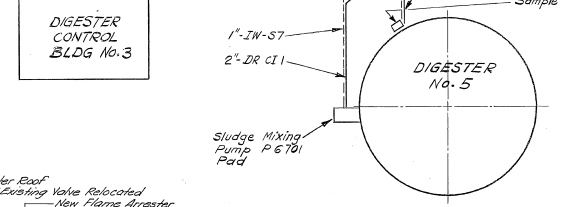

DETAIL A
N.T.S.
M-605




Note: Construct Draft Tube Assembly as per Mfrs. Approved Shop Drawings.




SECTION 4
N.T.S.




⚠ REVISED EQUIPMENT PIPING SCHEMATIC AT DIGESTER CONTROL BLDG. No. 3 & DIGESTER No. 5
N.T.S.

RECORD DRAWING

AS BUILT 10 31 82 WF

INDEXED WF

CAMP DRESSER & McKEE INC.
Southwestern Regional Office
283 South Lake Avenue
Pasadena California 91101

CDM
environmental engineers, scientists, planners & management consultants

| Approved by: | Date | Name |
|---|---|---|
| Structural | | |
| Architectural | | |
| Electrical | | |
| Instrumentation | | |
| HVAC | | |
| Project Engineer | 8/79 | A.S.R. |
| Project Manager | 8/79 | D.M.S. |

| MARK | REVISIONS | APPR. | DATE |
|---|---|---|---|
| 1 | RECORD DRAWING REVISIONS | RM | 12-23-91 |

DESIGNED BY AR/EL DRAWN BY EG CHECKED BY EL

CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT

APPROVED BY
PRINCIPAL ENGINEER
PARK DEPARTMENT
TRAFFIC DIVISION
CHIEF F. W. ENGINEER
PUBLIC WORKS DIRECTOR
DATE 12-28-90

PRIMARY - SECONDARY REPLACEMENT CAPACITY PROJECT

DIGESTER NO 5
SECTIONS & DETAILS I

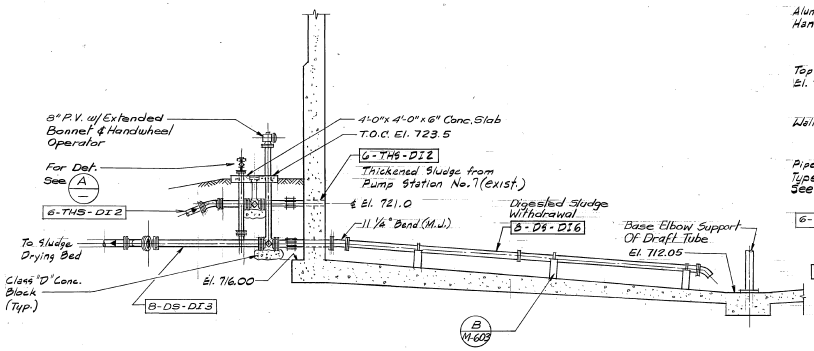
ACCOUNT NO. T-14

SHEET M-606 OF 55

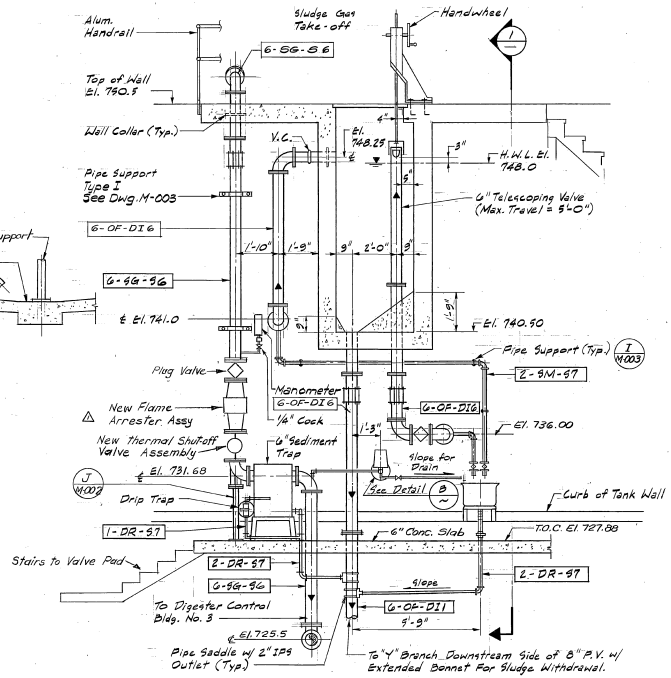
HORIZ. SCALE: AS SHOWN VERT. SCALE: 1" -

FILE NO. 2012-7-088

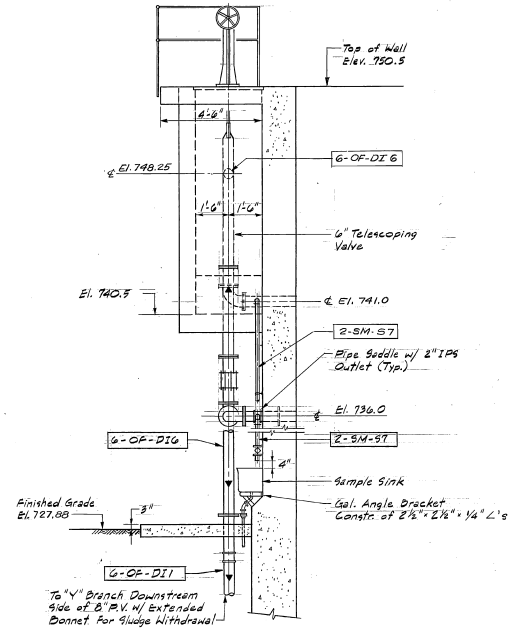
REGISTERED PROFESSIONAL ENGINEER CARLOS M. CAMPOLONGO NO. 13161 STATE OF CALIFORNIA



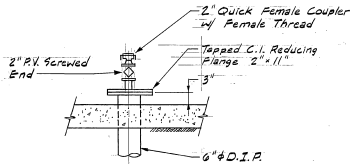

SECTION 3/16" = 1'-0" (3 M603)



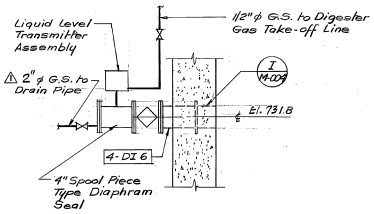

SECTION N.T.S. (6 M605)




SECTION 3/8" = 1'-0" (1)




DETAIL A N.T.S.




DETAIL B N.T.S.

**RECORD DRAWING**

AS BUILT 10-31-82 LF

INDEXED LF

**CAMP DRESSER & McKEE INC.**
Southwestern Regional Office
283 South Lake Avenue
Pasadena California 91101

**CDM**
environmental engineers, scientists, planners & management consultants

| Approved by: | Date | Name |
|---|---|---|
| Structural | | |
| Architectural | | |
| Electrical | | |
| Instrumentation | | |
| HVAC | | |
| Project Engineer | 8/79 | A.S.R. |
| Project Manager | 8/79 | D.M.S. |

| MARK | REVISIONS | APPR. | DATE |
|---|---|---|---|
| 1 | RECORD DRAWING REVISIONS | | 12-23-79 |

DESIGNED BY EVL DRAWN BY GW CHECKED BY LG

**CITY OF RIVERSIDE**
PUBLIC WORKS DEPARTMENT

APPROVED BY: PRINCIPAL ENGINEER, PARK DEPARTMENT, TRAFFIC DIVISION, CHIEF P. W. ENGINEER

APPROVED BY: PUBLIC WORKS DIRECTOR

DATE 12-28-79

PRIMARY - SECONDARY REPLACEMENT CAPACITY PROJECT

DIGESTER NO. 5

SECTIONS & DETAILS II

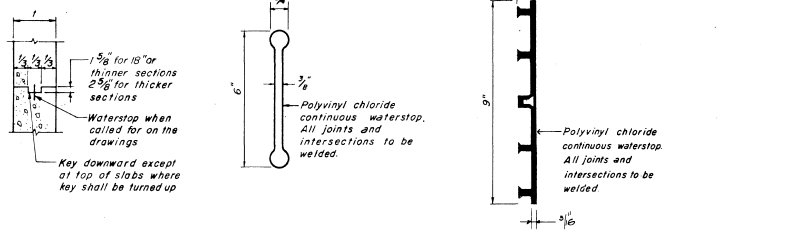
ACCOUNT NO. T-14

SHEET M-607 OF 55

HORIZ. SCALE: AS SHOWN VERT. SCALE: 1" -

FILE NO. 2012.7-089

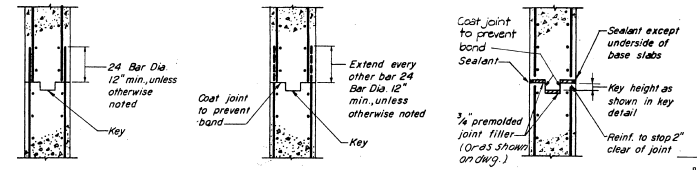
REGISTERED PROFESSIONAL ENGINEER CARLOS M. CAMPIZANO NO. 13161 CIVIL STATE OF CALIFORNIA




CONCRETE KEY
No scale

JOINT WATERSTOP
No scale

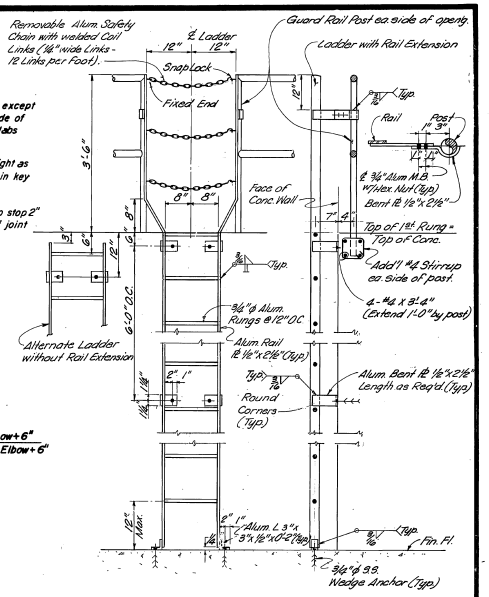
EXPANSION JOINT WATERSTOP
No scale

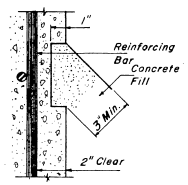



CONSTRUCTION JOINT
No scale

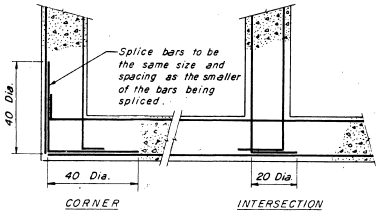
CONTROL JOINT
No scale

EXPANSION JOINT
No scale

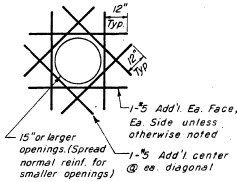


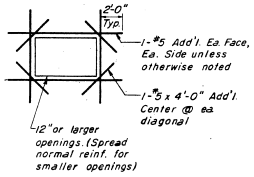

CONCRETE FILL KEY DETAIL
No scale



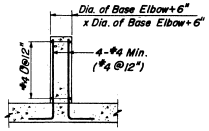

WALL REINFORCING DETAIL
No scale




REINFORCEMENT AT ROUND OPENINGS
No scale

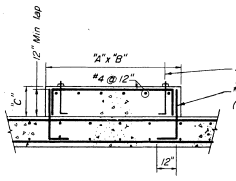



REINFORCEMENT AT RECTANGULAR OPENINGS
No scale



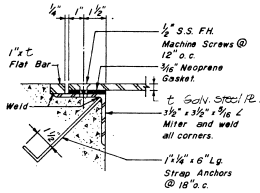

CONCRETE PIPE SUPPORT
(For Base Fittings)
No scale

FIXED LADDER SUPPORT
No Scale

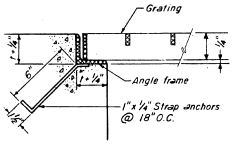



"A", "B" & "C" dimensions to suit equipment

EQUIPMENT PAD DETAIL
No scale

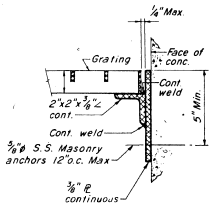



WATERTIGHT COVER PLATE DETAIL
No scale



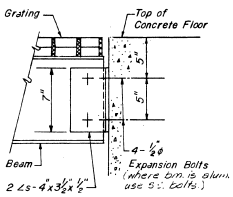

Note:
1. Band all edges and openings
2. Angle frame to be mitered and welded at corners.
3. Materials to Match Grating as Indicated on Plan.

GRATING SEAT DETAIL
No scale



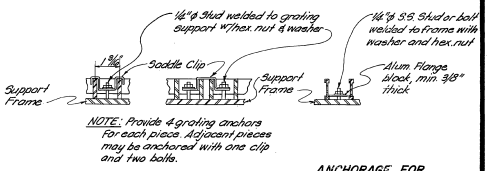

Note:
Materials to Match Grating as Indicated on Plan.

GRATING SUPPORT WITH BACK-UP PLATE
No scale




Note:
Materials as Indicated on Plan.

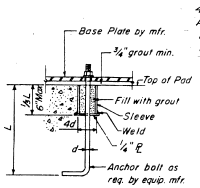
GRATING SUPPORT CONNECTION DETAIL
No scale



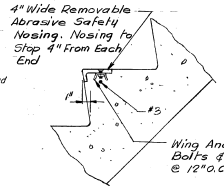

NOTE: Provide 4 grating anchors for each piece. Adjacent pieces may be anchored with one clip and two bolts.

ANCHORAGE FOR STEEL GRATING
No Scale

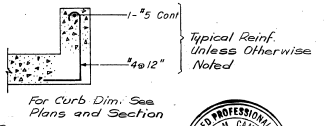
ANCHORAGE FOR ALUMINUM GRATING
No Scale



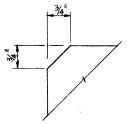

ANCHOR BOLT DETAIL
No scale




CONCRETE STEP DETAIL
(typical all concrete stairs)
No scale

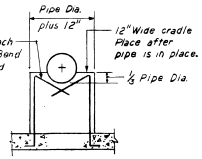



CURB DETAIL
No scale

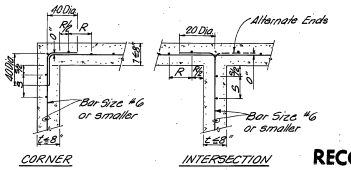



NOTE: All Exposed Conc. Corners to be chamfered unless otherwise noted.

CHAMFER DETAIL
N.T.S.

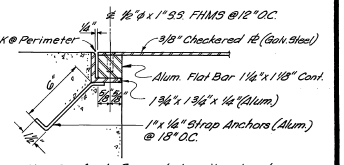



CONCRETE PIPE CRADLE
No scale




CORNER
INTERSECTION

C.M.U. OR CONCRETE WALL INTERSECTION DETAILS FOR SINGLE CURTAIN REINFORCEMENT
No Scale




NOTE: Angle Frame to be mitered and welded at corners.

FUTURE GRATING SEAT DETAILS
No Scale

RECORD DRAWING


CAMP DRESSER & McKEE INC.
Southwestern Regional Office
283 South Lake Avenue
Pasadena California 91101

CDM
environmental engineers, scientists, planners & management consultants


REGISTERED PROFESSIONAL ENGINEER CARLOS W. CAMPBELL NO. 13161 CIVIL STATE OF CALIFORNIA

| Approved by: | Date | Name |
|---|---|---|
| Structural | | |
| Architectural | | |
| Electrical | | |
| Instrumentation | | |
| HVAC | | |
| Project Engineer | 8/79 | ASR |
| Project Manager | 8/79 | DMS |

| MARK | REVISIONS | APPR | DATE |
|---|---|---|---|
| | | | |

DESIGNED BY EL DRAWN BY EG CHECKED BY VJP

CITY OF RIVERSIDE
PUBLIC WORKS DEPARTMENT

APPROVED BY: PRINCIPAL ENGINEER, PARK DEPARTMENT, TRAFFIC DIVISION, CHIEF P.W. ENGINEER

APPROVED BY: PUBLIC WORKS DIRECTOR
DATE 12-28-79

AS BUILT 10-31-82 WF

INDEXED WF

PRIMARY - SECONDARY REPLACEMENT CAPACITY PROJECT

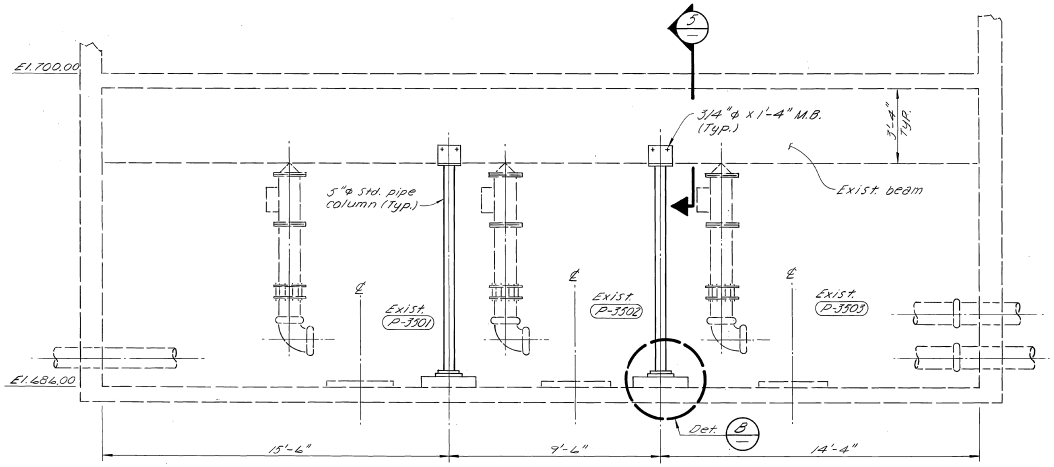
STRUCTURAL DETAILS I

HORIZ. SCALE: NO SCALE VERT. SCALE: 1" =

ACCOUNT NO. T-14

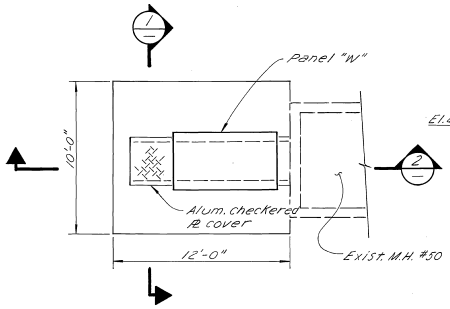
SHEET S-001 OF 18

FILE NO. 2012-7-099

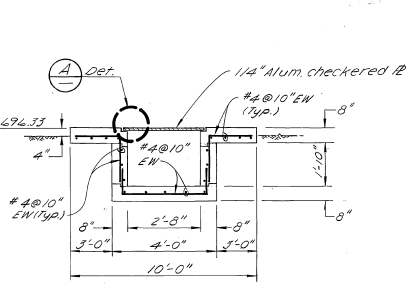



BLOWER / PUMP BLDG. NO. 1 (EXIST.) STRUCTURAL MODIFICATIONS
3/16"=1'-0"

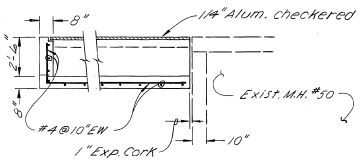
Note: Exist. piping & equipment are partially shown for clarity



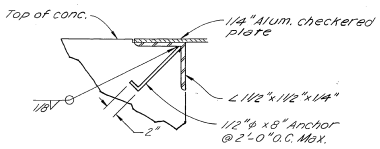

CONC. PAD FOR PANEL "W" - PLAN
N.T.S.



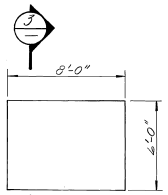

SECTION 1
N.T.S.



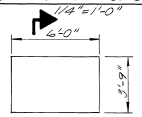

SECTION 2
N.T.S.



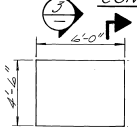

REBATE DETAIL A
N.T.S.



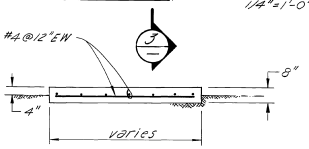

CONC. PAD FOR TRANSFORMER "T4"
1/4"=1'-0"



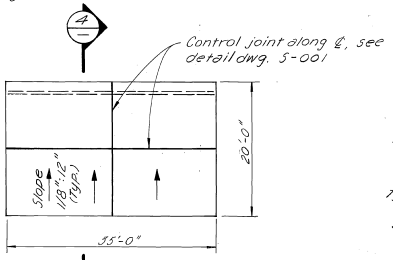

CONC. PAD FOR MCC "T"
1/4"=1'-0"



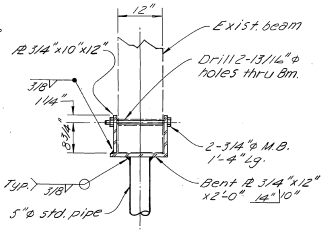

CONC. PAD FOR DAF CONT. PANEL
1/4"=1'-0"



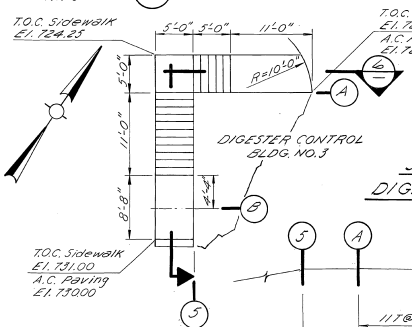

SECTION 3
N.T.S.



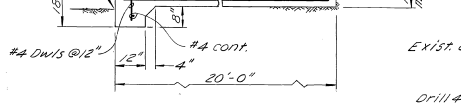

CONC. PAD FOR SWITCHGEAR "S6A" & "S6B" - PLAN
N.T.S.



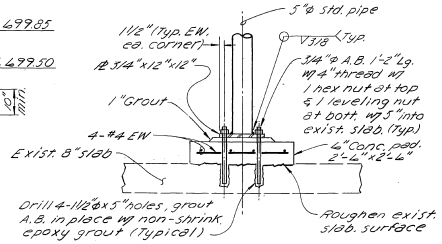

SECTION 5
3/4"=1'-0"



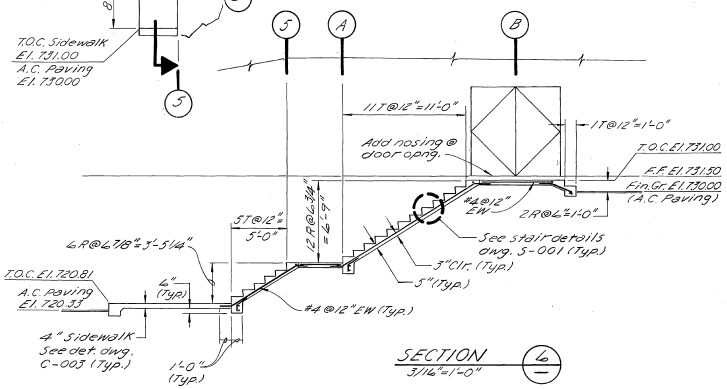

STAIRS PARTIAL PLAN
DIGESTER CONTROL BLDG. NO. 3




SECTION 4
N.T.S.




DETAIL 8
3/4"=1'-0"




SECTION 6
3/16"=1'-0"

Slope walkway 1/4":1'-0" to drain away from bldg.

**RECORD DRAWING**

#AS BUILT 10-31-82 LF

INDEXED LF

**CAMP DRESSER & McKEE INC.**
Southwestern Regional Office
283 South Lake Avenue
Pasadena California 91101

**CDM**
environmental engineers, scientists, planners & management consultants

| Approved by: | Date | Name |
|---|---|---|
| Structural | | |
| Architectural | | |
| Electrical | | |
| Instrumentation | | |
| HVAC | | |
| Project Engineer | 12/79 | ASR |
| Project Manager | 12/79 | DMS |

| MARK | REVISIONS | APPR. | DATE |
|---|---|---|---|
| | | | |

DESIGNED BY BW DRAWN BY EL CHECKED BY EC

**CITY OF RIVERSIDE**
PUBLIC WORKS DEPARTMENT

APPROVED BY: PRINCIPAL ENGINEER, PARK DEPARTMENT, TRAFFIC DIVISION, CHIEF P. W. ENGINEER

APPROVED BY: PUBLIC WORKS DIRECTOR
DATE 12-28-79

PRIMARY - SECONDARY REPLACEMENT CAPACITY PROJECT

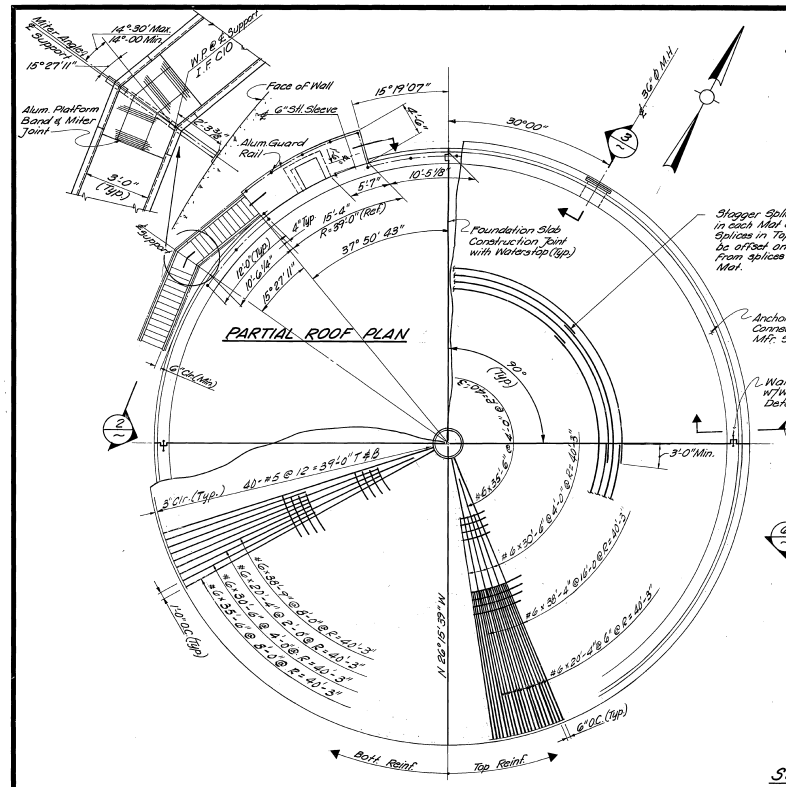
STRUCTURAL MISCELLANEOUS

ACCOUNT NO.

