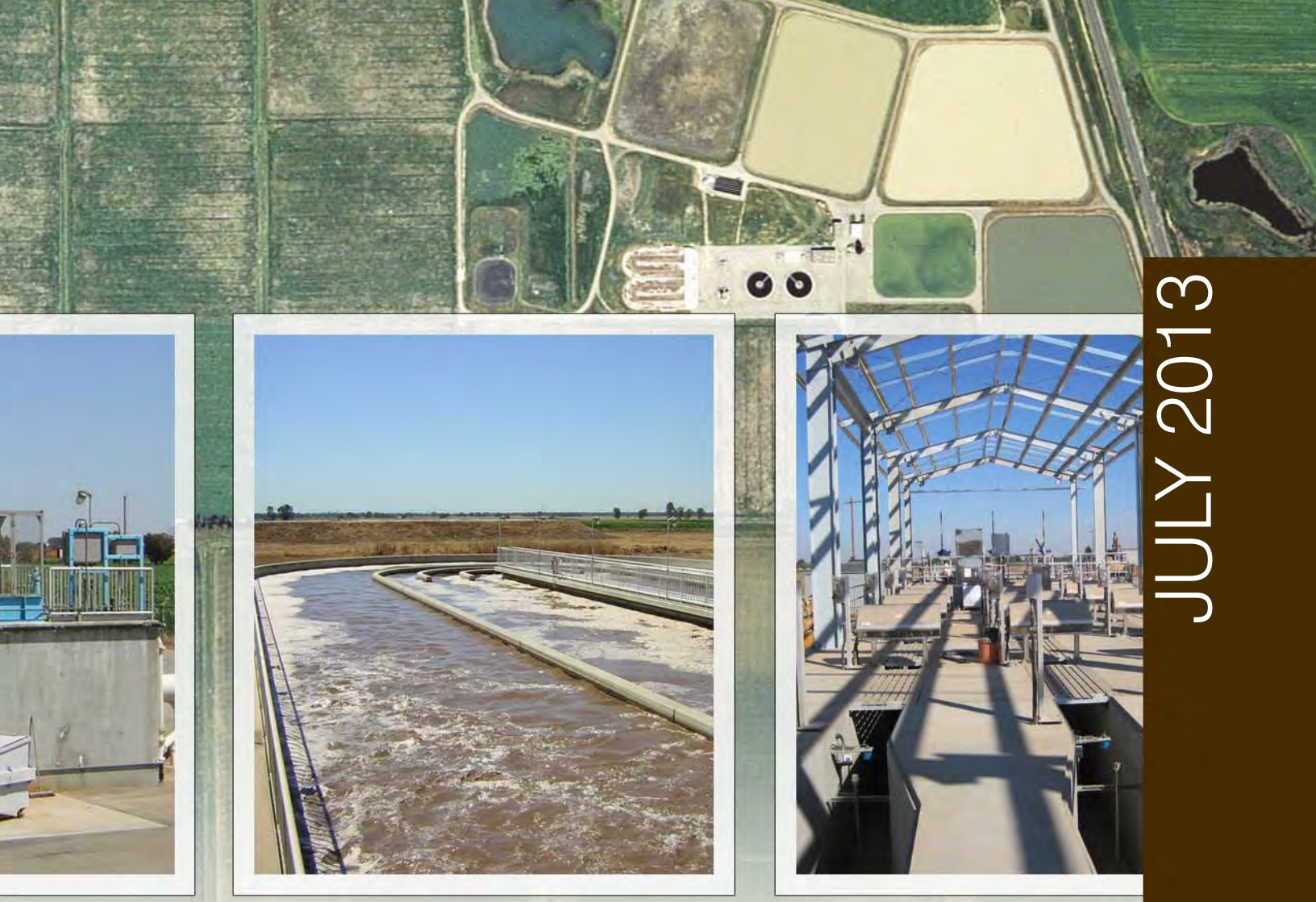

JULY 2013

CITY OF GALT

Wastewater Treatment Plant Facilities Master Plan

WEST YOST ASSOCIATES

City of GALT

Wastewater Treatment Plant Facilities Master Plan

Prepared for

City of Galt

July 2013

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Table of Contents

Executive Summary

ES.1 WWTP Overview ES-1

ES.2 Capacity Needs ES-1

ES.3 Near-Term Upgrade Improvements ES-2

ES.4 Long-Term Expansion Improvements ES-5

ES.5 Implementation Plan ES-7
- ES.5.1 Phase 2 Plan: Current Permitted Design Condition ES-7
- ES.5.2 Phase 3 Plan: Future Permitted Design Condition ES-7
- ES.5.3 Phase 4 Plan: Ultimate Buildout Design Condition ES-7
- ES.5.4 Arsenic Compliance Plan ES-8

ES.6 Site Layout and Costs ES-8

Chapter 1. Introduction

1.1 Facilities Master Plan Report Overview 1-1

1.2 WWTP Overview 1-2

1.3 Previous WWTP Planning and Upgrade Efforts 1-2

1.4 Permit Compliance Strategies Not Related to WWTP Upgrade Projects 1-3
- 1.4.1 Arsenic 1-3
- 1.4.2 Copper 1-3

1.5 Proposed Phasing Plan 1-4

1.6 References 1-5

Chapter 2. Basis of Design

2.1 Projected Average Dry Weather Flow 2-1
- 2.1.1 Previously Developed Flow Projections 2-1
- 2.1.2 Recommended Projection for WWTP Planning 2-4
- 2.1.3 Impacts of Water Conservation 2-6
- 2.1.4 Recommended Flow Projection 2-10

2.2 Design Flow Characteristics 2-11
- 2.2.1 Recommended Flow Peaking Factors 2-11
- 2.2.2 Design Flows 2-14

2.3 Design Load Characteristics 2-14
- 2.3.1 Historical WWTP Influent Load Data 2-15
- 2.3.2 Design Load Peaking Factors 2-16
- 2.3.3 Design Loads 2-17
- 2.3.4 Secondary Process Influent Load Characterization 2-18

2.4 Current Regulatory Requirements 2-23
- 2.4.1 Surface Water Discharge Effluent Limitations 2-23
- 2.4.2 Filtration Operational Requirements 2-25
- 2.4.3 UV Disinfection Operational Requirements 2-27
- 2.4.4 Biosolids Land Application Requirements 2-28
- 2.4.5 Groundwater Limitations 2-31

2.5 Reliability/Redundancy Requirements 2-32

2.6 References 2-36

Chapter 3. Existing Treatment Plant Performance and Condition

Chapter 3 3-1
3.1 WWTP Construction History 3-1
3.2 Liquid Treatment Processes 3-3
3.2.1 Headworks and Screening 3-4
3.2.2 Grit Removal 3-6
3.2.3 Oxidation Ditches 3-7
3.2.4 Secondary Clarifiers 3-9
3.2.5 Polymer Storage and Dosing Facility 3-11
3.2.6 Filter Feed and Utility Water Pump Station 3-12
3.2.7 Cloth Disc Filters 3-14
3.2.8 Tertiary Plant Drain Pump Station 3-15
3.2.9 UV Disinfection 3-16
3.2.10 Effluent Diversion Structure 3-18
3.2.11 Effluent Flow Metering 3-19
3.2.12 Chlorine Contact Chamber (Abandoned) 3-19
3.3 Solids Treatment Processes 3-20
3.3.1 Secondary Sludge Pump Station 3-20
3.3.2 Sludge Lagoons 3-22
3.3.3 Sludge Feed Pump Station 3-23
3.3.4 Biosolids Polymer Facility 3-24
3.3.5 Biosolids Dewatering Beds 3-25
3.3.6 Filtrate Pump Station 3-27
3.3.7 Biosolids Storage Area 3-28
3.4 Storage and Irrigation Systems 3-29
3.4.1 Storage Facilities 3-29
3.4.2 Auxiliary Basin Pump Station 3-31
3.4.3 Irrigation Facilities 3-31
3.4.4 Irrigation Pump Stations 3-33
3.4.5 WWTP Water Systems 3-33
3.4.6 High Pressure Utility Water (W3) 3-34
3.4.7 Non-potable Water (W2) System 3-34
3.5 Electrical and Control Systems 3-35
3.5.1 Utility Service 3-36
3.5.2 Electrical Switchgear 3-36
3.5.3 Standby Generators 3-37
3.5.4 Motor Control Centers 3-38
3.5.5 SCADA System 3-39
3.5.6 Solar Facility 3-40
3.6 Buildings and storage facilities 3-40
3.6.1 Operations & Electrical (O&E) Building 3-41
3.6.2 Control Building 3-42
3.6.3 Chemical / Maintenance Building 3-43
3.6.4 Vehicle Storage Building 3-44
3.6.5 Covered Equipment Storage 3-44
3.7 Other Ancillary Facilities 3-44
3.7.1 Compressed Air System 3-45
3.7.2 Fire Protection 3-45
3.7.3 Site Lighting 3-45

3.7.4 Site Access and Security 3-45
3.7.5 Site Drainage 3-46
3.7.6 Parking and Ingress/Egress 3-46
3.7.7 Non-Wastewater Related Site Uses 3-46
3.7.8 Landscaping 3-47
3.8 References 3-47

Chapter 4. Secondary Treatment Capacity Analysis and Plant Hydraulics

4.1 Oxidation Ditch Treatment Capacity 4-1
4.2 Secondary Clarifier Treatment Capacity 4-4
4.3 Hydraulic Capacity 4-10
4.3.1 Spreadsheet Model Development 4-10
4.3.2 Available Data and Assumptions 4-11
4.3.3 WWTP Hydraulic Capacity Results 4-12
4.4 References 4-16

Chapter 5. Biosolids Handling Facilities

5.1 Projected Biosolids Production 5-1
5.1.1 Current Solids Balance 5-1
5.1.2 Projected Production 5-2
5.2 Biosolids Dewatering Facilities Alternatives Evaluation 5-3
5.2.1 Existing Capacity Evaluation 5-4
5.2.2 Design Dewatering Bed Operating Assumptions 5-6
5.2.3 Evaluation of Alternatives 5-7
5.2.4 Comparison of Estimated Costs 5-10
5.2.5 Recommended Alternative 5-13
5.3 Biosolids Disposal Facilities Alternatives Evaluation 5-13
5.3.1 Existing Capacity Evaluation 5-14
5.3.2 Alternatives for Expansion 5-17
5.4 Summary and Conclusions 5-18
5.5 References 5-18

Chapter 6. WWTP Expansion Alternatives

6.1 Current Permitted Design Condition Upgrade Alternatives 6-1
6.1.1 Project Components Common to All Alternatives 6-1
6.1.2 Oxidation Ditch Upgrade Alternatives 6-6
6.1.3 Secondary Clarifier Capacity Needs 6-16
6.1.4 Tertiary Filtration and UV Disinfection Facilities Requirements 6-18
6.1.5 Summary of Potential Upgrade Alternatives 6-19
6.2 Future Permitted Design Condition (4.5 MGD ADWF) Expansion Alternatives 6-19
6.2.1 Project Components Common to All Alternatives 6-20
6.2.2 Oxidation Ditch Expansion Alternatives 6-22
6.2.3 Secondary Clarifier Capacity Needs 6-26
6.2.4 Tertiary Filtration and UV Disinfection Facilities Requirements 6-28
6.2.5 Summary of Potential Expansion Alternatives 6-28
6.3 Ultimate Buildout Design Condition (6.0 MGD ADWF) expansion Alternatives 6-29

6.3.1 Project Components Common to All Alternatives 6-29
6.3.2 Oxidation Ditch Expansion Alternatives 6-30
6.3.3 Secondary Clarifier Capacity Needs 6-33
6.3.4 Tertiary Filtration and UV Disinfection Facilities Requirements 6-35
6.3.5 Summary of Potential Expansion Alternatives 6-35

6.4 Approaches to Facilities Expansion 6-36

6.5 Estimated Upgrade and Expansion Project Costs 6-37
6.5.1 Capital Costs 6-37
6.5.2 NPV Costs 6-41
6.5.3 Major Maintenance Project Costs 6-44
6.5.4 Operations and Electrical Building Expansion 6-46

6.6 Comparative Evaluation Of Alternatives 6-48
6.6.1 Ranking Criteria Overview 6-48
6.6.2 Rating of Alternatives 6-50

6.7 References 6-57

Chapter 7. Recommended Facilities Implementation Plan

7.1 Implementation Strategy 7-1
7.1.1 Phase 1 Strategy: Current Permitted Design Condition 7-1
7.1.2 Phase 2 Strategy: Future Permitted Design Condition 7-2
7.1.3 Phase 3 Strategy: Ultimate Buildout Design Condition 7-4

7.2 Recommended Site Layout 7-6

7.3 Implementation Plan 7-6
7.3.1 Phase 1 Plan: Current Permitted Design Condition 7-7
7.3.2 Arsenic Compliance Plan 7-7
7.3.3 Phase 2 Plan: Future Permitted Design Condition 7-7
7.3.4 Phase 3 Plan: Ultimate Buildout Design Condition 7-8

List of Tables

Table ES-1. Rating of Alternatives at the Current Permitted Design Condition ES-3

Table ES-2. Rating of Alternatives at the Future Permitted Design Condition ES-6

Table ES-3. Estimated Costs for Phase 2 Upgrade Project Current Permitted Design Condition ... ES-9

Table ES-4. Estimated Costs for the Phase 3 Expansion Project Future Permitted Design Condition ES-9

Table ES-5. Estimated Costs for the Phase 4 Expansion Project Ultimate Buildout Design Condition ES-10

Table 2-1. Wastewater Collection System Master Plan Flow Coefficients 2-3

Table 2-2. Per Capita Flow History 2-4

Table 2-3. Residential Flow Factors for WWTP Planning 2-5

Table 2-4. Derivation of School Flow Factor (gpd Per City Resident) 2-7

Table 2-5. Derivation of Non-Residential Flow Factors 2-8

Table 2-6. Land-Use Based Flow Projection 2-9

Table 2-7. Galt WWTP Recommended Flow Peaking Factors 2-13

Table 2-8. Galt WWTP Design Flows 2-14

Table 2-9. Summary of Key Galt WWTP Influent Characteristics, Special Sampling Program, April/May 2012 2-16
Table 2-10. Design Load Peaking Factors from Similar Service Areas 2-17
Table 2-11. Proposed Design Loads for the Galt WWTP 2-18
Table 2-12. Influent Flow and Load Conditions Analyzed with Secondary Process Modeling of City of Galt Wastewater Treatment Plant 2-19
Table 2-13. Secondary Process Modeling Influent Concentrations 2-20
Table 2-14. Secondary Process Modeling Recycle Stream Concentrations 2-22
Table 2-15. Galt WWTP Final Effluent Limitations (Order No. R5-2010-0099) 2-24
Table 2-16. Additional Galt WWTP Effluent Limitations (Order No. R5-2010-0099) 2-25
Table 2-17. Comparison of Biosolids/Land Application Metals Limitations and Existing WWTP Biosolids/Land Application Conditions 2-30
Table 2-18. Reliability and Redundancy Requirements for Galt WWTP Facilities Plan 2-33
Table 4-1. Existing Facilities Performance Criteria Assumptions for Secondary Process Modeling 4-3
Table 4-2. Secondary Process Model Results 4-5
Table 4-3. Bases of State Point Analysis Input Parameters 4-6
Table 4-4. Secondary Clarifier State Point Analysis Summary - Current Flows 4-8
Table 4-5. Secondary Clarifier State Point Analysis Summary - Permitted Flows 4-9
Table 4-6. WWTP Flow vs. Process Units Assumed to be in Service 4-11
Table 4-7. Predicted Future Hydraulic Deficiencies and Recommended Improvements 4-13
Table 5-1. Influent and Effluent Wastewater Characteristics 5-1
Table 5-2. Process Characteristics 5-2
Table 5-3. Comparison of Solids Balance Results 5-2
Table 5-4. Biosolids Production Rates 5-3
Table 5-5. Typical Lagoon and Dewatering Bed Operation & Performance Variables 5-5
Table 5-6. Dewatering Bed Operation Assumptions Common to All Alternatives 5-6
Table 5-7. Solids Drying Bed Capacities and Deficits with Additional Dewatering Beds at the Current Permitted Design Condition 5-8
Table 5-8. Preliminary Mechanical Dewatering Sizing for the Current Permitted Design Condition 5-8
Table 5-9. Mechanical Dewatering Equipment Costs and Expected Performance for the Current Permitted Design Condition 5-9
Table 5-10. Estimated Project Costs for Solids Processing Alternatives for the Current Permitted Design Condition, Dollars 5-10
Table 5-11. Assumed Routine O&M Requirements for Dewatering Technologies 5-12
Table 5-12. Estimated NPV for Solids Processing Alternatives for the Current Permitted Design Condition, Million Dollars 5-13
Table 5-13. Agronomic Nitrogen Requirements for City-Owned Fields 5-14
Table 5-14. Estimate of Remaining Time for Land Application Based on Metals CPLRs 5-16
Table 6-1. Proposed Retrofits for Existing Secondary Clarifiers 6-2

Table 6-2. Comparison of Influent Peak Flows to Flows with Raw Wastewater Equalization 6-5

Table 6-3. Major Oxidation Ditch Performance Criteria Assumptions for Secondary Process Modeling 6-7

Table 6-4. Alternative A (Oxidation Ditch Only) Secondary Process Modeling Results Current Permitted Design Condition 6-8

Table 6-5. Alternative B (Separate Anoxic Basin) Secondary Process Modeling Results Current Permitted Design Condition 6-9

Table 6-6. Comparison of Major Operational Requirements for Microscreening and Conventional Primary Clarification 6-11

Table 6-7. Primary Microscreening Constituent Removal 6-12

Table 6-8. Alternative C (Primary Microscreening) Secondary Process Modeling Results Current Permitted Design Condition 6-13

Table 6-9. Alternative D (Separate Biosolids Treatment) Secondary Process Modeling Results Current Permitted Design Condition 6-14

Table 6-10. Alternative E (Clarifier Capacity) Secondary Process Modeling Results Current Permitted Design Condition 6-15

Table 6-11. Secondary Clarifier State Point Analysis Criteria Current Permitted Design Condition. 6-17

Table 6-12. Secondary Clarifier State Point Analysis of Alternatives Current Permitted Design Condition 6-18

Table 6-13. Treatment Process Unit Installed Quantities for Viable Alternatives at the Current Permitted Design Condition 6-19

Table 6-14. Alternative A (Oxidation Ditches Only) Secondary Process Modeling Results Future Permitted Design Condition 6-23

Table 6-15. Alternative F (C-D Hybrid) Secondary Process Modeling Results Future Permitted Design Condition 6-25

Table 6-16. Secondary Clarifier State Point Analysis Criteria Future Permitted Design Condition .. 6-27

Table 6-17. Secondary Clarifier State Point Analysis of Alternatives Future Permitted Design Condition 6-28

Table 6-18. Treatment Process Unit Installed Quantities for Viable Alternatives at the Future Permitted Design Condition 6-29

Table 6-19. Alternative F (C-D Hybrid) Secondary Process Modeling Results Ultimate Buildout Design Condition 6-32

Table 6-20. Secondary Clarifier State Point Analysis Criteria (Ultimate Buildout Design Condition) 6-34

Table 6-21. Secondary Clarifier State Point Analysis of Alternatives 6-35

Table 6-22. Treatment Process Unit Installed Quantities for Viable Alternatives at the Ultimate Buildout Design Condition 6-36

Table 6-23. Estimated Capital Costs of Treatment Train Alternatives Future Permitted Design Condition (3.0 MGD) 6-38

Table 6-24. Estimated Capital Costs of Treatment Train Alternatives Future Permitted Design Condition (4.5 MGD) 6-39

Table 6-25. Estimated Capital Costs of Treatment Train Alternatives Ultimate Buildout Design Condition (6.0 MGD) 6-40

Table 6-26. Estimated NPV for Treatment Alternatives 6-42

Table 6-27. Estimated Capital Cost of Major Maintenance Improvements 6-47
Table 6-28. Estimated Capital Cost for Operations and Electrical Building Expansion 6-48
Table 6-29. Rating of Alternatives at the Current Permitted Design Condition 6-52
Table 6-30. Rating of Alternatives at the Future Permitted Design Condition 6-54
Table 6-31. Rating of Alternatives at the Ultimate Buildout Design Condition 6-56
Table 7-1. Estimated Costs for Phase 1 Upgrade Project, million dollar Current Permitted Design Condition (3.0 mgd) 7-2
Table 7-2. Estimated Costs for the Phase 2 Expansion Project, million dollar Future Permitted Design Condition (4.5 mgd) 7-3
Table 7-3. Estimated Costs for the Phase 3 Expansion Project, million dollar Ultimate Buildout Design Condition (6.0 mgd) 7-6

List of Figures

Figure ES-1. Site Location Map ES-11
Figure ES-2. Existing Facilities Schematic ES-12
Figure ES-3. City of Galt WWTP Flow Projection ES-13
Figure ES-4. Flow Chart of the Recommended Facilities Master Plan Implementation Strategy .. ES-14
Figure ES-5. Preliminary Schedule for Upgrade and Expansion Projects ES-15
Figure ES-6. Site Layout for Alternative C: Microscreening ES-16
Figure ES-7. Site Layout for Alternative F: Microscreening/Solids Treatment Hybrid ES-17
Figure 1-1. Site Location Map 1-6
Figure 1-2. Existing Facilities Schematic 1-7
Figure 2-1. City of Galt WWTP Flow Projection 2-37
Figure 2-2. Annual BOD/TSS Concentrations, 2000-2011 2-38
Figure 2-3. Monthly Influent BOD Concentrations, 2000-2011 2-39
Figure 2-4. Monthly Influent TSS Concentration, 2000-2011 2-40
Figure 2-5. Basin Plan Groundwater Protection Requirements 2-41
Figure 3-1. Liquid Treatment Schematic (existing) 3-48
Figure 3-2. Solids Treatment Schematic (existing) 3-49
Figure 3-3. Pond Schematic 3-50
Figure 3-4. Storage Reservoir Capacity Curves 3-51
Figure 3-5. Irrigation Facilities Layout 3-52
Figure 3-6. Operations & Electrical Building Conceptual Drawing 3-53
Figure 3-7. Maintenance/Chemical Building and Control Building Conceptual Drawing 3-54
Figure 4-1. Clarifier Solids Loading Rates vs. SVI, Current Flows 4-17
Figure 4-2. Clarifier Solids Loading Rates vs. SVI, Permitted Flows 4-18
Figure 4-3. Existing Hydraulic Profile 1 4-19

Figure 4-4. Existing Hydraulic Profile 2 4-20
Figure 4-5. Future Hydraulic Profile 1 4-21
Figure 4-6. Future Hydraulic Profile 2 4-22
Figure 5-1. Land Required for Biosolids Application Based on Nitrogen Loadings (250 Pounds per Acre Nitrogen Application Rate) 5-19
Figure 5-2. Land Required for Biosolids Application Based on Nitrogen Loadings (400 Pounds per Acre Nitrogen Application Rate) 5-20
Figure 6-1. Alternative A Schematic, Current Permitted Design Condition (3.0 mgd ADWF) 6-58
Figure 6-2. Alternative C Schematic, Current Permitted Design Condition (3.0 mgd ADWF) 6-59
Figure 6-3. Alternative D Schematic, Current Permitted Design Condition (3.0 mgd ADWF) 6-60
Figure 6-4. Alternative A Schematic, Future Permitted Design Condition (4.5 mgd ADWF) 6-61
Figure 6-5. Alternative C Schematic, Future Permitted Design Condition (4.5 mgd ADWF) 6-62
Figure 6-6. Alternative D Schematic, Future Permitted Design Condition (4.5 mgd ADWF) 6-63
Figure 6-7. Alternative F (Hybrid C+D) Schematic, Future Permitted Design Condition (4.5 mgd ADWF) 6-64
Figure 6-8. Alternative A Schematic, Ultimate Buildout Design Condition (4.5 mgd ADWF) 6-65
Figure 6-9. Alternative C Schematic, Ultimate Buildout Design Condition (4.5 mgd ADWF) 6-66
Figure 6-10. Alternative D Schematic, Ultimate Buildout Design Condition (4.5 mgd ADWF) 6-67
Figure 6-11. Alternative F (Hybrid C+D) Schematic, Ultimate Buildout Design Condition (4.5 mgd) 6-68
Figure 6-12. Flow Chart of Potential Expansion Options 6-69
Figure 7-1. Flow Chart of the Recommended Facilities Master Plan Implementation Strategy 7-11
Figure 7-2. Site Layout for Alternative C: Microscreening 7-12
Figure 7-3. Site Layout for Alternative C: Microscreening/Biosolids Treatment Hybrid 7-13
Figure 7-4. Preliminary Schedule for Upgrade and Expansion Projects 7-14

List of Appendices

Appendix A: Water Quality Sampling Plan
Appendix B: WWTP Secondary Process Modeling
Appendix C: Arsenic Management Plan
Appendix D: Existing Facilities Design Basis
Appendix E: Existing Facilities Conditions Assessment
Appendix F: Pond Use Analysis

EXECUTIVE SUMMARY

The City of Galt (City) Wastewater Treatment Plant (WWTP) Facilities Master Plan defines near-term and long-term improvements for capacity, reliability, and regulatory compliance. The City faces near-term compliance challenges driven by new secondary treatment requirements that must be met by September 2015. In addition, the City must plan for long-term capacity and reliability needs to accommodate economic development consistent with the recently adopted General Plan. The Facilities Master Plan findings are summarized below in the following sections:

- WWTP Overview
- Capacity Needs
- Near-Term Improvements
- Long-Term Improvements
- Comparison of Alternatives
- Implementation Plan and Costs

ES.1 WWTP OVERVIEW

The WWTP and its properties are located at 10059 Twin Cities Road as shown on Figure ES-1. The major treatment facilities at the WWTP are depicted in schematically in Figure ES-2. These facilities consist of coarse bar screening, grit removal, activated sludge extended aeration in two oxidation ditches, solids settling in two secondary clarifiers, cloth media filtration, and ultraviolet (UV) disinfection. Biosolids generated from the secondary process are directed to two sludge lagoons, where they are held until they can be dewatered on drying beds and eventually land-applied on approximately 175 acres of City-owned agricultural properties that surround the central treatment plant site. The WWTP discharges year-round to Laguna Creek via Skunk Creek (see Figure ES-1). In addition, treated effluent is discharged to the Storage Reservoir from where it is used for irrigation on the City-owned agricultural fields. The filtration, UV disinfection, and biosolids drying beds were completed in 2011 as Phase 1 of the City's plan for compliance with new permit requirements.

ES.2 CAPACITY NEEDS

Projections of future influent flows and waste loads to the WWTP are necessary to adequately plan for serving growth through buildout as defined in the City's General Plan. For the purposes of WWTP planning, increasing capacity needs are expressed in terms of increasing average dry weather flow (ADWF). However, associated with any predicted future ADWF is a series of other parameters that also dictate treatment needs, such as: peak monthly, peak daily, and peak hourly flows. In addition, treatment needs are based on predictions of the amount of waste or "load" carried by the flow to the WWTP.

Based on an evaluation of historical flows, the anticipated population, and the land uses predicted in the General Plan, an ADWF of 6.0 million gallons per day (mgd) is anticipated at buildout of the General Plan. The construction of additional treatment facilities needed to accommodate this increase in flow, as well as comply with permit requirements will occur under the following phases:

- **Phase 1: Tertiary Improvements** phase, completed in 2011.
- **Immediate Improvements: Current Permitted Design Condition**, which will address the improvements needed to ensure reliable compliance when the ADWF is 3.0 mgd (i.e. the current permitted discharge flow rate).
- **Phase 2: Future Permitted Design Condition**, which will include the expansion projects needed to ensure reliable treatment as the ADWF increases to 4.5 mgd.
- **Phase 3: Ultimate Buildout Design Condition**, which will include the expansion projects needed to ensure reliable treatment as the ADWF increases to 6.0 mgd (i.e. the ADWF anticipated at the General Plan buildout).

Improvements required under the Current Permitted Design Condition must be completed by September 1, 2015, as required by the existing permit for the WWTP. A projection of when the future expansion projects will need to be completed was developed based on discussions with City staff and consideration of a potential range of flow growth projections. The resulting ADWF projection is presented in Figure ES-3. As shown, the Future Permitted Design Condition expansion should be planned for completion by 2020, and the Ultimate Buildout Design Condition expansion would not likely be needed until about 2032.

The timing of the two future expansion phases (Phases 2and 3) will ultimately be determined by the actual rate of growth in the City of Galt. For purposes of the Master Plan, it has been assumed that expansions will be triggered by increasing ADWF. However, if water conservation efforts reduce the amount of flow for a given population, the need for additional WWTP capacity will still be triggered, as the waste loads will not be affected by such decreases.

ES.3 NEAR-TERM UPGRADE IMPROVEMENTS

The City previously elected to complete upgrades to the WWTP for permit compliance in two different phases. In 2011, the City completed construction of the first phase. Under this first project, tertiary filtration and UV disinfection facilities were constructed to comply with the new filtration and disinfection byproduct standards that were placed on the City in 2003. These facilities were sized based on the WWTP's 3.0 mgd ADWF design capacity.

One of the primary objectives of this facilities planning effort was to determine the best approach for completing the immediate improvements needed for permit compliance. Specifically, the existing oxidation ditches were not designed to remove nitrogen (i.e. ammonia and nitrate) from the City's wastewater, and improvements are necessary to achieve this objective. The following five potential alternatives were evaluated for providing reliable nitrogen removal at the Current Permitted Design Condition:

- Alternative A: Construct one additional oxidation ditch
- Alternative B: Construct a separate anoxic basin for denitrification
- Alternative C: Provide primary microscreening to reduce loads to Oxidation Ditches
- Alternative D: Provide an alternative solids treatment approach to reduce demands on the Oxidation Ditches

- Alternative E: Expand the secondary clarifiers to allow higher loads to applied to the Oxidation Ditches

Chapter 6 (WWTP Expansion Alternatives) provides a detailed description of how the alternatives were evaluated and compared. Alternatives B and E were eliminated because they would require immediate construction of at least two additional clarifiers and (particularly in the case of Alternative E) they would result in potentially unstable operating conditions in the oxidation ditches. However, Alternatives A, C, and D were evaluated further, based on a number of factors important to the City.

The alternatives were evaluated based on both cost and non-cost factors, and the specific criteria were assigned weighting factors to reflect varying levels of importance, with the most important criteria receiving the highest weighting. For each criterion, the alternatives were rated 3 for "most favorable," 2 for "less favorable," and 1 for "unfavorable." Table ES-1 lists the criteria, their weighting, and the results of the comparison for the Current Permitted Design Condition. Based on this comparison, Alternative A (adding an oxidation ditch) is the recommended alternative for the near-term improvements.

Table ES-1. Rating of Alternatives at the Current Permitted Design Condition

Category	Weighting Factor	Alternative A Oxidation Ditch Only		Alternative C Primary Microscreening		Alternative D Separate Biosolids Treatment	
		Rating	Weighted Rating	Rating	Weighted Rating	Rating	Weighted Rating
Life Cycle Costs	15	3	45	4	60	2	30
Capital	—	1	—	2	—	1	—
O&M	—	2	—	2	—	1	—
O&M Considerations	6	9	54	4	24	5	30
Different Skills	—	3	—	2	—	1	—
Workload	—	3	—	1	—	2	—
Spare Parts	—	3	—	1	—	2	—
Ability to Meet Current and Future Permits	5	2	10	1	5	3	15
Reliability	5	3	15	1	5	2	10
Flexibility	7	3	21	2	14	1	7
Investment Life	2	3	6	2	4	3	6
Energy Efficiency	2	2	4	3	6	1	2
Risk of Negative Secondary Impacts	3	3	9	1	3	2	6
Total Weighted Rating	—	—	**164**	—	**121**	—	**106**

As indicated in Table ES-1, Alternative C ranked the highest with respect to costs (i.e., it was the lowest cost alternative). However, the overall ranking for this alternative was low because this approach relies on a relatively new technology called primary microscreening. Use of a new technology like primary microscreens at the City's WWTP would introduce a risk that the oxidation ditches and clarifiers would not perform as anticipated, resulting in difficulties meeting permit requirements. Nevertheless, as primary microscreening technology becomes more common and tested with oxidation ditch treatment systems, it may become a viable option for later phases of expansion in Galt.

Alternative D, which relies on adding a new solids treatment process at the WWTP, was ruled out for the Current Permitted Design phase because it also introduces process complexities that are not recommended at this time. (Alternative D also does not result in significant savings.) However, an option that incorporates an alternative solids treatment approach was considered for the future expansion projects.

In addition to constructing a new (third) oxidation ditch, the recommended plan includes: modifications to the existing oxidation ditches to provide the aeration control necessary for reliable nitrogen removal and the construction of a new (third) secondary clarifier. A number of improvements not directly related to the secondary process upgrade were also identified for the Current Permit Design Condition phase. These recommendations address various deficiencies described in Chapter 3 (Existing Facilities) and Chapter 4 (Existing Wastewater Treatment Plant Conditions Assessment), as well findings from the biosolids land application, handling and disposal assessment in Chapter 5 (Biosolids Handling Facilities). The recommended improvements include:

- supplemental mechanical dewatering equipment to increase solids handling reliability and flexibility, and preclude the need to construct additional liquid storage;
- sludge lagoon improvements including aerators, a vehicle ramp, and a pedestrian stairway to provide safer access;
- supervisory control and data acquisition (SCADA) improvements;
- modifications to the old Control Building to improve laboratory efficiency;
- improved vehicle access to the Maintenance Building;
- construction of covered storage to protect and extend the life of mobile equipment currently parked outdoors; and
- retrofits to the headworks to facilitate control of flow equalization and minimize the impact of peak flows on the performance of the secondary and tertiary processes.

In addition, the following activities beyond the WWTP upgrades recommended in this Facilities Master Plan are needed to ensure regulatory compliance with the limitations prescribed by the Regional Board for arsenic and copper:

- The City will need to modify selected existing wellhead water treatment systems to ensure reliable compliance with the arsenic limitations for the WWTP. A detailed strategy for completing this effort is provided in the Arsenic Management Plan (Appendix C) of this Facilities Master Plan.
- Recent effluent data collected by the City indicates that a Water Effects Ratio (WER) Study for copper is needed to obtain adjusted permit limits for this parameter.

Finally, a number of necessary items were identified that could be included in the Current Permitted Design Condition project or completed on a slightly extended schedule:

- Expansion of the Operations and Electrical Building to accommodate additional City water system operations staff at the WWTP.
- A number of major maintenance projects that need to be completed over the next five year period.

ES.4 LONG-TERM EXPANSION IMPROVEMENTS

The following three potential alternatives were evaluated for expanding the secondary process to meet the Future Permitted Design Condition and Ultimate Buildout Design Condition capacity needs:

- Alternative A: Continue adding oxidation ditches and clarifiers to meet the treatment needs
- Alternative C: Add primary microscreens to reduce loads to the oxidation ditches, thereby reducing the total number of ditches ultimately required
- Alternative D: Add an alternative solids treatment system to reduce demands on the oxidation ditches, thereby reducing the total number of ditches ultimately required

Although each of these alternatives was assessed separately for the Current Permitted Design Condition, the City could elect to move from one alternative to another under one of the future expansion projects. For example, although it is recommended that the City construct the facilities needed to implement Alternative A at the Current Permitted Design Condition, either Alternative C or Alternative D could be implemented at the Future Permitted Design Condition by adding either primary microscreening or separate solids treatment facilities, respectively. Similarly, a fourth hybrid option (Alternative F), which involves adding both primary microscreens and an alternative solids treatment system, could be considered for the Ultimate Buildout Design Condition. A flow chart summarizing the potential scenarios are illustrated in Figure ES-4.

Table ES-2 lists the results of the comparison of alternatives evaluated for the Future Permitted Design Condition (Phase 2). Based on this comparison, Alternative C (adding primary microscreening) is the recommended alternative for the Phase 2 improvements, presuming the technology has adequately matured and can be relied upon to produce consistent and predictable performance with respect to impacts on downstream processes.

A similar analysis was performed for the Ultimate Buildout Design Condition (Phase 3); however, several factors are likely to influence the final decision regarding improvements at the Ultimate Buildout Design Condition. Specifically, it is likely that emerging energy recovery and solids treatment technologies will influence future decisions regarding WWTP improvements. Based on the analysis presented in Chapter 6, either additional primary microscreening under Alternative C, or the hybrid approach employing both separate biosolids treatment and primary microscreening (Alternative F) are attractive in Phase 3.

Table ES-2. Rating of Alternatives at the Future Permitted Design Condition

		Alternative A		Alternative C		Alternative D	
		Oxidation Ditch Only		Primary Microscreening		Separate Biosolids Treatment	
	Weighting Factor	Rating	Weighted Rating	Rating	Weighted Rating	Rating	Weighted Rating
Life Cycle Costs	15	2	30	5	75	3	45
Capital	—	1	—	3	—	2	—
O&M	—	1	—	2	—	1	—
O&M Considerations	6	8	48	5	30	6	36
Different Skills	—	3	—	2	—	2	—
Workload	—	3	—	2	—	2	—
Spare Parts	—	2	—	1	—	2	—
Ability to Meet Current and Future Permits	5	2	10	1	5	3	15
Reliability	5	3	15	1	5	2	10
Flexibility	7	1	7	3	21	2	14
Investment Life	2	3	6	2	4	3	6
Energy Efficiency	2	2	4	3	6	1	2
Risk of Negative Secondary Impacts	9	3	9	2	6	3	9
Total Weighted Rating	—	—	**129**	—	**152**	—	**137**

Finally, as further detailed in Chapter 6 (WWTP Expansion Alternatives), the following improvements (common to all alternatives) are necessary to provide the expanded treatment capacity:

- Effluent Flow Metering Improvements
- Various Hydraulic Improvements (Pipe Replacements)
- Upgraded Electrical Service and Switchgear
- Additional Standby Power Generators

- Additional Biosolids Handling and Disposal Equipment
- Additional Tertiary Filtration and UV Disinfection Facilities

ES.5 IMPLEMENTATION PLAN

A preliminary schedule for the implementation of the three WWTP phases discussed in this Facilities Master Plan and for the Arsenic Compliance Plan discussed in Appendix C is presented in Figure ES-5. Additional discussion of the proposed schedule for each phase is provided below.

ES.5.1 Immediate Improvements Plan: Current Permitted Design Condition

The primary driver for the timeline of the Current Permitted Design Condition (Immediate Improvements) is the September 1, 2015, compliance date for meeting effluent ammonia, and nitrate/nitrite requirements in the City's discharge permit for the WWTP. However, even if the City immediately initiates design of the Immediate Improvements upon completion of this Facilities Master Planning effort, the Immediate Improvements will not likely be fully completed by September 2015. Nevertheless, with careful staging of the construction project, the oxidation ditch expansion and improvement projects necessary to meet the ammonia and nitrate/nitrite limitations should be completed by the compliance date.

ES.5.2 Phase 2 Plan: Future Permitted Design Condition

It is recommended that the final facilities planning efforts needed to implement the Future Permitted Design Condition project begin when average dry weather flows are approximately 85 percent of 3.0 mgd (or 2.6 mgd). As shown in Figure ES-5, if growth rates are as predicted, a microscreening technology and reliability review will need to begin in mid to late-2015, or around the time the Current Permitted Design Condition Project is being completed. This schedule provides an additional 3 years for the microscreening technology to mature and become established in the United States. Chapter 7 includes a list of topics that should be addressed as part of the microscreening technology review.