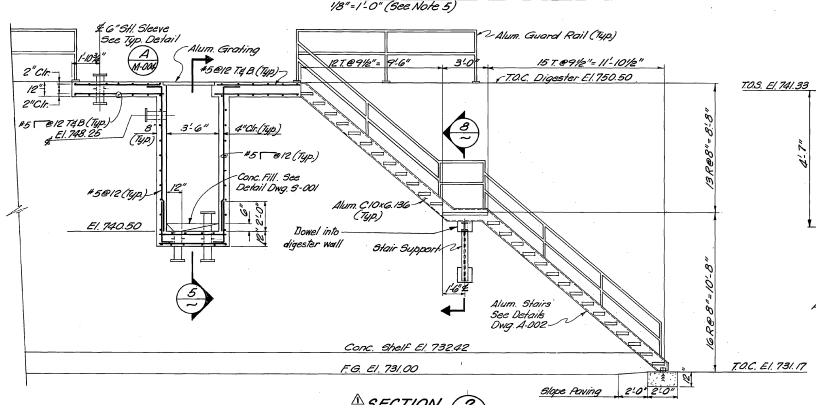
SHEET S-002 OF 18

HORIZ. SCALE: AS SHOWN VERT. SCALE: 1" -

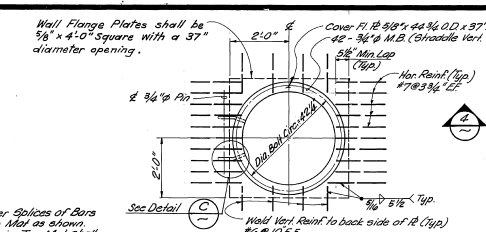
FILE NO. 2012-7-100



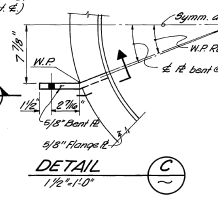

PARTIAL TANK FOUNDATION & ROOF PLAN
1/8"=1'-0" (See Note 5)



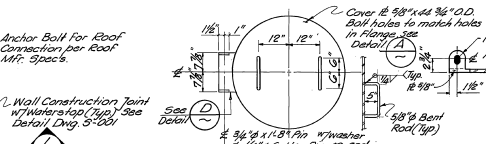

SECTION (2)
1/4"=1'-0"




M.H. COVER FLANGE Pl. DETAIL (A)
1/2"=1'-0"

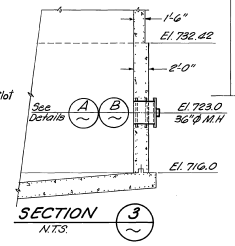



DETAIL (C)
1 1/2"=1'-0"

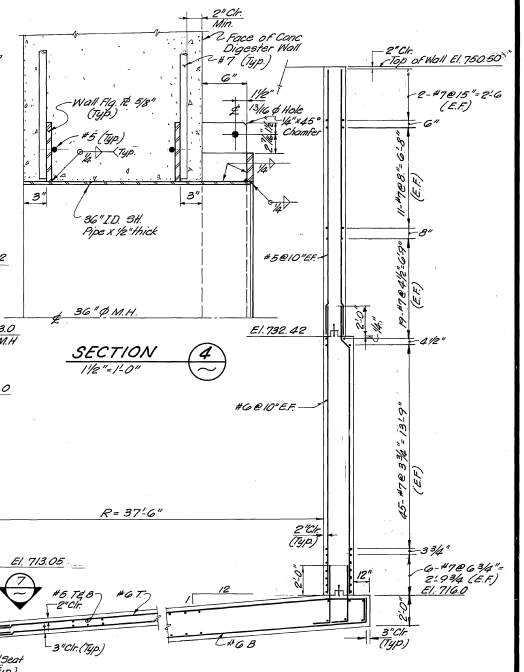



M.H. COVER Pl. DETAIL (B)
1/2"=1'-0"
(* SEE NOTE 7)

DETAIL (D)
1"=1'-0"



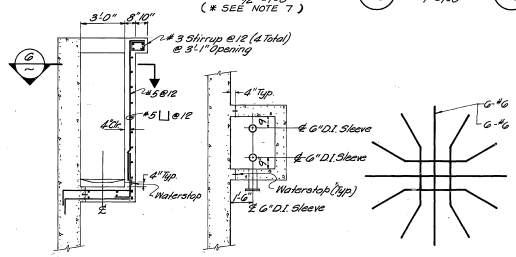

SECTION (3)
N.T.S.




SECTION (4)
1 1/2"=1'-0"

NOTE:
Install additional #6 & #7 Rebars x 6'-0" Horiz. at vert. construction Joints

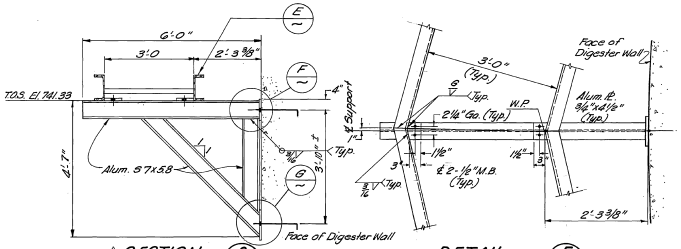
TYPICAL WALL SECTION (1)
1/4"=1'-0"




SECTION (5)
1/4"=1'-0"

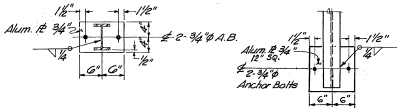
SECTION (6)
1/4"=1'-0"

SECTION (7)
1/4"=1'-0"




SECTION (8)
1/2"=1'-0"

DETAIL (E)
3/4"=1'-0"




DETAIL (F)
3/4"=1'-0"

DETAIL (G)
3/4"=1'-0"

NOTES: (1-4 Apply to 36" M.H.)

1. 1/8" Long Fiber Asbestos sheet Gasket 41 3/8" O.D. x 36" I.D. between Flange and Cover.
2. All Reinf. cut shall be welded to Wall Flange Pl. Actual Reinf. Location may vary from that shown.
3. Welds not shown shall be full penetration groove welds.
4. Hot-dip Galv. M.H. weldment. Galvanizing damaged by field welding shall be galvanized in field.
5. See Dwg. M-G05 for location of Pipe sleeves not shown.
6. Contractor may provide a water tight manhole conforming to AWWA D-100 Specifications. Either MH shown detailed or a Std. AWWA D-100 shall be tested for water tightness before installation.
7. Cover Plate material shall meet AWWA D-100 specification for finish and tolerances to provide a water tight connection with the MH Cover Flange Plate.
8. Digester interior vertical concrete walls & first foot of floor slab painted with 2 coats of coal tar epoxy.

**RECORD DRAWING**

* AS BUILT 10-31-82 LF

INCORPORATED LF

REGISTERED PROFESSIONAL ENGINEER CARLO M. CAMPOLONGO NO. 13161 CIVIL STATE OF CALIFORNIA

**CAMP DRESSER & McKEE INC.**
Southwestern Regional Office
283 South Lake Avenue
Pasadena California 91101

**CDM**
environmental engineers, scientists, planners & management consultants

| Approved by: | Date | Name |
|---|---|---|
| Structural | | |
| Architectural | | |
| Electrical | | |
| Instrumentation | | |
| HVAC | | |
| Project Engineer | 8/79 | ASR |
| Project Manager | 8/79 | DMS |

| MARK | REVISIONS | APPR. | DATE |
|---|---|---|---|
| 1 | AS BUILT REVISIONS | | 10-22-81 |

DESIGNED BY LBS DRAWN BY JTL CHECKED BY EC

**CITY OF RIVERSIDE**
PUBLIC WORKS DEPARTMENT

APPROVED BY: PRINCIPAL ENGINEER, PARK DEPARTMENT, TRAFFIC DIVISION, CHIEF P.W. ENGINEER

APPROVED BY: PUBLIC WORKS DIRECTOR
DATE 12-28-79

PRIMARY - SECONDARY REPLACEMENT CAPACITY PROJECT

DIGESTER NO. 5
PLAN & SECTIONS

ACCOUNT NO. T-14

SHEET S-601 OF 18

HORIZ. SCALE: AS SHOWN VERT. SCALE: 1" -

FILE NO. 2012-7-114

# SEISMIC CALCULATIONS

## Digester Result Summary

f'c = 4500 psi
fy = 50 ksi

$\phi_T$ = 0.9

## WALL HOOP STRESS

***Seismic Condition with Fixed Base)***

| depth from top | wall thick (in) | rebar ($in^2/ft$) | $\phi Tn$ (psi) | High Level Liquid = 32ft $T_{u,ASCE7-10}$ (psi) | $T_{u,BSE-1E}$ (psi) | **DCR ASCE 7-10** | **DCR BSE-1E** | Low Level Liquid = 26ft $T_{u,ASCE7-10}$ (psi) | $T_{u,BSE-1E}$ (psi) | **DCR ASCE 7-10** | **DCR BSE-1E** |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 - 3.00 | 18 | 0.96 | 200 | 70 | 64 | **0.35** | **0.32** | 42 | 38 | **0.21** | **0.19** |
| 3.00 - 10.33 | 18 | 1.80 | 375 | 267 | 253 | **0.71** | **0.67** | 108 | 102 | **0.29** | **0.27** |
| 10.33 - 17.08 | 18 | 3.20 | 667 | 470 | 447 | **0.71** | **0.67** | 292 | 278 | **0.44** | **0.42** |
| 17.08 - 31.28 | 24 | 3.84 | 600 | 547 | 521 | **0.91** | **0.87** | 390 | 373 | **0.65** | **0.62** |
| 31.28 - 34.5 | 24 | 2.13 | 333 | 205 | 195 | **0.62** | **0.59** | 142 | 136 | **0.43** | **0.41** |
| | | | | | | ***OK*** | ***OK*** | | | ***OK*** | ***OK*** |

***Seismic Condition with Pinned Base)***

| depth from top | wall thick (in) | rebar ($in^2/ft$) | $\phi Tn$ (psi) | High Level Liquid = 32ft $T_{u,ASCE7-10}$ (psi) | $T_{u,BSE-1E}$ (psi) | **DCR ASCE 7-10** | **DCR BSE-1E** | Low Level Liquid = 26ft $T_{u,ASCE7-10}$ (psi) | $T_{u,BSE-1E}$ (psi) | **DCR ASCE 7-10** | **DCR BSE-1E** |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 - 3.00 | 18 | 0.96 | 200 | 81 | 74 | **0.41** | **0.37** | 50 | 45 | **0.25** | **0.23** |
| 3.00 - 10.33 | 18 | 1.80 | 375 | 274 | 259 | **0.73** | **0.69** | 112 | 106 | **0.30** | **0.28** |
| 10.33 - 17.08 | 18 | 3.20 | 667 | 451 | 429 | **0.68** | **0.64** | 279 | 265 | **0.42** | **0.40** |
| 17.08 - 31.28 | 24 | 3.84 | 600 | 483 | 461 | **0.81** | **0.77** | 338 | 323 | **0.56** | **0.54** |
| 31.28 - 34.5 | 24 | 2.13 | 333 | 126 | 119 | **0.38** | **0.36** | 82 | 78 | **0.25** | **0.23** |
| | | | | | | ***OK*** | ***OK*** | | | ***OK*** | ***OK*** |

**Digester Result Summary**

## FOUNDATION RING STRESS

***Seismic Condition***

| | wall thick (in) | rebar ($in^2/ft$) | $\phi Tn$ (psi) | High Level Liquid = 32ft $T_{u,ASCE7-10}$ (psi) | $T_{u,BSE-1E}$ (psi) | DCR ASCE 7-10 | DCR BSE-1E | Low Level Liquid = 26ft $T_{u,ASCE7-10}$ (psi) | $T_{u,BSE-1E}$ (psi) | DCR ASCE 7-10 | DCR BSE-1E |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Fix-Base | 24 | 0.62 | 97 | 276 | 257 | **2.85** | **2.65** | 180 | 169 | **1.86** | **1.74** |
| Pin-Base | 24 | 0.62 | 97 | 247 | 235 | **2.55** | **2.43** | 199 | 191 | **2.05** | **1.97** |
| | | | | | | ***->NG*** | ***->NG*** | | | ***->NG*** | ***->NG*** |
| | | | | Liquid Level = 22ft | | | | Liquid Level = 16.5ft | | | |
| Fix-Base | 24 | 0.62 | 97 | 133 | 126 | **1.37** | **1.30** | 86 | 82 | **0.89** | **0.85** |
| Pin-Base | 24 | 0.62 | 97 | 174 | 168 | **1.80** | **1.73** | 149 | 145 | **1.54** | **1.50** |
| | | | | | | ***->NG*** | ***->NG*** | | | ***->NG*** | ***->NG*** |

## FOUNDATION SLAB BEARING PRESSURE (SERVICE LEVEL)

| | | Fixed-Base q (psi) | Fixed-Base q (psf) | Pinned-Base q (psi) | Pinned-Base q (psf) |
|---|---|---|---|---|---|
| **(ASCE 7-10)** | | | | | |
| High Level Liquid = 32ft | = | 32 | 4,666 | 35 | 5,026 |
| Low Level Liquid = 26ft | = | 25 | 3,643 | 28 | 4,018 |
| Liquid Level = 22ft | = | 22 | 3,125 | 25 | 3,528 |
| Liquid Level = 16.5ft | = | 18 | 2,534 | 21 | 3,010 |
| | = | | | | |
| **(BSE-1E)** | | | | | |
| High Level Liquid = 32ft | = | 31 | 4,522 | 34 | 4,882 |
| Low Level Liquid = 26ft | = | 25 | 3,571 | 27 | 3,946 |
| Liquid Level = 22ft | = | 21 | 3,067 | 24 | 3,470 |
| Liquid Level = 16.5ft | = | 17 | 2,506 | 21 | 2,981 |

## Digester Result Summary

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| concrete strength, $f'_c$ | = 4,500 | psi | Notes: | | Bar size | Diameter | Bar Area |
| reinf bar strength, $f_y$ | = 50,000 | psi | $t$ : | | # 3 | 0.375 | 0.11 |
| | | | $d_b$ : | Slab or Wall thickness | # 4 | 0.500 | 0.20 |
| Concrete Shear Stress, Vn | $= 2*(f'_c)^{0.5}$ | | $s$ : | Reinforcing bar diameter | # 5 | 0.625 | 0.31 |
| Rebar In-Plane Shear Stress, Vs | $= \rho * f_y$ | | $d$ : | Reinforcing bar spacing | # 6 | 0.750 | 0.44 |
| Tension Stress, Tn | $= \rho * f_y$ | | $A_s$ : | Distance to flexural reinforcing | # 7 | 0.875 | 0.60 |
| | | | $\rho$ : | Nominal reinforcing bar area | # 8 | 1.000 | 0.79 |
| Strength Reduction Factor for Bending, $\phi_B$ = | = 0.9 | | $M_n$ : | Reinforcement bar ratio = $A_s$ / ( 12 in * d ) | # 9 | 1.128 | 1.00 |
| Strength Reduction Factor for Shear, $\phi_v$ = | = 0.75 | | | Flexural capacity = $(\rho f_y d^2 * [1-(0.588 \rho f_y / f'_c)]$ | # 10 | 1.270 | 1.27 |
| | | | | | # 11 | 1.410 | 1.56 |

**OUT-OF-PLANE BENDING MOMENT**

| | Mu (kip-ft/ft) | t (in) | bar size | bar spacing (in) | bar cover (in) | $d_b$ (in) | $A_s$ ($in^2$/ft) | d (in) | ρ | $\phi M_n$ (kip-ft /ft) | DCR | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Wall Moment (Fixed-Base)*** | | | | | | | | | | | | |
| **(ASCE 7-10)** | | | | | | | | | | | | |
| High Liquid Level = 32ft | 72.9 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 1.72 | <-- NG |
| Low Liquid Level = 26ft | 53.2 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 1.25 | <-- NG |
| Liquid Level = 22ft | 44.2 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 1.04 | <-- NG |
| Liquid Level = 16.5ft | 39.5 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.93 | OK |
| **(BSE-1E)** | | | | | | | | | | | | |
| High Liquid Level = 32ft | 68.5 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 1.61 | <-- NG |
| Low Liquid Level = 26ft | 50.5 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 1.19 | <-- NG |
| Liquid Level = 22ft | 42.3 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 1.00 | OK |
| Liquid Level = 16.5ft | 39.2 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.92 | OK |
| ***Wall Moment (Pinned-Base)*** | | | | | | | | | | | | |
| **(ASCE 7-10)** | | | | | | | | | | | | |
| High Liquid Level = 32ft | 41.9 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.99 | OK |
| Low Liquid Level = 26ft | 32.4 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.76 | OK |
| Liquid Level = 22ft | 26.3 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.62 | OK |
| Liquid Level = 16.5ft | 17.7 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.42 | OK |
| **(BSE-1E)** | | | | | | | | | | | | |
| High Liquid Level = 32ft | 39.5 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.93 | OK |
| Low Liquid Level = 26ft | 30.6 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.72 | OK |
| Liquid Level = 22ft | 24.9 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.59 | OK |
| Liquid Level = 16.5ft | 16.6 | 24 | # 6 | 10 | 2.38 | 0.750 | 0.53 | 21.63 | 0.00204 | 42.42 | 0.39 | OK |

## Digester Result Summary

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Slab Bars in Radial Dir (Fixed-Base)*** | | | | | | | | | | | | |
| **(ASCE 7-10)** | | | | | | | | | | | | |
| High Liquid Level = 32ft (Bottom) | 61.7 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 1.74 | <-- NG |
| Low Liquid Level = 26ft (Bottom) | 49.3 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 1.39 | <-- NG |
| Liquid Level = 22ft (Bottom) | 45.9 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 1.30 | <-- NG |
| Liquid Level = 16.5ft (Bottom) | 42.8 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 1.21 | <-- NG |
| | | | | | | | | | | | | |
| High Liquid Level = 32ft (Top) | 34.3 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.49 | OK |
| Low Liquid Level = 26ft (Top) | 22.5 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.32 | OK |
| Liquid Level = 22ft (Top) | 17.2 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.25 | OK |
| Liquid Level = 16.5ft (Top)) | 12.5 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.18 | OK |
| | | | | | | | | | | | | |
| **(BSE-1E)** | | | | | | | | | | | | |
| High Liquid Level = 32ft (Bottom) | 57.9 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 1.63 | <-- NG |
| Low Liquid Level = 26ft (Bottom) | 47.7 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 1.35 | <-- NG |
| Liquid Level = 22ft (Bottom) | 44.8 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 1.26 | <-- NG |
| Liquid Level = 16.5ft (Bottom) | 42.2 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 1.19 | <-- NG |
| | | | | | | | | | | | | |
| High Liquid Level = 32ft (Top) | 31.9 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.46 | OK |
| Low Liquid Level = 26ft (Top) | 21.0 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.30 | OK |
| Liquid Level = 22ft (Top) | 16.3 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.23 | OK |
| Liquid Level = 16.5ft (Top)) | 12.0 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.17 | OK |
| | | | | | | | | | | | | |
| ***Slab Bars in Radial Dir (Pinned-Base)*** | | | | | | | | | | | | |
| **(ASCE 7-10)** | | | | | | | | | | | | |
| High Liquid Level = 32ft (Bottom) | 9.4 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 0.26 | OK |
| Low Liquid Level = 26ft (Bottom) | 5.5 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 0.16 | OK |
| Liquid Level = 22ft (Bottom) | 4.7 | | | | | | | | | | | |
| Liquid Level = 16.5ft (Bottom) | 3.9 | | | | | | | | | | | |
| | | | | | | | | | | | | |
| High Liquid Level - ASCE 7-10 (Top) | 41.4 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.59 | OK |
| Low Liquid Level - ASCE 7-10 (Top) | 31.5 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.45 | OK |
| Liquid Level = 22ft (Top) | 27.6 | | | | | | | | | | | |
| Liquid Level = 16.5ft (Top)) | 25.0 | | | | | | | | | | | |
| | | | | | | | | | | | | |
| **(BSE-1E)** | | | | | | | | | | | | |
| High Liquid Level - BSE-1E (Bottom) | 6.9 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 0.19 | OK |
| Low Liquid Level - BSE-1E (Bottom) | 5.4 | 24 | # 6 | 12 | 2.38 | 0.750 | 0.44 | 21.63 | 0.00170 | 35.43 | 0.15 | OK |
| Liquid Level = 22ft (Bottom) | 4.6 | | | | | | | | | | | |
| Liquid Level = 16.5ft (Bottom) | 3.9 | | | | | | | | | | | |
| | | | | | | | | | | | | |
| High Liquid Level - BSE-1E (Top) | 39.0 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.56 | OK |
| Low Liquid Level - BSE-1E (Top) | 30.1 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.43 | OK |
| Liquid Level = 22ft (Top) | 26.7 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.38 | OK |
| Liquid Level = 16.5ft (Top)) | 24.6 | 24 | # 6 | 6 | 2.38 | 0.750 | 0.88 | 21.63 | 0.00340 | 70.06 | 0.35 | OK |

## Digester Result Summary

**OUT-OF-PLANE SHEAR**

| | $V_{u,max}$ (psi) | $V_c$ (psi) | DCR | |
|---|---|---|---|---|
| ***Wall Shear (Fixed-Base)*** | | | | |
| **(ASCE 7-10)** | | | | |
| High Liquid Level = 32ft | 70 | 101 | **0.69** | **OK** |
| Low Liquid Level = 26ft | 56 | 101 | **0.55** | **OK** |
| **(BSE-1E)** | | | | |
| High Liquid Level = 32ft | 65 | 101 | **0.64** | **OK** |
| Low Liquid Level = 26ft | 53 | 101 | **0.52** | **OK** |
| ***Wall Shear (Pinned-Base)*** | | | | |
| **(ASCE 7-10)** | | | | |
| High Liquid Level = 32ft | 58 | 101 | **0.57** | **OK** |
| Low Liquid Level = 26ft | 44 | 101 | **0.43** | **OK** |
| **(BSE-1E)** | | | | |
| High Liquid Level = 32ft | 54 | 101 | **0.54** | **OK** |
| Low Liquid Level = 26ft | 41 | 101 | **0.41** | **OK** |

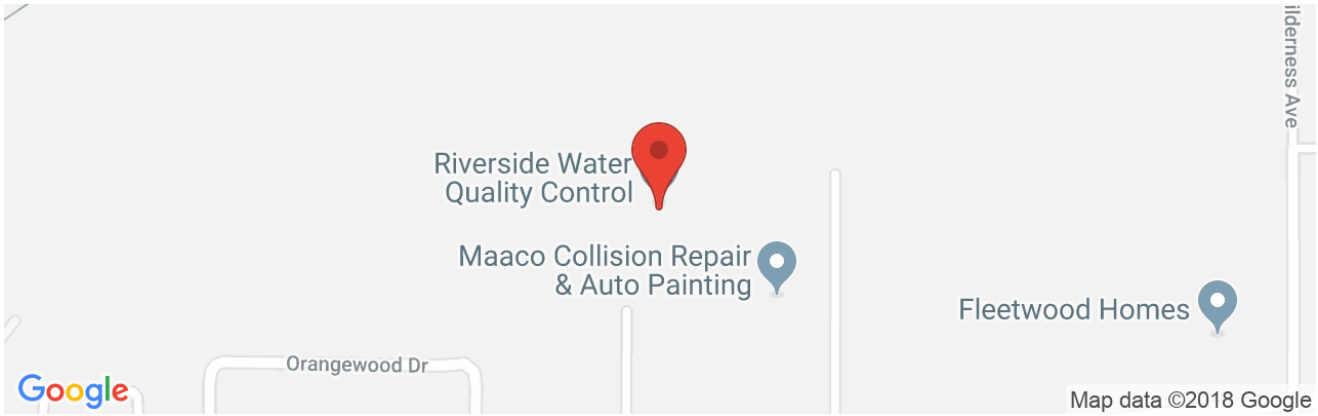
**IN-PLANE SHEAR STRESS CHECK**

| | $SXY_{max}$ (psi) | thick (in) | bar area ($in^2$) | bar spacing (in) | ρ | $V_s$ (psi) | $V_c$ (psi) | $\phi V_n$ (psi) | DCR | |
|---|---|---|---|---|---|---|---|---|---|---|
| ***Fixed Base*** | | | | | | | | | | |
| **(ASCE 7-10)** | | | | | | | | | | |
| High Liquid Level = 32ft | 197 | 24 | 2.13 | 12 | 0.0074 | 370 | 134 | 378 | **0.52** | **OK** |
| Low Liquid Level = 26ft | 144 | 24 | 2.13 | 12 | 0.0074 | 370 | 134 | 378 | **0.38** | **OK** |
| **(BSE-1E)** | | | | | | | | | | |
| High Liquid Level = 32ft | 183 | 24 | 2.13 | 12 | 0.0074 | 370 | 134 | 378 | **0.48** | **OK** |
| Low Liquid Level = 26ft | 134 | 24 | 2.13 | 12 | 0.0074 | 370 | 134 | 378 | **0.35** | **OK** |
| ***Pinned Base*** | | | | | | | | | | |
| **(ASCE 7-10)** | | | | | | | | | | |
| High Liquid Level = 32ft | 243 | 24 | 2.13 | 12 | 0.0074 | 370 | 134 | 378 | **0.64** | **OK** |
| Low Liquid Level = 26ft | 177 | 24 | 2.13 | 12 | 0.0074 | 370 | 134 | 378 | **0.47** | **OK** |
| **(BSE-1E)** | | | | | | | | | | |
| High Liquid Level = 32ft | 225 | 24 | 2.13 | 12 | 0.0074 | 370 | 134 | 378 | **0.60** | **OK** |
| Low Liquid Level = 26ft | 165 | 24 | 2.13 | 12 | 0.0074 | 370 | 134 | 378 | **0.44** | **OK** |