If the technology and reliability review reveals that microscreening is *not* a viable alternative, the City should move forward with the implementation of Alternative D2 – construction of a separate biosolids treatment system such as aerobic digestion. If the review indicates that microscreening is viable, the City will move forward with implementing Alternative C2 – construction of primary microscreening. In this case, pilot studies of a microscreening unit from one or more manufacturer(s) will need to be included as part of the predesign of the Phase 2 improvements.

ES.5.3 Phase 3 Plan: Ultimate Buildout Design Condition

Technologies for treatment of solids and/or using solids to generate energy are evolving rapidly. In addition, the City's land application area disposal capacity is limited and new regulations could impact the City's ability to dispose of solids onsite (and potentially at landfills). Therefore, final planning for the Ultimate Buildout Design Condition should begin with a review of regulations for solids disposal and an identification of whether new technologies for using solids to generate energy (e.g., pyrolysis and gasification) are a viable option for the WWTP.

If Alternative C2 is implemented in Phase 2 and the solids handling review indicates that continued landfill disposal is the preferred long-term strategy for solids that cannot be land-applied, then Alternative C2 (expansion of the primary microscreens) is the likely best approach for the Phase 3 also. However, if it is determined that modification to the solids treatment and disposal system is needed (or desired for energy generation purposes), then the City should consider a separate biosolids treatment system in Phase 3 (i.e. Alternative F2).

Based on what is known today about the microscreening technology, it is expected that by the time the City is ready to implement Phase 3, this technology will have matured and be well established in the U.S. Therefore, if Alternative D2 is implemented in Phase 2, it is expected that the City will install primary microscreening in Phase 3 (i.e. Alternative F3).

Depending on the number and complexity of solids handling options being considered, the evaluation of solids handling options could take up to one year to complete. Therefore, it is recommended that the facilities planning efforts for Phase 3 begin when flows are approximately 80 to 85 percent of 4.5 mgd, or 3.7 to 3.8 mgd.

ES.5.4 Arsenic Compliance Plan

The schedule shown in Figure ES-5 for the Arsenic Compliance Plan is consistent with the recommendations outlined in Appendix C. As shown, the schedule provides the City with the flexibility to determine and implement the best long-term strategy for handling the arsenic treatment backwash solids.

ES.6 SITE LAYOUT AND COSTS

Figures ES-6 and ES-7 illustrate the ultimate treatment plant layout for the most likely scenarios under the recommended implementation plan. The figures are color coded to indicate what facilities would be constructed in each phase. The order of Phase 2 and 3 facilities could be reversed for the layout shown on ES-7, but the layout would not change.

The estimated cost of the recommended Immediate Improvements alternative and for the likely Phases 2 and 3 scenarios are summarized in Tables ES-3, ES-4, and ES-5, respectively. These estimates are provided to assist the City in selecting between alternatives, and in establishing a reasonable basis for setting rates and fees. The basis for these planning level cost estimates, including critical assumptions regarding how the estimates were derived and the potential variability in actual costs are presented in Chapter 6.

The estimated costs include improvements to capture arsenic solids at the wellhead treatment sites, improvements for expansion of Operations Building and major maintenance items. An allowance of $100,000 has been included for completing the WER Study for copper compliance.

Table ES-3. Estimated Costs for Immediate Improvements Upgrade Project[a]
Current Permitted Design Condition (3.0 mgd)

Cost Component	Cost ($million)	
	Alternative A (recommended)	Alternative C
Secondary Treatment Improvements and Wellhead Arsenic Removal	18.0	11.2
Operation Building Expansion	0.8	0.8
Major Maintenance Projects	1.7	1.7
Subtotal, Estimate of Probable Construction Bid (December 2013 Dollars)	**20.5**	**13.7**
Project Allowances[b]	6.9	4.7
WER Study (allowance)	0.1	0.1
Total Project Cost	**27.5**	**18.5**
Annual O&M Cost (2014 – 2019)	1.23	1.21
Net Present Value (2012 Dollars)	**31.6**	**23.5**

(a) From Tables 6-23, 6-26, 6-27, and 6-28 . Estimated costs represent planning-level estimates using available information and the assumptions stated throughout the Facilities Master Plan. An inflation allowance has been included, which may be higher or lower than actual inflation and/or the effects of other market forces.

(b) Total Allowances are 33.5 percent, which include: City construction materials and services (0.5 percent); City design and construction period costs (0.5 percent); predesign (2 percent); design services (15 percent); construction period services including construction management, Operations and Maintenance (O&M) manuals, and record drawings (15 percent); and other City materials and services, including environmental (0.5 percent).

Table ES-4. Estimated Costs for the Phase 2 Expansion Project[a]
Future Permitted Design Condition (4.5 mgd)

Cost Component	Immediate Improvements >>	Cost ($million)	
		Alternative A	
	Phase 2 >>	Alternative C2	Alternative D2
Estimate of Probable Construction Bid (2018 Dollars, based on 3 percent annual inflation)		9.4	17.1
Project Allowances[b]		3.2	5.7
Total Project Cost		**12.6**	**22.8**
Annual O&M Cost (2020 – 2031)		1.67	1.78
Net Present Value (2012 Dollars)		**24.0**	**31.9**
Total Net Present Value for Immediate Improvements and Phase 2		**55.6**	**63.5**

(a) From Tables 6-24 and 6-26. Estimated costs represent planning-level estimates using available information and the assumptions stated throughout the Facilities Master Plan. An inflation allowance has been included, which may be higher or lower than actual inflation and/or the effects of other market forces.

(b) Total Allowances are 33.5 percent, which include: City construction materials and services (0.5 percent); City design and construction period costs (0.5 percent); predesign (2 percent); design services (15 percent); construction management, including O&M manuals and record drawings (15 percent); and other City materials and services, including environmental (0.5 percent).

Table ES-5. Estimated Costs for the Phase 3 Expansion Project[a]
Ultimate Buildout Design Condition (6.0 mgd)

Cost Component	Immediate Improvements >>	Cost ($million)		
		Alternative A		
	Phase 2>>	Alternative C2		Alternative D2
	Phase 3>>	Alternative C2	Alternative F2	Alternative F3
Estimate of Probable Construction Bid (2030 Dollars, based on 3 percent annual inflation)		14.6	18.4	11.1
Project Allowances[b]		4.9	6.2	3.7
Total Project Cost		**19.5**	**24.6**	**14.8**
Annual O&M Cost (20 years, 2032 – 2051)		2.09	2.10	2.10
Net Present Value		**31.2**	**33.2**	**29.5**
Total Net Present Value for All Phases		**86.8**	**88.8**	**93.0**

(a) From Tables 6-25 and 6-26. Estimated costs represent planning-level estimates using available information and the assumptions stated throughout the Facilities Master Plan. An inflation allowance has been included, which may be higher or lower than actual inflation and/or the effects of other market forces.

(b) Total Allowances are 33.5 percent, which include: City construction materials and services (0.5 percent); City design and construction period costs (0.5 percent); predesign (2 percent); design services (15 percent); construction management, including O&M manuals and record drawings (15 percent); and other City materials and services, including environmental (0.5 percent).

Source: Imagery provided by ESRI Online.

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Scale in Feet

FIGURE ES-1

City of Galt
WWTP Facilities Master Plan

SITE LOCATION MAP

WEST YOST ASSOCIATES
Consulting Engineers

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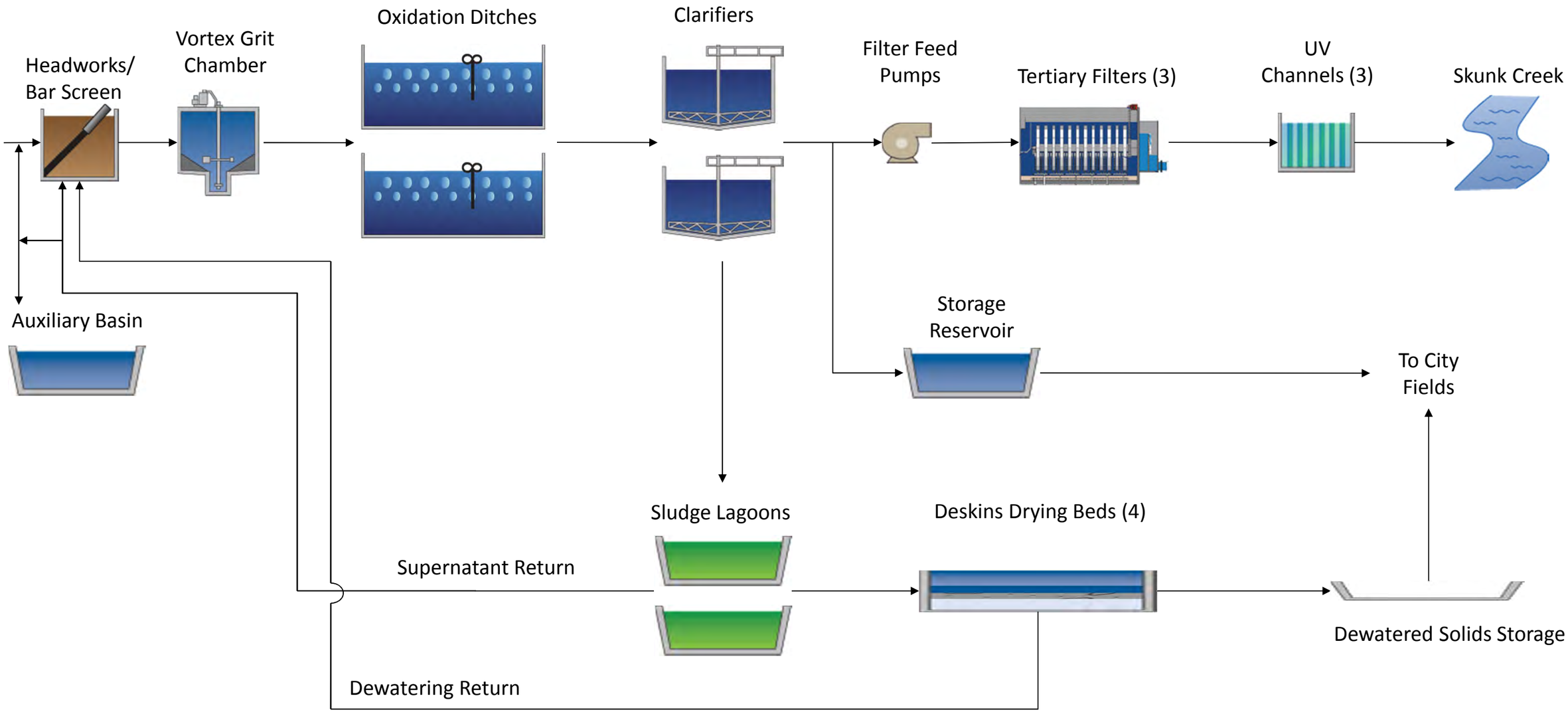

FIGURE ES-2

City of Galt
WWTP FACILITIES MASTER PLAN
EXISTING FACILITIES SCHEMATIC

WEST YOST ASSOCIATES
Consulting Engineers

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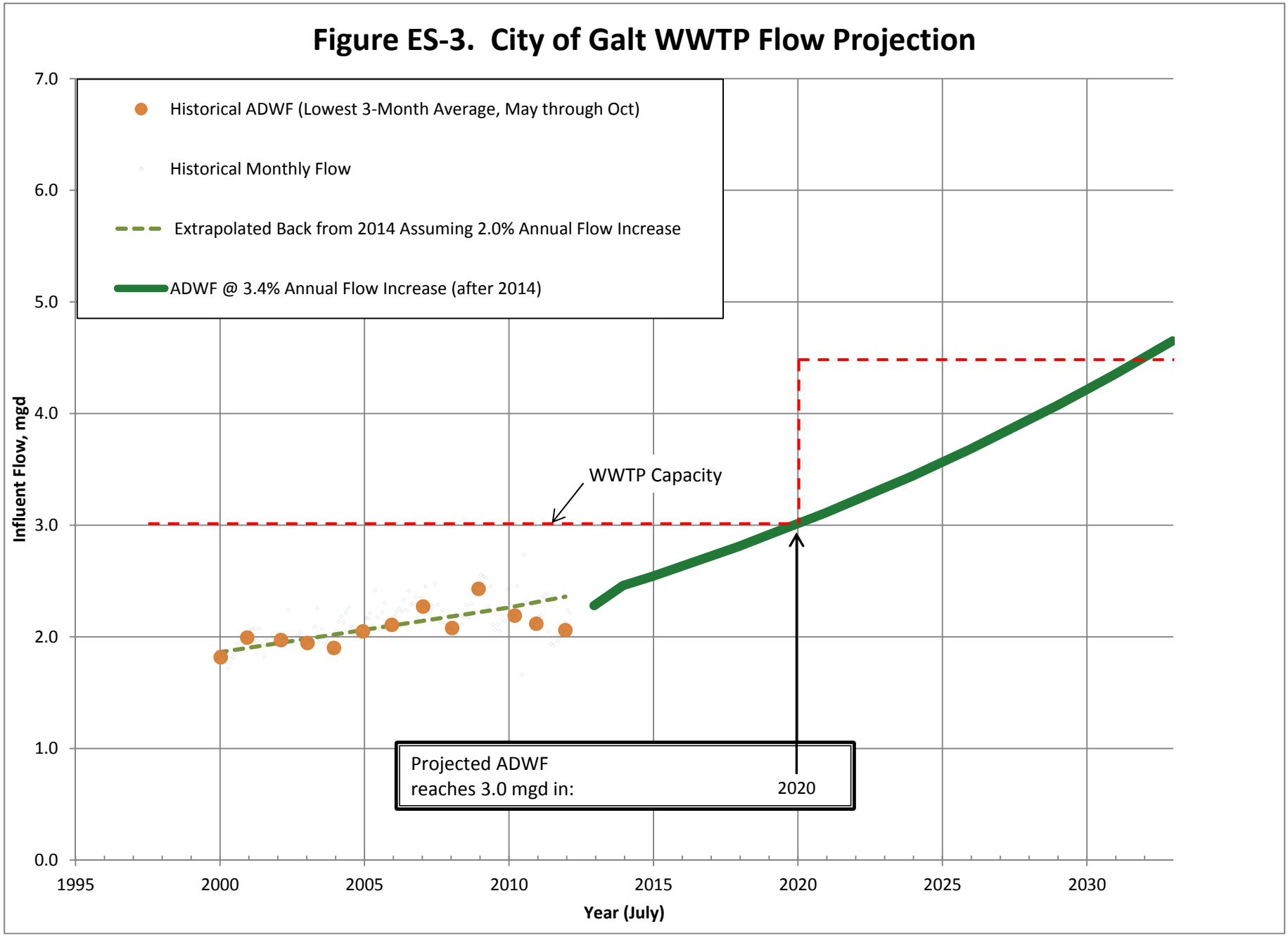


Figure ES-3. City of Galt WWTP Flow Projection
Historical ADWF (Lowest 3-Month Average, May through Oct)
Historical Monthly Flow
Extrapolated Back from 2014 Assuming 2.0% Annual Flow Increase
ADWF @ 3.4% Annual Flow Increase (after 2014)
WWTP Capacity
Projected ADWF
reaches 3.0 mgd in:
2020
Influent Flow, mgd
Year (July)
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
1995
2000
2005
2010
2015
2020
2025
2030