# Riverside - Digester

## 5950 Acorn St, Riverside, CA 92504, USA

**Latitude, Longitude: 33.962132, -117.45361919999999**

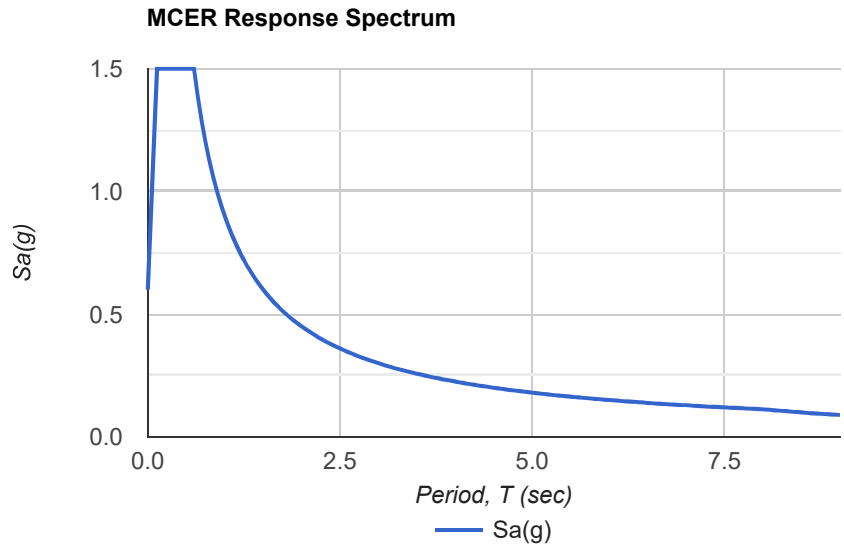


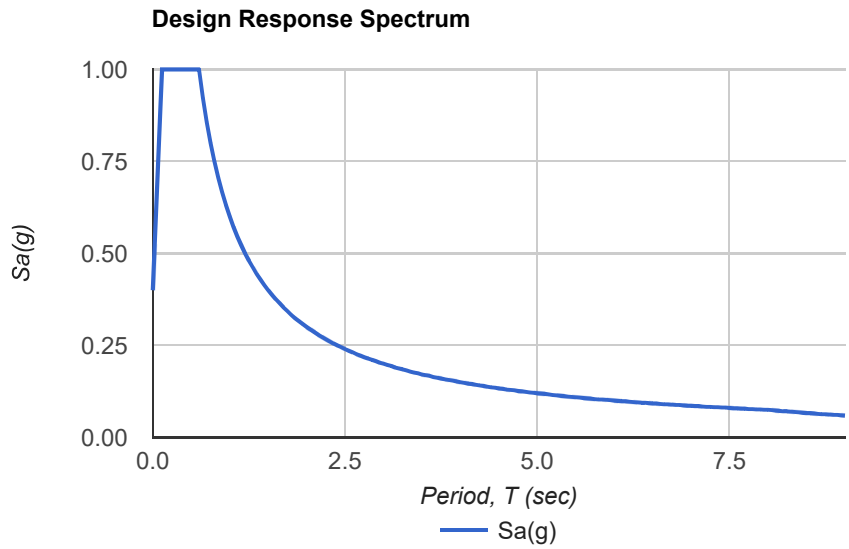

| | |
|---|---|
| **Date** | 12/5/2018, 10:46:39 AM |
| **Design Code Reference Document** | ASCE7-10 |
| **Risk Category** | IV |
| **Site Class** | D - Stiff Soil |

| Type | Value | Description |
|---|---|---|
| $S_S$ | 1.5 | $MCE_R$ ground motion. (for 0.2 second period) |
| $S_1$ | 0.6 | $MCE_R$ ground motion. (for 1.0s period) |
| $S_{MS}$ | 1.5 | Site-modified spectral acceleration value |
| $S_{M1}$ | 0.9 | Site-modified spectral acceleration value |
| $S_{DS}$ | 1 | Numeric seismic design value at 0.2 second SA |
| $S_{D1}$ | 0.6 | Numeric seismic design value at 1.0 second SA |

| Type | Value | Description |
|---|---|---|
| SDC | D | Seismic design category |
| $F_a$ | 1 | Site amplification factor at 0.2 second |
| $F_v$ | 1.5 | Site amplification factor at 1.0 second |
| PGA | 0.5 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 1 | Site amplification factor at PGA |
| $PGA_M$ | 0.5 | Site modified peak ground acceleration |
| $T_L$ | 8 | Long-period transition period in seconds |
| SsRT | 1.821 | Probabilistic risk-targeted ground motion. (0.2 second) |
| SsUH | 1.634 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration |
| SsD | 1.5 | Factored deterministic acceleration value. (0.2 second) |
| S1RT | 0.695 | Probabilistic risk-targeted ground motion. (1.0 second) |
| S1UH | 0.644 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration. |
| S1D | 0.6 | Factored deterministic acceleration value. (1.0 second) |
| PGAd | 0.5 | Factored deterministic acceleration value. (Peak Ground Acceleration) |

| Type | Value | Description |
|---|---|---|
| $C_{RS}$ | 1.114 | Mapped value of the risk coefficient at short periods |
| $C_{R1}$ | 1.079 | Mapped value of the risk coefficient at a period of 1 s |

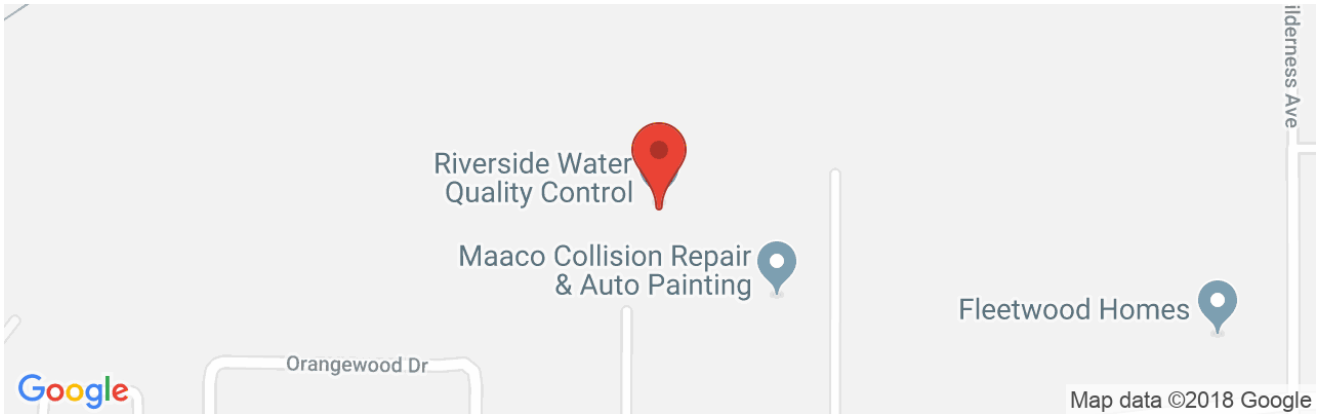

MCER Response Spectrum
1.5
1.0
0.5
0.0
Sa(g)
0.0
2.5
5.0
7.5
Period, T (sec)
Sa(g)


Design Response Spectrum
1.00
0.75
0.50
0.25
0.00
Sa(g)
0.0
2.5
5.0
7.5
Period, T (sec)
Sa(g)

# Riverside - Digester

## 5950 Acorn St, Riverside, CA 92504, USA

**Latitude, Longitude: 33.962132, -117.45361919999999**




| | |
|---|---|
| **Date** | 12/5/2018, 11:11:02 AM |
| **Design Code Reference Document** | ASCE41-13 |
| **Custom Probability** | |
| **Site Class** | D - Stiff Soil |

| Type | Description | Value |
|---|---|---|
| Hazard Level | | BSE-2N |
| $S_S$ | spectral response (0.2 s) | 1.5 |
| $S_1$ | spectral response (1.0 s) | 0.6 |
| $S_{XS}$ | site-modified spectral response (0.2 s) | 1.5 |
| $S_{X1}$ | site-modified spectral response (1.0 s) | 0.9 |
| $F_a$ | site amplification factor (0.2 s) | 1 |
| $F_v$ | site amplification factor (1.0 s) | 1.5 |
| ssuh | max direction uniform hazard (0.2 s) | 1.634 |
| crs | coefficient of risk (0.2 s) | 1.114 |
| ssrt | risk-targeted hazard (0.2 s) | 1.821 |
| ssd | deterministic hazard (0.2 s) | 1.5 |
| s1uh | max direction uniform hazard (1.0 s) | 0.644 |
| cr1 | coefficient of risk (1.0 s) | 1.079 |
| s1rt | risk-targeted hazard (1.0 s) | 0.695 |
| s1d | deterministic hazard (1.0 s) | 0.6 |

| Type | Description | Value |
|---|---|---|
| Hazard Level | | BSE-1N |
| $S_{XS}$ | site-modified spectral response (0.2 s) | 1 |
| $S_{X1}$ | site-modified spectral response (1.0 s) | 0.6 |

| Type | Description | Value |
|---|---|---|
| Hazard Level | | BSE-2E |
| $S_S$ | spectral response (0.2 s) | 1.255 |
| $S_1$ | spectral response (1.0 s) | 0.49 |
| $S_{XS}$ | site-modified spectral response (0.2 s) | 1.255 |
| $S_{X1}$ | site-modified spectral response (1.0 s) | 0.74 |
| $f_a$ | site amplification factor (0.2 s) | 1 |
| $f_v$ | site amplification factor (1.0 s) | 1.51 |

| Type | Description | Value |
|---|---|---|
| Hazard Level | | BSE-1E |
| $S_S$ | spectral response (0.2 s) | 0.77 |
| $S_1$ | spectral response (1.0 s) | 0.292 |
| $S_{XS}$ | site-modified spectral response (0.2 s) | 0.918 |
| $S_{X1}$ | site-modified spectral response (1.0 s) | 0.531 |
| $F_a$ | site amplification factor (0.2 s) | 1.192 |
| $F_v$ | site amplification factor (1.0 s) | 1.815 |

| Type | Description | Value |
|---|---|---|
| Hazard Level | | T-Sub-L Data |
| T-Sub-L | Long-period transition period in seconds | 8 |

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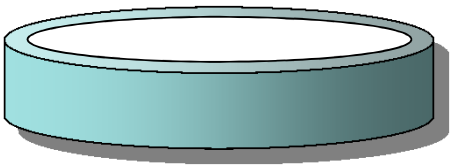
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | High Liquid; Wall Thick 24"; I=1.0 | | | |

**Hydrostatic and Hydrodynamic Seismic Analysis of a Circular Tank per ASCE 7-10 and the 2012 IBC code:**

Does groundwater exist in which to consider buoyancy? **No Groundwater**

**( Note: Response spectra values shall be strength level. )**

| | | | | | |
|---|---|---|---|---|---|
| tank inside diameter, D = | 75 | ft | | | |
| tank inside radius, R = | **37.5** | ft | tank wall mass, $W_w$ = | 2503.7 | kip |
| tank wall thickness, $t_w$ = | **24** | inch | wall c.g. relative to base, $h_w$ = | 17.250 | ft |
| tank wall height to underside of roof = | **34.5** | ft | | | |
| roof thickness = | **0** | inch | tank roof weight = | 0.0 | kip |
| misc roof weights included with seismic = | **0.05** | ksf | total misc roof weight = | 245.1 | kip |
| | | | total roof mass, $W_r$ = | 245.1 | kip |
| liquid height, $H_L$ = | **32** | ft | roof c.g. relative to base, $h_r$ = | 34.500 | ft |
| liquid specific gravity = | **1** | | | | |
| liquid density, $\gamma_L = (sp.gr.)*\gamma_w$ = | 0.0624 | $k/ft^3$ | liquid mass, $W_L = \pi R^2 * H_L * \gamma_L$ = | 8821.6 | kip |
| acceleration due to gravity, g = | 32.17 | $ft/sec^2$ | | | |
| liquid mass density, $\rho_L = \gamma_L / g$ = | 0.00194 | $k\text{-}sec^2/ft^4$ | | | |
| concrete strength, $f'_c$ = | **4.5** | ksi | | | |
| concrete density, $\gamma_c$ = | 0.150 | $k/ft^3$ | | | |
| concrete modulus of elasticity, $E_c$ = | 3823.7 | ksi | | | |
| concrete mass density, $\rho_c = \gamma_c / g$ = | 0.00466 | $k\text{-}sec^2/ft^4$ | | | |
| Seismic: | | | | | |
| Structure Risk Category = | **2** | | | | |
| Importance factor, I = | **1** | | | | |
| Response modification factor, $R_i$ = | **1.874** | | | | |
| Response modification factor, $R_c$ = | **1.609** | | | | |



tank inside diameter, D = 75 ft

( acceleration values from a maximum considered earthquake )

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, $S_{DS}$ = **1** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, $S_{D1}$ = **0.6** *g

1). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$C_w = 0.09375 + 0.2039\left(\frac{H_L}{D}\right) - 0.1034\left(\frac{H_L}{D}\right)^2 - 0.1253\left(\frac{H_L}{D}\right)^3 + 0.1267\left(\frac{H_L}{D}\right)^4 - 0.03186\left(\frac{H_L}{D}\right)^5 = 0.15594$$

$C_I = C_w * 10*((t_w/12)/R)^{1/2}$ = 0.1559*10*(24/12/37.5)^½ = 0.3601

$\omega_i = C_I * 12/H_L*(E_c / \rho_c)^{1/2}$ = 0.3601*12 / 32*( 3823.7 / 0.00466 )^½ = 122.2949 rad/sec

impulsive period of oscillation, $T_i = 2\pi / \omega_i = 2\pi /$ 122.2949 = 0.0514 sec

design factored spectral response acceleration for impulsive mass ( 5% damping ), $S_{ai} = S_{DS}$ = 1 g

$$\lambda = \sqrt{3.68\ g\ \tanh\left(3.68\left(\frac{H_L}{D}\right)\right)} =$$ (3.68*32.17*tanh(3.68*(32/75))^½ = 10.4195

convective circular frequency, $\omega_c = \frac{\lambda}{\sqrt{D}} =$ 10.4195 / ( 75)^½ = 1.2031 rad/sec

convective period of sloshing, $T_c = 2\pi / \omega_c = 2\pi /$ 1.2031 = 5.2223 sec

Long transition period (from map figure 22-12 ASCE 7), $T_L$ = **8** sec.

design spectral response acceleration for convective mass ( 0.5% damping ), $S_{ac}$ = 1.5 * Sd1 / Tc = 0.1723 g

effective mass coeff., $\varepsilon = 0.0151\left(\frac{D}{H_L}\right)^2 - 0.1908\left(\frac{D}{H_L}\right) + 1.021$ , but $\leq$ 1.0 = 0.6568

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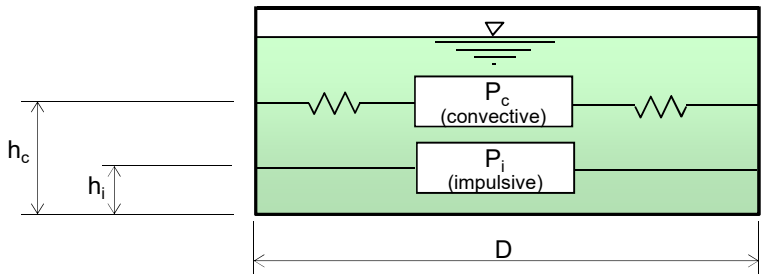
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | High Liquid; Wall Thick 24"; I=1.0 | | | |




Dynamic Model

| | | |
|---|---|---|
| D = | 75 | ft |
| $H_L$ = | 32 | ft |
| $W_L$ = | 8821.6 | kip |
| $D / H_L$ = | 2.34375 | |
| $H_L / D$ = | 0.42667 | |

2). lateral fluid impulsive force:

equivalent impulsive mass component, $W_i = W_L \left( \dfrac{\tanh\left(0.866 \dfrac{D}{H_L}\right)}{0.866 \dfrac{D}{H_L}} \right) =$ 4198.8 kip

height above base to the impulsive lateral force, hi (EBP) = HL * 0.375 = 12 ft

hi (IBP) = HL * {{(0.866*D/HL)/(2*tanh(0.866*D/HL))} -1/8 } = 29.616 ft

impulsive force, $P_i = \left( \dfrac{S_{ai}\ I}{R_i} \right) W_i =$ (1 * 1 / 1.874) * 4198.8 = 2240.6 kip

impulsive force moment excluding bottom pressure , $M_{i(EBP)} = P_i * h_{i(EBP)}$ = 2240.6 * 12 = 26887.2 ft-k

impulsive force moment including bottom pressure , $M_{i(IBP)} = P_i * h_{i(IBP)}$ = 2240.6 * 29.616 = 66357.6 ft-k

3). lateral fluid convective force:

equivalent convective mass component, $W_c = W_L \left( 0.23 \left( \dfrac{D}{H_L} \right) \tanh\left( 3.68 \left( \dfrac{H_L}{D} \right) \right) \right) =$ 4360.9 kip

height above base to convective lateral force, $h_{c\ (EBP)} = H_L \left( 1 - \dfrac{\cosh\left(3.68\left(\dfrac{H_L}{D}\right)\right) - 1}{3.68\left(\dfrac{H_L}{D}\right)\sinh\left(3.68\left(\dfrac{H_L}{D}\right)\right)} \right) =$ 18.638 ft

$h_{c(IBP)} = H_L \left( 1 - \dfrac{\cosh\left(3.68\left(\dfrac{H_L}{D}\right)\right) - 2.01}{3.68\left(\dfrac{H_L}{D}\right)\sinh\left(3.68\left(\dfrac{H_L}{D}\right)\right)} \right) =$ 27.59 ft

convective force, $P_c = \left( \dfrac{S_{ac}\ I}{R_c} \right) W_c =$ ( 0.1723 * 1 / 1.609 ) * 4360.9 = 467.0 kip

convective force moment excluding bottom pressure , $M_{c(EBP)} = P_c * h_{c(EBP)}$ = 467 * 18.638 = 8703.9 ft-k

convective force moment including bottom pressure , $M_{c(IBP)} = P_c * h_{c(IBP)}$ = 467 * 27.59 = 12884.5 ft-k

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|---|---|---|---|---|---|
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| **CHKD:** | | **DESCRIPTION:** | Digester Evaluation | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | High Liquid; Wall Thick 24"; I=1.0 | | |

4). lateral inertia force of the accelerating wall:

tank wall mass, $W_w$ = 2503.7 kip

wall c.g. relative to base, $h_w$ = 17.250 ft

wall inertia force, $P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_i}\right) W_w =$ ( 1 * 1 * 0.6568 / 1.874 ) * 2503.7 = 877.4 kip

wall inertia force moment, $M_w = P_w * h_w$ = 877.4 * 17.25 = 15135.2 ft-k

5). lateral inertia force of the accelerating roof:

total roof mass, $W_r$ = 245.1 kip

roof c.g. relative to base, $h_r$ = 34.5 ft

roof inertia force, $P_r = \left(\frac{S_{ai}\ I}{R_i}\right) W_r =$ ( 1 * 1 / 1.874 ) * 245.1 = 130.8 kip

roof inertia force moment, $M_r = P_r * h_r$ = 130.8 * 34.5 = 4512.6 ft-k

6). total base shear:

$$V = \sqrt{(P_i + P_w + P_r)^2 + P_c^2}$$

V = ( ( 2240.6 + 877.4 + 130.8 )² + ( 467 )² )^½ = 3282.2 kip

7). total moment at the base excluding bottom pressure (EBP):

$$M_b = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

Mb = (( 26887.2 + 15135.2 + 4512.6 )² + ( 8703.9 )² )^½ = 47342.0 ft-k

8). total moment at the base including bottom pressure (IBP):

$$M_o = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

Mo = (( 66357.6 + 15135.2 + 4512.6 )² + ( 12884.5 )² )^½ = 86965.2 ft-k

9). maximum wave slosh height displacement: ( see ASCE-10, 15.7.6.1 notes c and d )

( Risk Category = 2 ) I = 1 ,use TL = 4 ,Sd1 = 0.6 ,Tc = 5.2223

$S_{ac}$ = 1.5 * Sd1 * TL / Tc² = 0.132 *g

$d_{(max)}$ = 0.42 ( D )( $S_{ac}$ I ) = 0.42 * ( 75 ) * ( 0.132 * 1 ) = 4.16 ft

( minimum freeboard see table 15.7-3 of ASCE 7 ) , d(min) = No minimum req'd

Wave height is greater than the freeboard of 2.5-ft. Check effects of wave spillage.

10). Vertical acceleration:

design horizontal accereration, $S_{DS}$ = 1 *g

period of vibration, $T_v = 2\pi*(\gamma_L*D*H_L^2 / (24g*t_w*E_c))^{1/2}$ = 0.0517 sec

Ts = $S_{D1}$ / $S_{DS}$ = 0.6 / 1 = 0.6 sec

vertical acceleration (per ACI 350 para 9.4.3), for $T_V \le T_S$ then $C_t = S_{DS}$, for $T_V \succ T_S$ then $C_t = \frac{S_{D1}}{T_V}$

therefore, vertical spectral response acceleration, $S_{av} = C_t$ = 1.0000 *g

per ASCE 7-10 para. 15.7.7.2(b), use I = $R_i$ = b = 1.0

Design vertical acceleration, ü = $\frac{S_{av}\ I\ b}{R_i}$ = 1*1*1/1 = 1.0000 g

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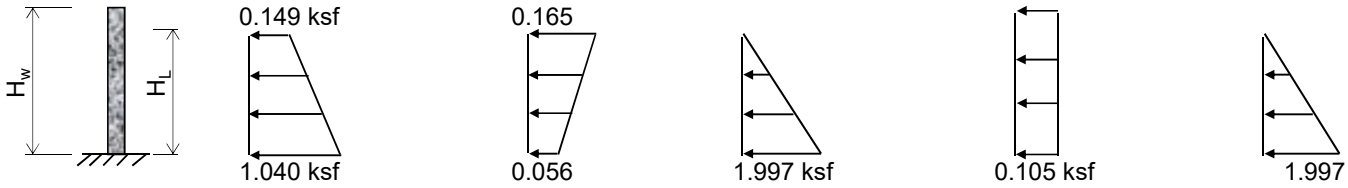
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | High Liquid; Wall Thick 24"; I=1.0 | | | |

12). vertical pressure distribution on a unit width using the linear distribution of ACI 350 sec 5.3:




$H_w$ = 34.5
$H_L$ = 32

impulsive | convective | vertical acceleration | wall inertia | hydrostatic

impulsive pressure:

$$p_{iy} = \frac{2\left(\frac{P_i}{2}\right)\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{\pi R H_L^2} \cos\theta =$$

use θ = 0°

| | | |
|---|---|---|
| impulsive force, $P_i$ = | 2240.6 | kip |
| $h_i$ = | 12 | ft |
| at y = $H_L$, $p_{iy}$ = | 0.149 | ksf |
| at base y = 0, $p_{iy}$ = | 1.040 | ksf |

convective pressure:

$$p_{cy} = \frac{16\left(\frac{P_c}{2}\right)\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{9\,\pi\,R\,H_L^2} \cos\theta =$$

use θ = 0°

| | | |
|---|---|---|
| convective force, $P_c$ = | 467.0 | kip |
| $h_c$ = | 18.638 | ft |
| at y = $H_L$, $p_{cy}$ = | 0.165 | ksf |
| at base y = 0, $p_{cy}$ = | 0.056 | ksf |

vertical acceleration pressure:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

| | | |
|---|---|---|
| vertical acceleration, ü = | 1 | g |
| at y = $H_L$, $p_{vy}$ = | 0 | ksf |
| at base y = 0, $p_{vy}$ = | 1.997 | ksf |

wall inertia pressure:

$$p_{wy} = \frac{S_{ai}\ I\ \varepsilon\,\gamma_c\,(t_w/12)}{R_i} =$$

| | | |
|---|---|---|
| $p_{wy}$ = | 0.3505 * ( $\gamma_c$ * $t_w$ ) | |
| at y = $H_w$, $p_{wy}$ = | 0.105 | ksf |
| at base y = 0, $p_{wy}$ = | 0.105 | ksf |

hydrostatic pressure:

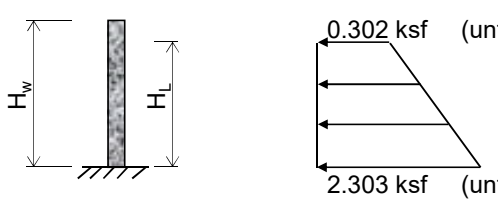
$$q_{hy} = \gamma_L\,(H_L - y) =$$

| | | |
|---|---|---|
| at y = $H_L$, $q_{hy}$ = | 0 | ksf |
| at base y = 0, $q_{hy}$ = | 1.997 | ksf |

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

| | | |
|---|---|---|
| at y = $H_w$, $p_y$ = | 0.302 | ksf |
| at base y = 0, $p_y$ = | 2.303 | ksf |




(unfactored load = 0.302 / 1.4 = 0.216 ksf)

(unfactored load = 2.303 / 1.4 = 1.645 ksf)

resultant dynamic pressures

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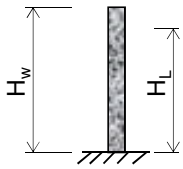
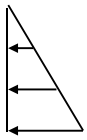
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | High Liquid; Wall Thick 24"; I=1.0 | | | |

13). load cases:

a). hydrostatic water load case:

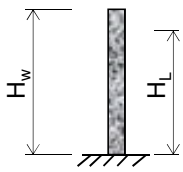


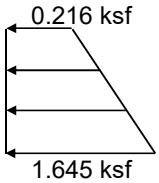



q1 = 1.997 ksf
hydrostatic

triangular pressure = 1.997 ksf

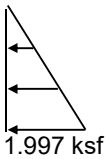
b). seismic load case:
equivalent unfactored dynamic + static pressure loadings… equivalent loading ( unfactored )



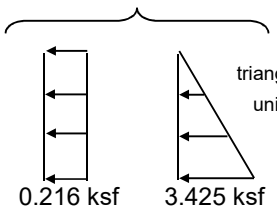

1.645 ksf
dynamic

+



1.997 ksf
hydrostatic

=



0.216 ksf
q2

3.425 ksf
q3

triangular pressure = 3.425 ksf
uniform pressure = 0.216 ksf

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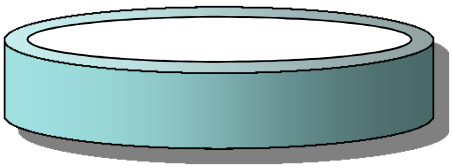
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | Low Liquid; Wall Thick 24"; I=1.0 | | | |

**Hydrostatic and Hydrodynamic Seismic Analysis of a Circular Tank per ASCE 7-10 and the 2012 IBC code:**

Does groundwater exist in which to consider buoyancy? **No Groundwater**

**( Note: Response spectra values shall be strength level. )**

| | | | | | |
|---|---|---|---|---|---|
| tank inside diameter, D = | 75 | ft | | | |
| tank inside radius, R = | **37.5** | ft | tank wall mass, $W_w$ = | 2503.7 | kip |
| tank wall thickness, $t_w$ = | **24** | inch | wall c.g. relative to base, $h_w$ = | 17.250 | ft |
| tank wall height to underside of roof = | **34.5** | ft | | | |
| roof thickness = | **0** | inch | tank roof weight = | 0.0 | kip |
| misc roof weights included with seismic = | **0.05** | ksf | total misc roof weight = | 245.1 | kip |
| | | | total roof mass, $W_r$ = | 245.1 | kip |
| liquid height, $H_L$ = | **26** | ft | roof c.g. relative to base, $h_r$ = | 34.500 | ft |
| liquid specific gravity = | **1** | | | | |
| liquid density, $\gamma_L = (sp.gr.)*\gamma_w$ = | 0.0624 | $k/ft^3$ | liquid mass, $W_L = \pi R^2 * H_L * \gamma_L$ = | 7167.5 | kip |
| acceleration due to gravity, g = | 32.17 | $ft/sec^2$ | | | |
| liquid mass density, $\rho_L = \gamma_L / g$ = | 0.00194 | $k\text{-}sec^2/ft^4$ | | | |
| concrete strength, $f'_c$ = | **4.5** | ksi | | | |
| concrete density, $\gamma_c$ = | 0.150 | $k/ft^3$ | | | |
| concrete modulus of elasticity, $E_c$ = | 3823.7 | ksi | | | |
| concrete mass density, $\rho_c = \gamma_c / g$ = | 0.00466 | $k\text{-}sec^2/ft^4$ | | | |
| Seismic: | | | | | |
| Structure Risk Category = | **2** | | | | |
| Importance factor, I = | **1** | | | | |
| Response modification factor, $R_i$ = | **2** | | | | |
| Response modification factor, $R_c$ = | **1.5** | | | | |



tank inside diameter, D = 75 ft

( acceleration values from a maximum considered earthquake )

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, $S_{DS}$ = **1** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, $S_{D1}$ = **0.6** *g

1). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$C_w = 0.09375 + 0.2039\left(\frac{H_L}{D}\right) - 0.1034\left(\frac{H_L}{D}\right)^2 - 0.1253\left(\frac{H_L}{D}\right)^3 + 0.1267\left(\frac{H_L}{D}\right)^4 - 0.03186\left(\frac{H_L}{D}\right)^5 = 0.14846$$