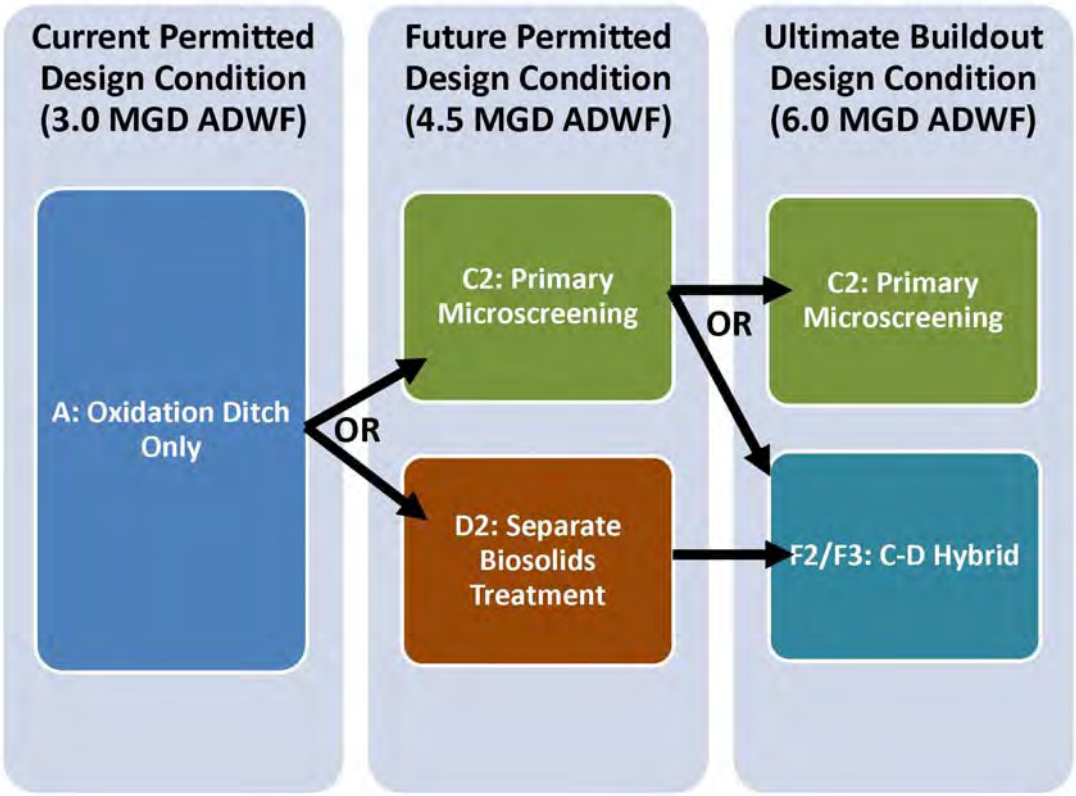
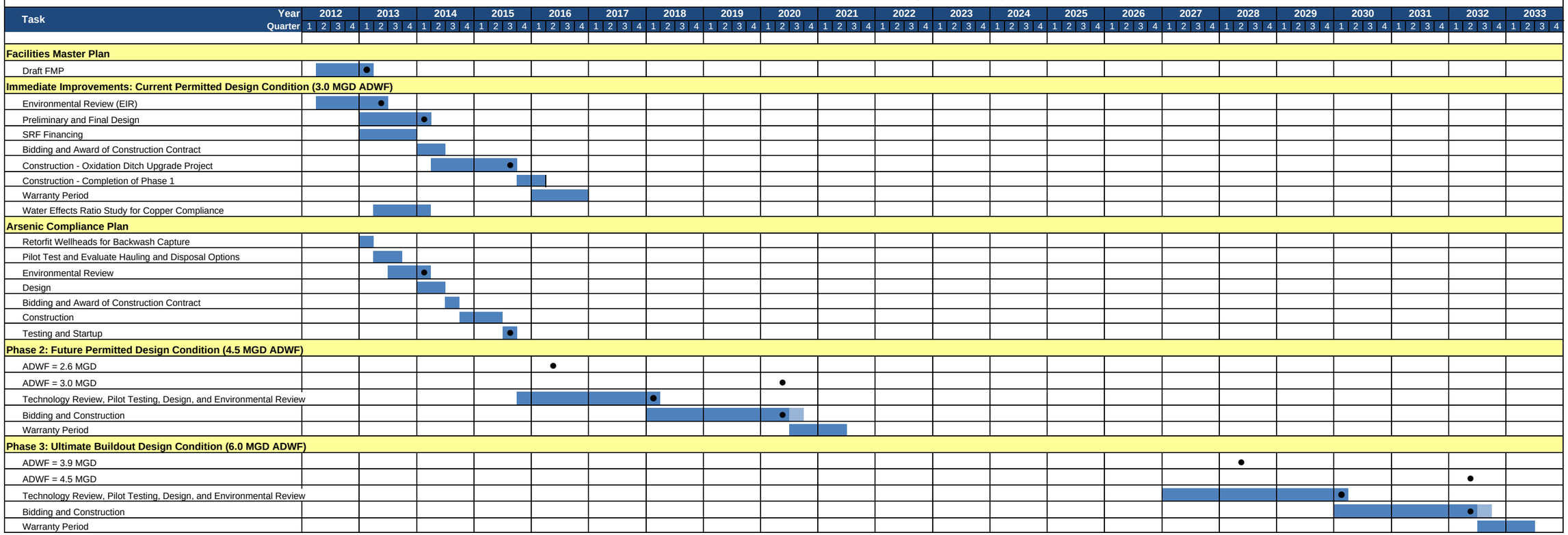
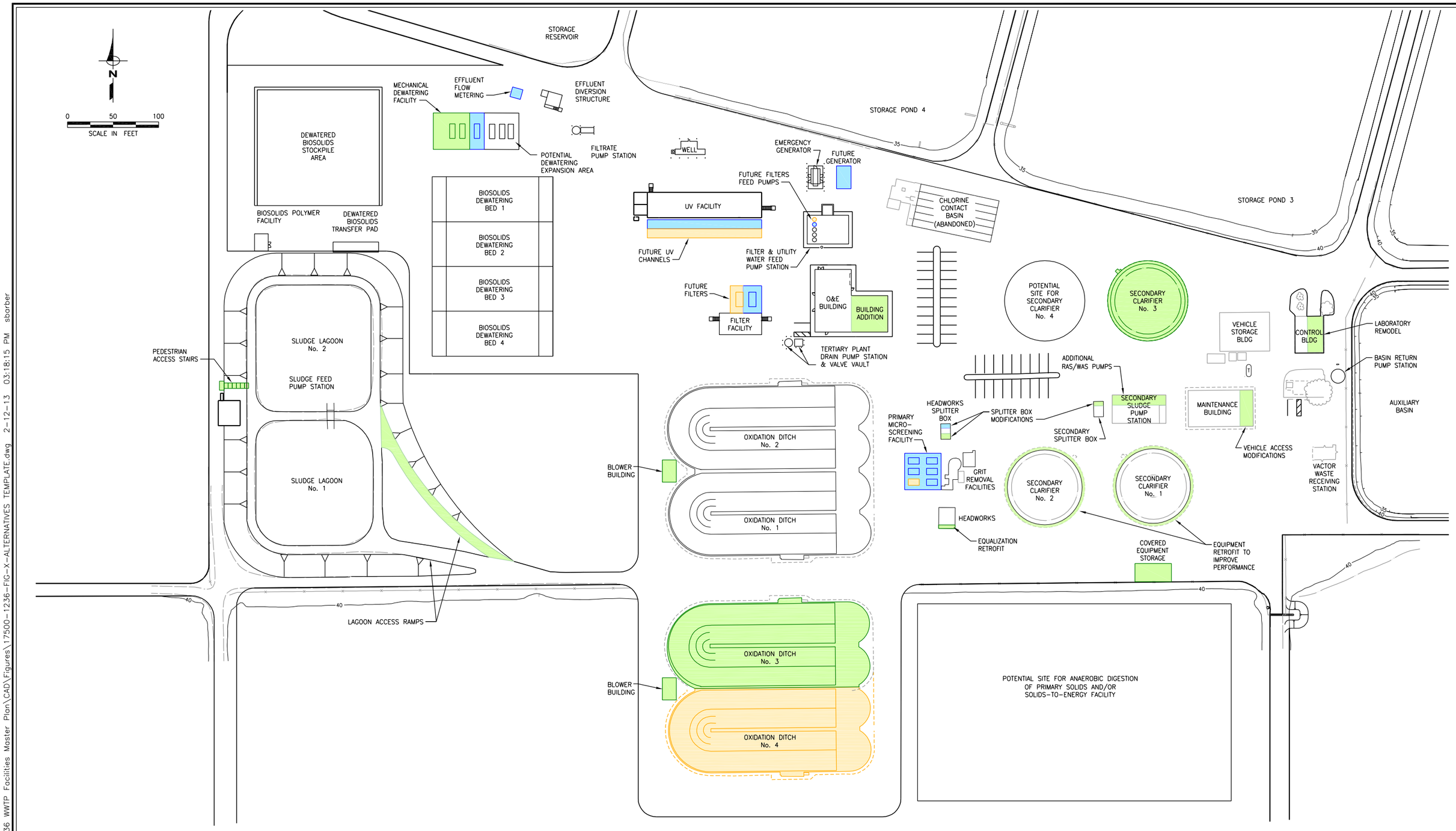


Figure ES-4. Flow Chart of the Recommended Facilities Master Plan Implementation Strategy

Figure ES-5. Preliminary Schedule for Upgrade and Expansion Projects





LEGEND

- FACILITIES INSTALLED FOR CURRENT PERMITTED DESIGN CONDITION
- FACILITIES INSTALLED FOR FUTURE PERMITTED DESIGN CONDITION
- FACILITIES INSTALLED FOR ULTIMATE BUILD-OUT DESIGN CONDITION

Figure ES-6

City of Galt
WWTP Facilities Master Plan
SITE LAYOUT FOR ALTERNATIVE C: MICROSCREENING

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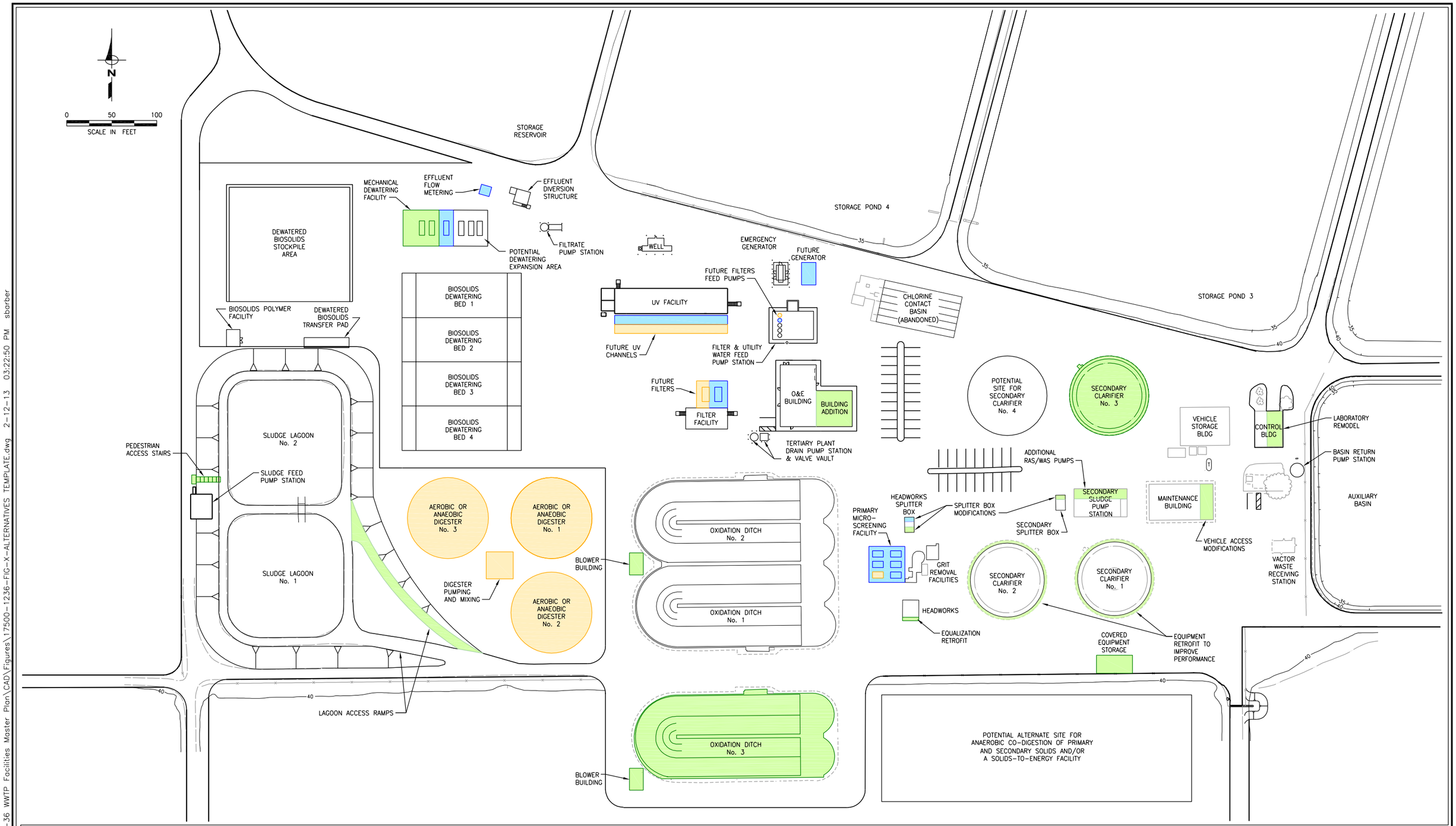

LEGEND

- FACILITIES INSTALLED FOR CURRENT PERMITTED DESIGN CONDITION
- FACILITIES INSTALLED FOR FUTURE PERMITTED DESIGN CONDITION
- FACILITIES INSTALLED FOR ULTIMATE BUILD-OUT DESIGN CONDITION

Figure ES-7
City of Galt
WWTP Facilities Master Plan
SITE LAYOUT FOR ALTERNATIVE F
(MICROSCREENING/SOLIDS TREATMENT HYBRID)

WEST YOST ASSOCIATES
Consulting Engineers

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This Chapter presents an overview of and introduction to the City of Galt (City) Wastewater Treatment Plant (WWTP) Facilities Mater Plan. The following specific topics are addressed:

- Facilities Master Plan Report Overview
- WWTP Overview
- Previous WWTP Facilities Planning Efforts
- Proposed Phasing Plan

1.1 FACILITIES MASTER PLAN REPORT OVERVIEW

The purpose of this WWTP Facilities Master Plan is to define a roadmap to cost effective capacity, reliability, and regulatory compliance for the City's WWTP. The City faces near-term compliance challenges that are driven by treatment requirements that must be met by September 2015. In addition, the City must plan for long-term capacity and reliability needs that will arise from new economic development within the City, and consistent with the recently adopted General Plan (Mintier Harnish et. al., 2009).

The chapters of this Facilities Master Plan are as follows:

- **Chapter 1. Introduction**: Provides this overview of the WWWTP Facilities Master Plan
- **Chapter 2. Basis of Design**: Details the principal design criteria for the WWTP
- **Chapter 3. Existing Treatment Plant Performance and Condition**: Provides a detailed evaluation of the function, performance, and condition of each major existing WWTP unit process and associated ancillary facilities.
- **Chapter 4**. **Secondary Treatment Capacity Analysis and Plant Hydraulics:** Presents an evaluation of the current capacity of the WWTP secondary processes, and the hydraulic capacities of the liquid stream processes.
- **Chapter 5**. **Biosolids Handling Facilities:** Summarizes the results of an evaluation of biosolids dewatering and land application facility needs of the WWTP.
- **Chapter 6. WWTP Expansion Alternatives:** Presents an evaluation of potential treatment train upgrade and expansion alternatives for the WWTP under the three design phases evaluated in this Facilities Master Plan.
- **Chapter 7. Recommended Facilities Implementation Plan:** Provides the overall implementation plan for completing the recommended upgrade and expansion projects.

1.2 WWTP OVERVIEW

The WWTP and its properties are located at 10059 Twin Cities Road, along the west side of the Union Pacific Railroad and west of Highway 99. The location of the City of Galt WWTP is shown on Figure 1-1.

The major treatment facilities at the WWTP are depicted in schematically in Figure 1-2. These facilities consist of coarse bar screening, grit removal, activated sludge extended aeration in two oxidation ditches, solids settling in two secondary clarifiers, cloth media filtration, ultraviolet disinfection. Solids generated from the secondary process are directed to two sludge lagoons, where they are held until they can be dewatered on drying beds.

The WWTP discharges year-round to Laguna Creek via Skunk Creek (see Figure 1-1). In addition, treated effluent is discharged to the Storage Reservoir from where it is applied to approximately 175 acres of City-owned agricultural fields that surround the central treatment plant site. The dewatered biosolids are also applied to the City-owned agricultural fields.

1.3 PREVIOUS WWTP PLANNING AND UPGRADE EFFORTS

The secondary treatment facilities at the WWTP were constructed in 1991 and were designed based on an Average Dry Weather Flow (ADWF) capacity of 3.0 million gallons per day (mgd). Until 2003, the Galt WWTP operated under an NPDES permit that required treatment to secondary standards with seasonal discharge to Laguna Creek. Operational requirements were comparatively limited, and the only effluent limitations included standard technology-based limits for biochemical oxygen demand (BOD), total suspended solids (TSS), settleable solids, chlorine residual, and total coliform bacteria. The secondary treatment facilities constructed in 1991 could comply with such limits.

In 2003, the Central Valley Regional Water Quality Control Board (Regional Board) issued new, more stringent, requirements for the WWTP. The City subsequently invested considerable efforts to identify the preferred long-term compliance strategy, including the development of the *Phase 1 Feasibility Study* (West Yost Associates, January 2005) and the *NPDES Permit Compliance Action Plan* (West Yost Associates, December 2005). The City evaluated a total of 17 different alternatives for compliance ranging from year-round reuse, to discharging the effluent at a larger WWTP and/or a different water body. Through this planning process, the preferred approach to compliance was determined to be upgrading the WWTP to allow for year-round discharge to Skunk Creek/Laguna Creek.

The City elected to complete upgrades to the WWTP in two different phases. In 2010, the City completed construction of the first phase. Under this project, tertiary filtration and UV disinfection facilities were constructed to comply with the new filtration and disinfection byproduct standards that were placed on the City in 2003. These facilities were sized based on the WWTP's 3.0 mgd ADWF design capacity.

The final compliance date for meeting the remaining new permit limitations is September 1, 2015. One of the primary objectives of this facilities planning effort is to determine the best approach for completing the second phase of upgrade projects needed for permit compliance. Specifically, the existing oxidation ditches were not designed to remove nitrogen (i.e. ammonia and nitrate) from the City's wastewater. Therefore, secondary treatment improvements are necessary to meet the effluent limitations for ammonia and nitrate.

1.4 PERMIT COMPLIANCE STRATEGIES NOT RELATED TO WWTP UPGRADE PROJECTS

This section briefly discusses the activities beyond the WWTP upgrades recommended in this Facilities Master Plan that are needed to ensure long-term regulatory compliance with the limitations prescribed by the Regional Board for arsenic and copper. Compliance with these limitations must also be achieved by September 1, 2015.

1.4.1 Arsenic

Arsenic enters the WWTP collection system via the water supply. The City has had arsenic removal equipment installed and operating at three of the City's groundwater well sites since 2009 in an effort to meet the drinking water requirement.[1] In addition, several of the City's groundwater well sites have iron and manganese treatment systems that use different filtration media but can also remove some arsenic. While these water treatment systems remove arsenic from the potable water via media filtration, they require backwashing of the media to ensure continued performance. The treatment systems discharge the backwash water with elevated arsenic levels to the sanitary sewer system, so the arsenic loads eventually enter the WWTP where they cannot be effectively removed. The City will need to modify the existing water treatment systems to ensure reliable compliance with the arsenic limitations for the WWTP. A detailed strategy for completing this effort is provided in the Arsenic Management Plan (Appendix C) of this Facilities Master Plan. The Arsenic Management Plan calls for capturing and removing the backwash solids at one or more of the wellhead water treatment systems, before the backwash is discharged to the sanitary sewer system.

1.4.2 Copper

Reliable compliance with copper effluent limitations was anticipated following the completion of the 2010 Tertiary Improvement Project. Indeed, the effluent was in compliance with the copper limitations for the first six months following installation of the effluent filtration facilities. However, more recent data indicates that continuous compliance may not be achievable. If the cause of recent exceedances of the copper limitations cannot be identified or resolved, the City may need to develop a Translator Study and/or a Water Effects Ratio (WER) Study for copper.

[1] The USEPA arsenic drinking water standard is also the governing criterion for maximum arsenic levels in WWTP effluent set in the WDRs.

Metals translators are conversion factors used to relate aquatic toxicity-based water quality standards (expressed as dissolved metals) to discharge permit effluent limits (expressed as total metals). A WER is defined as a correction factor used to adjust a water quality standard to account for differences in the toxicity of a specific pollutant between laboratory water and site water. Per State regulatory guidelines, site-specific metals translators and WERs may be derived which supersede conservatively estimated default values, thereby resulting in an increase of the applicable effluent limitation for the given discharge. To ensure reliable compliance with the copper limitation, it is recommended that the City complete a WER study for copper.

1.5 PROPOSED PHASING PLAN

The WWTP improvement projects described in this Facilities Master Plan are expected to be implemented in three phases. The current permitted ADWF capacity of the treatment plant is 3.0 mgd, which corresponds with its original design. However, as noted above, some improvements are needed by September 2015 to ensure reliable compliance with the ammonia and nitrate limitations that have been mandated for the WWTP. The basis of design for these facility improvements will hereinafter be referred to as the Current Permitted Design Condition, or the "Immediate Improvements" phase.

The City's current discharge permit also allows for increasing ADWF discharges to 4.5 mgd upon expansion of the treatment facilities to provide the additional capacity. The improvements needed to provide this added capacity will be constructed under a second phase, hereinafter referred to as the Future Permitted Design Condition, or Phase 2.

Finally, the WWTP must ultimately be able to accommodate the wastewater flows anticipated at buildout of the City's General Plan. The improvements needed to provide this added capacity will be constructed under a third, and final, phase, hereinafter referred to as the Ultimate Buildout Design Condition, or Phase 3.

1.6 REFERENCES

Central Valley Regional Water Quality Control Board, *Order No. R5-2010-0099, NPDES No. CA0081434, Waste Discharge Requirements for the City of Galt, City of Galt Wastewater Treatment Plant and Reclamation Facility, Sacramento County*. September 23, 2010.

Mintier Harnish, ESA, Applied Development Economics, and Omni-means, *2030 Galt General Plan Policy Document, Final*. April 2009.

West Yost Associates, *Phase I Feasibility Study*, City of Galt Wastewater Treatment Plant. December 2004.

West Yost Associates, *NPDES Permit Compliance Action Plan, City of Galt Wastewater Treatment Plant*. December 2005.

Source: Imagery provided by ESRI Online.