$C_I = C_w * 10*((t_w/12)/R)^{1/2}$ = 0.1485*10*(24/12/37.5)^½ = 0.3429

$\omega_i = C_I * 12/H_L*(E_c / \rho_c)^{1/2}$ = 0.3429*12 / 26*( 3823.7 / 0.00466 )^½ = 143.2962 rad/sec

impulsive period of oscillation, $T_i = 2\pi / \omega_i = 2\pi /$ 143.2962 = 0.0438 sec

design factored spectral response acceleration for impulsive mass ( 5% damping ), $S_{ai} = S_{DS}$ = 1 g

$$\lambda = \sqrt{3.68\ g\ \tanh\left(3.68\left(\frac{H_L}{D}\right)\right)} =$$ (3.68*32.17*tanh(3.68*(26/75))^½ = 10.0628

convective circular frequency, $\omega_c = \frac{\lambda}{\sqrt{D}} =$ 10.0628 / ( 75)^½ = 1.1620 rad/sec

convective period of sloshing, $T_c = 2\pi / \omega_c = 2\pi /$ 1.162 = 5.4074 sec

Long transition period (from map figure 22-12 ASCE 7), $T_L$ = **8** sec.

design spectral response acceleration for convective mass ( 0.5% damping ), $S_{ac}$ = 1.5 * Sd1 / Tc = 0.1664 g

effective mass coeff., $\varepsilon = 0.0151\left(\frac{D}{H_L}\right)^2 - 0.1908\left(\frac{D}{H_L}\right) + 1.021$ , but $\leq$ 1.0 = 0.5963

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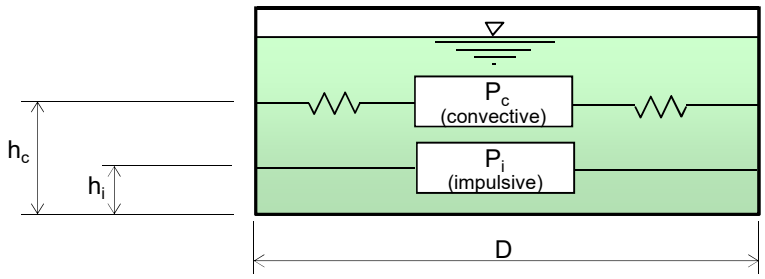
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | Low Liquid; Wall Thick 24"; I=1.0 | | | |




Dynamic Model

| | | |
|---|---|---|
| D = | 75 | ft |
| $H_L$ = | 26 | ft |
| $W_L$ = | 7167.5 | kip |
| D / $H_L$ = | 2.88462 | |
| $H_L$ / D = | 0.34667 | |

2). lateral fluid impulsive force:

equivalent impulsive mass component, $W_i = W_L \left( \dfrac{\tanh\left(0.866 \dfrac{D}{H_L}\right)}{0.866 \dfrac{D}{H_L}} \right) =$ 2830.7 kip

height above base to the impulsive lateral force, hi (EBP) = HL * 0.375 = 9.75 ft

hi (IBP) = HL * {{(0.866*D/HL)/(2*tanh(0.866*D/HL))} -1/8 } = 29.667 ft

impulsive force, $P_i = \left( \dfrac{S_{ai}\ I}{R_i} \right) W_i =$ (1 * 1 / 2) * 2830.7 = 1415.4 kip

impulsive force moment excluding bottom pressure , $M_{i(EBP)} = P_i * h_{i(EBP)}$ = 1415.4 * 9.75 = 13800.2 ft-k

impulsive force moment including bottom pressure , $M_{i(IBP)} = P_i * h_{i(IBP)}$ = 1415.4 * 29.667 = 41990.7 ft-k

3). lateral fluid convective force:

equivalent convective mass component, $W_c = W_L \left( 0.23 \left( \dfrac{D}{H_L} \right) \tanh\left( 3.68 \left( \dfrac{H_L}{D} \right) \right) \right) =$ 4067.5 kip

height above base to convective lateral force, $h_{c\ (EBP)} = H_L \left( 1 - \dfrac{\cosh\left(3.68\left(\dfrac{H_L}{D}\right)\right) - 1}{3.68\left(\dfrac{H_L}{D}\right)\sinh\left(3.68\left(\dfrac{H_L}{D}\right)\right)} \right) =$ 14.517 ft

$h_{c(IBP)} = H_L \left( 1 - \dfrac{\cosh\left(3.68\left(\dfrac{H_L}{D}\right)\right) - 2.01}{3.68\left(\dfrac{H_L}{D}\right)\sinh\left(3.68\left(\dfrac{H_L}{D}\right)\right)} \right) =$ 26.984 ft

convective force, $P_c = \left( \dfrac{S_{ac}\ I}{R_c} \right) W_c =$ ( 0.1664 * 1 / 1.5 ) * 4067.5 = 451.2 kip

convective force moment excluding bottom pressure , $M_{c(EBP)} = P_c * h_{c(EBP)}$ = 451.2 * 14.517 = 6550.1 ft-k

convective force moment including bottom pressure , $M_{c(IBP)} = P_c * h_{c(IBP)}$ = 451.2 * 26.984 = 12175.2 ft-k

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| | | | | | |
|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | Digester Evaluation | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | Low Liquid; Wall Thick 24"; I=1.0 | | | | |

4). lateral inertia force of the accelerating wall:

tank wall mass, $W_w$ = 2503.7 kip

wall c.g. relative to base, $h_w$ = 17.250 ft

wall inertia force, $P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_i}\right) W_w =$ ( 1 * 1 * 0.5963 / 2 ) * 2503.7 = 746.4 kip

wall inertia force moment, $M_w = P_w * h_w$ = 746.4 * 17.25 = 12875.4 ft-k

5). lateral inertia force of the accelerating roof:

total roof mass, $W_r$ = 245.1 kip

roof c.g. relative to base, $h_r$ = 34.5 ft

roof inertia force, $P_r = \left(\frac{S_{ai}\ I}{R_i}\right) W_r =$ ( 1 * 1 / 2 ) * 245.1 = 122.6 kip

roof inertia force moment, $M_r = P_r * h_r$ = 122.6 * 34.5 = 4229.7 ft-k

6). total base shear:

$$V = \sqrt{(P_i + P_w + P_r)^2 + P_c^2}$$

V = ( ( 1415.4 + 746.4 + 122.6 )² + ( 451.2 )² )^½ = 2328.5 kip

7). total moment at the base excluding bottom pressure (EBP):

$$M_b = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

Mb = (( 13800.2 + 12875.4 + 4229.7 )² + ( 6550.1 )² )^½ = 31591.8 ft-k

8). total moment at the base including bottom pressure (IBP):

$$M_o = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

Mo = (( 41990.7 + 12875.4 + 4229.7 )² + ( 12175.2 )² )^½ = 60337.0 ft-k

9). maximum wave slosh height displacement: ( see ASCE-10, 15.7.6.1 notes c and d )

( Risk Category = 2 ) I = 1 ,use TL = 4 ,Sd1 = 0.6 ,Tc = 5.4074

$S_{ac}$ = 1.5 * Sd1 * TL / Tc² = 0.1231 *g

$d_{(max)}$ = 0.42 ( D )( $S_{ac}$ I ) = 0.42 * ( 75 ) * ( 0.1231 * 1 ) = 3.88 ft

( minimum freeboard see table 15.7-3 of ASCE 7 ) , d(min) = No minimum req'd

10). Vertical acceleration:

design horizontal accereration, $S_{DS}$ = 1 *g

period of vibration, $T_v = 2\pi*(\gamma_L*D*H_L^2 / (24g*t_w*E_c))^{1/2}$ = 0.0420 sec

Ts = $S_{D1}$ / $S_{DS}$ = 0.6 / 1 = 0.6 sec

vertical acceleration (per ACI 350 para 9.4.3), for $T_V \le T_S$ then $C_t = S_{DS}$, for $T_V \succ T_S$ then $C_t = \frac{S_{D1}}{T_V}$

therefore, vertical spectral response acceleration, $S_{av} = C_t$ = 1.0000 *g

per ASCE 7-10 para. 15.7.7.2(b), use I = $R_i$ = b = 1.0

Design vertical acceleration, ü = $\frac{S_{av}\ I\ b}{R_i}$ = 1*1*1/1 = 1.0000 g

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| | | | | | | | |
|---|---|---|---|---|---|---|---|
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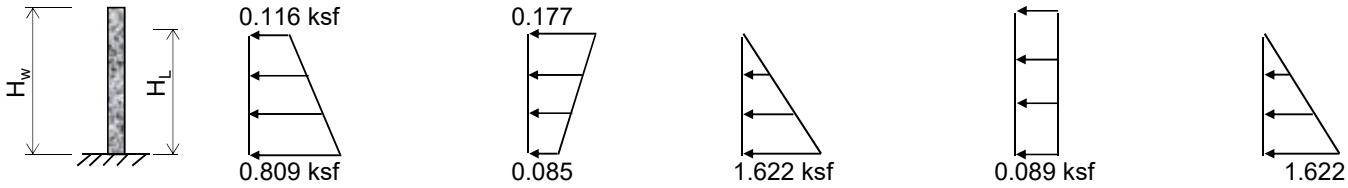
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | Low Liquid; Wall Thick 24"; I=1.0 | | | |

12). vertical pressure distribution on a unit width using the linear distribution of ACI 350 sec 5.3:




$H_w$ = 34.5
$H_L$ = 26

impulsive | convective | vertical acceleration | wall inertia | hydrostatic

impulsive pressure:

$$p_{iy} = \frac{2\left(\frac{P_i}{2}\right)\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{\pi R H_L^2} \cos\theta =$$

use θ = 0°

| | | |
|---|---|---|
| impulsive force, $P_i$ = | 1415.4 | kip |
| $h_i$ = | 9.75 | ft |
| at y = $H_L$, $p_{iy}$ = | 0.116 | ksf |
| at base y = 0, $p_{iy}$ = | 0.809 | ksf |

convective pressure:

$$p_{cy} = \frac{16\left(\frac{P_c}{2}\right)\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{9\,\pi\,R\,H_L^2} \cos\theta =$$

use θ = 0°

| | | |
|---|---|---|
| convective force, $P_c$ = | 451.2 | kip |
| $h_c$ = | 14.517 | ft |
| at y = $H_L$, $p_{cy}$ = | 0.177 | ksf |
| at base y = 0, $p_{cy}$ = | 0.085 | ksf |

vertical acceleration pressure:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

| | | |
|---|---|---|
| vertical acceleration, ü = | 1 | g |
| at y = $H_L$, $p_{vy}$ = | 0 | ksf |
| at base y = 0, $p_{vy}$ = | 1.622 | ksf |

wall inertia pressure:

$$p_{wy} = \frac{S_{ai}\ I\ \varepsilon\,\gamma_c\,(t_w/12)}{R_i} =$$

| | | |
|---|---|---|
| $p_{wy}$ = | 0.2981 * ( $\gamma_c$ * $t_w$ ) | |
| at y = $H_w$, $p_{wy}$ = | 0.089 | ksf |
| at base y = 0, $p_{wy}$ = | 0.089 | ksf |

hydrostatic pressure:

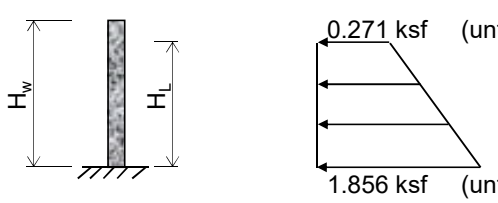
$$q_{hy} = \gamma_L\,(H_L - y) =$$

| | | |
|---|---|---|
| at y = $H_L$, $q_{hy}$ = | 0 | ksf |
| at base y = 0, $q_{hy}$ = | 1.622 | ksf |

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

| | | |
|---|---|---|
| at y = $H_w$, $p_y$ = | 0.271 | ksf |
| at base y = 0, $p_y$ = | 1.856 | ksf |




0.271 ksf (unfactored load = 0.271 / 1.4 = 0.193 ksf)

1.856 ksf (unfactored load = 1.856 / 1.4 = 1.326 ksf)

resultant dynamic pressures

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| | | | | | | | |
|---|---|---|---|---|---|---|---|
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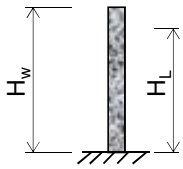
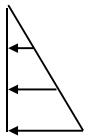
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | Low Liquid; Wall Thick 24"; I=1.0 | | | |

13). load cases:

a). hydrostatic water load case:

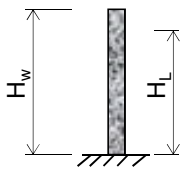


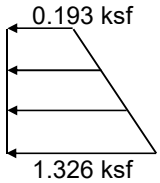



q1 = 1.622 ksf
hydrostatic

triangular pressure = 1.622 ksf

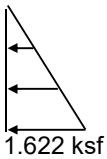
b). seismic load case:
equivalent unfactored dynamic + static pressure loadings… equivalent loading ( unfactored )



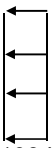

1.326 ksf
dynamic

+

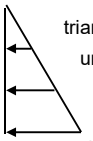


1.622 ksf
hydrostatic

=



0.193 ksf
q2



2.755 ksf
q3

triangular pressure = 2.755 ksf
uniform pressure = 0.193 ksf

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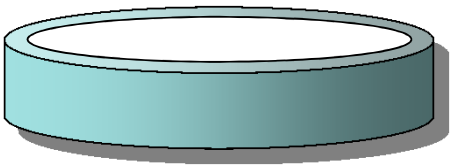
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | 22ft Liquid; Wall Thick 24"; I=1.0 | | | |

**Hydrostatic and Hydrodynamic Seismic Analysis of a Circular Tank per ASCE 7-10 and the 2012 IBC code:**

Does groundwater exist in which to consider buoyancy? **No Groundwater**

**( Note: Response spectra values shall be strength level. )**

| | | | | | |
|---|---|---|---|---|---|
| tank inside diameter, D = | 75 | ft | | | |
| tank inside radius, R = | **37.5** | ft | tank wall mass, $W_w$ = | 2503.7 | kip |
| tank wall thickness, $t_w$ = | **24** | inch | wall c.g. relative to base, $h_w$ = | 17.250 | ft |
| tank wall height to underside of roof = | **34.5** | ft | | | |
| roof thickness = | **0** | inch | tank roof weight = | 0.0 | kip |
| misc roof weights included with seismic = | **0.05** | ksf | total misc roof weight = | 245.1 | kip |
| | | | total roof mass, $W_r$ = | 245.1 | kip |
| liquid height, $H_L$ = | **22** | ft | roof c.g. relative to base, $h_r$ = | 34.500 | ft |
| liquid specific gravity = | **1** | | | | |
| liquid density, $\gamma_L = (sp.gr.)*\gamma_w$ = | 0.0624 | $k/ft^3$ | liquid mass, $W_L = \pi R^2 * H_L * \gamma_L$ = | 6064.8 | kip |
| acceleration due to gravity, g = | 32.17 | $ft/sec^2$ | | | |
| liquid mass density, $\rho_L = \gamma_L / g$ = | 0.00194 | $k\text{-}sec^2/ft^4$ | | | |
| concrete strength, $f'_c$ = | **4.5** | ksi | | | |
| concrete density, $\gamma_c$ = | 0.150 | $k/ft^3$ | | | |
| concrete modulus of elasticity, $E_c$ = | 3823.7 | ksi | | | |
| concrete mass density, $\rho_c = \gamma_c / g$ = | 0.00466 | $k\text{-}sec^2/ft^4$ | | | |
| Seismic: | | | | | |
| Structure Risk Category = | **2** | | | | |
| Importance factor, I = | **1** | | | | |
| Response modification factor, $R_i$ = | **2** | | | | |
| Response modification factor, $R_c$ = | **1.5** | | | | |



tank inside diameter, D = 75 ft

( acceleration values from a maximum considered earthquake )

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, $S_{DS}$ = **1** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, $S_{D1}$ = **0.6** *g

1). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$C_w = 0.09375 + 0.2039\left(\frac{H_L}{D}\right) - 0.1034\left(\frac{H_L}{D}\right)^2 - 0.1253\left(\frac{H_L}{D}\right)^3 + 0.1267\left(\frac{H_L}{D}\right)^4 - 0.03186\left(\frac{H_L}{D}\right)^5 = 0.14237$$

$C_I = C_w * 10*((t_w/12)/R)^{1/2}$ = 0.1424*10*(24/12/37.5)^½ = 0.3288

$\omega_i = C_I * 12/H_L*(E_c / \rho_c)^{1/2}$ = 0.3288*12 / 22*( 3823.7 / 0.00466 )^½ = 162.4040 rad/sec

impulsive period of oscillation, $T_i = 2\pi / \omega_i = 2\pi /$ 162.404 = 0.0387 sec

design factored spectral response acceleration for impulsive mass ( 5% damping ), $S_{ai} = S_{DS}$ = 1 g

$$\lambda = \sqrt{3.68\ g\ \tanh\left(3.68\left(\frac{H_L}{D}\right)\right)} =$$ (3.68*32.17*tanh(3.68*(22/75))^½ = 9.6892

convective circular frequency, $\omega_c = \frac{\lambda}{\sqrt{D}} =$ 9.6892 / ( 75)^½ = 1.1188 rad/sec

convective period of sloshing, $T_c = 2\pi / \omega_c = 2\pi /$ 1.1188 = 5.6160 sec

Long transition period (from map figure 22-12 ASCE 7), $T_L$ = **8** sec.

design spectral response acceleration for convective mass ( 0.5% damping ), $S_{ac}$ = 1.5 * Sd1 / Tc = 0.1603 g

effective mass coeff., $\varepsilon = 0.0151\left(\frac{D}{H_L}\right)^2 - 0.1908\left(\frac{D}{H_L}\right) + 1.021$ , but $\leq$ 1.0 = 0.5460

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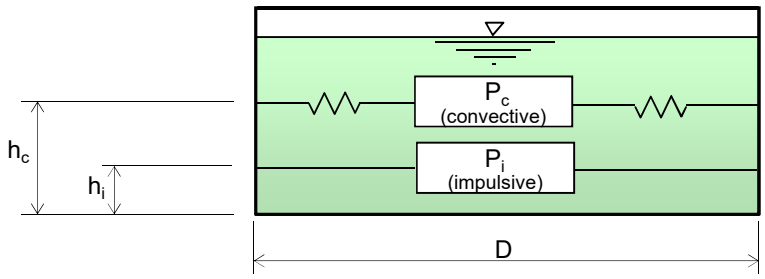
| **CHKD:** | | **DESCRIPTION:** | Digester Evaluation | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | 22ft Liquid; Wall Thick 24"; I=1.0 | | | | |




Dynamic Model

| | | |
|---|---|---|
| D = | 75 | ft |
| $H_L$ = | 22 | ft |
| $W_L$ = | 6064.8 | kip |
| D / $H_L$ = | 3.40909 | |
| $H_L$ / D = | 0.29333 | |

2). lateral fluid impulsive force:

equivalent impulsive mass component, $W_i = W_L \left( \dfrac{\tanh\left(0.866 \dfrac{D}{H_L}\right)}{0.866 \dfrac{D}{H_L}} \right) =$ 2043.1 kip

height above base to the impulsive lateral force, hi (EBP) = HL * 0.375 = 8.25 ft

hi (IBP) = HL * {{(0.866*D/HL)/(2*tanh(0.866*D/HL))} -1/8 } = 29.903 ft

impulsive force, $P_i = \left( \dfrac{S_{ai}\ I}{R_i} \right) W_i =$ (1 * 1 / 2) * 2043.1 = 1021.6 kip

impulsive force moment excluding bottom pressure , $M_{i(EBP)} = P_i * h_{i(EBP)}$ = 1021.6 * 8.25 = 8428.2 ft-k

impulsive force moment including bottom pressure , $M_{i(IBP)} = P_i * h_{i(IBP)}$ = 1021.6 * 29.903 = 30548.9 ft-k

3). lateral fluid convective force:

equivalent convective mass component, $W_c = W_L \left( 0.23 \left( \dfrac{D}{H_L} \right) \tanh\left( 3.68 \left( \dfrac{H_L}{D} \right) \right) \right) =$ 3771 kip

height above base to convective lateral force, $h_{c\ (EBP)} = H_L \left( 1 - \dfrac{\cosh\left(3.68\left(\dfrac{H_L}{D}\right)\right) - 1}{3.68\left(\dfrac{H_L}{D}\right)\sinh\left(3.68\left(\dfrac{H_L}{D}\right)\right)} \right) =$ 11.957 ft

$h_{c(IBP)} = H_L \left( 1 - \dfrac{\cosh\left(3.68\left(\dfrac{H_L}{D}\right)\right) - 2.01}{3.68\left(\dfrac{H_L}{D}\right)\sinh\left(3.68\left(\dfrac{H_L}{D}\right)\right)} \right) =$ 27.771 ft

convective force, $P_c = \left( \dfrac{S_{ac}\ I}{R_c} \right) W_c =$ ( 0.1603 * 1 / 1.5 ) * 3771 = 403.0 kip

convective force moment excluding bottom pressure , $M_{c(EBP)} = P_c * h_{c(EBP)}$ = 403 * 11.957 = 4818.7 ft-k

convective force moment including bottom pressure , $M_{c(IBP)} = P_c * h_{c(IBP)}$ = 403 * 27.771 = 11191.7 ft-k

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| **CHKD:** | | **DESCRIPTION:** | Digester Evaluation | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | 22ft Liquid; Wall Thick 24"; I=1.0 | | | | |

4). lateral inertia force of the accelerating wall:

tank wall mass, $W_w$ = 2503.7 kip

wall c.g. relative to base, $h_w$ = 17.250 ft

wall inertia force, $P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_i}\right) W_w =$ ( 1 * 1 * 0.546 / 2 ) * 2503.7 = 683.6 kip

wall inertia force moment, $M_w = P_w * h_w =$ 683.6 * 17.25 = 11792.1 ft-k

5). lateral inertia force of the accelerating roof:

total roof mass, $W_r$ = 245.1 kip

roof c.g. relative to base, $h_r$ = 34.5 ft

roof inertia force, $P_r = \left(\frac{S_{ai}\ I}{R_i}\right) W_r =$ ( 1 * 1 / 2 ) * 245.1 = 122.6 kip

roof inertia force moment, $M_r = P_r * h_r =$ 122.6 * 34.5 = 4229.7 ft-k

6). total base shear:

$$V = \sqrt{(P_i + P_w + P_r)^2 + P_c^2}$$

V = ( ( 1021.6 + 683.6 + 122.6 )² + ( 403 )² )^½ = 1871.7 kip

7). total moment at the base excluding bottom pressure (EBP):

$$M_b = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

Mb = (( 8428.2 + 11792.1 + 4229.7 )² + ( 4818.7 )² )^½ = 24920.3 ft-k

8). total moment at the base including bottom pressure (IBP):

$$M_o = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

Mo = (( 30548.9 + 11792.1 + 4229.7 )² + ( 11191.7 )² )^½ = 47896.6 ft-k

9). maximum wave slosh height displacement: ( see ASCE-10, 15.7.6.1 notes c and d )

( Risk Category = 2 ) I = 1 ,use TL = 4 ,Sd1 = 0.6 ,Tc = 5.616

$S_{ac}$ = 1.5 * Sd1 * TL / Tc² = 0.1141 *g

$d_{(max)}$ = 0.42 ( D )( $S_{ac}$ I ) = 0.42 * ( 75 ) * ( 0.1141 * 1 ) = 3.59 ft

( minimum freeboard see table 15.7-3 of ASCE 7 ) , d(min) = No minimum req'd

10). Vertical acceleration:

design horizontal accereration, $S_{DS}$ = 1 *g

period of vibration, $T_v = 2\pi*(\gamma_L*D*H_L^2 / (24g*t_w*E_c))^{1/2}$ = 0.0355 sec

Ts = $S_{D1}$ / $S_{DS}$ = 0.6 / 1 = 0.6 sec

vertical acceleration (per ACI 350 para 9.4.3), for $T_V \le T_S$ then $C_t = S_{DS}$, for $T_V \succ T_S$ then $C_t = \frac{S_{D1}}{T_V}$

therefore, vertical spectral response acceleration, $S_{av} = C_t$ = 1.0000 *g

per ASCE 7-10 para. 15.7.7.2(b), use I = $R_i$ = b = 1.0

Design vertical acceleration, ü = $\frac{S_{av}\ I\ b}{R_i}$ = 1*1*1/1 = 1.0000 g

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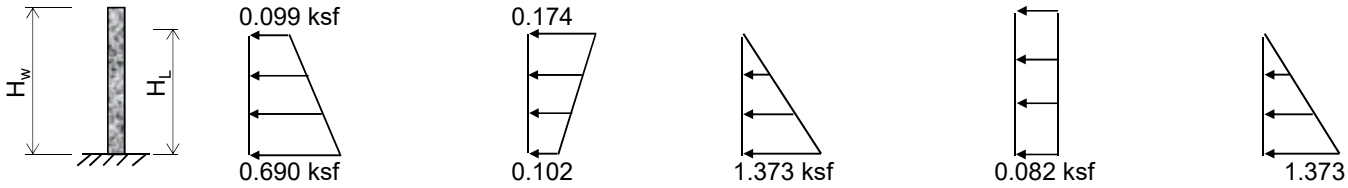
| **DESIGN TASK:** | | | | 22ft Liquid; Wall Thick 24"; I=1.0 | | | |

12). vertical pressure distribution on a unit width using the linear distribution of ACI 350 sec 5.3:




$H_w$ = 34.5
$H_L$ = 22

impulsive | convective | vertical acceleration | wall inertia | hydrostatic

impulsive pressure:

$$p_{iy} = \frac{2\left(\frac{P_i}{2}\right)\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{\pi R H_L^2} \cos\theta =$$

use θ = 0°

| | | |
|---|---|---|
| impulsive force, $P_i$ = | 1021.6 | kip |
| $h_i$ = | 8.25 | ft |
| at y = $H_L$, $p_{iy}$ = | 0.099 | ksf |
| at base y = 0, $p_{iy}$ = | 0.690 | ksf |

convective pressure:

$$p_{cy} = \frac{16\left(\frac{P_c}{2}\right)\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{9\,\pi\,R\,H_L^2} \cos\theta =$$

use θ = 0°

| | | |
|---|---|---|
| convective force, $P_c$ = | 403.0 | kip |
| $h_c$ = | 11.957 | ft |
| at y = $H_L$, $p_{cy}$ = | 0.174 | ksf |
| at base y = 0, $p_{cy}$ = | 0.102 | ksf |

vertical acceleration pressure:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

| | | |
|---|---|---|
| vertical acceleration, ü = | 1 | g |
| at y = $H_L$, $p_{vy}$ = | 0 | ksf |
| at base y = 0, $p_{vy}$ = | 1.373 | ksf |

wall inertia pressure:

$$p_{wy} = \frac{S_{ai}\ I\ \varepsilon\,\gamma_c\,(t_w/12)}{R_i} =$$

| | | |
|---|---|---|
| $p_{wy}$ = | 0.2730 * ( $\gamma_c$ * $t_w$ ) | |
| at y = $H_w$, $p_{wy}$ = | 0.082 | ksf |
| at base y = 0, $p_{wy}$ = | 0.082 | ksf |

hydrostatic pressure:

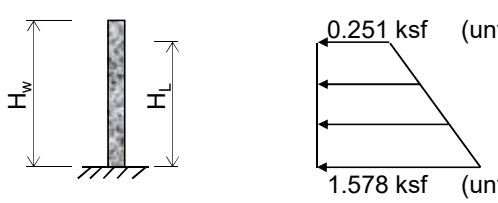
$$q_{hy} = \gamma_L\,(H_L - y) =$$

| | | |
|---|---|---|
| at y = $H_L$, $q_{hy}$ = | 0 | ksf |
| at base y = 0, $q_{hy}$ = | 1.373 | ksf |

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

| | | |
|---|---|---|
| at y = $H_w$, $p_y$ = | 0.251 | ksf |
| at base y = 0, $p_y$ = | 1.578 | ksf |




(unfactored load = 0.251 / 1.4 = 0.179 ksf)

(unfactored load = 1.578 / 1.4 = 1.127 ksf)

resultant dynamic pressures

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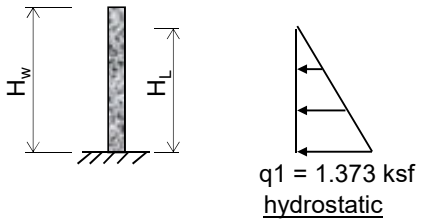
| **DESIGN TASK:** | | | | 22ft Liquid; Wall Thick 24"; I=1.0 | | | |

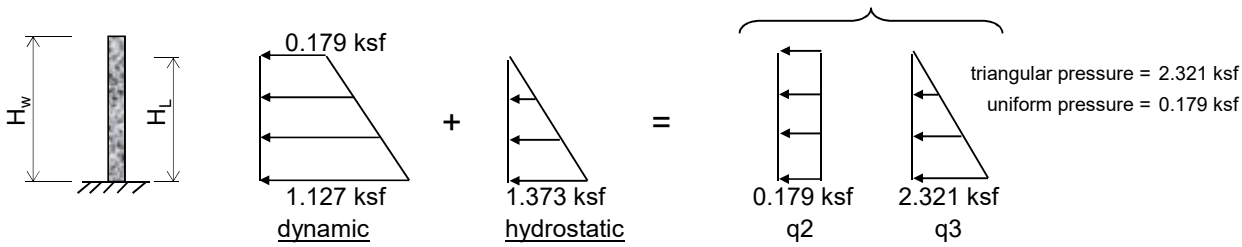
13). load cases:

a). hydrostatic water load case:




triangular pressure = 1.373 ksf

b). seismic load case:
equivalent unfactored dynamic + static pressure loadings… equivalent loading ( unfactored )



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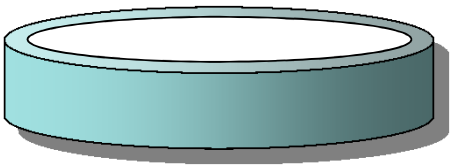
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | 16.5ft Liquid; Wall Thick 24"; I=1.0 | | | |

**Hydrostatic and Hydrodynamic Seismic Analysis of a Circular Tank per ASCE 7-10 and the 2012 IBC code:**

Does groundwater exist in which to consider buoyancy? **No Groundwater**

**( Note: Response spectra values shall be strength level. )**

| | | | | | |
|---|---|---|---|---|---|
| tank inside diameter, D = | 75 | ft | | | |
| tank inside radius, R = | **37.5** | ft | tank wall mass, $W_w$ = | 2503.7 | kip |
| tank wall thickness, $t_w$ = | **24** | inch | wall c.g. relative to base, $h_w$ = | 17.250 | ft |
| tank wall height to underside of roof = | **34.5** | ft | | | |
| roof thickness = | **0** | inch | tank roof weight = | 0.0 | kip |
| misc roof weights included with seismic = | **0.05** | ksf | total misc roof weight = | 245.1 | kip |
| | | | total roof mass, $W_r$ = | 245.1 | kip |
| liquid height, $H_L$ = | **16.5** | ft | roof c.g. relative to base, $h_r$ = | 34.500 | ft |
| liquid specific gravity = | **1** | | | | |
| liquid density, $\gamma_L = (sp.gr.)*\gamma_w$ = | 0.0624 | $k/ft^3$ | liquid mass, $W_L = \pi R^2 * H_L * \gamma_L$ = | 4548.6 | kip |
| acceleration due to gravity, g = | 32.17 | $ft/sec^2$ | | | |
| liquid mass density, $\rho_L = \gamma_L / g$ = | 0.00194 | $k\text{-}sec^2/ft^4$ | | | |
| concrete strength, $f'_c$ = | **4.5** | ksi | | | |
| concrete density, $\gamma_c$ = | 0.150 | $k/ft^3$ | | | |
| concrete modulus of elasticity, $E_c$ = | 3823.7 | ksi | | | |
| concrete mass density, $\rho_c = \gamma_c / g$ = | 0.00466 | $k\text{-}sec^2/ft^4$ | | | |
| Seismic: | | | | | |
| Structure Risk Category = | **2** | | | | |
| Importance factor, I = | **1** | | | | |
| Response modification factor, $R_i$ = | **2** | | | | |
| Response modification factor, $R_c$ = | **1.5** | | | | |



tank inside diameter, D = 75 ft

( acceleration values from a maximum considered earthquake )

Deisgn, 5% damped, spectral response acceleration at the short period of 0.2-second, $S_{DS}$ = **1** *g

Deisgn, 5% damped, spectral response acceleration at a period of 1-second, $S_{D1}$ = **0.6** *g

1). Dynamic properties, Spectral amplification factors, and Effective mass coefficient:

$$C_w = 0.09375 + 0.2039\left(\frac{H_L}{D}\right) - 0.1034\left(\frac{H_L}{D}\right)^2 - 0.1253\left(\frac{H_L}{D}\right)^3 + 0.1267\left(\frac{H_L}{D}\right)^4 - 0.03186\left(\frac{H_L}{D}\right)^5 = 0.13255$$

$C_I = C_w * 10*((t_w/12)/R)^{1/2}$ = 0.1325*10*(24/12/37.5)^½ = 0.3061

$\omega_i = C_I * 12/H_L*(E_c / \rho_c)^{1/2}$ = 0.3061*12 / 16.5*( 3823.7 / 0.00466 )^½ = 201.6024 rad/sec

impulsive period of oscillation, $T_i = 2\pi / \omega_i = 2\pi /$ 201.6024 = 0.0312 sec

design factored spectral response acceleration for impulsive mass ( 5% damping ), $S_{ai} = S_{DS}$ = 1 g

$$\lambda = \sqrt{3.68\ g\ \tanh\left(3.68\left(\frac{H_L}{D}\right)\right)} =$$ (3.68*32.17*tanh(3.68*(16.5/75))^½ = 8.9019

convective circular frequency, $\omega_c = \frac{\lambda}{\sqrt{D}} =$ 8.9019 / ( 75)^½ = 1.0279 rad/sec

convective period of sloshing, $T_c = 2\pi / \omega_c = 2\pi /$ 1.0279 = 6.1126 sec

Long transition period (from map figure 22-12 ASCE 7), $T_L$ = **8** sec.

design spectral response acceleration for convective mass ( 0.5% damping ), $S_{ac}$ = 1.5 * Sd1 / Tc = 0.1472 g

effective mass coeff., $\varepsilon = 0.0151\left(\frac{D}{H_L}\right)^2 - 0.1908\left(\frac{D}{H_L}\right) + 1.021$ , but $\leq$ 1.0 = 0.4657

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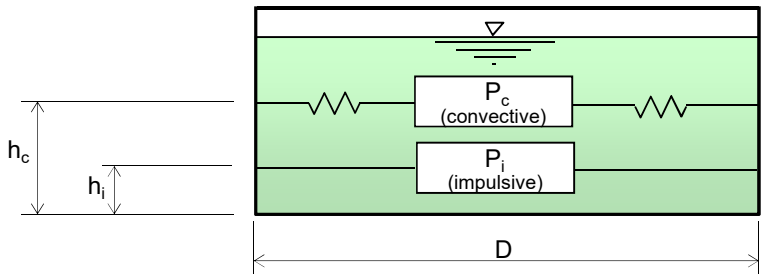
| **DESIGN TASK:** | 16.5ft Liquid; Wall Thick 24"; I=1.0 | | | | |




Dynamic Model

| | | |
|---|---|---|
| D = | 75 | ft |
| $H_L$ = | 16.5 | ft |
| $W_L$ = | 4548.6 | kip |
| $D / H_L$ = | 4.54545 | |
| $H_L / D$ = | 0.22000 | |

2). lateral fluid impulsive force:

equivalent impulsive mass component, $W_i = W_L \left( \dfrac{\tanh\left(0.866 \dfrac{D}{H_L}\right)}{0.866 \dfrac{D}{H_L}} \right) =$ 1154.7 kip

height above base to the impulsive lateral force, hi (EBP) = HL * 0.375 = 6.188 ft

hi (IBP) = HL * {{(0.866*D/HL)/(2*tanh(0.866*D/HL))} -1/8 } = 30.437 ft

impulsive force, $P_i = \left( \dfrac{S_{ai}\ I}{R_i} \right) W_i =$ (1 * 1 / 2) * 1154.7 = 577.4 kip

impulsive force moment excluding bottom pressure , $M_{i(EBP)} = P_i * h_{i(EBP)}$ = 577.4 * 6.188 = 3573.0 ft-k

impulsive force moment including bottom pressure , $M_{i(IBP)} = P_i * h_{i(IBP)}$ = 577.4 * 30.437 = 17574.3 ft-k

3). lateral fluid convective force:

equivalent convective mass component, $W_c = W_L \left( 0.23 \left( \dfrac{D}{H_L} \right) \tanh\left( 3.68 \left( \dfrac{H_L}{D} \right) \right) \right) =$ 3183.1 kip

height above base to convective lateral force, $h_{c\ (EBP)} = H_L \left( 1 - \dfrac{\cosh\left(3.68\left(\dfrac{H_L}{D}\right)\right) - 1}{3.68\left(\dfrac{H_L}{D}\right)\sinh\left(3.68\left(\dfrac{H_L}{D}\right)\right)} \right) =$ 8.673 ft

$h_{c(IBP)} = H_L \left( 1 - \dfrac{\cosh\left(3.68\left(\dfrac{H_L}{D}\right)\right) - 2.01}{3.68\left(\dfrac{H_L}{D}\right)\sinh\left(3.68\left(\dfrac{H_L}{D}\right)\right)} \right) =$ 31.519 ft

convective force, $P_c = \left( \dfrac{S_{ac}\ I}{R_c} \right) W_c =$ ( 0.1472 * 1 / 1.5 ) * 3183.1 = 312.4 kip

convective force moment excluding bottom pressure , $M_{c(EBP)} = P_c * h_{c(EBP)}$ = 312.4 * 8.673 = 2709.4 ft-k

convective force moment including bottom pressure , $M_{c(IBP)} = P_c * h_{c(IBP)}$ = 312.4 * 31.519 = 9846.5 ft-k

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| **CHKD:** | | **DESCRIPTION:** | Digester Evaluation | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | 16.5ft Liquid; Wall Thick 24"; I=1.0 | | | | |

4). lateral inertia force of the accelerating wall:

tank wall mass, $W_w$ = 2503.7 kip

wall c.g. relative to base, $h_w$ = 17.250 ft

wall inertia force, $P_w = \left(\frac{S_{ai}\ I\ \varepsilon}{R_i}\right) W_w =$ ( 1 * 1 * 0.4657 / 2 ) * 2503.7 = 583.0 kip

wall inertia force moment, $M_w = P_w * h_w$ = 583 * 17.25 = 10056.8 ft-k

5). lateral inertia force of the accelerating roof:

total roof mass, $W_r$ = 245.1 kip

roof c.g. relative to base, $h_r$ = 34.5 ft

roof inertia force, $P_r = \left(\frac{S_{ai}\ I}{R_i}\right) W_r =$ ( 1 * 1 / 2 ) * 245.1 = 122.6 kip

roof inertia force moment, $M_r = P_r * h_r$ = 122.6 * 34.5 = 4229.7 ft-k

6). total base shear:

$$V = \sqrt{(P_i + P_w + P_r)^2 + P_c^2}$$

V = ( ( 577.4 + 583 + 122.6 )² + ( 312.4 )² )^½ = 1320.5 kip

7). total moment at the base excluding bottom pressure (EBP):

$$M_b = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

Mb = (( 3573 + 10056.8 + 4229.7 )² + ( 2709.4 )² )^½ = 18063.8 ft-k

8). total moment at the base including bottom pressure (IBP):

$$M_o = \sqrt{(M_i + M_w + M_r)^2 + M_c^2}$$

Mo = (( 17574.3 + 10056.8 + 4229.7 )² + ( 9846.5 )² )^½ = 33347.6 ft-k

9). maximum wave slosh height displacement: ( see ASCE-10, 15.7.6.1 notes c and d )

( Risk Category = 2 ) I = 1 ,use TL = 4 ,Sd1 = 0.6 ,Tc = 6.1126

$S_{ac}$ = 1.5 * Sd1 * TL / Tc² = 0.0963 *g

$d_{(max)}$ = 0.42 ( D )( $S_{ac}$ I ) = 0.42 * ( 75 ) * ( 0.0963 * 1 ) = 3.03 ft

( minimum freeboard see table 15.7-3 of ASCE 7 ) , d(min) = No minimum req'd

10). Vertical acceleration:

design horizontal accereration, $S_{DS}$ = 1 *g

period of vibration, $T_v = 2\pi*(\gamma_L*D*H_L^2 / (24g*t_w*E_c))^{1/2}$ = 0.0266 sec

Ts = $S_{D1}$ / $S_{DS}$ = 0.6 / 1 = 0.6 sec

vertical acceleration (per ACI 350 para 9.4.3), for $T_V \le T_S$ then $C_t = S_{DS}$, for $T_V \succ T_S$ then $C_t = \frac{S_{D1}}{T_V}$

therefore, vertical spectral response acceleration, $S_{av} = C_t$ = 1.0000 *g

per ASCE 7-10 para. 15.7.7.2(b), use I = $R_i$ = b = 1.0

Design vertical acceleration, ü = $\frac{S_{av}\ I\ b}{R_i}$ = 1*1*1/1 = 1.0000 g

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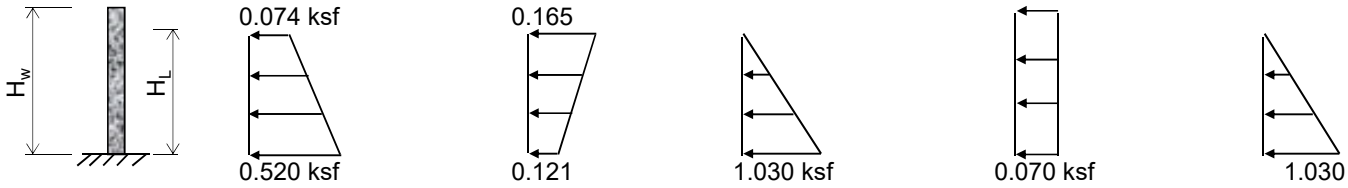
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | 16.5ft Liquid; Wall Thick 24"; I=1.0 | | | |

12). vertical pressure distribution on a unit width using the linear distribution of ACI 350 sec 5.3:




$H_w$ = 34.5
$H_L$ = 16.5

impulsive | convective | vertical acceleration | wall inertia | hydrostatic

impulsive pressure:

$$p_{iy} = \frac{2\left(\frac{P_i}{2}\right)\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{\pi R H_L^2} \cos\theta =$$

use θ = 0°

| | | |
|---|---|---|
| impulsive force, $P_i$ = | 577.4 | kip |
| $h_i$ = | 6.188 | ft |
| at y = $H_L$, $p_{iy}$ = | 0.074 | ksf |
| at base y = 0, $p_{iy}$ = | 0.520 | ksf |

convective pressure:

$$p_{cy} = \frac{16\left(\frac{P_c}{2}\right)\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{9\,\pi\,R\,H_L^2} \cos\theta =$$

use θ = 0°

| | | |
|---|---|---|
| convective force, $P_c$ = | 312.4 | kip |
| $h_c$ = | 8.673 | ft |
| at y = $H_L$, $p_{cy}$ = | 0.165 | ksf |
| at base y = 0, $p_{cy}$ = | 0.121 | ksf |

vertical acceleration pressure:

$$p_{vy} = \ddot{u}\,\gamma_L\,(H_L - y) =$$

| | | |
|---|---|---|
| vertical acceleration, ü = | 1 | g |
| at y = $H_L$, $p_{vy}$ = | 0 | ksf |
| at base y = 0, $p_{vy}$ = | 1.030 | ksf |

wall inertia pressure:

$$p_{wy} = \frac{S_{ai}\ I\ \varepsilon\,\gamma_c\,(t_w/12)}{R_i} =$$

| | | |
|---|---|---|
| $p_{wy}$ = | 0.2329 * ( $\gamma_c$ * $t_w$ ) | |
| at y = $H_w$, $p_{wy}$ = | 0.070 | ksf |
| at base y = 0, $p_{wy}$ = | 0.070 | ksf |

hydrostatic pressure:

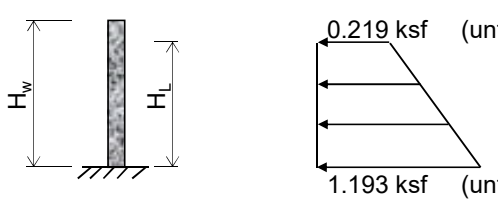
$$q_{hy} = \gamma_L\,(H_L - y) =$$

| | | |
|---|---|---|
| at y = $H_L$, $q_{hy}$ = | 0 | ksf |
| at base y = 0, $q_{hy}$ = | 1.030 | ksf |

combine the effects of the dynamic pressures on the wall:

$$p_y = \sqrt{(p_{iy} + p_{wy})^2 + p_{cy}^2 + p_{vy}^2} =$$

| | | |
|---|---|---|
| at y = $H_w$, $p_y$ = | 0.219 | ksf |
| at base y = 0, $p_y$ = | 1.193 | ksf |




(unfactored load = 0.219 / 1.4 = 0.156 ksf)

(unfactored load = 1.193 / 1.4 = 0.852 ksf)

resultant dynamic pressures

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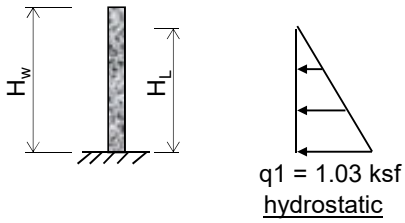
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **BY:** | C. Che | **DATE:** | Dec-18 | **CLIENT:** | Riverside | **SHEET:** | |
| **CHKD:** | | **DESCRIPTION:** | | Digester Evaluation | | **JOB NO:** | 10495A.00 |
| **DESIGN TASK:** | | | | 16.5ft Liquid; Wall Thick 24"; I=1.0 | | | |

13). load cases:

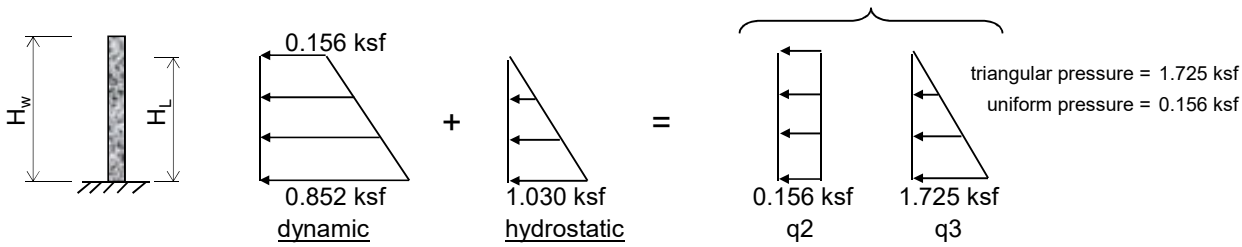
a). hydrostatic water load case:




triangular pressure = 1.030 ksf

b). seismic load case:

equivalent unfactored dynamic + static pressure loadings… equivalent loading ( unfactored )



Attachment C

# DETAILED COST ESTIMATES

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# PROJECT SUMMARY

| | | | |
|---|---|---|---|
| | | Estimate Class: | |
| Project: | Food Waste Evaluation - Digester #5 | PIC: | GJG |
| Client: | City of Riverside | PM: | CT |
| Location: | Riverside | Date: | May 13, 2019 |
| Zip Code: | 92507 | By: | MAE |
| Carollo Job # | 10495A.00 | Reviewed: | CT |

| NO. | DESCRIPTION | | TOTAL |
|---|---|---|---|
| 01 | Recoating of digester walls | | $199,862 |
| 02 | Structural retrofit of foundations and digester walls per Figure 1 | | $254,609 |
| 03 | Instrumentation and Valves | | $50,000 |
| 04 | Uninstalled stand-by mixing pump | | $95,000 |
| | Installation Mechanical, pipe, supports, etc. | | $100,000 |
| | Installation Electrical | | $85,000 |
| | **TOTAL DIRECT COST** | | **$784,471** |
| | Contingency | 30.0% | $235,341 |
| | Subtotal | | **$1,019,812** |
| | General Conditions and Contractor Overhead, Profit & Risk | 25.0% | $254,953 |
| | Subtotal | | **$1,274,765** |
| | Escalation to Mid-Point | 10.5% | $133,850 |
| | Subtotal | | **$1,408,616** |
| | Sales Tax | 8.8% | $123,254 |
| | Subtotal | | **$1,531,870** |
| | Bid Market Allowance | 0.0% | $0 |
| | **TOTAL ESTIMATED CONSTRUCTION COST** | | **$1,531,870** |
| | Engineering, Legal & Administration Fees | 30.0% | $459,561 |
| | **TOTAL ESTIMATED PROJECT COST** | | **$1,991,430** |

*The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion*

*The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.*

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**Project: Food Waste Evaluation - Digester #5**
**Client: City of Riverside**
**Location: Riverside**
**Carollo Job # 10495A.00**

**May 13, 2019**

**Estimate Class:**

| SPEC. DIVISION/ ELEMENT DESCRIPTION | DIV. 00 PROC CTRC | DIV. 01 GEN REQTS | DIV. 02 EXIST COND | DIV. 03 CONC | DIV. 21 FIRE SUPP | DIV. 22 PLUMB | ELEMENT TOTALS | ELEMENT % of Total | TOTAL ESTIMATED CONST COSTS |
|---|---|---|---|---|---|---|---|---|---|
| 01 Figure digester retrofits | | | | $213,170 | | | $213,170 | **100.00%** | $213,170 |
| | | | | | | | | | |
| | | | | | | | | | |
| **Total Direct Cost** | **$0** | **$0** | **$0** | **$213,170** | **$0** | **$0** | **$213,170** | | **$213,170** |
| **Percent of Total** | **0.00%** | **0.00%** | **0.00%** | **100.00%** | **0.00%** | **0.00%** | **100.00%** | | |

COMMENTS / NOTES

1. Note that the above Divisional costs DO NOT include all of the applicable mark-ups for the total construction or project cost. The far right-hand columns provide the for each Element and the Total Estimated Construction Costs. However, any other Program Indirect Costs are not included. Refer to the PROJECT SUMMARY for

# UNIT COST DEVELOPMENT (UCD)



LOCATION FACTOR: 1.065

Date : May 13, 2019

By : MAE

Reviewd: CT

Project: Riverside Digester #5
Client: City of Riverside
Location: Riverside
Carollo Job # 10495A.00

| ITEM NO. (Carollo Code) | SPEC. NO. | DESCRIPTION | UNIT | MATERIAL UNIT COST | LABOR UNIT COST | CONST EQUIP UNIT COST | SUB UNIT COST | OTHER UNIT COST | TOTAL DIRECT UNIT COST | RESOURCE/COMMENTS |
|---|---|---|---|---|---|---|---|---|---|---|
| **0330040000** | **03_30_00 / 03300** | **CONCRETE WALLS** | | | | | | | | |
| 0330040030 | 03_30_00 / 03300 | 12" CURVED WALL, 31'-50' DIA, >8' HIGH | CY | $322.94 | $694.95 | $28.52 | $157.00 | $0.00 | $1,281.63 | |
| **0330020000** | **03_30_00 / 03300** | **CONCRETE SLABS ON GRADE** | | | | | | | | |
| 0330020019 | 03_30_00 / 03300 | 12" SLOPED SLAB ON GRADE (TO 30%) | CY | $292.06 | $5.88 | $26.86 | $0.86 | $0.00 | $346.83 | |
| **030000XX000** | **03_00_00** | **Non-Inventory Item - Spec 030000** | | | | | | | | |
| 030000XX001 | 03_00_00 | Epoxy Bonded Dowel | EA | $55.00 | | | | | $58.58 | Non-Inventory Item - Based on 12" embed with #6 dowel. Per RSMeans |
| 030000XX002 | 03_00_00 | Concrete Surface Prep | SF | $0.25 | $14.00 | $7.00 | | | $22.63 | Non-Inventory Item - assume 2 man team can prep 150 sq.ft. per hour, at $65 per man per hour. Assumes crane and operator required for 2 days to remove any material from roughening process. Assume necessary equipment includes air compressor, spray gun, replacement heads, etc. - this is probably conservative |

# QUANTITY TAKEOFF WORKSHEET

**Project:** Food Waste Evaluation - Digester #5
**Client:** City of Riverside
**Location:** Riverside
**Zip Code:** 92507
**Element**

**Date:** May 13, 2019
**By :** MAE
**Reviewed:** CT
**Format:** MASTER FORMAT 50

| MF50 / SPEC NO. | DRAWING # / DESCRIPTION | # of PLACES | Resulting UNIT | LENGTH in Feet | WIDTH, HEIGHT or DEPTH | THICKNESS in Feet | DIAMETER in Feet | LBS per LF | TOTAL QTY | | NOTES | Item No. (Carollo Code) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | (Leave this row blank) | | | | | | | | | | | |
| 03_30_00 / 03300 | 12" Curved Wall, 31'-50' Dia, >8' High | 1 | CY | 236 | 14 | 1 | | | 122.37 | CY | | 0330040030 |
| 03_30_00 / 03300 | 12" Sloped Slab On Grade (To 30%) | 1 | CY | | | 1 | 46.3 | | 62.36 | CY | This is not slab on grade but | 0330020019 |
| 03_00_00 | Epoxy Bonded Dowel | 1300 | EA | | | | | | 1300 | EA | Non-Inventory Item | 030000XX001 |
| 03_00_00 | Concrete Surface Prep | 1 | SF | 235.5 | 37.5 | | | | 8831.25 | SF | Non-Inventory Item | 030000XX002 |

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# DETAILED COST ESTIMATE

For Allowances, make sure "Spec No." is entered as TEXT.