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Scale in Feet

FIGURE 1-1

City of Galt
WWTP Facilities Master Plan

SITE LOCATION MAP

WEST YOST ASSOCIATES
Consulting Engineers

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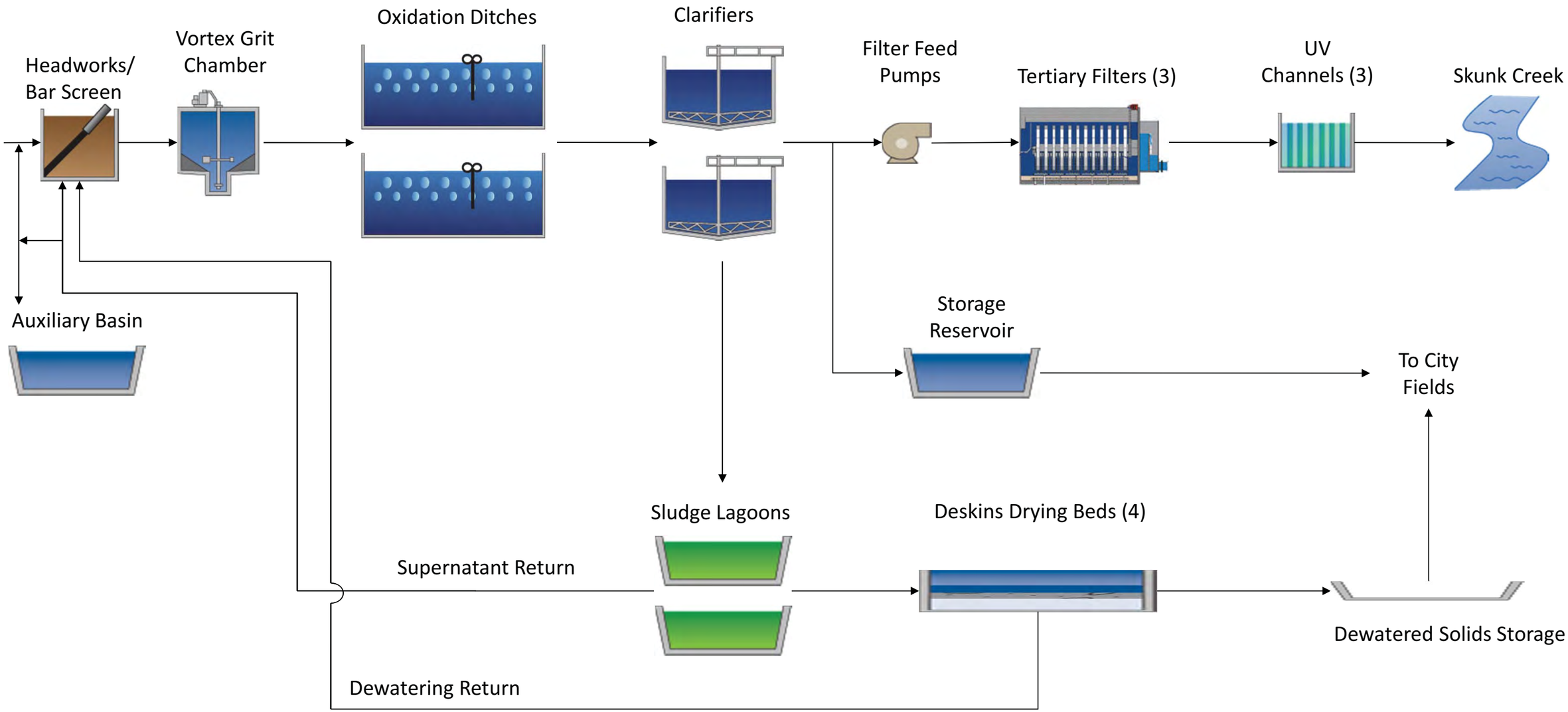

FIGURE 1-2

City of Galt
WWTP FACILITIES MASTER PLAN
EXISTING FACILITIES SCHEMATIC

WEST YOST ASSOCIATES
Consulting Engineers

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This chapter details the principal design criteria (basis of design) for the WWTP. The major topics covered in this chapter include the following:

- Projected Average Dry Weather Flow
- Design Flow Characteristics
- Design Load Characteristics
- Secondary Process Influent Characterization
- Current Regulatory Requirements
- Reliability/Redundancy Requirements

2.1 PROJECTED AVERAGE DRY WEATHER FLOW

Projections of future influent flows and waste loads to the WWTP are necessary to adequately plan WWTP facilities needed to serve the planned buildout population defined in the City's General Plan (Mintier Harnish et. al., 2009). Two differing wastewater flow projections for the buildout conditions were recently prepared by others. This section includes a review of these two projections, which differ substantially, and a recommended projection of Average Dry Weather Flow (ADWF) to be used for WWTP planning purposes.

2.1.1 Previously Developed Flow Projections

The wastewater flow projection that was prepared for the General Plan update and the flow project developed for the City's Wastewater Collection System Master Plan (Carollo Engineers, 2005; referred to herein as "Collection System Master Plan") are discussed below.

2.1.1.1 General Plan Wastewater Flow Projection

An estimate of WWTP influent flows at buildout of the General Plan is presented in Chapter 6 of the City's General Plan Draft Environmental Impact Report (ESA, 2008). The estimate is summarized in Table 6-2 of that document and is based on the following assumptions:

- Future residential dwelling units will each produce 326 gallons per day (gpd).
- Non-residential growth, except for parks, will produce 7.4 equivalent dwelling units (EDU) per acre at 250 gpd/EDU.
- Parks will produce 0.50 EDU per acre at 326 gpd/EDU.
- Growth within the existing City limits will produce 1,870 future residential dwelling units, 10 acres of parks, and 346 other non-residential acres.
- Growth outside the existing City limits will produce 7,865 future residential dwelling units (including mixed use units), 140 acres of parks, and 1,235 other non-residential acres.

The growth used as the basis for projecting future flow increases therefore totals 9,735 dwelling units, 150 acres of parks, and 1,581 other non-residential acres. The stated assumptions result in a growth flow projection, which, when added to the then-current (2007) average dry weather flow (ADWF) of about 2.3 million gallons per day (mgd), produced a total estimated flow of 8.4 mgd at General Plan buildout.

2.1.1.2 Wastewater Collection System Master Plan Flow Projection

The Collection System Master Plan included a projection of wastewater influent flows based on the acres in each General Plan land use category and flow coefficients calibrated using flow data from the collection system and the WWTP. The acreage used to project flow excluded currently undeveloped areas that lie within the 100-year floodplain. The following land uses were assumed to have no flow to the WWTP:

- Rural residential areas (1,236 acres)
- Roads, railroads, and canals, which were allocated approximately 20 percent (1,572 acres) of the total gross acreage
- The WWTP area (296 acres)
- Parks (190 acres)
- Open Space (78 acres)

The total flow-generating acreage for the Collection System Master Plan included 2,945 residential and mixed residential/commercial acres, plus 1,742 non-residential acres. Flow coefficients used to estimate flows from both existing and growth areas are summarized in Table 2-1.

The stated assumptions are consistent with the 2007 average daily flow of 2.3 mgd, and result in a General Plan Buildout flow projection of 5.6 mgd. The Collection System Master Plan notes that there is very little difference between the average daily flow (which is the 12-month annual average flow) and the ADWF reported at the WWTP each year when a four month average (June through September) is used for the ADWF.

Table 2-1. Wastewater Collection System Master Plan Flow Coefficients

Land Use Designation	Flow Coefficient[(a)], gpd/acre
Rural Residential	0
Residential Estates	300
Low Density Residential	1,385
Medium Density Residential	1,900
Medium High Density Residential	2,200
High Density Residential	2,300
Commercial	800
Office Professional	800
Light Industrial	800
Public/Quasi-Public	800
Mixed Use	800
Open Space	0
Park	0
Meadowview Park	0
Roads/Railroads/Canals	0
WWTP	0

[(a)] Source: Carollo Engineers, 2010. Dwelling-unit based residential flow factors, and higher flow factors in the Office Professional and Light Industrial categories are used for WWTP planning, as discussed in this section.

2.1.1.3 Comparison of Estimated Buildout Per-Capita Flows to Historical Values

A Per capita flow value, which is defined as the total ADWF (i.e. both residential and non-residential flow flows) divided by a given population, are often used in wastewater planning as the basis for projecting flow increases with increased populations. A comparison of the per-capita flow values associated with the two previously developed flow projections discussed above to historic values is presented below.

The General Plan flow projection anticipated that, with a buildout population is 51,291 and an ADWF of .4 mgd, the per capita flow rate would be 168 gpd per person.

The Collection System Master Plan included a preliminary estimated buildout population of 50,094. Using this population value, the anticipated Collection System Master Plan buildout flow of 5.6 mgd would result in a per capita flow value of 118 gpd/person. The Collection System Master Plan documented a historical per capita flow averaging 92 gpd/person from 2004 through 2008, with a maximum of 97 gpd/person in 2007. It is noted in the Collection System Master Plan that employment-related (i.e., flow-producing non-residential) land uses will increase from eight percent of the total developed land in 2007 to 16 percent at buildout, and that this increase explains the increase in per capita flow from an average of 92 to the predicted 118 gpd/person.

Population and ADWF data from 2000 through 2012, along with the calculated per capita ADWF, are summarized in Table 2-2. For the purposes of the current planning efforts, ADWF has been defined in accordance with the discharge permit for the WWTP, Waste Discharge Requirements Order No R5-2010-0099 (WDRs), which says that compliance with the ADWF limitation at the WWTP will be determined annually based on the average daily flow over three consecutive dry weather months. The ADWF shown in Table 2-2 is based on the lowest three-month average reported influent flow from the months of May through October of each year.

Table 2-2. Per Capita Flow History

Year	ADWF[(a)], mgd	Population[(b)]	Calculated Per Capita Flow, gpd/person
2000	1.82	19,472	93
2001	1.99	20,047	99
2002	1.97	20,925	94
2003	1.94	21,843	89
2004	1.90	21,952	90
2005	2.05	22,485	91
2006	2.11	22,628	93
2007	2.27	22,996	99
2008	2.08	23,371	89
2009	2.43	23,584	103
2010	2.19	23,647	93
2011	2.12	23,757	89
2012	2.19	24,076	91

(a) ADWF = Average dry weather flow (lowest three-month average).
(b) Population as of January 1, except for years 2000 and 2010 (April 1). Sources: Department of Finance, 2012a; Department of Finance, 2012b.

2.1.2 Recommended Projection for WWTP Planning

Future flows may be estimated using a population based projection to the degree that the ratio of residential to non-residential flows is expected to remain constant. However, if non-residential flows are expected to increase relative to residential flows (as concluded in the Collection System Master Plan), an increasing per capita flow may be justified. Therefore, to predict General Plan buildout flows, it is recommended that the flow projection be based on land use.

A land use based flow projection requires a quantification of future land uses, and wastewater flow generation factors (flow factors). A land-use approach was used for the Collection System Master Plan and (as shown in Table 2-1) the flow factors were expressed in units of gallons per acre per day for non-residential categories. However, for wastewater treatment facilities master planning, a more refined approach to developing specific flow factors for each General Plan land use category has been developed from the Collection System Master Plan recommendations. The specific flow factors recommendations are as follows:

Residential Land Uses. The Collection System Master Plan used an acreage-based flow factor for predicting future residential flows. The General Plan projects future residential land use in terms of both acreage and dwelling units. Dwelling units are recommended as a more accurate method of predicting flows associated with residential growth. The non-residential portion of flow in 2007 was estimated to equal approximately 312,000 gpd ADWF. However, the Collection System Master Plan did not differentiate school-based flows from other public and quasi-public flows; all non-residential flows were based on a factor of 800 gpd/acre. For the purposes of WWTP planning, school-based flows will be predicted using a per-student factor (as described below), which results in a revised estimate of 2007 non-residential flows equal to 386,000 gpd. Given a total ADWF of 2,310,000 gpd, the total residential flow for 2007 is estimated to be 1,924,000 gpd. The residential population in 2007 was 22,996, so the estimate flow was equal to 83.67 gpd per person. For predicting future residential flows, a factor of 80 gpd per person will be used in combination with the General Plan projections for persons per household. The resulting per-dwelling unit factors are presented in Table 2-3.

Table 2-3. Residential Flow Factors for WWTP Planning

Land Use Category a	Persons per Household[(a)]	Per-dwelling unit ADWF[(b)], gpd/du
Estates (K-RE)	2.75	220
Low Density (B-LDR)	3.00	240
Medium Density (C-MDR)	3.00	240
Medium High Density (L-MHD)	3.00	240
High Density (D-HDR)	3.25	260
Mixed Use (M-MU) – Residential Portion	3.25	260
Rural Residential (A-RR)	Not included in WWTP service area.	

(a) Galt General Plan.
(b) Based on 80 gpd per person (excluding all non-residential flows).

School Flows. School flows will be predicted using a factor of 6.17 gpd per City resident. This factor was derived by reviewing the historical school student population relative to the total City population, and estimating flows based on 20 gpd per elementary school student and 25 gpd per secondary school student. Table 2-4 summarizes the derivation of the per-resident school flow factor.

Other Non-Residential Flows. The Collection System Master Plan used a single non-residential flow factor of 800 gpd/acre. For WWTP planning, it is recommended that higher flow factors in the Office Professional and Light Industrial land use categories be used, as presented in Table 2-5. The WWTP planning factors account for an assumed future average floor-to-area ratio (FAR) representing the density of building floor space relative to the total parcel acreage, as well as an assumed density of employees per square foot of floor area. The assumed future average FAR is significantly lower than the maximum FAR allowed by the General Plan for each category, as noted in the table.

Exceptions to the recommended flow factors may be considered on a case-by-case basis, and large industrial dischargers (if anticipated) must be evaluated separately from acreage-based flow projections. However, City staff have indicated that WWTP planning should not incorporate an allowance for a future large "special case" industrial discharger at this time.

Table 2-6 summarizes the WWTP flow projection for buildout of the General Plan, including the acres, dwelling unit count, and flow factors used to develop the projection. The acreages have been reconciled to the General Plan Land Use Diagram, and data from the City geographical information system provided by City staff. **The total projected flow at buildout of the General Plan is 5,996,000 gpd, or 6.00 mgd.**

2.1.3 Impacts of Water Conservation

Successful water conservation measures in Galt may serve to reduce the per capita flow rate. However, in WWTP planning, peak flows and waste loads are generally of primary concern. Water conservation will not have an effect on influent loads to the WWTP, and conservation would only result in a marginal reduction to the peak flows. Moreover, it is difficult to make accurate predictions on how water conservation will reduce base flows, as the majority of water conservation measures are directed toward irrigation water uses, which have no impact on collection system flows. Therefore, water conservation is not accounted for in this facilities planning effort. Nevertheless, to the extent water conservation measures are effective, energy used for wastewater pumping and some treatment processes will be reduced and the life of hydraulically-limited structures and processes will be extended. Therefore, the City should re-evaluate the current and anticipated flows as a first step to evaluating improvements under the various project design phases.

Table 2-4. Derivation of School Flow Factor (gpd Per City Resident)

School Yr Ending	City Population	Students[a]			Flow[b], gpd		gpd per resident
		Elementary	Secondary	Total	Elementary	Secondary	
2000	19,472	3,838	1,817	5,655	76,760	45,425	6.27
2001	20,047	4,004	1,890	5,894	80,080	47,250	6.35
2002	20,925	4,089	1,992	6,081	81,780	49,800	6.29
2003	21,843	4,257	2,074	6,331	85,140	51,850	6.27
2004	21,952	4,348	2,131	6,479	86,960	53,275	6.39
2005	22,485	4,384	2,184	6,568	87,680	54,600	6.33
2006	22,628	4,382	2,251	6,633	87,640	56,275	6.36
2007	22,996	4,394	2,343	6,737	87,880	58,575	6.37
2008	23,371	4,292	2,475	6,767	85,840	61,875	6.32
2009	23,584	4,190	2,405	6,595	83,800	60,125	6.10
2010	23,647	4,122	2,292	6,414	82,440	57,300	5.91
2011	23,757	3,972	2,270	6,242	79,440	56,750	5.73
2012	24,076	3,855	2,287	6,142	77,100	57,175	5.58
						13 year average:	6.17

Note:
(a) Source: California Department of Education, http://dq.cde.ca.gov/dataquest/DQ/EnrTimeRpt.aspx?Level=District&cYear=2011-12&cname=GALT%20JOINT%20UNION%20ELEMENTARY&cCode=3467348 (downloaded 7/3/2012).
(b) Based on 20 gpd/elementary school student and 25 gpd/secondary student (as used in Vacaville, California for sewer planning purposes), given City population and school enrollment from 2000 through 2010.

Table 2-5. Derivation of Non-Residential Flow Factors

Land Use Category	General Plan (Table LU-2)		Ratio of Average FAR to Max FAR[a]	Assumed Avg. FAR	Assumed gpd/empl[b]	Employment gpd/acre	Other Process Flow, gpd/acre	Recommended WWTP Planning Factor		Notes
	Max Allowed FAR	Employee Density, sf/empl								
Commercial (E-C)	3.0	550	0.165	0.50	20	800	—	800	gpd/acre	Factor is consistent with 2010 Wastewater Collection System Master Plan findings.
Office Professional (F-OP)	2.0	283	0.20	0.40	20	1,200	—	1,200	gpd/acre	Allows for higher future flows than 2010 Wastewater Collection System Master Plan.
Industrial (G-LI)	1.0	439	0.50	0.50	25	1,200	—	1,200	gpd/acre	Allows for higher future flows than 2010 Wastewater Collection System Master Plan.
Industrial (G-LI) - large user	—	—	—	—	—	—	—	—	—	No separate allowance for a large industrial discharger is included.
Mixed Use (M-MU), commercial portion	3.0	550	0.165	0.50	20	800	—	800	gpd/acre	Assume employee density and FAR similar to "Commercial". Factor is consistent with 2010 Wastewater Collection System Master Plan findings.
Public/Quasi-Public (H-PQP)										
Schools	—	—	—	—	—	—	—	6.17	gpd/City Resident	See Table 2-4 for derivation of factor.
Hospitals	—	—	—	—	—	—	—	—	—	Future hospital use is included in the general "Other" factor. Per-acre flow will be above the average for the H-PQP category, which may be important for collection system capacity planning.
Other	1.0	283	0.25	0.25	20	800	—	800	gpd/acre	Assume employee density similar to "Office Professional". Factor is consistent with 2010 Wastewater Collection System Master Plan findings.

Notes:
(a) Floor-to-Area Ratio (FAR) used to calculate number of employees is the listed fraction of the maximum allowable FAR stated in the General Plan.
(b) Sacramento Area Sewer District's basis for industrial discharger's domestic flow charges is 20 gpd per office employee and 25 gpd per plant operations employee (California Department of General Services Central Plant, Sacramento).

Table 2-6. Land-Use Based Flow Projection

Land Use Category	General Plan			WWTP Planning Factors			WWTP Planning Acres			Buildout Flow, gpd		
	General Plan Land Use Diagram[(a)], acres	DU	Persons Per Household	Existing gpd/acre	Growth, gpd/acre	Existing & Growth gpd/DU	Existing	Growth	Total at Buildout[(a)]	Flow from Existing, gpd	Flow from Growth, gpd	Total Flow at Buildout
Residential												
Rural Residential (A-RR)	1,451	—	—	—	—	0	0.1	1,236	1,236	—	—	—
Estates (K-RE)	188	188	2.75	—	—	220	-	172	172	—	41,000	41,000
Low Density (B-LDR)	2,465	9,860	3.00	—	—	240	990	1,213	2,203	1,320,000	1,046,000	2,366,000
Medium Density (C-MDR)	366	2,196	3.00	—	—	240	208	113	321	380,000	147,000	527,000
Medium High Density (L-MHD)	83	913	3.00	—	—	240	—	63	63	—	219,000	219,000
High Density (D-HDR)	198	3,564	3.25	—	—	260	101	70	171	223,000	704,000	927,000
Mixed Use (M-MU) - Res Portion	Included	376	3.25	—	—	260	—	—	—	—	98,000	98,000
SUBTOTAL, RESIDENTIAL	**3,300**	**17,097**	—	—	—	—	**1,299**	**1,631**	**2,930**	**1,923,000**	**2,255,000**	**4,178,000**
Employment Related and Other												
Commercial (E-C)	682	—	—	800	800	—	101	521	622	81,000	417,000	498,000
Office Professional (F-OP)	197	—	—	800	1,200	—	3	176	179	2,000	211,000	213,000
Light Industrial (G-LI)	651	—	—	800	1,200	—	120	419	539	96,000	503,000	599,000
Light Industrial (G-LI) - large user	—	—	—	—	—	—	—	—	—	—	—	—
Mixed Use (M-MU)	21	—	—	800	800	—	4	15	19	3,000	12,000	15,000
Public/Quasi-Public (H-PQP), Schools	200	—	—	—	—	—	90	110	200	146,000	171,000	317,000
Public/Quasi-Public (H-PQP), Other	282	—	—	800	800	—	72	147	219	58,000	118,000	176,000
SUBTOTAL, NON-RESIDENTIAL	**2,033**	—	—	—	—	—	**390**	**1,388**	**1,778**	**386,000**	**1,432,000**	**1,818,000**
Categories Not Served by the Collection System and/or Not Generating Flow												
Open Space (I-OS)	572	—	—	—	—	—	11	550	561	—	—	—
Park (I-PK)	205	—	—	—	—	—	70	121	191	—	—	—
Roads/Railroads/Canals	Included	—	—	—	—	—	702	870	1,572	—	—	—
Flood Plain Areas Outside the City Limit	—	—	—	—	—	—	—	253	253	—	—	—
Wastewater Treatment Plant	296	—	—	—	—	—	293	3	296	—	—	—
Totals	**7,857**	**17,097**	—	—	—	—	**2,765**	**6,052**	**8,817**	**2,309,000**	**3,687,000**	**5,996,000**

(a) City staff confirmed that the total area encompassed by the General Plan Planning Area is 8,817 acres. The GP Land Use Diagram identifies 7,857 acres, representing the net parcel area within the current City Limits plus the gross parcel area outside the City Limits. Therefore the total WWTP Planning Acres at buildout (net parcel area within and without the current City Limits) will be lower than the GP Land Use Diagram acreage in the flow-producing categories.