**Project:** Food Waste Evaluation - Digester #5
**Client:** City of Riverside
**Location:** Riverside
**Element:** Digester repairs

**Format:** MASTER FORMAT 50
**Date :** May 13, 2019
**By :** MAE
**Reviewed:** CT

| SPEC. NO. | DESCRIPTION | QUANTITY | UNIT | UNIT COST | SUBTOTAL | TOTAL | COMMENTS | ITEM NO (Carollo Code) |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | |
| 03_30_00 / 03300 | 12" Curved Wall, 31'-50' Dia, >8' High | 122.37 | CY | $1,281.63 | $156,833 | | | 0330040030 |
| 03_30_00 / 03300 | 12" Sloped Slab On Grade (To 30%) | 62.36 | CY | $346.83 | $21,628 | | | 0330020019 |
| 03_00_00 | Epoxy Bonded Dowel | 1300 | EA | $58.58 | $76,148 | | Non-Inventory Item | 030000XX001 |
| 03_00_00 | Concrete Surface Prep | 8831.25 | SF | $22.63 | $199,862 | | Non-Inventory Item | 030000XX002 |

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# PROJECT SUMMARY

| | | | |
|---|---|---|---|
| | | Estimate Class: | |
| Project: | Food Waste Evaluation - Digester #5 | PIC: | GJG |
| Client: | City of Riverside | PM: | CT |
| Location: | Riverside | Date: | January 16, 2019 |
| Zip Code: | 92507 | By: | MAE |
| Carollo Job # | 10495A.00 | Reviewed: | CT |

| NO. | DESCRIPTION | | TOTAL |
|---|---|---|---|
| 1 | Food Waste Receiving Station - Equipment | | $521,800 |
| 1.1 | Electrical Installation | | $156,540 |
| 1.2 | Mechanical Installation | | $313,080 |
| | **TOTAL DIRECT COST** | | **$521,800** |
| | Contingency | 30.0% | $156,540 |
| | Subtotal | | **$678,340** |
| | General Conditions and Contractor Overhead, Profit & Risk | 25.0% | $169,585 |
| | Subtotal | | **$847,925** |
| | Escalation to Mid-Point | 10.5% | $89,032 |
| | Subtotal | | **$936,957** |
| | Sales Tax | 8.8% | $81,984 |
| | Subtotal | | **$1,018,941** |
| | Bid Market Allowance | 0.0% | $0 |
| | **TOTAL ESTIMATED CONSTRUCTION COST** | | **$1,018,941** |
| | Engineering, Legal & Administration Fees | 30.0% | $305,682 |
| | **TOTAL ESTIMATED PROJECT COST** | | **$1,324,623** |

*The cost estimate herein is based on our perception of current conditions at the project location. This estimate reflects our professional opinion of accurate costs at this time and is subject to change as the project design matures. Carollo Engineers have no control over variances in the cost of labor, materials, equipment; nor services provided by others, contractor's means and methods of executing the work or of determining prices, competitive bidding or market conditions, practices or bidding strategies. Carollo Engineers cannot and does not warrant or guarantee that proposals, bids or actual construction costs will not vary from the costs presented as shown.*

**City of Riverside**
**Food Waste Receiving Facility Equipment List**

| EQUIPMENT NAME | Recommended Manufacturers | Model Number | Quantity | Size | HP | Unit costs | Costs |
|---|---|---|---|---|---|---|---|
| **Slurry Receiving Facility** | | | | | | | |
| Equalization/Storage Tank | Xerxes | FRP | 4 | 30,000 gal | | $ 40,000 | $ 160,000 |
| Tank Mixing/Chopper Pump | GasMix | | 4 | 200 gpm | 45 | $ 45,000 | |
| | Landia | Horizontal Chopper Pump | | 200 gpm | 15 | $ 20,000 | $ 260,000 |
| Food Waste Slurry Metering Pump | Boerger | Rotary Lobe-Blueline AL | 4 | 30 gpm | 10 | $ 10,000 | $ 40,000 |
| Instrumentation: COD analyzer | HACH | UVAS | 1 | | | $ 25,000 | $ 25,000 |
| Flow meters | various | | 2 | | | $ 5,000 | $ 10,000 |
| Isolation valves | Plug Valve: DeZurik or Milliken | PEF | 8 | 6 in | | $ 2,100 | $ 16,800 |
| Odor Control | CARBTROL | Carbon canister, 3,000 lbs | 1 | | | $ 10,000 | $ 10,000 |
| | Calgon Carbon Corp. | Carbon Canister | | | | | |
| **Total** | | | | | 73.00 | | **$ 521,800** |

Appendix 11B

# RWQCP LEVEE REHABILITATION 100% DESIGN

# Riverside Regional Water Quality Control Plant
# Levee Rehabilitation
# Riverside, California

## 100% Design Report

*Prepared for:*

CITY OF RIVERSIDE

*Prepared by:*

TETRA TECH

17885 Von Karman, Suite 500
Irvine, California 92614

**March 2016**

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# EXECUTIVE SUMMARY

This Design Report presents the results of the 100% design of the RWQCP levee rehabilitation along the Santa Ana River. The City of Riverside Regional Water Quality Control Plant (RWQCP) is currently constructing the Plant Rehabilitation/Expansion Project. The RWQCP current NPDES permit (Appendix H), requires the adjacent Levee (RWQCP Levee), on the north side of the plant, to provide the plant protection for a 100-year storm event. A levee-deficiency analysis report was completed for the City which indicated the levee is deficient in freeboard and embankment protection (Appendix A), per Federal Emergency Management Agency (FEMA) and standard flood control engineering design requirements.

The overall levee rehabilitation has been separated into two independent phases. Phase 1 improvements consist of floodwall protection improvements to protect from the 100-year flood water surface elevation. Phase 2 improvements are to remedy the scour protection deficiency along the levee for the 100-year flood event. It should be noted that until Phase 2 improvements are designed and constructed the Phase 1 project is susceptible to failure due to scour and loss of levee embankment. The Phase 2 Improvements are to be designed and completed via a separate contract and schedule due to environmental permitting and funding.

The Design Report provides an overview of the project and issues to the City, including the project approach, initial project issues, anticipated problems, a review of possible environmental resource agency issues, and components of the RWQCP Levee systems for the Phase 1 project.

Due to the plant expansion project, the improvements to raise the existing levee are constrained by treatment facilities on the protected side of the levee and environmental limitations on the river side of the levee. These constraints impact a straightforward design solution of raising the earthen levee embankment due to the restrictions on the expansion of the levee footprint. Additionally, these constraints impact the design alternatives to mitigate the embankment protection improvements required to protect the levee foundation from scour during flood events. At this time, the project consists of the raising of the existing levee from 1’ to 6’ with the construction of a levee embankment and maximum 4’ high floodwall. The City’s goal is to obtain a better understanding of the problem and development an acceptable design solution to provide the 100-year flood protection and raise the existing levee in a cost effective manner that can be designed, permitted and constructed by August 2017.

This design focuses on the construction of a floodwall along the top of the levee to provide the hydraulic protection needed (Phase 1 improvements). Using the results of the refined hydraulics, two floodwall sections (for various heights) were reviewed and preliminarily calculated.

A Preliminary Design Report (Tetra Tech April 2015) was prepared to explore floodwall alternatives for the proposed levee rehabilitation. Four alternatives where explored as part of the analysis. They consisted a floodwalls placed along the south side (Alternative 1) of the Santa Ana River Trail (SART), so that the public can view the river, or along the north side (Alternative 2) so that the public will view the RWQCP. In addition to the two alternatives, Tetra Tech explored a partial levee raise of 6 feet with a smaller floodwall, up to 4 feet, place along the south side (Alternative 3) and the north side (Alternative 4) of the SART.

The PDR alternatives were presented to the RWQCP staff that included potential project benefits, costs, project phasing, and right-of-way constraints, maintenance, and SART public issues was

performed and determine Alternative 3 to be the recommended plan, which was the most cost effective, environmentally sound, and technically feasible alternative.

Based upon the selected plan Tetra Tech proceeded to the 100% design including, plans, project specifications, and cost estimate. The Design Report presented the analysis and documents in this report. The construction cost for the improvements is $2,861,000.

It should be noted that preliminary Phase 2 cost, which consist of grouting of the levee revetment from station 27+00 to 44+00, will be approximately $3,885,000 plus environmental mitigation cost and permitting. Alternative protection methods where reviewed, such as a sheet pile cutoff wall, but due to the geotechnical issues and limitations the design of a sheet pile wall was determined to be infeasible from an engineering standpoint. Future work associated with Phase 2 will consist of open trench excavation and grouting of the revetment.

# PERTINENT DATA

Purpose: Flood Risk Management

| Item | Description |
|---|---|
| RWQCP Levee Length | Approx. 4,800 lf |
| 100-Year Peak Discharge | 144,300 cfs |
| Minimum Design Freeboard | 3 feet |
| Floodwall/ Levee Raise Length | 2,108 lf |
| Max Floodwall Height | 4 feet |
| Max Levee Embankment Increase | 6 feet |

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# 1. INTRODUCTION

## GENERAL

1.1 This Design Report presents the results of the design of the City of Riverside, Regional Water Quality Control Plant (RWQCP) levee rehabilitation along the Santa Ana River (Figure 1.1). The RWQCP is currently constructing the Plant Rehabilitation/Expansion Project. As part of the City's current NPDES permit (Appendix H), the RWQCP Levee shall provide the plant protection for a 100-year storm event. A levee-deficiency analysis report was completed for the City which indicated the levee is deficient in freeboard and embankment protection (Appendix A), per Federal Emergency Management Agency (FEMA) and standard flood control engineering design requirements.

## PURPOSE

1.2 The purpose of this report is to provide an overview of the project and issues to the City, including the project approach, initial project issues, anticipated problems, a review of possible environmental resource agency issues, and preliminary design of the Phase 1 RWQCP Levee. The design is based upon Alternative 3 as presented in the Preliminary Design Report (Tetra Tech, April 2015).

## SCOPE OF STUDIES

1.3 This Design Report presents the design for the selected floodwall alternatives, supplemental geotechnical investigations, refined hydraulic model of the Santa Ana River, structural analysis, cost estimates and environmental review.

1.4 The overall levee rehabilitation has been separated into two independent phases. Phase 1 improvements consist of floodwall protection improvements to protect from the 100-year flood water surface elevation. Phase 2 improvements are to remedy the scour protection deficiency along the levee for the 100-year flood event. It should be noted that until Phase 2 improvements are designed and constructed the Phase 1 project is susceptible to failure due to scour and loss of levee embankment. The Phase 2 improvements are to be designed and completed via a separate contract and schedule due to environmental permitting and funding.

1.5 As mentioned in the Preliminary Design Report (PDR), the initial design alternatives focused on a construction of a floodwall along the top of the levee to provide the hydraulic protection needed. Two floodwall sections (for various heights) were to be placed along the south side (Alternative 1) of the Santa Ana River Trail (SART), so that the public can view the river, or along the north side (Alternative 2) so that the public will view the RWQCP. Tetra Tech then expanded the study to include a partial levee raise of 6 feet with a smaller floodwall, up to 4 feet, place along the south side (Alternative 3) and the north side (Alternative 4) of the SART.

1.6 Alternative 3 was selected as the recommended plan, which was the most cost effective, environmentally sound, and technically feasible alternative. The 100% design is presented in this report to support the preparation of the 100% design including, plans, project specifications, and cost estimate.

Figure 1.1 Location Map

## Surveying and Mapping

1.7 The mapping is based on LiDAR method aerial topography for the City of Riverside as part of the RWQCP Plant Rehabilitation/Expansion Project. The LiDAR data was provided as point data in XYZ format text files with associated one-foot contours by the City of Riverside. Horizontal control is based on the North American Datum (NAD) of 1983, California State Plane 5; vertical control is based on the North American Vertical Datum (NAVD) of 1988. Additional site survey was performed by Coast Surveying (June 2015), which reflected site modifications including newly constructed facilities and grading as part of the plant expansion.

## Site Explorations

1.8 A supplemental subsurface investigations was performed by Tetra Tech, for the design of the RWQCP Levee Project. A report titled "Geotechnical Engineering Report Levee and Floodwall Evaluation, Riverside Regional Water Quality Control Plant", dated March 9, 2015 compiles the analyses and results presented in Appendix B.

## Coordination with Others

1.9 Coordination of the design of the project will be conducted by the project team. Items that would be discussed include mapping, as-built plans, rights-of-way (ROW), O&M, easements, utility relocations, disposal sites, and maintenance features.

Ongoing coordination with the City of Riverside.

Contact Person: Ernest Marquez, Jr., P.E.
Principle Engineer
City of Riverside, Public Works Department
Regional Water Quality Control Plant
5950 Acorn Street
Riverside, CA 92504
(951) 826-5409

- Rights-of-Way (ROW).

The real estate interests required for and/or impacted by the project are owned and/or held by private owners, county governments or municipalities, and public utilities. ROW requirements will be established in detail prior to completion of plans and specifications.

- Utility Relocations.

The project would require relocation of publicly and privately owned utilities. Interfering utilities include a 6" water line and a 20" High pressure gas line which crosses the Santa Ana River at the eastern portion of RWQCP. Two storm drain lines will need to relocate; 42" RCP and 10" PVC force main. Where possible, relocations will be avoided or accomplished in advance of the construction. The affected utilities are identified in Table 1.1.

Table 1.1 Affected Utilities

| Location | Approx. Station | Type | Owner |
|---|---|---|---|
| Located within Levee Section | Sta 20+00 to 45+00 | Reclaimed Water Line | The Toro Company |
| Located within Levee Section | Sta 27+50 to 28+00 | 42” RCP – Storm Drain | RWQCP |
| Located within Levee Section | Sta 26+40 to 32+45 | 10” PVC Storm Drain Force Main | RWQCP |
| Santa Ana River Trail (SART) | Along the RWQCP Levee | Bike/Trail Path | City of Riverside/ Riverside County Parks and Recreation |

Table 1.2 Potential Affected Utilities

| Location | Approx. Station | Type | Owner |
|---|---|---|---|
| Crossing Above Levee | Sta 46+50 | High Pressure Gas Line | Sempra Energy |

The existing reclaimed water line will need to be relocated vertically to account for the levee raising. Although it is preferred to remove any obstructions or encroachments, specifically pipelines, from the levee prism, due to the amount of conflicts with alternative alignments and the additional cost. It was determined that the water line will need to be remain within the levee prism as it is not economical feasible to relocated outside the levee prism. Manhole structure with gate valves are installed to allow for emergency shut off if needed. Additionally, to assist in leak detection two water meters, one at the plant near the connection with the 24” reclaimed water line and the other near the end user (Toro) are installed. The meters will be monitored by the City to determine if there is any leakage within the pipeline. If it is determined that a leakage is occurring, the City will commence to repair the pipeline appropriately.

# 2. SELECTED PLAN

## GENERAL

2.1 The Selected Plan consists Alternative 3 – This alternative assumes raising the existing levee by up to 6 feet and installing a floodwall on top of the newly raised levee on the plant side. The floodwall height varies from 0 to 4 feet high with the top of footing flush with the top of new levee elevations. The new slopes would require slope paving to be placed as slope protection. After completion of the raised levee and new floodwall, a bike trail and chain link fence would be installed. This alternative also requires the installation of a five foot high retaining wall in two sections within the plant area to increase access road widths and offsets from proposed buildings as part of the plant expansion. A typical cross section is provided below.

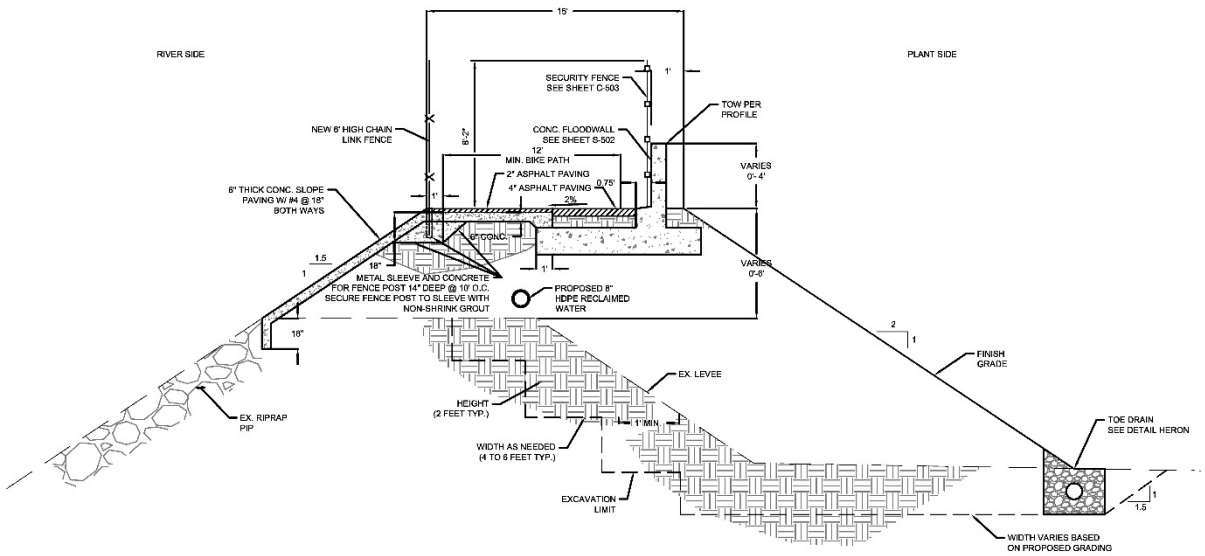


Figure 2.1 Selected Alternative

2.2 As part of the 90 percent design submittal, Tetra Tech performed a design refinement review. One of the changes incorporated into the design (in addition to the partial levee raise) is the re-alignment of the upstream levee to avoid the high pressure gas line. From station 45+00 to 47+30 the levee improvements will turn perpendicular to the existing levee and join high ground to the east as shown on Appendix E, Sheet 13.

## STATION TO STATION IMPROVEMENTS

The improvements described below are segmented between levee sections and changes in profile and construction requirements. The stations provided in the descriptions are based upon the 100% design plans.

**Station 0+00 to 8+00 (Levee Adjacent to Van Buren Blvd.)**

2.3 No levee improvements are proposed for this section of the existing levee. The existing levee in this location possesses the necessary 3 feet of freeboard above the flood water surface elevations and the existing security fencing has been constructed along the plant side of the levee access road.

**Station 8+00 to 26+25**

2.4 No levee improvements are proposed for this section of the existing levee. The existing levee in this location possesses the necessary 3 feet of freeboard above the flood water surface elevations. At this time the existing chain link fencing along the plant side of the levee access road will be removed and replaced with security fencing along the entire length of the reach. Additionally, security lighting will be added along this reach. The design proposes to protect in place the existing access ramp at approximate station 19+50 with the addition of a double swing security gate. The security fencing will join the existing fencing at approximate Station 8+00.

**Station 26+25 to 35+00**

2.5 Beginning at Station 26+25, the existing levee embankment will be raised from 0' to 6' at approximate station 31+40. The embankment raising will encroach into the existing plant, but will avoid any conflicts with existing facilities or the proposed plan expansion improvements. The embankment will continue to increase from 5' to a maximum 6' (approximate station 31+40 to station 35+00). The increased levee height will contain the 100-yr flood event with a minimum of 3 feet of freeboard.

2.6 Security fencing and lighting will continue along this segment of the levee improvements. The existing access ramp located at station 32+00 will be regraded to maintain a maximum 10% grade with the addition of a double swing security gate.

2.7 The existing reclaimed water line will be removed and relocated 3 feet below the top of levee. Manhole vault with a measuring meter will be installed to determine the amount of flow distributed. The relocation of the reclaimed water line outside of the levee prism was discussed with the City of Riverside, but it was not economically feasible. An existing 42" RCP and 10" storm drain force main will be relocated to be outside the levee prism. There are 3 utility vaults adjacent to the 42" RCP, these will be protected in place.

2.8 Due to the embankment raise, the existing chain link fencing along the riverside of the access road will be removed and replaced along with the SART. SART will be repaved with 4" asphalt and restriped with a 10' minimum width bike path.

**Station 35+00 to 47+00**

2.9 Through this segment of the levee, the levee embankment will be raised 6', beginning at Station 35+00, the addition of a floodwall will be constructed along the plant side of the levee access road to provide the additional freeboard for the 100-yr event. The floodwall will vary in height from 1.0' to 2.24' from approximate station 35+00 to 40+00 and 2.25' to 4.0' from 40+00

to 44+00. From Station 44+00 to 47+12 the floodwall varies from 4' to 1.0' and will turn along the existing right of way to join the existing high ground. The modification will follow the existing right-of-way to avoid any possible impact to the existing high pressure gas line.

2.10 Security fencing and lighting will continue along this segment of the levee improvements. The access road will join the existing access road located along the northeast side of the plant.

2.11 The embankment raising will encroach into the existing plant. Due to the encroachment, two 5' high retaining walls are provided at approximate station 36+50 and from station 37+50 to 40+00 to maintain clearance from newly constructed facilities.

2.12 Due to the embankment raise, the existing chain link fencing along the river side of the access road will be removed and replaced along with the SART. SART will be repaved with a 2" asphalt overlay over the proposed floodwall footing and concrete slope paving. The SART will be restriped with a 10' minimum width bike path. Since the levee embankment is raised by approximately 6', the profile of the SART at the upstream end of the levee improvements will be regraded to join the existing trail just before the high pressure gas line crossing.

2.13 The existing reclaimed water line will be removed and relocated 3 feet below the top of levee. The relocation of the water outside of the levee prism was discussed with the City of Riverside, but it was not economically feasible. An existing 10" storm drain force main will be relocated to be outside the levee prism.

2.14 The SART is a Riverside County owned regional trail/bike path that is highly utilized by cyclist and the public. Temporary closure of the SART will be required for construction purposes and an approved detour along Jurupa Avenue will be provided for the duration of construction. The City is currently planning to restripe Jurupa to add bike lanes along the proposed detour path. This will help reduce the impacts to the closure and provide a safe detour route through the construction period.

2.15 The proposed design will also allow for the future Phase 2 construction work which consist of grouting the existing riprap embankment from approximate station 27+00 to upstream of the project limits. The work associated with this will also require the grouting of the existing toe scour protection below the channel invert.

2.16 There are potential environmental impacts, (see Section 6.5) about potential nesting birds within the existing vegetation adjacent to the levee.

## MATERIAL REQUIRED FOR CONSTRUCTION

2.17 Through advance planning, the City of Riverside has stockpiled approximately 24,000 cy of suitable fill material for use in the raising of the levee embankment. This material has been set aside within the southwest portion of the plant for the use in this project.

# 3. HYDROLOGIC AND HYDRAULIC BASIS FOR DESIGN

3.1 This section summarizes the previous hydraulic analyses conducted for the RWQCP levee and describes the hydraulic analysis that was conducted to support the project design. The analysis described here includes hydraulic modeling of the proposed conditions. Descriptions of the assumptions, inputs, methodologies, and results of this analysis are provided below.

3.2 A levee-deficiency analysis report was completed by Albert A. Webb Associates in July 2011 (Appendix I). A summary of the levee deficiencies, per FEMA requirements, is provided in Table 3.1. In order to mitigate freeboard deficiency, the preferred alternative consisted of (1) an additional fill reach to raise the existing levee along its downstream portion, and (2) construction of a floodwall above the 100-year water surface elevations, with FEMA required freeboard, on the top of the existing levee along its upper portion.

Table 3.1 Summary of RWQCP Levee Deficiency

| Item | Deficiency (Yes/No) |
|---|---|
| Freeboard | Yes |
| Closures | No |
| Embankment Protection | Yes |
| Levee and Foundation Stability | No |
| Settlement | No |
| Interior Drainage | No |

3.3 In May 2010, Tetra Tech conducted a scour and sediment-transport analyses for the preferred alternative presented in the levee-deficiency analysis report (Webb, 2011) to provide levee toe protection against potential scour due to high flow velocity.

3.4 Hydraulic analysis of the design plan was conducted using the HEC-River Analysis System (HEC-RAS) program, Version 4.1.0 (January 2010), developed by the U.S. Army Corps of Engineers (USACE). The hydraulic model performed in the *Final Hydraulic, Scour, and Sediment-Transport Analyses Report* (Tetra Tech, 2012, Appendix A) was used as the basis for the current hydraulic analyses. The boundary of hydraulic model was from approximately one-half mile downstream of Van Buren Boulevard to approximately 1.4 miles upstream from the Metropolitan Water District (MWD) pipeline crossing (see Figure 3.1).

3.5 Flow rates for the Santa Ana River were obtained from the Riverside County Flood Control and Water Conservation District (RCFC&WCD). Flow rates of 83,000; 144,300; and 316,300 cubic feet per second (cfs) are corresponding to 50-, 100-, and 500-year flood events, respectively. These flow rates account for the flow reduction provided by the upstream Seven Oaks Dam (USACE, 1991).

3.6 FEMA utilized an *n*-value of 0.06 to represent the channel and overbank roughness for the entire reach of the Santa Ana River in Riverside County in preparation of the Flood Insurance Study (FEMA, 2014). Based upon reviewing of historical photographs and field visits, vegetation would be washed out during more frequent storm events (less than 100-year event) due to the existing sand bed channel. Therefore, a reduced *n*-value is more representative of the reach along the RWQCP levee for the 100-year design peak flow.

3.7 Based on Section 5-3 of the USACE Engineering Manual (EM) 1110-2-1601 (USACE, 1994), the estimated *n*-values for the incised channel/adjacent floodplain and overbank areas are presented in Tables 3.2 and 3.3, respectively. The selected Manning's *n*-values are 0.035 and 0.060 for the incised channel/adjacent floodplain and overbank areas, respectively.

Table 3.2 Estimation of Manning's n-values for Incised Channel and Adjacent Floodplain

| Variable | Channel Conditions | Recommended Value | Min. Value | Selected Value | Max. Value |
|---|---|---|---|---|---|
| Basic, $n_b$ | | | | | |
| | Earth | 0.020 | 0.020 | 0.020 | 0.020 |
| | Rock cut | 0.025 | | | |
| | Fine gravel | 0.024 | | | |
| | Coarse gravel | 0.028 | | | |
| Degree of Surface Irregularity, $n_1$ | | | | | |
| | Smooth | 0.000 | | | |
| | Minor | 0.005 | 0.005 | 0.005 | |
| | Moderate | 0.010 | | | 0.010 |
| | Severe | 0.020 | | | |
| Variation in Shapes and Size of Channel Cross Section, $n_2$ | | | | | |
| | Gradual | 0.000 | 0.000 | 0.000 | |
| | Alternating occasionally | 0.005 | | | 0.005 |
| | Alternating frequently | 0.010 - 0.015 | | | |
| Relative Effect of Obstructions, $n_3$ | | | | | |
| | Negligible | 0.000 | 0.000 | 0.005 | 0.005 |
| | Minor | 0.010 - 0.015 | | | |
| | Appreciable | 0.020 - 0.030 | | | |
| | Severe | 0.040 - 0.060 | | | |
| Vegetation, $n_4$ | | | | | |
| | Low | 0.005 - 0.010 | 0.005 | 0.005 | 0.005 |
| | Medium | 0.010 - 0.025 | | | |
| | High | 0.025 - 0.050 | | | |
| | Very High | 0.050 - 0.100 | | | |
| Degree of Meandering, m | | | | | |
| | Minor | 1.00 | 1.000 | 1.000 | 1.000 |
| | Appreciable | 1.15 | | | |
| | Severe | 1.30 | | | |
| $n = (n_b + n_1 + n_2 + n_3 + n_4)\cdot m$ | | | 0.030 | 0.035 | 0.045 |

Reference: Table 5-5 of Open Channel Hydraulics (V.T. Chow, 1959)

Table 3.3 Estimation of Manning's n-values for Overbank Areas

| Variable | Channel Conditions | Recommended Value | Min. Value | Selected Value | Max. Value |
|---|---|---|---|---|---|
| Basic, $n_b$ | | | | | |
| | Earth | 0.020 | 0.020 | 0.020 | 0.020 |
| | Rock cut | 0.025 | | | |
| | Fine gravel | 0.024 | | | |
| | Coarse gravel | 0.028 | | | |
| Degree of Surface Irregularity, $n_1$ | | | | | |
| | Smooth | 0.000 | 0.000 | | |
| | Minor | 0.005 | | 0.005 | |
| | Moderate | 0.010 | | | 0.010 |
| | Severe | 0.020 | | | |
| Variation in Shapes and Size of Channel Cross Section, $n_2$ | | | | | |
| | Gradual | 0.000 | 0.000 | 0.000 | 0.000 |
| | Alternating occasionally | 0.005 | | | |
| | Alternating frequently | 0.010 - 0.015 | | | |
| Relative Effect of Obstructions, $n_3$ | | | | | |
| | Negligible | 0.000 | | | |
| | Minor | 0.010 - 0.015 | 0.010 | 0.015 | |
| | Appreciable | 0.020 - 0.030 | | | 0.020 |
| | Severe | 0.040 - 0.060 | | | |
| Vegetation, $n_4$ | | | | | |
| | Low | 0.005 - 0.010 | 0.015 | | |
| | Medium | 0.010 - 0.025 | | 0.020 | |
| | High | 0.025 - 0.050 | | | 0.025 |
| | Very High | 0.050 - 0.100 | | | |
| Degree of Meandering, m | | | | | |
| | Minor | 1.00 | 1.000 | 1.000 | 1.000 |
| | Appreciable | 1.15 | | | |
| | Severe | 1.30 | | | |
| $n = (n_b + n_1 + n_2 + n_3 + n_4) \cdot m$ | | | 0.045 | 0.060 | 0.075 |

Reference: Table 5-5 of Open Channel Hydraulics (V.T. Chow, 1959)

3.8 The downstream hydraulic control for this model was based on normal depth calculations. The Santa Ana River has an invert slope of 0.004 (feet/feet) at the downstream end of the project limit and was used in the hydraulic model as the downstream hydraulic control in HEC-RAS.

3.9 Under the proposed conditions, the computed 100-year water surface elevations along RWQCP Levee are shown on Figure 3.2 for the minimum, selected, and maximum Manning's n-values with top of levee elevations that satisfied the minimum of 3 feet freeboard. It should be noted that during the generation of alternatives it was decided that the levee would be designed to maintain a minimum freeboard to meet the NDPES permit requirements, since FEMA levee certification was not obtainable without major reconstruction of the embankment due to liquefaction and post seismic embankment stability (See geotechnical section). The computed channel hydraulics are summarized in Table 3.4.

Table 3.4 Computed 100-year Channel Hydraulics

| | | Water Surface Elevation (ft) | | | Average Flow Velocity (ft/sec) | | | Minimum Top of Levee[1] (ft) |
|---|---|---|---|---|---|---|---|---|
| Station ID | Channel Thalweg (ft) | Minimum n-values | Selected n-values | Maximum n-values | Minimum n-values | Selected n-values | Maximum n-values | |
| 50 | 720.00 | 731.02 | 732.08 | 733.83 | 8.01 | 6.87 | 5.47 | |
| 49 | 720.00 | 730.36 | 731.50 | 733.28 | 7.18 | 6.21 | 5.11 | |
| 48 | 716.00 | 730.08 | 731.19 | 732.93 | 5.27 | 4.75 | 4.11 | |
| 47 | 716.00 | 730.01 | 731.09 | 732.78 | 4.38 | 4.05 | 3.59 | |
| 46 | 712.00 | 729.76 | 730.83 | 732.51 | 4.97 | 4.64 | 4.19 | |
| 45 | 712.00 | 729.40 | 730.47 | 732.14 | 6.11 | 5.76 | 5.21 | |
| 44 | 708.00 | 728.00 | 729.15 | 730.99 | 11.37 | 10.47 | 9.11 | |
| 43 | 708.00 | 725.56 | 726.59 | 728.67 | 13.17 | 12.46 | 11.03 | |
| 42 | 704.00 | 724.92 | 725.75 | 727.52 | 11.19 | 10.73 | 9.82 | |
| Union Pacific Railroad | | | | | | | | |
| 41 | 704.00 | 722.90 | 724.02 | 726.05 | 12.36 | 11.53 | 10.28 | |
| 40 | 704.00 | 721.22 | 722.84 | 725.25 | 15.01 | 13.38 | 11.45 | |
| 39 | 704.00 | 722.01 | 723.26 | 725.28 | 9.83 | 9.21 | 8.25 | |
| 38 | 700.11 | 722.13 | 723.29 | 725.17 | 7.46 | 7.05 | 6.38 | |
| 37 | 700.00 | 721.77 | 722.94 | 724.82 | 8.07 | 7.58 | 6.83 | |
| 36 | 700.00 | 721.48 | 722.59 | 724.40 | 8.32 | 7.91 | 7.24 | |
| 35 | 700.00 | 721.01 | 722.12 | 723.91 | 9.28 | 8.85 | 8.14 | |
| 34 | 696.00 | 720.67 | 721.78 | 723.51 | 9.58 | 9.11 | 8.46 | |
| 33 | 696.00 | 719.38 | 720.52 | 722.33 | 12.66 | 12.12 | 11.19 | |
| 32 | 696.00 | 718.32 | 719.48 | 721.22 | 13.98 | 13.31 | 12.34 | |
| 31 | 696.00 | 718.54 | 719.60 | 721.16 | 11.33 | 10.83 | 10.13 | |
| MWD Pipeline | | | | | | | | |
| 30 | 693.00 | 716.10 | 717.58 | 719.45 | 14.73 | 13.73 | 12.66 | |
| 29 | 693.00 | 716.55 | 717.91 | 719.61 | 11.46 | 10.77 | 10.01 | |
| 28 | 692.00 | 716.60 | 717.89 | 719.47 | 10.25 | 9.67 | 9.01 | |
| 27 | 691.00 | 716.76 | 717.99 | 719.47 | 8.69 | 8.23 | 7.71 | |
| 26 | 691.00 | 716.83 | 718.02 | 719.44 | 7.76 | 7.37 | 6.93 | |
| 25 | 690.00 | 716.96 | 718.11 | 719.48 | 6.51 | 6.24 | 5.89 | |
| 24[1] | 689.00 | 716.67 | 717.84 | 719.19 | 7.35 | 7.02 | 6.64 | 722.19 |
| 23[1] | 688.00 | 715.56 | 716.75 | 718.17 | 11.16 | 10.81 | 10.17 | 721.17 |
| 22[1] | 687.00 | 714.10 | 715.25 | 716.71 | 14.83 | 14.51 | 13.70 | 719.71 |
| 21[1] | 687.00 | 711.91 | 713.40 | 714.90 | 17.40 | 16.42 | 15.39 | 717.90 |
| 20[1] | 685.00 | 710.93 | 712.16 | 713.53 | 19.05 | 18.49 | 17.37 | 716.53 |
| 19[1] | 685.00 | 710.12 | 710.87 | 712.04 | 20.01 | 20.20 | 18.92 | 715.04 |
| 18[1] | 684.00 | 705.82 | 706.71 | 708.16 | 23.94 | 23.49 | 21.67 | 711.16 |
| 17[1] | 683.00 | 707.02 | 707.08 | 707.35 | 18.87 | 19.60 | 19.08 | 710.35 |
| 16[1] | 682.00 | 702.30 | 702.27 | 702.27 | 21.19 | 21.35 | 21.33 | 705.30 |
| 15[1] | 681.00 | 698.04 | 698.14 | 698.93 | 18.57 | 18.71 | 17.32 | 701.93 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 14[1] | 681.00 | 696.79 | 697.60 | 699.14 | 13.96 | 13.68 | 11.97 | 702.14 |
| 13[1] | 678.00 | 696.78 | 697.46 | 698.74 | 10.55 | 10.27 | 9.26 | 701.74 |
| 12[1] | 678.00 | 696.42 | 696.94 | 698.14 | 10.97 | 10.96 | 9.87 | 701.14 |
| 11[1] | 678.00 | 695.73 | 696.14 | 697.30 | 11.44 | 11.33 | 10.19 | 700.30 |
| 10[1] | 676.00 | 695.61 | 695.95 | 696.87 | 9.18 | 8.94 | 8.33 | 699.87 |
| 9[1] | 674.00 | 694.03 | 694.33 | 695.27 | 11.98 | 11.69 | 10.85 | 698.27 |
| Van Buren Boulevard | | | | | | | | |
| 8 | 674.00 | 690.20 | 691.02 | 692.63 | 16.75 | 15.37 | 13.21 | |
| 7 | 674.00 | 689.54 | 690.05 | 691.53 | 15.12 | 14.48 | 12.57 | |
| 6 | 673.00 | 687.87 | 688.53 | 690.45 | 16.12 | 15.04 | 12.22 | |
| 5 | 674.00 | 688.05 | 688.87 | 690.22 | 11.32 | 10.82 | 9.53 | |
| 4 | 673.00 | 686.70 | 687.56 | 689.06 | 13.01 | 12.30 | 10.58 | |
| 3 | 672.00 | 685.99 | 686.76 | 688.23 | 11.62 | 11.18 | 9.68 | |
| 2 | 672.00 | 685.38 | 686.08 | 687.49 | 10.79 | 10.28 | 8.95 | |
| 1 | 671.00 | 685.03 | 685.62 | 686.91 | 9.74 | 9.34 | 8.32 | |
| -1 | 669.71 | 683.61 | 684.38 | 685.88 | 11.90 | 10.94 | 9.38 | |
| -2 | 672.00 | 682.88 | 683.66 | 685.18 | 11.66 | 10.72 | 9.21 | |
| -3 | 670.86 | 682.22 | 682.95 | 684.41 | 11.30 | 10.54 | 9.26 | |
| -4 | 668.00 | 681.00 | 681.79 | 683.35 | 12.69 | 11.76 | 10.26 | |
| -5 | 668.00 | 679.50 | 680.45 | 682.19 | 14.36 | 13.11 | 11.25 | |

1. Minimum top of levee elevation that satisfied the minimum 3 feet of freeboard requirement.

3.10 Details of channel scour analysis should be referred to the *Final Hydraulic, Scour, and Sediment-Transport Analyses Report* (Tetra Tech, 2012, Appendix A). Based upon the above mentioned scour analysis (Tetra Tech, 2012), it was recommended that the levee revetment be extended either to bedrock or to the computed total scour depths as presented in Table 3.5, along with adequate levee armoring measure(s), and an O&M program to assure levee stability during the occurrence of a design 100-year flood event.

Table 3.5 Summary of Channel Scour Analysis

| Segment No. | From HEC-RAS Station ID[1] | To HEC-RAS Station ID[1] | Scour Depth (ft) |
|---|---|---|---|
| Downstream Reach | 9 | 15 | 12 |
| Middle Reach | 16 | 21 | 25 |
| Upstream Reach | 22 | 24 | 20 |

1. See Figure 4.1 for HEC-RAS Station ID locations.

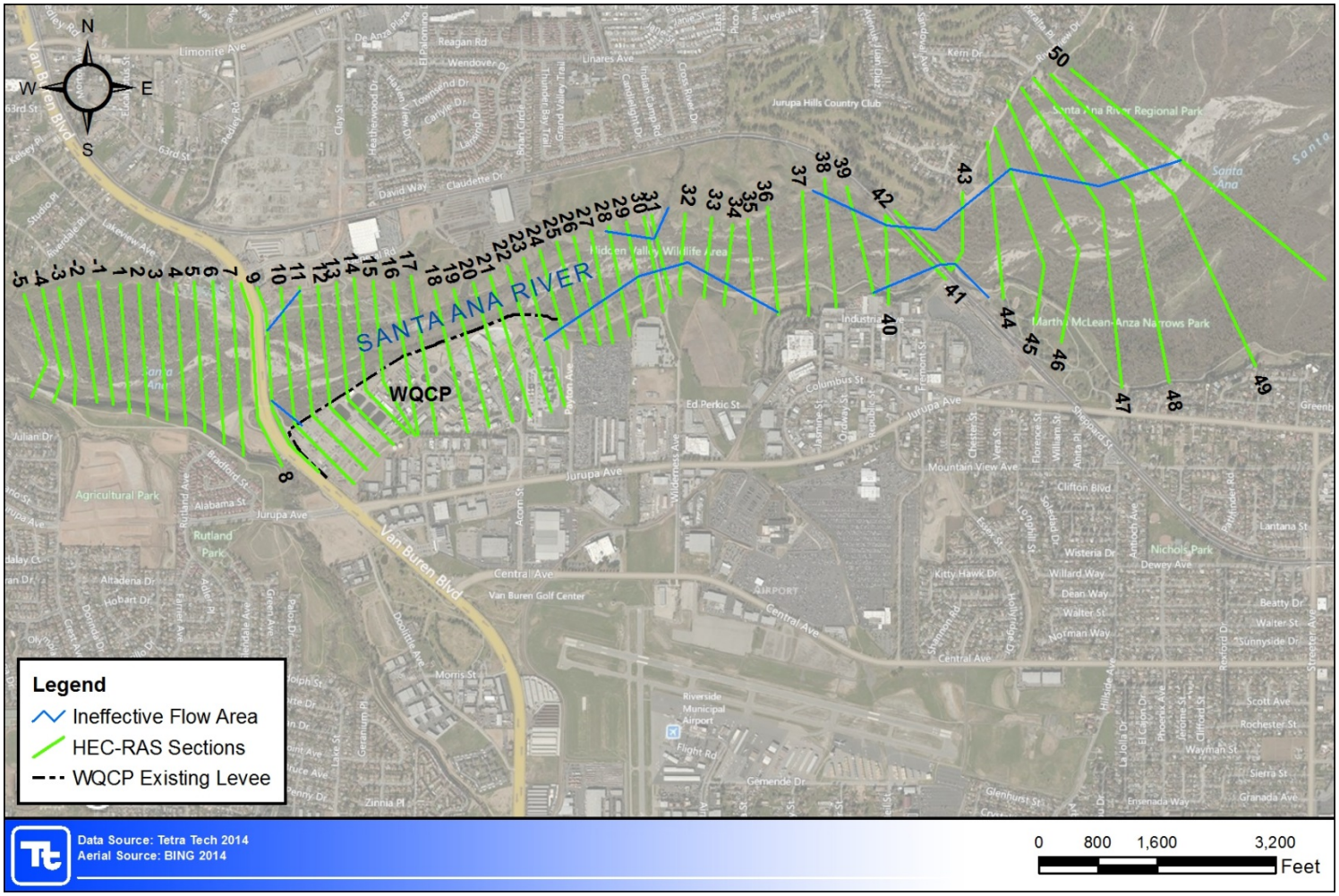


Figure 3.1 HEC-RAS Cross Section Location Map

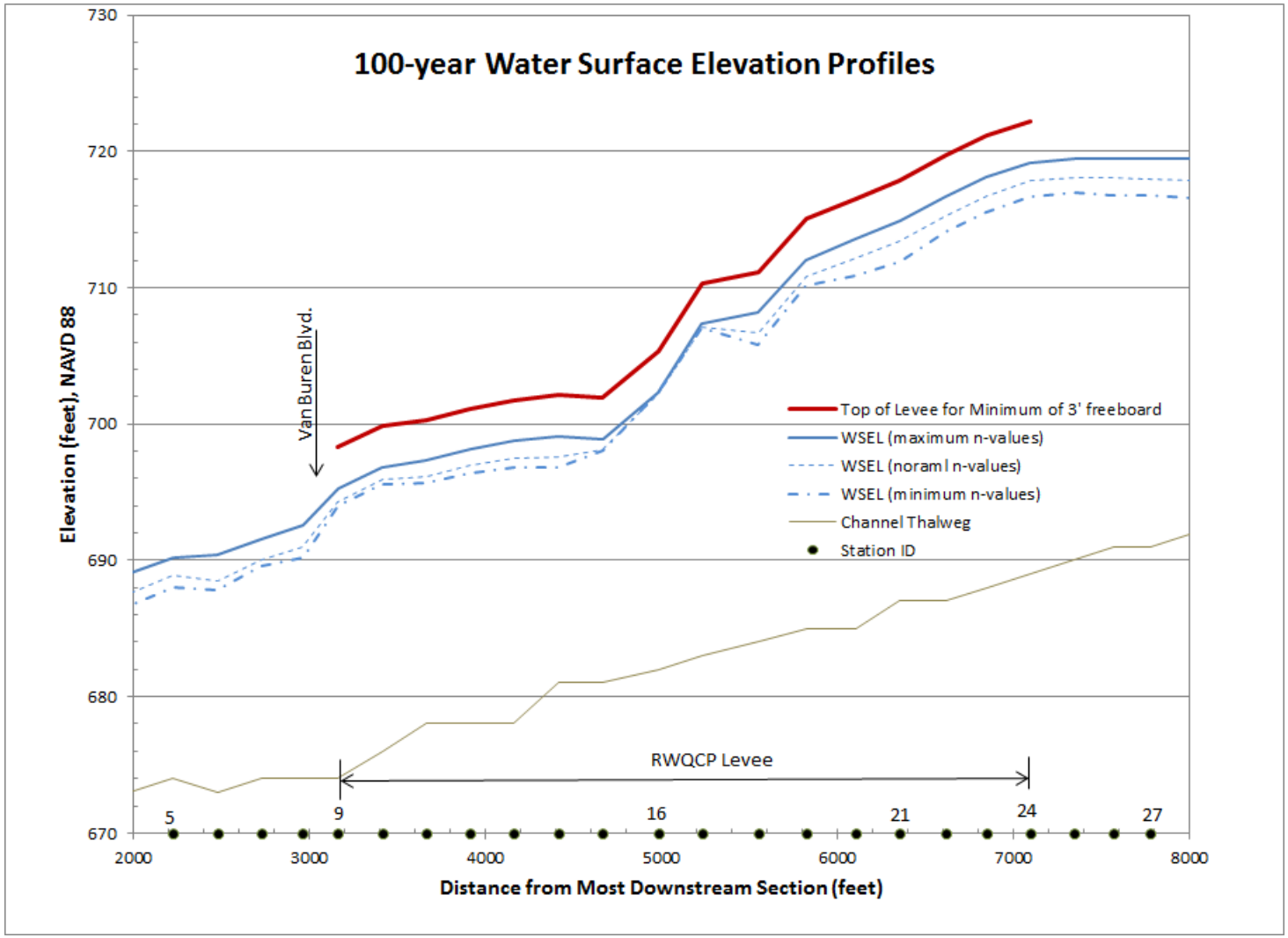


Figure 3.2 Computed 100-year Water Surface Elevation Profiles

# 4. GEOTECHNICAL BASIS FOR DESIGN

## GENERAL

4.1 This section presents the design criteria from the geotechnical evaluation of the RWQCP Levee. The geotechnical evaluation included review of previous geotechnical investigations and performance of supplemental exploration and testing.

4.2 The purpose of this study was to evaluate the general soil/foundation conditions along the existing levee alignment and to provide a professional opinion with regard to the ability of the proposed improvements to contain the 100-year flood event (1 percent annual chance of exceedance) from a geotechnical perspective.

## DESIGN CRITERIA

4.3 The geotechnical design for the RWQCP Levee was performed in general accordance with the criteria set forth in Engineering Manuals (EM), Engineering Technical Letters (ETL), and Engineering Records (ER) published by the Office of the Chief of Engineers of the USACE and accepted engineering practice.

## SUBSURFACE INVESTIGATION

4.4 A geotechnical investigation was previously performed by C.H.J. Incorporated (CHJ) and summarized in their report dated March 18, 2011 (Appendix F). That investigation included five (5) exploratory borings drilled to depths ranging from 40 to 60.5 feet. Subsurface conditions as described in the report consisted of 10 to 18 feet of levee fill overlying native alluvial soils. The levee fill was described as fine to coarse-grained silty sand with gravel. The alluvium was described as gray to reddish brown silty sand underlain by gray sand units that include gravel intervals. Bedrock was not encountered in any of the borings. Groundwater was encountered at depths ranging from 21.5 to 24 feet below the levee crest.

4.5 Tetra Tech conducted field exploration activities on October 22 and 23, 2014 to assess the current geotechnical conditions along the levee. Exploration included drilling of four (4) hollow stem auger borings to depths of 41.5 to 70 feet below the levee crest.

4.6 The subsurface conditions encountered during the 2014 field exploration consisted of levee fill soils extending to depths of approximately 19 to 24 feet below the levee crest elevation, underlain by sandy alluvial soils. At two boring locations, borings B-3 and B-4, granitic bedrock was encountered below the alluvium at a depth of 62 and 48 feet, respectively.

4.7 The underlying alluvial soils encountered in the borings consisted of deposits of silty sand, poorly graded sand, and well graded sand. SPT blowcounts indicated that the alluvial soils are generally loose to medium dense in relative density. A few layers of dense silty sand were encountered in boring B-3.

4.8 The bedrock encountered in borings B-3 and B-4 consisted of weathered, moderately hard, gray quartz monzodiorite. Based on the depth of the bedrock encountered in the borings drilled within the levee crest and the bedrock depths as interpreted from the riverside toe excavations

shown on the construction record drawings, it appears that the top of the bedrock contact is somewhat undulating in the direction of the levee alignment and likely rises in elevation going northward from the levee alignment.

4.9 Groundwater was encountered in all of the test borings performed by Tetra Tech as well as the borings drilled for the CHJ investigation. A summary of groundwater levels is presented in Table 4.1.

Table 4.1 Summary of Groundwater Depths

| Boring | Approx. Surface Elevation (ft) | Groundwater Depth (ft) | Groundwater Elevation (ft) |
|---|---|---|---|
| Tetra Tech B-1 | 702 | 26.8 | 675.2 |
| Tetra Tech B-2 | 702 | 29.0 | 673.0 |
| Tetra Tech B-3 | 703.5 | 26.5 | 677.0 |
| Tetra Tech B-4 | 706.8 | 24.4 | 682.4 |
| CHJ B-1 | 698 | 21.5 | 676.5 |
| CHJ B-2 | 701 | 24.0 | 677.0 |
| CHJ B-3 | 702 | 24.0 | 678.0 |
| CHJ B-4 | 703 | 22.5 | 680.5 |
| CHJ B-5 | 705 | 22.5 | 682.5 |

## GEOTECHNICAL / GEOLOGIC FINDINGS

4.10 The closest faults to the site which are considered active are the San Jacinto fault, mapped approximately 10 miles northeast of the site, the Chino fault, mapped approximately 11 miles southwest of the site, and the Whittier-Elsinore fault, mapped 11.5 miles southwest of the site. The San Andreas Fault is located about 17.5 miles to the northeast of the site. Other nearby active faults meeting the State of California definition include the Sierra Madre-Cucamonga fault, located approximately 17.5 miles north-northwest of the site.

4.11 A summary of the estimated mean peak ground acceleration and selected spectral acceleration values for various return periods are presented in Table 4.2.

Table 4.2 Estimated Peak Ground and Spectral Accelerations

| Return Period | Peak Ground Acceleration | Spectral Acceleration | | |
|---|---|---|---|---|
| | | 0.2 second | 0.3 second | 1 second |
| 108 years | 0.27g | 0.59g | 0.58g | 0.32g |
| 144 years | 0.30g | 0.66g | 0.64g | 0.36g |
| 475 years | 0.45g | 0.97g | 0.96g | 0.56g |
| 949 years | 0.54g | 1.18g | 1.17g | 0.70g |

4.12 Seismic analysis indicated that significant liquefaction potential exists for the on-site alluvial soils. An estimate of potential ground settlement caused by ground shaking and soil liquefaction is presented below in Table 4.3.

Table 4.3 Results of Liquefaction Analyses (based on 475-year return period, PGA = 0.45g)

| Boring No. | Assumed Groundwater Depth | Liquefiable Zone Depth | Factor of Safety $FS_{liq}$ | Liquefaction Settlement | Settlement of Dry Sands | Combined Seismic Settlement |
|---|---|---|---|---|---|---|
| | (ft) | (ft) | – | (inch) | (inch) | (in) |
| B-1 | 20 | 24 – 40 | 0. 36 –0.73 | 4.1 | 0.1 | 4.2 |
| B-2 | | 25 – 39 | 0.35 – 0.60 | 3.7 | 0.1 | 3.8 |
| B-3 | | 25 – 50<br>55 - 62 | 0.42 – 0.86 | 5.9 | 0.1 | 6.0 |
| B-4 | | 25 - 48 | 0.33 – 0.64 | 6.2 | 0.5 | 6.7 |

4.13 Based on field and laboratory testing the following parameters were estimated for the on-site soils.

Table 4.4 Summary of Soil Shear Strength Parameters

| Soil Properties | | Levee Fill | Foundation Alluvial Soil | Riprap |
|---|---|---|---|---|
| Total Unit Weight (pcf) | | 120 | 115 | 130 |
| Effective Shear Strength | Cohesion (psf) | 50 | 0 | 0 |
| | Friction Angle (degrees) | 34 | 31 | 45 |

Table 4.5 Summary of Saturated Hydraulic Conductivity Parameters

| Soil Type | Saturated Hydraulic Conductivity (cm/s) | |
|---|---|---|
| | Vertical | Horizontal |
| Levee Fill | $3 \times 10^{-5}$ | $1.5 \times 10^{-4}$ |
| Alluvial Soil | $5 \times 10^{-2}$ | $5 \times 10^{-2}$ |
| Riprap | $1.5 \times 10^{-1}$ | $1.5 \times 10^{-1}$ |

## DESIGN ALTERNATIVES

4.14 Several design alternatives were geotechnically evaluated. A basic description of the various alternatives is outlined below:

Table 4.6 Summary of Design Alternatives

| Alternative No. | Floodwall Location | | Raise of Existing Levee Fill | Top of Footing | Floodwall Height (ft) | Remarks |
|---|---|---|---|---|---|---|
| | Plant side | Riverside | | | | |
| 1 | X | -- | No | At grade | 0 - 10 | Plant side slope at 2(H):1(V) or flatter |
| 2 | | X | No | both below and at grade | 0 - 10 | |
| 3 | X | -- | Yes – up to 6 feet | At grade | 0 - 4 | Plant side slope at 2(H):1(V) or flatter |
| 4 | | X | Yes – up to 6 feet | At grade | 0 - 4 | Plant side slope at 2(H):1(V) or flatter |

## SETTLEMENT ANALYSIS

4.15 The post–construction total static settlement of the floodwall / levee was estimated to be up to 2 inches. The design of the freeboard capacity of the floodwall should include consideration for this potential settlement. Due to the potential variability of the new fill thickness and the underlying soil conditions along the levee alignment, the floodwalls are recommended to be designed for a differential settlement of 1 inch over a distance of 20 feet.