2.1.4 Recommended Flow Projection

As discussed in Chapter 1 (Introduction), the construction of expanded treatment facilities will occur under the following three phases:

- Current Permitted Design Condition phase, which will address the improvements needed assure reliable compliance when ADWF is 3.0 mgd (i.e. the current permitted discharge flow rate).
- Future Permitted Design Condition phase, which will include the expansion projects needed to ensure reliable treatment as the ADWF increases to 4.5 mgd.
- Ultimate Buildout Design Condition phase, which will include the expansion projects needed to ensure reliable treatment as the ADWF increases to 6.0 mgd (i.e. the ADWF anticipated at the General Plan buildout)

The timing of the two future expansion phases will ultimately be determined by the actual rate of growth in the City of Galt. Expansion will be triggered either by increasing flow, or if water conservation results in decreased flow per person and higher influent waste concentrations, an expansion may be triggered by increased waste loads to the plant.

The General Plan is based on reaching a buildout population of 51,291 by the year 2030. This constituted an average annual growth rate of about 3.4 percent from 2007. However, the actual growth rate since the General Plan projections were prepared has been much lower. Given recent trends, it is unlikely that the population predicted in the General Plan will not be reached by 2015, and even a return to growth rates experienced in the early years of the current century will not produce the expected buildout population by 2030. Because an over-estimation of growth rates could overinflate City budgeting need, an alternative flow projection is recommended for WWTP facilities planning purposes.

For WWTP planning purposes, it is assumed that over the next two year period, it is expected that the ADWF will rebound by about 0.3 mgd to account for the fact that at least a portion of the recent ADWF decrease to the WWTP is associated with reduced economic activity. This assumption implies that the ADWF in 2014 will be about 2.4 mgd. After 2014, several growth rate scenarios were evaluated for the period from 2014 through 2030. The three scenarios, as well as historical population information, are depicted on Figure 2-1, as follows:

Population Growth. The General Plan population projection is shown in five year increments beginning in 2015. The projection represents a 3.4 percent annual growth rate beginning with a 2015 population of 32,779. Using the same annual growth rate, the projected population is extrapolated backwards to the year 2000. Actual population as reported by the California Department of Finance is also shown from 2000 through 2012.

Historical Flow. WWTP monthly influent flow and ADWF are shown from 2000 through late 2012. Over the 10 year period from 2000 through 2009, the average annual increase in ADWF was about 2 percent.

2030 Buildout Scenario. If the projected buildout ADWF (6.0 mgd) were to be reached by 2030, the average rate of flow increase after 2014 would be about 5.73 percent per year. This far exceeds the historical average growth rate (prior to 2010). Under this scenario, and ADWF of 3.0 mgd would be reached by about 2017, and 4.5 mgd would be reached in 2025.

General Plan Growth Rate Scenario. Applying the 3.4 percent annual growth rate after 2014 results in an ADWF of 3.0 mgd by 2020 and 4.5 mgd sometime after 2030. Under this scenario one expansion to the future permitted design flow of 4.5 mgd would be adequate for the foreseeable future.

Historical Flow Growth Rate Scenario. Using the historical growth rate of 2 percent per year after 2014, expansion beyond 3.0 mgd would not be triggered until 2024.

Based on discussions with City staff and consideration of the presented range of flow growth projections, this WWTP Facilities Plan is based on the General Plan Growth Rate Scenario. The next expansion should therefore be planned for completion by 2020. Expansion beyond 4.5 mgd would not likely be needed until about 2032.

Influent flows and loads must be evaluated annually to assess whether or not this predicted schedule should be adjusted. A five-year planning, design and construction window should be used, such that initial expansion planning activities are initiated five years prior to the anticipated date of need. The flow projection presented in Figure 2-1 and its underlying assumptions should therefore be reviewed (and updated, as appropriate) in 2015.

2.2 DESIGN FLOW CHARACTERISTICS

In addition to the ADWF, treatment facilities must be designed to accommodate peak flows that may occur periodically during the life of the treatment plant, as well as the daily and seasonal fluctuations in flow (and waste load) that can affect treatment process performance. It is convenient to consider the ratio of the various important flow conditions relative to the predicted ADWF. This ratio is called a “peaking factor”.

2.2.1 Recommended Flow Peaking Factors

The following influent flow values are important:

- ADF: Average Daily Flow (average daily flow over the course of a calendar year)
- ADWF: Average Dry Weather Flow (the average daily flow over the three consecutive low-flow dry weather months)
- ADMMF: Average Day Maximum Month Flow (the average daily flow during the month with the maximum monthly flow)
- MDDWF: Maximum Day Dry Weather Flow (the highest flow for a calendar day during dry weather months (June through September)
- MDWWF: Maximum Day Wet Weather Flow (the highest flow for a calendar day during wet weather months (October through May)

- PDWF: Peak Hour Dry Weather Flow (the highest hourly flow during dry weather)
- PHWWF: Peak Hour Wet Weather Flow (the highest hourly flow during wet weather)

As noted above, influent flows reported prior to July 17, 2008, should not be used to calculate averages on intervals shorter than several months, since the reported influent flow was a calculated value equal to 1.01 times either the reported effluent flow or the reported irrigation water flow. Both of these data sources (effluent flow and irrigation flow) have been influenced by storage in the reservoir and ponds.

From July 18, 2008 to the present, influent flow is assumed to be based on the influent flow meter. Therefore, daily peaking characteristics should be based on data since July 2008. Hourly data was not recorded until late December 2010; therefore, historical hourly influent peaking characteristics can only be based on data from January 2011 to present.

Furthermore, maximum flow to the WWTP is limited to the maximum pumping capacity of the Live Oak Pump Station, which is a constant-speed pump station. Based on hourly influent flow meter data from 2011, the peak hourly pumping capacity of Live Oak Pump Station appears to be about 5.2 mgd. This flow limitation will be eliminated in 2013 upon completion of a replacement of the Live Oak Pump Station with a higher capacity station. The initial peak hour wet weather flow (PHWWF) design capacity of the new station is 9.5 mgd, and the station is expandable to a capacity of 14.0 mgd to accommodate General Plan buildout (year 2030) flows (Kennedy/Jenks Consultants, 2010). The new pump station will be equipped with variable speed drives.

Given the limited amount of data useful for calculating short-interval (hourly, daily, monthly) influent flow, an alternative basis for estimating peaking factors for the various flow values is advised. The Collection System Master Plan presented peaking factors representing the ratio between the given flow and ADF, noting that data from 2004 through 2008 indicates that there is little difference between average day (ADF), average dry (ADWF) and average wet weather flow (AWWF). For consistency with the WDRs, this WWTP Facilities Master Plan has been based on a projection of ADWF rather than ADF; however, the ratios between average and peak flows have been assumed to be the same as predicted in the Collection System Master Plan using ADF as a basis.

The flow peaking factors recommended for design of the WWTP are described in Table 2-7. These peaking factors can be used to translate projected ADWF values for a given year into peak flow values for the same year.

Table 2-7. Galt WWTP Recommended Flow Peaking Factors

Flow Characteristic	Peaking Factor	Basis of Peaking Factor
ADWF	1.0	Collection System Master Plan
ADMMF	1.3	Calculated based on monthly flow data from June 2000 through September 2011
MDDWF	1.7	Calculated value (1.74) based on daily flows reported by the influent flow meter August 2008 through September 2011
MDWWF	2.0	Calculated value (2.02) based on daily flows reported by the influent flow meter August 2008 through September 2011. Collection System Master Plan also estimated the value to be 2.0 based on data from 2007.
PDWF	1.9	Hourly data from 2011 only.[a]
PHWWF and sustained peak influent flow rate (over a 4-hour period)	3.0[b]	Collection System Master Plan

[a] Hourly data is not available prior to January 2011. The highest hourly flow during dry weather months in 2011 was about 4.0 mgd, which occurred twice, representing a peaking factor of about 1.9.
[b] Derived using wastewater collection system modeling. Hourly flow data was not available at the WWTP prior to 2011. The Live Oak Pump Station preliminary design report based design of the pump station on a peaking factor of 2.7 times the future ADWF.

As shown in Table 2-8, the Collection System Master Plan presented peaking factors for several flow values, including MDWWF (2.0) and PHWWF (3.0). These factors were developed using a calibrated model simulating flows that could occur during a 10-year, 24-hour design storm. The design storm has a total depth of precipitation equal to 3.26 inches, with a peak intensity of 0.62 inches/hour (Carollo Engineers, 2010). The remaining peaking factors for ADMMF, MDDWF, and PDWF were derived from available data, where possible.

The PHWWF peaking factor presented in the Collection System Master Plan was derived from the wastewater collection system model and judged to be consistent with the range of peaking factors typical for WWTPs in similar climates. A value of 3.0 is at the low end of typical values, however, it is reasonable and is consistent with the modeling predictions. At the WWTP, the highest recorded hourly flow in 2011 was 5.23 mgd, which occurred over a single hour on February 18. Flows of about 5.2 mgd (the apparent pump station capacity) were sustained for a six-hour period on March 24. Therefore, influent flows to the Live Oak Pump Station therefore likely exceeded 5.2 mgd for a sustained period of time on March 24. Based on a peaking factor of 3.0, the PHWWF would be 6.3 mgd, which is a plausible peak hour value for March 24, 2011, but not verifiable with available data.

The storm that occurred on March 24, 2011 has a return period somewhat less than that of a 2-year, 3-hour storm, based on the Sacramento County *Hydrology Standards* (Sacramento County, 1996). Larger storms, especially heavy storms preceded by extended periods of wet weather, have the potential to produce higher infiltration and inflow rates, and corresponding higher flows to the Live Oak Pump Station. One of the largest storms in recent history occurred on December 31, 2005, with a return period of 5 years at a duration of 14-hours. If available, continuous flow records from Live Oak Pump Station could be inspected to determine the duration of operation at pump station capacity as a second point of comparison. Based on the data analyzed to date, a PHWWF peaking factor of 3.0 has been used for WWTP facilities master planning purposes, but it has also been conservatively assumed that the PHWWF could be sustained over a 4-hour period.

2.2.2 Design Flows

Design flows for each of the three design phases, along with the corresponding 2011 flows, have been determined based on the flow peaking factors described above and are summarized in Table 2-8.

Table 2-8. Galt WWTP Design Flows

Flow Characteristic	Peaking Factor	2011 Flow, mgd	Current Permitted Design Condition (3.0 mgd ADWF), mgd	Future Permitted Design Condition (4.5 mgd ADWF), mgd	Ultimate Buildout Design Condition[a] (6.0 mgd ADWF), mgd
ADWF	1.0	2.11	3.0	4.5	6.0
ADMMF	1.3	2.7	3.9	5.9	7.9
MDDWF	1.7	3.6	5.1	7.7	10.3
MDWWF	2.0	4.2	6.0	9.0	12.1
PDWF	1.9	4.0	5.7	8.6	11.5
PHWWF[b]	3.0	6.3	9.0	13.5	18.0

[a] Buildout flow is for a population of 51,291 (Mintier Harnish et. al., 2009).
[b] Also assumed to represent a sustained peak influent flow rate (over a 4-hour period).

2.3 DESIGN LOAD CHARACTERISTICS

For WWTP planning, influent waste loads must also be considered in addition to influent flow. The secondary treatment process and solids treatment train capacity requirements are dependent primarily on influent loads, not on flow. Successful water conservation measures in the City may serve to reduce the gallons of water received at the WWTP per resident, but conservation will not affect the quantity of waste (in pounds) to be treated. Accordingly, this section includes a presentation of design load criteria for the WWTP. The presented criteria are for master planning purposes, and are thus subject to reevaluation and modification as additional data become available over the course of time. The major topics covered in this document include:

- Historical WWTP Influent Load Data
- Design Load Peaking Factors
- Design Loads

2.3.1 Historical WWTP Influent Load Data

Annual average influent BOD and TSS concentrations for the WWTP are shown on Figure 2-2. Monthly average influent BOD and TSS concentrations are shown on Figure 2-3 and Figure 2-4, respectively. As indicated on Figure 2-2, the highest annual average BOD concentration was 255 mg/L, and the lowest was 127 mg/L. The spread is even greater for TSS, where the annual average concentration was 312 mg/L and the lowest 95 mg/L. Monthly averages show an even more dramatic spread. The highest monthly average BOD concentration was 446 mg/L, and the lowest was 83 mg/L, as indicated on Figure 2-3. The highest monthly average TSS concentration was 693 mg/L, and the lowest was 50 mg/L, as indicated on Figure 2-4. The validity of these ranges and the need for corrections is discussed in the following paragraphs.

According to *Wastewater Engineering* (Metcalf & Eddy, Inc., 1991), for untreated domestic wastewater, BOD values can be expected to be in the range of 100 mg/L to 400 mg/L, with typical values around 220 mg/L, while TSS values can be expected to be in the range of 100 mg/L to 350 mg/L, with typical values also around 220 mg/L. Moreover, variability is not expected to be high in wastewater dominated by residential/commercial inputs, especially where infiltration and inflow is not significant.

These results suggest that the influent data for the period in question have shown unrealistic levels of variability, and also appear to exhibit long-term biases in both directions. Specifically, the results appear unrealistically high at times in the early to mid-2000s, and unrealistically low at times from the late 2000s onward. Unrealistically high results could be obtained if the influent sampler tended to collect flow from a part of the influent channel that was prone to accumulating solids, whereas unrealistically low results could be obtained if the influent sampler inlet tended to get ragged up for prolonged periods. Biases arising from the influence of plant recycle flows may also have been a factor in past results.

Given these results, it does not appear that pre-2012 data are usable for this analysis. However, corrections have since been made to the influent sampling methodology, and an intensive Special Sampling Program was performed during late April and early May 2012 that provided two weeks of apparently reliable daily 24-hour influent composite sample data uninfluenced by recycle streams. Sampling was performed based on the Sampling Plan TM (Appendix A). The Special Sampling Program results for BOD, TSS, and ammonia are summarized in Table 2-9.

Table 2-9. Summary of Key Galt WWTP Influent Characteristics, Special Sampling Program, April/May 2012

Parameter	Influent Concentration, mg/L		
	Average	Minimum	Maximum
BOD	206	161	262
TSS	207	172	294
Ammonia	25.3	24	28
TKN	36.4	35	38

2.3.2 Design Load Peaking Factors

For the current analysis, significant seasonal variability or other longer-term sources of variation are not expected. The average values indicated in Table 2-9 are thus assumed to be representative of long-term average conditions of the WWTP influent. However, due to the lack of long-term data available, projecting max month and max day loads (as is required for design of secondary facilities) is not possible using the existing WWTP influent data alone. Therefore, for purposes of this Facilities Master Plan, the maximum month and maximum day load peaking factors have been determined from other, similar facilities.

Three other facilities with similar service areas are used in this analysis as a basis for determining load peaking factors. Specifically, previously published design criteria for the cities of Lodi, Vacaville, Davis, and Woodland are used to estimate max month and max day load factors for the WWTP. Typical load peaking factors for each city are shown in Table 2-10. (Note that the results for Davis may be amplified since a significant student population is present during peak loading periods but not during baseline periods.)

Given the results shown in Table 2-10, and given the limited amount of reliable data for the WWTP influent, the following conservatively-assumed load peaking factors will be used in this analysis for all four constituents of interest: BOD, TSS, ammonia, and TKN.

- Max Month: 1.3
- Max Day: 1.6

Table 2-10. Design Load Peaking Factors from Similar Service Areas

Parameter	Vacaville[a]	Lodi[b]	Davis[c]	Woodland[d]
BOD				
Max Month	1.12	1.22	1.29	1.3
Max Day	1.27	1.61	1.64	1.7
TSS				
Max Month	1.15	1.28	1.34	not estimated
Max Day	1.42	1.61	1.82	not estimated
Ammonia				
Max Month	1.13	not estimated	1.36	not estimated
Max Day	1.26	not estimated	1.72	not estimated
TKN				
Max Month	1.16	not estimated	not estimated	not estimated
Max Day	1.21	not estimated	not estimated	not estimated

(a) Source: West Yost, 2009.
(b) Source: West Yost, 2001.
(c) Source: Carollo Engineers, 2005.
(d) Source: ECO:LOGIC Consulting Engineers, 2001.

2.3.3 Design Loads

Buildout loads should be projected based on historical waste concentrations and the projected flow, assuming no water conservation. In this manner, the WWTP facilities can be adequately planned to accommodate the future loads, whether or not the per capita flow decreases. Using this approach, the projection of future flows - assuming no reduction in per capita flow due to water conservation- should accurately predict the timing of future improvements because waste load triggers for plant expansion will occur regardless of water conservation measures.

Design loads have been determined for average conditions from the average observed concentrations shown in Table 2-9. Design loads for the maximum month and maximum day conditions were determined from these average load values and the load peaking factors. Design loads for the three design conditions of interest are shown in Table 2-11.

Table 2-11. Proposed Design Loads for the Galt WWTP

Design Condition	BOD, lb/day	TSS, lb/day	Ammonia, lb/day	TKN, lb/day
Current Permitted Design Condition (3.0 mgd ADWF)				
Average Day	5,100	5,200	600	900
Max Month	6,700	6,800	800	1,200
Max Day	8,200	8,400	1,000	1,500
Future Permitted Design Condition (4.5 mgd ADWF)				
Average Day	7,700	7,800	900	1,400
Max Month	10,000	10,100	1,200	1,800
Max Day	12,300	12,400	1,500	2,200
Ultimate Buildout Design Condition (6.0 mgd ADWF)				
Average Day	10,400	10,400	1,300	1,800
Max Month	13,500	13,600	1,700	2,400
Max Day	16,600	16,700	2,000	2,900

2.3.4 Secondary Process Influent Load Characterization

In addition to the characterization discussed above of the influent flows and loads for BOD, TSS, and nitrogen, a detailed characterization of the secondary process water quality is needed to understand potential impacts of key constituents and diurnal trends on the secondary process performance. Secondary process modeling has been performed with EnviroSim's BioWin software. A detailed discussion of the data collected – per the Sampling Plan (Appendix A) – and the secondary process evaluations completed for this Facilities Plan are provided in the WWTP Secondary Process Modeling TM (Appendix B).

Various combinations of flow conditions, load conditions, and temperature were analyzed through the secondary process modeling to determine the "worst-case" scenarios with respect to effluent concentrations and other operating conditions. The eleven combinations that were analyzed are summarized in Table 2-12.

Table 2-12. Influent Flow and Load Conditions Analyzed with Secondary Process Modeling of City of Galt Wastewater Treatment Plant

Flow Condition[b]	Load Condition[a]			
	Average Day Load (ADL)	Maximum Month Load (MML) [= 1.3 x ADL]	MML with Recycle Streams[b]	Maximum Day Load (MDL) [=1.6 x ADL]
Average Dry Weather Flow (ADWF)	21.6°C/ 17.0°C	21.6°C/17.0°C	17.0°C	
Average Day Maximum Month Flow (ADMMF) [= 1.3 x ADWF]		21.6°C/17.0°C		21.6°C/14.5°C
Maximum Day Dry Weather Flow (MDDWF) [= 1.7 x ADWF]				21.6°C/14.5°C

(a) Values shown are temperature values used in the analyzed scenarios. Temperatures shown represent the following based on the City's June 2010 through March 2012 (weekly) influent temperature data: Average temperature = 21.6°C; Lowest monthly average temperature = 17.0°C; Lowest temperature = 14.5°C.

(b) The City can operate the biosolids dewatering facility and sludge lagoon such that recycle streams have minimal or no flow during MDL conditions, but recycle streams are likely to have significant flows during a month with MML, so the impact of the recycle streams under MML conditions needs to be evaluated.

From an analysis of the eleven conditions listed above, three scenarios were identified to represent the range of design conditions for the WWTP, as follows:

- **Annual Average:** ADWF and ADL conditions at the average monthly average temperature of 21.6°C, including recycle stream flows and loads.
- **Maximum Month:** ADWF and MML conditions at the minimum monthly average temperature of 17.0°C, including recycle stream flows and loads
- **Maximum Day:** ADMMF and MDL conditions at a minimum historic temperature of 14.5°C

The raw wastewater influent concentrations for key constituents used in the secondary process evaluation are shown in Table 2-13 for the three design conditions.

Table 2-13. Secondary Process Modeling Influent Concentrations

Parameter	Units	Concentration		
		Annual Average	Maximum Month	Maximum Day
Organics				
BOD-total	mg/L	206	267	253
BOD-soluble	mg/L	69	89	84
COD-total	mg/L	498	648	613
COD-soluble	mg/L	142	184	174
Solids/Alkalinity				
TSS	mg/L	210	273	258
VSS	mg/L	183	238	226
ISS	mg/L	26	33.9	32
Alkalinity (as $CaCO_3$)[a]	mg/L	239		
pH[a]	--	7.3		
Nitrogen				
Ammonia as N	mg/L	25	33	31
Nitrate as N	mg/L	0.01	0.013	0.012
TKN-total	mg/L	36	47	45
TKN-soluble	mg/L	27	35	33
Phosphorus				
Phosphorus-total	mg/L	4.5	5.8	5.5
Phosphorus-soluble	mg/L	2.7	3.6	3.4
Other Required Parameters				
Calcium[b]	mg/L	80		
Magnesium[b]	mg/L	15		
Dissolved Oxygen[b]	mg/L	0.0		

[a] Values for alkalinity and pH are based on measured values from the Sampling Program.

[b] Values for calcium, magnesium, and dissolved oxygen are assumed based on typical domestic wastewater values.

In addition to the influent flows, the secondary process modeling considers high-strength (but low flow) return flows from the solids handling processes. For purposes of this WWTP Facilities Master Plan, the solids handling return flows were conservatively included in the steady-state modeling, as follows:

- **Sludge Lagoon Supernatant Flows:** Supernatant flow is estimated based on the annual average solids production rate and an assumed concentration of solids from 0.8 percent to 2.5 percent during the settling process. The resulting value of 0.0408 mgd was used for the Current Permitted Design Conditions condition and divided evenly per oxidation ditch modeled. For other design conditions, this flow was adjusted proportional to the influent flow increase above 3.0 mgd.
- **Dewatering System Filtrate Flows:** The volume and timing for filtrate return to the treatment process will vary depending on the selected solids processing alternative. For purposes of developing the preliminary steady-state modeling presented in this WWTP Facilities Master Plan, the peak flow was conservatively estimated as 0.374 mgd (including "clean water" used in the Deskin Beds). This value is based on one pump at a flow of 260 gallons per minute and represents a worst-case peak flow condition. This flow was divided evenly per oxidation ditch modeled. This flow was not adjusted for the increase in ADWF from existing conditions to the Current Permitted Design Condition (3.0 mgd) but was adjusted for the other design conditions proportional to the influent flow increase above 3.0 mgd.

This WWTP Facilities Master Plan addresses potential options for modifying the solids handling facilities. In addition, the improvements needed to allow the City to use the Auxiliary Storage Basin to equalize return flows under all future operations, thus allowing for return of high strength loads during off-peak periods, are recommended. Therefore, a more detailed dynamic modeling analysis will need to be performed for the preferred alternative during preliminary and final design that accounts for the solids handling and equalization approach identified for the selected expansion alternative.

The concentrations of key constituents in the recycle stream flows used in the secondary process evaluation for the three design conditions are shown in Table 2-14. (Note that total and soluble phosphorus were not measured as part of the sampling program and influent values were assumed for the solids return stream flows.)

Table 2-14. Assumed Secondary Process Modeling Recycle Stream Concentrations

Parameter	Unit	Filtrate Return[a]	Supernatant
Organics			
BOD-total	mg/L	125	314
BOD-soluble[b]	mg/L	92	277
COD-total	mg/L	230	467
COD-soluble	mg/L	170	413
Solids/Alkalinity			
TSS	mg/L	53	80
VSS	mg/L	36	51
ISS[c]	mg/L	21	30
Alkalinity (as $CaCO_3$)	mg/L	449	798
Nitrogen			
Ammonia (as N)	mg/L	74	147
Nitrate (as N)	mg/L	63	0.01
Nitrite (as N)	mg/L	2.9	0.002
TKN-total	mg/L	79	156
TKN-soluble[b]	mg/L	58	137
Phosphorus			
Phosphorus-total[d]	mg/L	4.5	
Phosphorus-soluble[d]	mg/L	2.7	
Other Required Parameters			
Calcium[d]	mg/L	80	
Magnesium[d]	mg/L	15	
Dissolved Oxygen[d]	mg/L	0.0	

(a) Based on Deskin Bed filtrated data. Concentrations may be higher for an alternative mechanical dewatering system where dilution water is not added.
(b) TKN-soluble and BOD-soluble levels were not measured directly. Average levels for these have been estimated based on the average TKN-total and BOD-total levels, respectively, multiplied by the ratio of average COD-soluble and COD-total levels.
(c) ISS data have been calculated as the difference between paired TSS and VSS data. The first two sets of data (5/15/12 10:10 AM and 11:10 AM had VSS values slightly greater than TSS values, so ISS values have not been calculated for those data.)
(d) Assumed to be equal to average influent concentrations.

2.4 CURRENT REGULATORY REQUIREMENTS

The WDRs, issued by the Central Valley Regional Water Quality Control Board (Regional Board) on September 23, 2010, include specific regulatory requirements for liquid and solid discharges from the WWTP that should be incorporated into the design criteria. In addition, regulations for the treatment and land application of Class B biosolids are enforced by the United States Environmental Protection Agency (USEPA) through the Code of Federal Regulation, Title 40, Part 503 (40 CFR 503). This section thus provides a discussion of the following specific regulatory requirements that should be incorporated into the design criteria for the WWTP facilities:

- Surface Water Discharge Effluent Limitations
- Filtration Operational Requirements
- Ultraviolet (UV) Disinfection Operational Requirements
- Biosolids Land Application Requirements
- Groundwater Limitations

2.4.1 Surface Water Discharge Effluent Limitations

Regulatory effluent limitations are the primary drivers for upgrading the WWTP and its subsequent operation. The effluent limitations prescribed in the WDRs are shown in Table 2-15 and Table 2-16.

Table 2-15. Galt WWTP Final Effluent Limitations (Order No. R5-2010-0099)

Constituent	Units	Effluent Limitation				
		Average Annual	Average Monthly	Average Weekly	Daily Maximum	Daily Minimum
Biochemical Oxygen Demand (5-day @ 20°C)	mg/L	—	10	15	20	—
Total Suspended Solids	mg/L	—	10	15	20	—
Ammonia Nitrogen, Total (as N)	mg/L	—	1.7	—	3.3	—
Nitrate plus Nitrite, Total (as N)	mg/L	—	10	—	—	—
pH[(a)]	s.u.	—	—	—	8.2	6.5
Aluminum, Total Recoverable	µg/L	200	—	—	—	—
Arsenic	µg/L[(b)]	—	10	—	—	—
Copper, Total Recoverable	µg/L	—	3.1	—	4.3	—
Iron, Total Recoverable	µg/L	300	—	—	—	—
Lead, Total Recoverable	µg/L	—	0.6	—	1.0	—
Manganese, Total Recoverable	µg/L	50	—	—	—	—
Chlorodibromomethane	µg/L	—	0.41	—	0.83	—
Dichlorobromomethane	µg/L	—	0.56	—	1.3	—
Bis (2-ethylhexyl) phthalate	µg/L	—	1.8	—	3.6	—
Carbon Tetrachloride	µg/L	—	0.25	—	0.5	—
Cyanide, Total (as CN)	µg/L	—	3.4	—	9.6	—

(a) Effluent limitations for pH are instantaneous minimum and maximum values.
(b) The WDRs list the units for the arsenic effluent limitation as “mg/L”, but it is assumed the correct units are “µg/L”.

Table 2-16. Additional Galt WWTP Effluent Limitations (Order No. R5-2010-0099)

Constituent	Units	Effluent Limitation	
		Type	Value
Biochemical Oxygen Demand (5-day @ 20°C)	%	Minimum Average Monthly Removal	85
Total Suspended Solids	%	Minimum Average Monthly Removal	85
Acute Whole Effluent Toxicity	%	Minimum of any one bioassay	70
		Minimum Median of any three consecutive bioassays	90
Total Residual Chlorine	mg/L	4-day Average	0.01
		1-hour Average	0.02
Total Coliform Organisms	MPN per 100 mL	7-day Median	2.2
		Maximum in any 30-day period	23
		Maximum, at any time	240

2.4.2 Filtration Operational Requirements

Special Provision VI.C.6.a of the WDRs requires that effluent discharged to Skunk Creek "shall be oxidized, coagulated, filtered, and adequately disinfected pursuant to California Department of Public Health (DPH) reclamation criteria, Title 22 CCR [California Code of Regulations], Division 4, Chapter 3, (Title 22) or equivalent." "Filtered wastewater" is defined in Title 22 (Section 60301.320) as follows:

> *"…an oxidized wastewater that meets the criteria in subsection (a) or (b) below:*
>
> *(a) Has been coagulated and passed through natural undisturbed soils or a bed of filter media pursuant to the following:*
>
> > *(1) At a rate that does not exceed 5 GPM/ft2 in mono, dual or mixed media gravity or pressure filtration systems, or does not exceed 2 GPM/ft^2 in traveling bridge automatic backwash filters; and*
> >
> > *(2) The turbidity does not exceed any of the following:*
> >
> > > *(A) An average of 2 NTU* [Nephelometric Turbidity Units] *within a 24-hour period;*
> > >
> > > *(B) 5 NTU more than 5 percent of the time within a 24-hour period; and*
> > >
> > > *(C) 10 NTU at any time.*
>
> *(b) Has been passed through a micro. nano. or R.O. membrane following which the turbidity does not exceed any of the following:*
>
> > *(1) 0.2 NTU more than 5 percent of the time within a 24-hour period; and*
> >
> > *(2) 0.5 NTU at any time."*

Regarding coagulation, Sections 60304 ("Use of Recycled Water for Irrigation") and 60307 (Use of Recycled Water for Other Purposes) of Title 22 further state the following:

> *"… except that for filtration being provided pursuant to Section 60301.320(a) coagulation need not be used as part of the treatment process provided that the filter effluent turbidity does not exceed 2 NTU, the turbidity of the influent to the filters is continuously measured, the influent turbidity does not exceed 5 NTU for more than 15 minutes and never exceeds 10 NTU, and that there is the capability to automatically activate chemical addition or divert the wastewater should the filter influent turbidity exceed 5 NTU for more than 15 minutes." Sections 60305 ("Use of Recycled Water for Impoundments") and 60306 (Use of Recycled Water for Cooling") of Title 22 do not contain this exception."*

While Section 60301.320 defines turbidity requirements only for two specific types of filtration, Title 22, Section 60320.5 ("Other Methods of Treatment") states the following:

> *"Methods of treatment other than those included in this chapter and their reliability features may be accepted if the applicant demonstrates to the satisfaction of the State Department of Health that the methods of treatment and reliability features will assure an equal degree of treatment and reliability."*

To achieve tertiary filtration at the WWTP, the City currently uses Hydrotech polyester media filters from Veolia Water/Kruger (previously U.S. Filter Kruger Products). In a letter dated October 2, 2003, DPH (then known as Department of Health Services) staff indicated the following acceptance of this filter as meeting Title 22 criteria:

> *"Based on a review of the materials submitted, the [DPH] conditionally accepts the use of the U.S. Filter Kruger Product's Hydrotech Filter as a filtration technology for use in compliance with [Title 22 requirements], subject to the following special provisions* [Listed are only provisions relevant to filtration design.]*:*
>
> 1. *Loading rates shall not exceed 6 gpm/ft^2.*
> 2. *Turbidity in the filtered water shall not exceed an average of 2 NTU within a 24-hour period, 5 NTU more than 5 percent of the time within a 24-hour period, and 10 NTU at any time.*
> 3. *Acceptance of this technology is contingent on it being complimented with a disinfection process which is compliant with Section 60301.230 (Title 22).*
> 4. *Acceptance is for the filter fabric specified as "PET, monofilament, 2:2 twill weave, 11 micron (+/-2.0) mesh opening, 523.2 (n/inch) mesh count, 60-micron thickness, weight rating of 1.48 oz./sq.yd* (sic) *with a stabilized finish" which was assessed in the report noted above. Other cloth materials and fabrication will require additional demonstration studies prior to individual acceptance by the* [DPH].

5. *Pretreatment processes should be designed and operated to ensure that the turbidity of the influent to the* [cloth-media discfilter] *does not exceed 10 NTU more than five-percent of the time within a 24-hour period and never exceeds 15 NTU."*

As a means to monitor filtration performance at the WWTP, Special Provision VI.C.4.b of the WDRs specifically limits effluent turbidity to the standard Title 22 filter media levels detailed above (i.e., 2, 5, and 10 NTU).

2.4.3 UV Disinfection Operational Requirements

Section 60301.230 of Title 22 describes as follows the disinfection facilities to be provided following tertiary treatment systems:

"(a) The filtered wastewater has been disinfected by either:

(1) A chlorine disinfection process following filtration that provides a CT (the product of total chlorine residual and modal contact time measured at the same point) value of not less than 450 milligram minutes per liter at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow; or

(2) A disinfection process that, when combined with the filtration process, has been demonstrated to inactivate and/or remove 99.999 percent of the 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."

This section of Title 22 also indicates that "disinfected tertiary recycled water" must meet specific levels for total coliform organisms in the disinfected effluent. The specified levels are equivalent to the effluent limitations for total coliform organisms detailed in Table 2-16.

In addition to the general Title 22 requirements for disinfection facilities, Special Provision VI.C.4.c of the WDRs includes the following operational requirements for the UV disinfection system at the WWTP [listed are only those requirements relevant to disinfection design.]:

"[The City] *shall operate the UV disinfection system to provide a minimum UV dose of 100 millijoules per square centimeter (mJ/cm2) at peak daily flow, unless otherwise approved by the [DPH].*

i. [The City] *shall provide continuous, reliable monitoring of flow, UV transmittance, UV power, and turbidity.*

…

iii. The UV transmittance (at 254 nanometers) in the wastewater exiting the UV disinfection system shall not fall below 55 percent of maximum at any time.

2.4.4 Biosolids Land Application Requirements

Regulatory requirements for biosolids include numerous operational requirements such as containment, runoff control, crop and grazing restrictions. The following requirements, applicable to the land application of biosoilds on the City's property, are addressed in this WWTP Facilities Master Plan:

- Pathogen Reduction Requirements
- Vector Attraction Requirements
- Metals Loading Limits
- Nitrogen Loading Limits

2.4.4.1 Pathogen Reduction Requirements

Specific requirements are included in 40 CFR 503 for reducing pathogens in biosolids used for land application. To meet Class B pathogen reduction requirements, either regular testing for fecal coliform must be completed or a specific "Process to Significantly Reduce Pathogens" (PSRP) that has been approved by the USEPA, or equivalent, must be used. The City has historically maintained an MCRT in the oxidation ditches greater than 30 days to ensure adequate stabilization of the biosolids prior to discharge to the biosolids storage basins. This process for meeting Class B standards is not a USEPA-approved PSRP nor has it been defined as a "Process Equivalent to PSRP." Therefore, the City monitors the biosolids for pathogens to demonstrate that the biosolids meet the Class B pathogen reduction requirements.

To date, the City has been able to demonstrate reliable compliance with the Class B pathogen requirements through the pathogen testing. However, a reduction in the MCRT significantly below a 25-day period could result in non-compliance unless an additional solids treatment process that is designed to meet Class B requirements is provided. Aerobic digestion of waste activated sludge (WAS) from the oxidation ditches, for example, could provide the additional solids treatment by keeping solids under aeration for a longer period of time (typically 60 days) prior to sending the digested sludge to the biosolids dewatering facilities.

2.4.4.2 Vector Attraction Reduction Requirements

Specific requirements are included in 40 CFR 503 for reducing the attraction of vectors (disease-carrying pests) in biosolids used for land application. Ten alternatives are presented in 40 CFR 503 for meeting these requirements. Since the City constructed the additional biosolids dewatering facilities, all biosolids applied to the City-owned property have been tilled into the applied land within six hours following land application, which satisfies Treatment Option 10 of 40 CFR 503. It has been assumed for purposes of this analysis that the City will continue this practice with all land-applied biosolids.

2.4.4.3 Metals Loading Limits

In accordance with 40 CFR 503, the metals concentrations in Class B biosolids must fall below the "ceiling concentration", and the total metal loading to a land application site resulting from biosolids cannot exceed cumulative pollutant loading rates (CPLRs). The 40 CFR 503 requirement states that CPLR limits are not to be exceeded for all biosolids applications that have occurred since July 20, 1993, and the CPLR limits are to be used to determine the maximum amount of biosolids that can be applied to a given area. If the CPLR limits have been reached for a specific area, the applier must not add additional biosolids to the site.

The 40 CFR 503 ceiling concentration limits and CPLR limits are also included in the City's WDRs. The WDRs further clarify that the 40 CFR 503 CPLRs need to be adjusted for background concentrations to establish a Background-Adjusted Cumulative Loading Rate (BC), which then becomes the regulatory limit for biosolids application. The BC is calculated as follows:

$$BC = CPLR - 1.8*(BS)$$

where:

BC = Background-Adjusted Cumulative Loading Rate (lb/ac)

CPLR = CFR 503. Cumulative Pollutant Loading Rate (lb/ac)

BS = Site Background Soil concentration (mg/kg)

However, the WDRs do not include details on how the site background soil concentrations are to be determined. Moreover, the loadings to date, based on the City's calculations of CPLRs since 2000, are well below the CPLR limits. Therefore, for purposes of the analysis presented in this chapter, the background concentrations are assumed to be negligible.

These specific metals limits are summarized in Table 2-17, which also includes historic and recent biosolids metals data collected by the City. Comparing the metals data to the limits indicates that metals concentrations are well below the ceiling concentrations and cumulative loadings well below the CPLRs.