## SEEPAGE ANALYSIS

4.16 The seepage analyses were initially run as steady state models. For some of the cases, especially with higher floodwall, the steady state seepage condition yielded pore pressure values within the plant side slope that resulted in stability factors of safety less than required by USACE criteria. In reviewing the design flood hydrographs for the project it is evident that the flood event is of limited duration, with the river levels being within 5 feet of the peak for only approximately 5 hours and within 15 feet of the peak for approximately 24 hours. With this condition the use of transient analysis is considered an appropriate method to evaluate seepage conditions within the levee embankment.

4.17 The results of the transient seepage analysis were incorporated into the appropriate stability case as discussed in the following section. The seepage analysis did indicate that during peak flood conditions significantly high exit gradients could develop at the plant side toe of the levee. Gradients as high as approximately 0.45 to 0.5 were estimated in the area where the peak flood level is 6 feet above the current levee crest elevation. In order to mitigate this condition it is recommended that a toe drain be constructed at the plant side toe in order to intercept potential seepage and reduce exit gradients. The toe drain should extend over the length of levee where the design flood level is above the current crest elevation and the plant side slope is steeper than 3(H):1(V).

## STABILITY ANALYSIS

4.18 Results of the stability analyses performed on various levee and floodwall alternatives are summarized in the following table.

Table 4.7 Summary of Slope Stability Factors of Safety

| Stability Case USACE EM1110-2-1913 | Minimum Allowable Value | Calculated Minimum Factor of Safety | | | |
|---|---|---|---|---|---|
| | | Alt 1 | Alt 2 | Alt 3 | Alt 4 |
| Case I - End of Construction | 1.4[1] | 1.78 | 1.81 | 1.73 | 1.58 |
| Case II - Rapid Drawdown | 1.1[2] | 1.19 | 1.18 | 1.19 | 1.19 |
| Case III - Critical Flood Stage [3] | 1.4 | 1.64 | 1.65 | 1.56 | 1.54 |
| Case III - Full Flood | 1.4 | 1.41 | 1.50 | 1.56 | 1.54 |
| Case IV - Earthquake (k = 0.15g) No Liquefaction | 1.1[4] | 1.35 | 1.39 | 1.22 | 1.20 |
| Case IV Post-Earthquake with Liquefaction | 1.1[4] | 0.84 | 0.85 | 0.66 | 0.66 |

1 USACE EM 1110-2-1913 recommends 1.3 for end of construction, however, 1.4 is considered more appropriate for existing conditions.
2 USACE EM 1110-2-1913 recommends a factor of safety of 1.0 to 1.2. A value of 1.1 was used because significant scour was assumed in the drawdown model.
3 USACE EM 1110-2-1913 no longer requires analysis of this case.
4 USACE EM 1110-2-1913 does not currently specify an acceptable factor of safety. A value of 1.1 is a typical standard of practice in southern California.

## POST LIQUEFACTION CONDITIONS

4.19 In the event that soil liquefaction occurs during a major seismic event, fairly extensive soil liquefaction could occur. The estimated post-liquefaction factors of safety are summarized below. The analyses indicate that, in the event of earthquake liquefaction of the foundations soils, the post-earthquake factors of safety will be well below unity.

Table 4.8 Summary of Post-Earthquake Liquefaction Slope Stability (Liquefaction Considered)

| Alternative | Factor of Safety | |
|---|---|---|
| | Plant side Slope | River side Slope |
| 1 | 0.87 | 0.84 |
| 2 | 1.06 | 0.85 |
| 3 | 0.79 | 0.66 |
| 4 | 0.79 | 0.66 |

4.20 Mitigation of liquefaction potential could be provided by a number of remediation methods as outlined in the Geotechnical Report. Any of these methods would be expensive and would require construction access to both the plant side and river side areas of the levee.

4.21 If the City elects not to mitigate the liquefaction potential of the foundation soils, emergency response and repair of damage caused by seismically induced deformation would be a necessary component of the Operations and Maintenance (O&M) protocol for the levee. The protocol should require that following a significant earthquake event in the region, an experienced geologist or seismologist be consulted. Data regarding a seismic event (magnitude, peak ground acceleration, etc.) can be found at the internet site http://earthquake.usgs.gov/earthquakes/. Inspection of the levee slopes should be performed after significant seismic events including:

- A magnitude 5.5 or greater earthquake with an epicenter located within a 50 mile radius of the levee embankments.
- Any seismic event that generates a peak ground acceleration of 0.10g or greater in the proximity of the levee embankments.
- Any seismic event causing ground cracking or deformation in the vicinity of the floodwall or levee embankments.

Where any damage is encountered that would inhibit the flood protection ability of the levee or floodwalls, immediate repair should be implemented.

4.22 If the City has future plans for obtaining FEMA certification of the levee, then liquefaction remediation will need to be implemented.

## FLOODWALL FOUNDATIONS

4.23 Foundation design parameters for floodwall are presented below.

Table 4.9 Floodwall Geotechnical Design Parameters (Continuous Spread Footing Foundations)

| | |
|---|---|
| **Foundation Embedment** | • At least 24 inches from the levee crest to bottom of footing |
| **Allowable Bearing Capacity** | • 2,000 psf – FEMA Flood Load (3 feet of freeboard)<br>• 2,600 psf – Maximum Flood (flood level at top of wall) |
| **Estimated Settlement** | • Where levee embankment raise is performed, up to 2-inches total settlement. For other alternatives, less than 1-inch total settlement.<br>• Where levee embankment raise is performed, 1-inch differential settlement over 20 feet. For other alternatives, up to 0.5-inch differential settlement over 20 feet |
| **Ultimate Coefficient of Friction** | • 0.45 (concrete vs. soil), 0.60 (soil vs. soil) |

| | |
|---|---|
| **Ultimate Lateral Passive Resistance** | The ultimate lateral passive resistance should be calculated with the following parameters:<br>• Ultimate passive earth pressure coefficient $K_p$ = 3.54.<br>• Unit weight of 120 pcf for compacted backfill.<br>• Where bare ground (no concrete or asphalt cover) is present adjacent to the foundation passive resistance should be ignored for the upper 12 inches below grade, therefore, a trapezoidal pressure distribution should be used starting at 12 inches below grade.<br>• A minimum horizontal distance of 5 feet, measured at the top of footing, shall be provided between the near face of the footing and the face of the finished slope. |
| **Uplift** | • Uplift should be assumed to exist below the base of the footing on the riverside of the key. Where no key is present uplift should be assumed on the riverward 1/3 of the footing. The magnitude of the uplift can be estimated based on the full flood head. |

## RETAINING WALL FOUNDATIONS

4.24 Foundation design parameter for retaining walls are presented below:

Table 4.10 Retaining Walls Geotechnical Design Parameters (Continuous Spread Footing Foundations)

| | |
|---|---|
| **Foundation Embedment** | • At least 24 inches below lowest adjacent ground |
| **Allowable Bearing Capacity** | • 2,000 psf |
| **Estimated Settlement** | • Up to 1-inch total settlement.<br>• Up to 0.5-inch differential settlement over 20 feet |
| **Ultimate Coefficient of Friction** | • 0.45 (concrete vs. soil) |
| **Ultimate Lateral Passive Resistance** | The ultimate lateral passive resistance should be calculated with the following parameters:<br>• Ultimate passive earth pressure coefficient $K_p$ = 3.12.<br>• Unit weight of 120 pcf for alluvium.<br>• Passive resistance should be ignored for the upper 12 inches below grade, therefore, a trapezoidal pressure distribution should be used starting at 12 inches below grade. |

4.25 Recommended earth pressures against retaining walls are presented below:

Table 4.11 Summary of Earth Pressure

| **Design Condition** | **Equivalent Fluid Density (pcf)** |
|---|---|
| Active Lateral Earth Pressure | 51 |
| At-Rest Pressure | 80 |
| Seismic Earth Force (100 year return period) | 35.7 |
| Seismic Earth Force (144 year return period) | 39.3 |
| Seismic Earth Force (475 year return period) | 59.2 |
| Seismic Earth Force (949 year return period) | 70.8 |
| Seismic Earth Force (MCE) | 57.7 |

## CORROSION POTENTIAL

4.26 Based on the measured water soluble sulfate results the exposure of buried concrete to sulfate attack should be considered “not applicable”, i.e., exposure class S0 per ACI 318, Table 4.2.1. Consequently, injurious sulfate attack is not a concern for concrete with a minimum 28-day compressive strength of 2,500 psi. Per 2013 CBC, Section 1904.1, concrete reinforcement should be protected from corrosion and exposure to chlorides in accordance with ACI 318, Section 4.3.

4.27 The minimum resistivity from the CHJ testing was 1,880 and 4,400 ohm-cm and the pH values were 7.4 and 8.0. This testing indicate that the soils at the site may be classified as having a moderate to high corrosivity potential for buried mild steel. Consequently a preference should be given to a plastic piping and connections. A corrosion specialist should be consulted regarding suitable types of piping and necessary protection for underground metal conduits.

# 5. STRUCTURAL BASIS FOR DESIGN

## GENERAL

5.1 This section presents the design criteria for the structural design of the RWQCP Levee Floodwall and Levee Retaining Walls. Final analyses and design of structures have been performed to refine shape, quantities, and cost estimate, as well as develop detailing. The structures consist of reinforced concrete floodwall, and reinforced concrete retaining walls supporting the levee toe of slope on the protected side, with sections of varying height. The final structural analyses and design calculations are included in the Appendices.

## DESIGN CRITERIA

5.2 The structural design of the RWQCP Levee Floodwall and Levee Retaining Walls are in accordance with the criteria set forth in Engineering and Design Manuals (EM) published by USACE Headquarters, including *EM 1110-2-2502, Retaining and Flood Walls (1989), EM 1110-2-2100, Stability Analysis of Concrete Structures (2005),* and *EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures (2003).*

5.3 The floodwall and retaining walls were analyzed for stability and strength in accordance with EM 1110-2-2502, with seismic load conditions, seismic analysis coefficients and loading classifications in accordance with EM 1110-2-2100. The floodwall was also analyzed for wind load in accordance with ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures.* Structural design of the various floodwall and retaining wall sections is in accordance with *EM 1110-2-2502, EM 1110-2-2104,* and *ACI 318-11, Building Code Requirements for Structural Concrete,* using the computer program, "Retain Pro".

5.4 Load cases for the design of the floodwall and retaining walls are summarized in the following table:

Table 5.1 Summary of Wall Design Load Cases

| Floodwall | Retaining Walls |
|---|---|
| I1 – Infrequent/100-Year Flood | R1 – Normal Operating (NO) |
| I2 – Maximum Design Flood | R2 – NO + Short Duration Loads (Incl. Flood) |
| I3a – Operational Basis Earthquake (OBE) | R3a – NO + OBE |
| I3b – Maximum Credible Earthquake (MCE) | R3b – NO + Max. Design Earthquake (MDE) |
| I4 – Construction | |
| Wind | |

## CONSTRUCTION MATERIALS

5.5 Concrete design strength for structural elements is based on a minimum 28-day compressive strength, f'c = 4000 psi, for concrete floodwall and retaining wall structures.

5.6 Reinforcing steel for the floodwall and retaining walls is ASTM A615, Grade 60, with a yield strength, fy = 60,000 psi.

## RECOMMENDED SOIL DESIGN VALUES

5.7 The design soil parameters are presented in Section 4, *Geotechnical Basis for Design*, of this Design Report.

## STABILITY ANALYSES

5.8 For load conditions that include flood loading, floodwall and retaining wall stability analyses were performed utilizing the USACE computer program, "CTWALL". Stability analyses for load conditions that did not include flood loading were performed using the computer program, "Retain Pro". The floodwall and retaining walls were designed for stability in accordance with EM 1110-2-2502, with stability criteria as shown in Tables 5.2 and 5.3, respectively, below.

Table 5.2 Floodwall Stability Criteria

| Load Case | Sliding Factor of Safety (Min.) | Overturning Factor of Safety (Min.) | Min. % of Base in Compression | Soil Bearing Pressure (Max.) |
|---|---|---|---|---|
| I1 | 1.5 | 1.5 | 100 | 2000 psf |
| I2 | 1.33 | 1.33 | 75 | 2600 psf |
| I3a | 1.3 | 1.3 | 75 | 2000 psf |
| I3b | 1.1 | 1.1 | Resultant within base | 2000 psf |
| I4 | 1.5 | 1.5 | 75 | 2000 psf |
| Wind | 1.5 | 1.5 | 75 | 2000 psf |

Table 5.3 Retaining Wall Stability Criteria

| Load Case | Sliding Factor of Safety (Min.) | Overturning Factor of Safety (Min.) | Min. % of Base in Compression | Soil Bearing Pressure (Max.) |
|---|---|---|---|---|
| R1 | 1.5 | 1.5 | 100 | 2000 psf |
| R2 | 1.33 | 1.33 | 75 | 2000 psf |
| R3a | 1.3 | 1.3 | 75 | 2000 psf |
| R3b | 1.1 | 1.1 | Resultant within base | 2000 psf |

# 6. PRELIMINARY ENVIRONMENTAL ASSESSMENT

## SUMMARY OF KEY ENVIRONMENTAL ISSUES

6.1 The proposed project would occur alongside a reach of the Santa Ana River corridor that supports sensitive biological resources, including habitat for special status species and jurisdictional wetland resources as follows:

- Perennial stream habitat for the Santa Ana sucker (*Catostomus santaanae*), which is federally listed threatened by the U.S. Fish and Wildlife Service (USFWS) under the Endangered Species Act (ESA) and also designated a California State species of special concern by the California Department of Fish and Wildlife (CDFW).
- Riparian habitat for the least Bell's vireo (*Vireo bellii pusillus*), which is federally listed endangered by the USFWS under the ESA and California State listed endangered by the CDFW under the California Endangered Species Act (CESA).
- Critical habitat designated by the USFWS for the Santa Ana sucker and least Bell's vireo.
- Stream and riparian habitat for other non-listed California State species of special concern, including arroyo chub (*Gila orcuttii*), Santa Ana speckled dace (*Rhinichthys osculus* ssp. *3*), and western mastiff bat (*Eumops perotis californicus*).
- Nesting habitat for migratory birds protected under the federal Migratory Bird Treaty Act (MBTA) and California Fish and Game Code (CFG Code).
- Wetland and non-wetland waters of the U.S. subject to regulation by the U.S. Army Corps of Engineers (USACE) pursuant to Section 404 of the federal Clean Water Act (CWA).
- Wetland and non-wetland waters of the State subject to regulation by the Regional Water Quality Control Board (RWQCB) pursuant to Section 401 of the CWA.
- Unvegetated streambed and riparian habitat subject to regulation by the CDFW pursuant to Sections 1600 *et seq*. of CFG Code.

6.2 The proposed project has been specifically designed to avoid and minimize impacts to sensitive biological resources. The current design restricts the proposed levee improvements to disturbed and developed upland areas on the RWQCP side of the existing levee structure. As such, direct impacts to sensitive resources on the Santa Ana River corridor side of the levee would be completely avoided.

6.3 As currently designed, the proposed project is expected to have no impacts, direct or indirect, on perennial stream habitat in the Santa Ana River and therefore no effect on the Santa Ana sucker or its designated critical habitat. Similarly, the project would have no effect on other special status fishes, including arroyo chub and speckled dace.

6.4 The project is expected to have no direct impact on riparian habitat or special status species potentially occupying riparian habitat, including least Bell's vireo. Further, the project would have no effect on designated critical habitat for the least Bell's vireo. However, the project could result in adverse indirect impacts on vireo if the species is actively breeding in the immediate area during construction. Construction equipment that could generate noise in excess of 60 dB leq, as measured from the location of an active nest, could inadvertently disrupt breeding behavior and cause an active nest to fail, which could be considered "take" of the species. To the extent possible, project construction should avoid the breeding season for least Bell's vireo, which is defined as occurring

between March 1 and September 15. If the breeding season cannot be avoided, implementation of noise attenuation measures (e.g., sound walls, blankets, bafflers, etc.) and consultation and permitting with the USFWS and CDFW could be required.

6.5 Limited tree and shrub vegetation occurs on the RWQCP side of the existing levee that could be impacted by the project. This vegetation could provide suitable habitat for nesting birds protected under the MBTA and CFG Code. Therefore, direct impacts to nesting birds could occur during project construction. To the extent possible, removal of potential nesting habitat should occur outside of the general bird breeding season, which is defined as occurring between January 15 and September 15. If removal of nesting habitat cannot occur outside of the bird breeding season, a pre-construction (pre-removal) nesting bird survey should be conducted within three days prior to the activities to ensure no active nests occur within the habitat to be removed.

6.6 As currently designed, the project would avoid impacts to jurisdictional waters and wetlands subject to regulation by the USACE, RWQCB, and CDFW. The extent of potential USACE and RWQCB jurisdiction is expected to extend across the active floodplain for the Santa Ana River, from the toe-of-slope on the northern embankment to the toe-of-slope on the southern embankment. The project includes levee rehabilitation at the very top of the southern embankment for the Santa Ana River. No activities would occur down at the toe-of-slope or across the active floodplain areas for the Santa Ana River. As such, no fill, discharge, or dredge activities would occur within potential USACE and RWQCB jurisdiction. The extent of potential CDFW jurisdiction is expected to extend across the bankfull width of the Santa Ana River, from the top-of-slope on the northern embankment to the top-of-slope on the southern embankment. Potential CDFW jurisdiction would extend beyond the top-of-slope where riparian vegetation occurs. The project would build onto the top-of-slope on the southern embankment for the Santa Ana River. There is no riparian vegetation that extends beyond the top-of-slope on the southern embankment. Therefore, no alteration of streambed or removal of riparian habitat would occur within potential CDFW jurisdiction.

## POTENTIAL PERMITTING REQUIREMENTS

6.7 As currently designed, and assuming project construction can avoid the least Bell's vireo breeding season (March 1 – September 15), the project is not expected to require consultation or permits from the USFWS or CDFW. If the vireo breeding season cannot be avoided, implementation of noise attenuation measures (e.g., sound walls, blankets, bafflers, etc.) and consultation and permitting with the USFWS and CDFW could be required pursuant to the ESA and CESA. Similarly, as currently designed and assuming project construction would avoid jurisdictional wetland resources, the project is not expected to require permits from the USACE, RWQCB, or CDFW.

6.8 If the design changes or temporary encroachment into the Santa Ana River side of the existing levee is necessary during construction, then agency consultation and permitting would be required. Encroachment would require notification and permitting with the USFWS, USACE, RWQCB, and/or CDFW, depending on the extent of encroachment. If encroachment is limited to the existing riprap streambank above the toe-of-slope, then notification and permitting would only be required with CDFW. Under this scenario, a standard 5-year Streambed Alteration Agreement would be issued by CDFW for the activities. If encroachment is required down at the toe-of-slope

and north into the active floodplain, then notification and permitting would be required with USACE, RWQCB, and CDFW. Under this scenario, a CWA Section 404 permit [e.g., Nationwide Permit 3 (Maintenance), 13 (Bank Stabilization), or 31 (Maintenance of Existing Flood Control Facilities)], CWA Section 401 Water Quality Certification, and standard 5-year Streambed Alteration Agreement would be issued by the USACE, RWQCB, and CDFW, respectively. Further, if the encroachment would impact perennial stream or riparian habitat, impacts to the Santa Ana River and least Bell's vireo could occur, including designated critical habitat. Under this scenario, potential effects on sucker and vireo (and critical habitat) would require formal consultation between the USACE and USFWS pursuant to Section 7 of the ESA. The USFWS would issue a Biological Opinion and Incidental Take Statement for the project that would become attached to the USACE's CWA Section 404 permit. A consistency analysis would then likely be issued for impacts to vireo by CDFW pursuant to Section 2080.1 of the CESA.

# 7. COST ESTIMATES

## BASIS OF COST ESTIMATES

7.1 Cost estimates for recommended plan have been develop in accordance with the design information referenced in this document. The detailed cost estimates and quantity take-offs are presented in Appendix D.

7.2 Unit prices were derived from previously estimated unit costs for similar work in the Riverside area along with the RSMeans cost database. The cost estimates were developed for analysis of construction cost, and therefore are not total project cost estimates. Thus, the cost estimates do not account for real estate acquisitions; planning, or construction management costs.

7.3 The construction cost for the 100% Improvements is $2,861,000.

# 8. RECOMMENDATIONS

8.1 This Design Report describes in detail the 100% design of the preferred plan (alternative 3), including project issues and permitting requirements. It is recommended that this report provide the basis for the development of plans and specifications for the construction of the RWQCP Levee.

8.2 The estimated total construction cost of the project is approximately $2,861,000.

## REPORTS PREVIOUSLY ISSUED

Reports previously issued related to the project are as follows:

a. *Tetra Tech. 2014. Preliminary Design Report for the Riverside Regional Water Quality Control Plant Levee Rehabilitation. April 2015.*

b. *Albert A. Webb Associates. 2011. Water Quality Control Plant Levee Deficiency Analysis, revised July 2011.*

c. *Tetra Tech. 2012. Final Hydraulic, Scour, and Sediment-Transport Analyses Report. May 2012.*

d. *Federal Emergency Management Agency (FEMA). 2014. Flood Insurance Study, Riverside County, California and Incorporated Areas. August 2014.*

# REFERENCES


1. Design Memorandum No. 2, "Santa Ana River Mainstem Project Including Santiago Creek Feature Design - Seven Oaks Dam Floodway Delineation", U.S. Army Corps of Engineers, August, 1991.

2. EM 1110-2-1601, *"Hydraulic Design of Flood Control Channels"*, U.S. Army Corps of Engineers, 30 June, 1994.

3. EM 1110-2-1902, *"Slope Stability"*, U.S. Army Corps of Engineers, 31 October, 2003.

4. EM 1110-2-1913, *"Design and Construction of Levees"*, U.S. Army Corps of Engineers, 30 April, 2000.

5. EM 1110-2-2000, *"Standard Practice for concrete for Civil Works Structures"*, U.S. Army Corps of Engineers, Change 2, 31 March 2001.

6. EM 1110-2-2007, *"Structural Design of Concrete Lined Flood Control Channels"*, U.S. Army Corps of Engineers, 30 April 1995.

7. EM 1110-2-2100, *"Stability Analysis of Concrete Structures"*, U.S. Army Corps of Engineers, 1 December, 2003.

8. EM 1110-2-2102, *"Waterstops and Other Preformed Joint Materials for Civil Works Structures"*, 30 September 1995.

9. EM 1110-2-2104, *"Strength Design for Reinforced-Concrete Hydraulic Structures"*, 20 August, 2003.

10. EM 1110-2-2502, *"Retaining and Flood Walls"*, 29 September 1989.

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14. ETL 1110-2-322, *"Retaining and Flood Walls"*, October 1990.

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16. American Concrete Institute, *"Building Code Requirements for Reinforced Concrete, ACI 318M-95 and Commentary, ACI 318RM-95"*.

17. American Institute of Steel Construction (AISC), *"Manual of Steel Construction"*, ASD, Ninth Edition, 1989.

18. Waterways Experimental Station (WES), Corps of Engineers Computer Program, *"Concrete Strength Investigation and Design (CASTR)"*, May 1987.

19. Waterways Experimental Station (WES), Corps of Engineers Computer Program, *"Analysis of Retaining and Flood Walls (CTWALL)"*, 30 October 1993.

20. Albert A. Webb Associates. 2011. *Water Quality Control Plant Levee Deficiency Analysis*, revised July 2011.

21. CHJ, Inc., Geotechnical Investigation, Proposed Riverside Regional Water Quality Control Plant Levee, Van Buren Boulevard, Riverside County, California, Project No. 11708-3, Dated March 2011.

22. Chow, V. T. 1959, *Open Channel Hydraulics*, McGraw-Hill, New York.

23. Federal Emergency Management Agency (FEMA). 2014. Flood Insurance Study, Riverside County, California and Incorporated Areas. August 2014.

24. USACE. January 2010, HEC-RAS River Analysis System, version 4.1.0. Hydrologic Engineering Center. Davis, California.

25. Tetra Tech. 2012. *Final Hydraulic, Scour, and Sediment-Transport Analyses Report*. May 2012.

26. Tetra Tech. 2015. *Riverside Regional Water Quality Control Plant Levee Rehabilitation Preliminary Design Report (PDR)*. May 2015.

Appendix 11C

# RWQCP LEVEE REHABILITATION COST ESTIMATE

Engineers...Working Wonders With Water®

# CITY OF RIVERSIDE PUBLIC WORKS

## UPDATED INTEGRATED MASTER PLAN FOR WASTEWATER COLLECTION AND TREATMENT FACILITIES PROJECT

**TASK :** 7 - CIP and Financial Plan and User Rates and Fees
**JOB # :** 10495A.00
**LOCATION :** Riverside, CA
**TITLE :** Levee Rehabilitation Phase II

**ESTIMATE PREPARATION DATE :** 3/1/2016
**PREPARED BY :** TetraTech
**REVIEWED BY :**

| ITEM NO. | DESCRIPTION | QTY | UNIT | UNIT COST | SUBTOTAL | TOTAL |
|---|---|---|---|---|---|---|
| | | | | | | |
| 1 | **Mobilization** | | | | | |
| | Mobilization | 1.0 | LS | $85,654 | $85,700 | |
| | Total | | | | | $85,700 |
| | | | | | | |
| 2 | **Site Preparation** | | | | | |
| | Site Preparation | 1 | LS | $41,783 | $41,800 | |
| | Total | | | | | $41,800 |
| | | | | | | |
| 3 | **Earthwork** | | | | | |
| | Structural Excavation | 81925 | CY | $30 | $2,457,800 | |
| | Grouted Riprap | 2102 | CY | $200 | $420,400 | |
| | Total | | | | | $2,878,200 |
| | | | | | | |
| 4 | **Permitting** | | | | | |
| | Permitting | 1.0 | LS | $1,000,000 | $1,000,000 | |
| | Total | | | | | $1,000,000 |
| | | | | | | |
| | | | | | | |
| | **ITEM NO. 1,2,3,and 4 SUBTOTAL** | | | | | **$4,119,121** |
| | | | | | | |
| 5 | **Allowances** | | | | | |
| | | | | | | |
| | Construction Difficulty | 5 | % | | $205,956 | |
| | Total | | | | | $206,000 |
| | | | | | | |
| | **SUBTOTAL** | | | | | **$4,325,121** |
| | | | | | | |
| | Estimating Contingency | 30 | % | | | $1,298,000 |
| | **SUBTOTAL** | | | | | **$5,623,121** |
| | | | | | | |
| | General Conditions | 10 | % | | | $562,312 |
| | **SUBTOTAL** | | | | | **$6,185,434** |
| | | | | | | |
| | General Contractor Overhead & Profit | 15 | % | | | $646,659 |
| | **SUBTOTAL** | | | | | **$6,269,780** |
| | | | | | | |
| | Escalation to June 2018 | 3 | % | | | $188,093 |
| | **SUBTOTAL** | | | | | **$6,457,874** |
| | | | | | | |
| | Sales Tax on 50% of Subtotal Above | 8.75 | % | | | $549,000 |
| | **CONSTRUCTION COST SUBTOTAL** | | | | | **$6,818,780** |
| | | | | | | |
| | Engineering, Management, and Legal | 30 | % | | | $2,046,000 |
| | Phave II Preliminary Design Report Allowance | | | | | $250,000 |
| | | | | | | |
| | **PROJECT COST** | | | | | **$9,114,800** |