Table 2-17. Comparison of Biosolids/Land Application Metals Limitations and Existing WWTP Biosolids/Land Application Conditions

Constituent	Ceiling Concentration, mg/kg	Recent Biosolids Quality Data,[a] mg/kg	Median of Historic (2006-2008) Biosolids Quality Data,[b] mg/kg	CPLR lb/ac	Max. Field Cumulative Loading to Date,[c] lb/ac	Limiting Field Number[c]
Arsenic	75	11	15	36	2.0	11
Cadmium	85	J 0.31	2.1	34	2.1	15
Copper	4,300	48	221	1,336	6.6	11
Lead	840	2.1	21	267	3.0	19
Mercury	57	0.30	1.6	15	0.3	15
Molybdenum[d]	75	2.7	15	16	0.3	11
Nickel	420	3.7	29	374	6.7	15
Selenium	100	J 2.3	11	89	0.3	15
Zinc	7,500	100	529	2,494	13.7	11

(a) Data from February 23, 2012 sample. Values preceded by a "J" are for detected but not quantified (DNQ) testing results, where the concentration was below the reporting level for the particular constituent. The estimated value is shown following the "J".

(b) The City only recently began operating the drying bed facilities, and prior to their use, metals data was collected only for liquid biosolids. This previous data may not be representative of current biosolids quality. However, given the limited data set for dewatered biosolids, the historic median for liquid biosolids, from a more robust data set, has been used for the current analysis.

(c) The "Maximum Field Cumulative Loading to Date" is the maximum of the individual total metals loadings for each field since 2000. The maximum loadings for each constituent correspond with the fields indicated by the "Limiting Field Number."

(d) The indicated limits for molybdenum are included in the City's WDRs but not in CFR 503.

2.4.4.4 Nitrogen Loading Limits

The rate of biosolids land application, in terms of nitrogen content, is required by 40 CFR 503 to be equal to or less than the agronomic rate for nitrogen. The WDRs also state that "*total nitrogen loading to any individual Reuse Area…shall not exceed the agronomic rate for plant available nitrogen (PAN) for the type of crop to be grown, as specified in the most recent edition of the Western Fertilizer Handbook.*" Both the 40 CFR 503 guidance and the WDRs require that the City consider the total applied nitrogen from the PAN associated with the biosolids, any nitrogen fertilizers that are applied, and all the nitrogen applied with the City's recycled water when determining compliance with the nitrogen loading requirements.

The USEPA provides the following equations for calculating the PAN associated with biosolids applications:

Current Year Biosolids PAN = $[NH_3\text{-}N]*Kv+[Org\text{-}N]*F_{0\text{-}1}+[NO_3\text{-}N]$,

Year 1 Previous Bioavailable PAN = $[Org\text{-}N]*F_{1\text{-}2}$

Year 2 Previous Bioavailable PAN = $[Org\text{-}N]*F_{2\text{-}3}$

Where:

$[NH_3\text{-}N]$ = *concentration of ammonia of the applied biosolids (mg/kg)*

Kv = ammonia volatilization factor, which is 0.5 for dewatered biosolids (applied in any manner)

[Org-N] = concentration of organic nitrogen (mg/kg)

$F_{0\text{-}1}$ = *Fraction of Org-N mineralized during the first year following application, which is 0.3 for aerobically digested biosolids*

$F_{1\text{-}2}$ = *Fraction of Org-N mineralized during the second year following application, which is 0.15 for aerobically digested biosolids*

$F_{2\text{-}3}$ = *Fraction of Org-N mineralized during the third year following application, which is 0.08 for aerobically digested biosolids*

$[NO_3\text{-}N]$ = *concentration of nitrate nitrogen (mg/kg)*

2.4.5 Groundwater Limitations

Groundwater regulations applied to the City's land application area are dictated by the State's Antidegradation policy, *Resolution 68-16* (State Water Resources Control Board, 1968). This policy requires the maintenance of the existing quality of a water and that any change in groundwater quality must:

- Be consistent with "maximum benefit to the people of the State".
- Not unreasonably affect beneficial uses.
- Not result in water quality that exceeds the applicable objectives or background water quality, whichever is greater.

Numeric and narrative water quality objectives are provided in the Sacramento-San Joaquin River Basin Plan (Basin Plan) to protect the beneficial uses of groundwater. A decision tree that shows how the Basin Plan is used by the Regional Board to regulate groundwater is provided on Figure 2-5.

As shown on the figure, the Basin Plan requires that the quality of the applied water that reaches (or has the potential to impact) groundwater will not result in an impact on the water quality found upstream of the discharge, or "background conditions." In addition, the Basin Plan requires that numeric and narrative water quality objectives applicable to the beneficial uses of groundwater represent the most stringent criteria that may be applicable to a given groundwater basin. If it is demonstrated that a given project does not cause a change in groundwater quality, then no additional controls are needed. However, if a change in water quality is expected, additional controls (referred to as Best Practicable Treatment or Controls, or BPTCs) are necessary to demonstrate that a project "will not unreasonably affect beneficial uses." Such controls may include, but are not necessarily limited to, lining storage ponds and major conveyances, application only at agronomic rates, and/or higher levels of wastewater/biosolids treatment. In addition, water quality based effluent limitations may be applied in a discharge permit to ensure groundwater protection.

Data collected by the City to date indicate that the nitrate levels measured in some of the City's onsite groundwater monitoring wells exceed both the water quality objective of 10 mg/L as N and the background concentration. Therefore, additional BPTCs (beyond what has historically been employed) may be needed.

Up until recently, the effluent total inorganic nitrogen concentrations have been less than the 10 mg/L as N human health criterion for nitrate. However, recent operational changes have resulted in effluent inorganic nitrogen concentrations greater than 20 mg/L as N. Therefore, recent increases in monitoring well nitrate levels could be a result of this change in operation. Nevertheless, part of the WWTP facilities upgrade project will result in improved nitrogen removal at the WWTP such that the final effluent total inorganic nitrogen levels are less than 10 mg/L as N. Once these improvements are in place, the liquid treatment train of the WWTP will provide an appropriate level of BPTC for nitrogen.

In addition, the City has historically applied biosolids and irrigation water nitrogen at theoretical agronomic rates, a practice that also provides an appropriate level of BPTC. Should ongoing groundwater monitoring indicate nitrate levels do not decrease once the WWTP upgrades are in place, the City may need to further evaluate appropriate nitrogen application rates for the City-owned fields given site-specific cropping, soils and yield information. Such an evaluation could result in an overall reduction in the land application area disposal capacity discussed in Chapter 5 (Biosolids Handling Facilities).

Finally, one well (WSM-2) appears to have the highest nitrate concentrations of all the onsite wells. Given its proximity to the City's biosolids storage lagoons, a leak in the biosolids storage lagoon liner may be impacting this well. Therefore, the City should drain, clean, and inspect both lagoons to ensure such an impact is not occurring.

2.5 RELIABILITY/REDUNDANCY REQUIREMENTS

A summary of the reliability and redundancy requirements applicable to the major project components discussed in this chapter are provided in Table 2-18.

Table 2-18. Recommended Reliability/Redundancy Criteria for Galt Wastewater Treatment Facilities

Process Area	Potential Consequence of Failure	Reliability/Redundancy Criteria	
		General Criteria	Mechanical/Electrical Equipment
Pumping Facilities			
Filter Feed Pump Station	Excessive flow directed to Storage Ponds and available storage capacity exceeded.	1. Total capacity to match total capacity of filtration and disinfection processes. 2. Firm capacity to match or exceed maximum day wet weather flow. 3. Flows not pumped overflow by gravity to Storage Reservoir.	• Standby power capacity to operate all non-redundant equipment
Irrigation Pumps	Inability to irrigate City fields.	1. Total installed capacity to meet or exceed peak irrigation demand. 2. Complete spare pump packages to be stocked.	• None
Auxiliary Basin Return Pumps	Inability to return flows to the treatment process, limiting equalization capacity.	1. Total capacity adequate to return one basin volume within maximum of 7 days. 2. Complete spare pump package to be stocked.	• Standby power capacity to operate all non-redundant equipment
Storage Ponds			
Flow Equalization	Flows to the secondary treatment processes that exceed the capacity of the secondary clarifiers, causing an exceedance of final effluent limitations for suspended solids and other parameters, or excessive loading of filters.	1. Provide for automatic diversions of all flow that exceeds a defined set point (set point to be varied depending on operating conditions). 2. Independent from other storage facilities. 3. All flows directed to this facility would be returned to the treatment process as soon as possible.	• Facilities to measure and control return flow to the headworks. • Standby power capacity to operate all non-redundant equipment
Secondary Effluent Long-Term Storage / Reuse	Flows to the filtration and disinfection systems that exceed the design capacity, potentially causing an exceedance of effluent criteria for turbidity and/or an exceedance of effluent limitations for coliforms.	1. Anticipated maximum cumulative diversions must be contained within storage ponds without relying on the return of flows to the treatment process. 2. Provide the flexibility to return flow to the headworks. 3. Acceptable diversions include: • Dry weather period diversion if one filtration or disinfection unit is out of service. • Emergency wet-season conditions (i.e. unplanned unit outage in the wet-season months). • Annual peak flow diversions associated with peak I&I conditions.	• Facilities to measure and control return flow to the headworks.
Final Effluent Emergency Storage	Discharge of improperly treated wastewater.	1. Provide for automatic diversions of flow if any continuously monitored effluent criteria are exceeded. 2. Gravity flow from a location downstream of the UV facilities, but upstream of the final outfall. 3. No planned return of flows to the treatment process, but include the flexibility to return flow to the headworks.	• Facilities to measure and control return flow to the headworks. • Diversion gates at final effluent structure have a back-up power source that is independent of the WWTP power supply.
Secondary Treatment			
Oxidation Ditches	BOD, ammonia (and possibly total nitrogen) waste discharge violations	1. With all treatment trains in service, capable of meeting effluent BOD and ammonia criteria at design maximum month load and minimum-day winter temperatures. 2. With all treatment trains in service, capable of meeting effluent BOD and ammonia criteria at design maximum day load and minimum-month winter temperatures. 3. With one unit out of service the system must satisfy effluent BOD, ammonia, and nitrate criteria at the annual average conditions. If the MCRT requirements needed to meet Class B biosolids standards cannot be satisfied with one unit out of service, the City would temporarily haul solids to a landfill for disposal. 4. To reduce the likelihood of needing to take a ditch out of service, consider retrofitting with aeration equipment that can be removed and/or maintained while the treatment unit remains in service.	• For blower-based aeration systems, standby blower equipment to meet peak aeration requirements with the largest unit out of service. • Standby power capacity to operate all non-redundant equipment.
Secondary Clarifiers	Suspended solid waste discharge violations	1. With all units in service and at the equalized peak wet-weather design flow for the secondary process (plus 50% RAS rate), the maximum allowable solids loading rate is 26 lb/ft^2/day based on maximum design MLSS concentrations and an SVI of 250. 2. With one unit out of service and at the WWTP influent design peak hour dry-weather flow without equalization (plus 50% RAS rate), the maximum solids loading rate is 30 lb/ft^2/day based on maximum design MLSS concentrations and an SVI of 200. 3. The design flow for the clarifiers can account for diversions to equalization storage in accordance with the design criteria listed above.	• Standby power capacity to operate all equipment.

Table 2-18. Recommended Reliability/Redundancy Criteria for Galt Wastewater Treatment Facilities

Process Area	Potential Consequence of Failure	Reliability/Redundancy Criteria	
		General Criteria	Mechanical/Electrical Equipment
RAS/WAS Pumping	Suspended solid waste discharge violations	1. Each clarifier fitted with an independent RAS/WAS variable speed pumping system that can deliver up to 1.94 mgd of flow.	• Each clarifier pumping system capable of handling maximum design RAS and WAS flow with largest unit out of service. • Standby power capacity to operate all non-redundant equipment
Tertiary Treatment			
Chemical Addition	Turbidity, suspended solids or metals waste discharge violations	1. Ability to feed coagulant either to secondary clarifiers or just prior to filters.	• Polymer/coagulant dosing equipment capable of providing anticipated dosage range at peak secondary treatment design flows with the largest unit out of service. • Standby power capacity to operate all non-redundant equipment
Filtration	Turbidity, suspended solids or metals waste discharge violations	1. With all units in service, capable of meeting CDPH-approved Title 22 loading rates at the filtration/disinfection system peak wet-weather design flow. 2. Filtration/disinfection system peak wet-weather design flow can account for diversions to long-term storage in accordance with the design criteria for long-term storage of secondary treated waters.	• Standby power capacity to operate all non-redundant equipment.
UV Disinfection			
Disinfection	Coliform waste discharge violations	1. With all units in service, capable of meeting CDPH-approved Title 22 UV disinfection criteria at the filtration/disinfection system peak wet-weather design flow. 2. Filtration/disinfection system peak wet-weather design flow can account for diversions to long-term storage in accordance with the design criteria for long-term storage of secondary treated waters.	• Standby power capacity to operate all non-redundant equipment.
Solids Handling			
Aerobic Digestion (if installed in future)	Insufficient digestion time to produce Class B solids	1. Minimum of two units. 2. With all digesters in service and one oxidation ditch out of service, capable of meeting EPA Part 503 Biosolids Rule criteria for Class B biosolids at design maximum month load. 3. Either capable of meeting EPA Part 503 criteria for Class B biosolids at the average loading conditions and average temperature.	• Standby aeration equipment to meet peak aeration requirements with largest unit out of service. • Sludge withdrawal pumping system capable of conveying the peak WAS loading rate with largest unit out of service. • Supernatant pumping system capable of conveying the peak supernatant flow with largest unit out of service. • Standby power capacity to operate all non-redundant equipment.
Dewatering System	Exceedance of un-dewatered biosolids storage capacity; resulting solids overflow to wastewater treatment process could impact sludge settling and further process upset	1. Must be capable of handling average biosolids production rate with all units in service. 2. Provide supplemental mechanical dewatering equipment designed for 6 hours/day, five days/week operation, that can temporarily be operated for more hours per day, more days per week, or possibly higher loading rates, to compensate for lost dewatering capacity if one dewatering bed is temporarily out of service. (Mechanical dewatering units operated at higher loading rates would likely result in a temporary increase in the moisture content of dewatered solids.) 3. At the Current Permitted Design Condition, a redundant supplemental mechanical dewatering unit is not needed provided that at least two mechanical dewatering units are installed. (Each unit would have at least one-half the necessary supplemental capacity.) 4. At the Future Permitted and Ultimate Buildout Design Conditions, a redundant mechanical dewatering is required.	• Standby power capacity to operate all non-redundant equipment.

Table 2-18. Recommended Reliability/Redundancy Criteria for Galt Wastewater Treatment Facilities

Process Area	Potential Consequence of Failure	Reliability/Redundancy Criteria	
		General Criteria	Mechanical/Electrical Equipment
Biosolids Storage/Drying Area	Failure to meet specified waste discharge requirements for biosolids storage design and operational practices	1. No specific reliability/redundancy criteria. System must be capable of storing at least 6 months of dewatered biosolids. 2. The biosolids storage/drying area is not expected to require substantive maintenance over the life of the WWTP facilities upgrade project. Should maintenance, such as re-paving, be necessary, the City could temporarily retain the services of a third-party contractor to haul the biosolids to an offsite location for disposal or develop a temporary onsite storage location.	N/A
Biosolids Storage Lagoons	Solids overflow to wastewater treatment process could impact sludge settling	1. The number and size of storage lagoons should allow for one lagoon to be taken out of service during the summer for maintenance and/or repair. 2. Ensure adequate freeboard by running mechanical dewatering equipment, as needed, for more hours per day, or more days per week.	N/A
Biosolids Storage Lagoon Mixers	Inadequately mixed solids, reduced dewatering efficiency / increase cost	1. Redundant unit(s) not required. Stock spare parts sufficient to avoid prolonged outages.	N/A
Land Application Area	Insufficient area for land application, accumulation of biosolids at WWTP site, or excess application potentially violating nitrogen or metals loading requirements	1. With all fields in service, able to apply total annual mass of biosolids generated in the design year. 2. If a land application field is out of service for a significant period of time, temporarily retain the services of a contractor to haul a portion of the generated biosolids to an offsite location for disposal. 3. Long-term, evaluate opportunities to acquire additional land area relative to potential cost of off-hauling.	N/A

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Figure 2-1. City of Galt WWTP Flow Projection

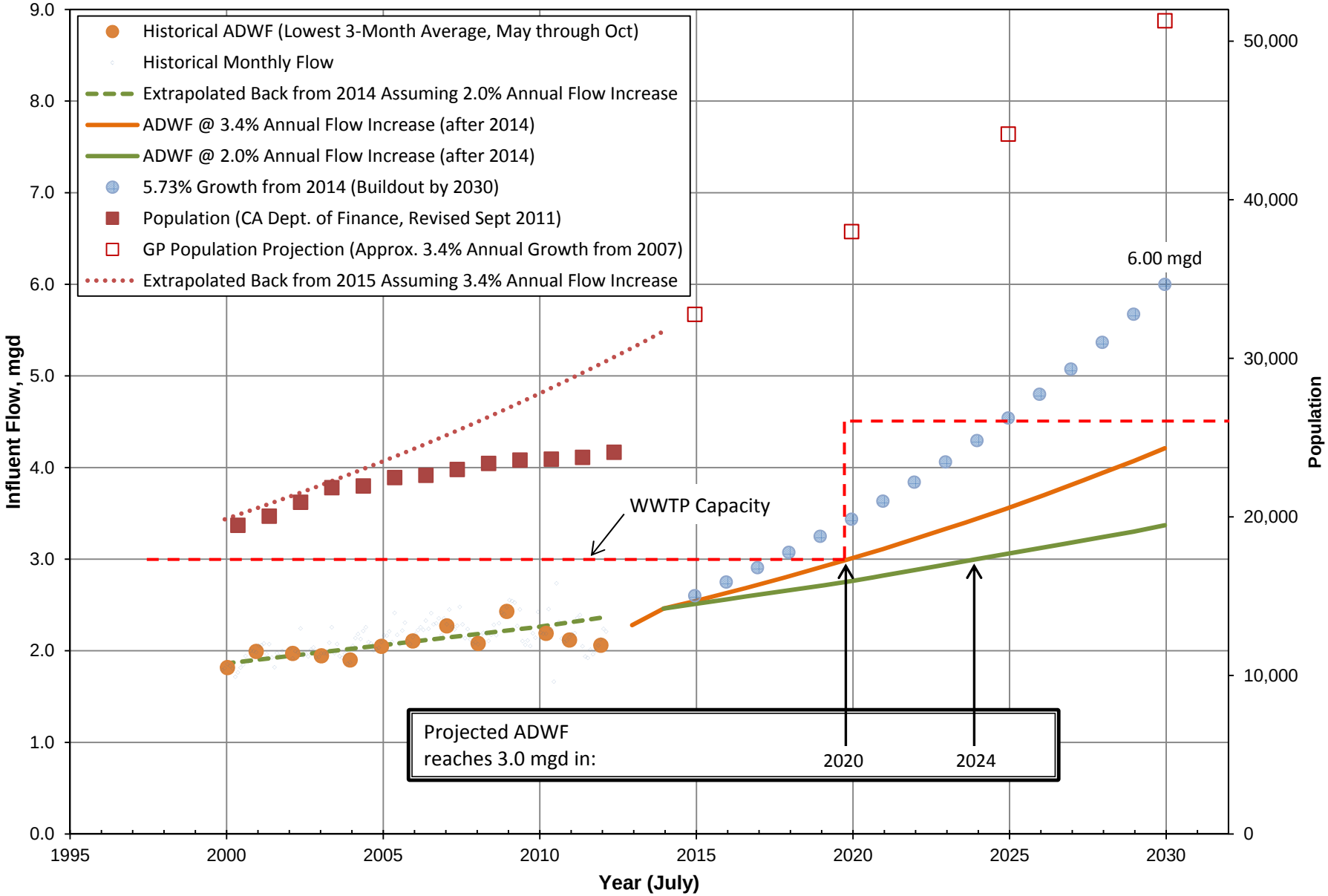


Figure 2-2. Annual BOD/TSS Concentrations, 2000–2011

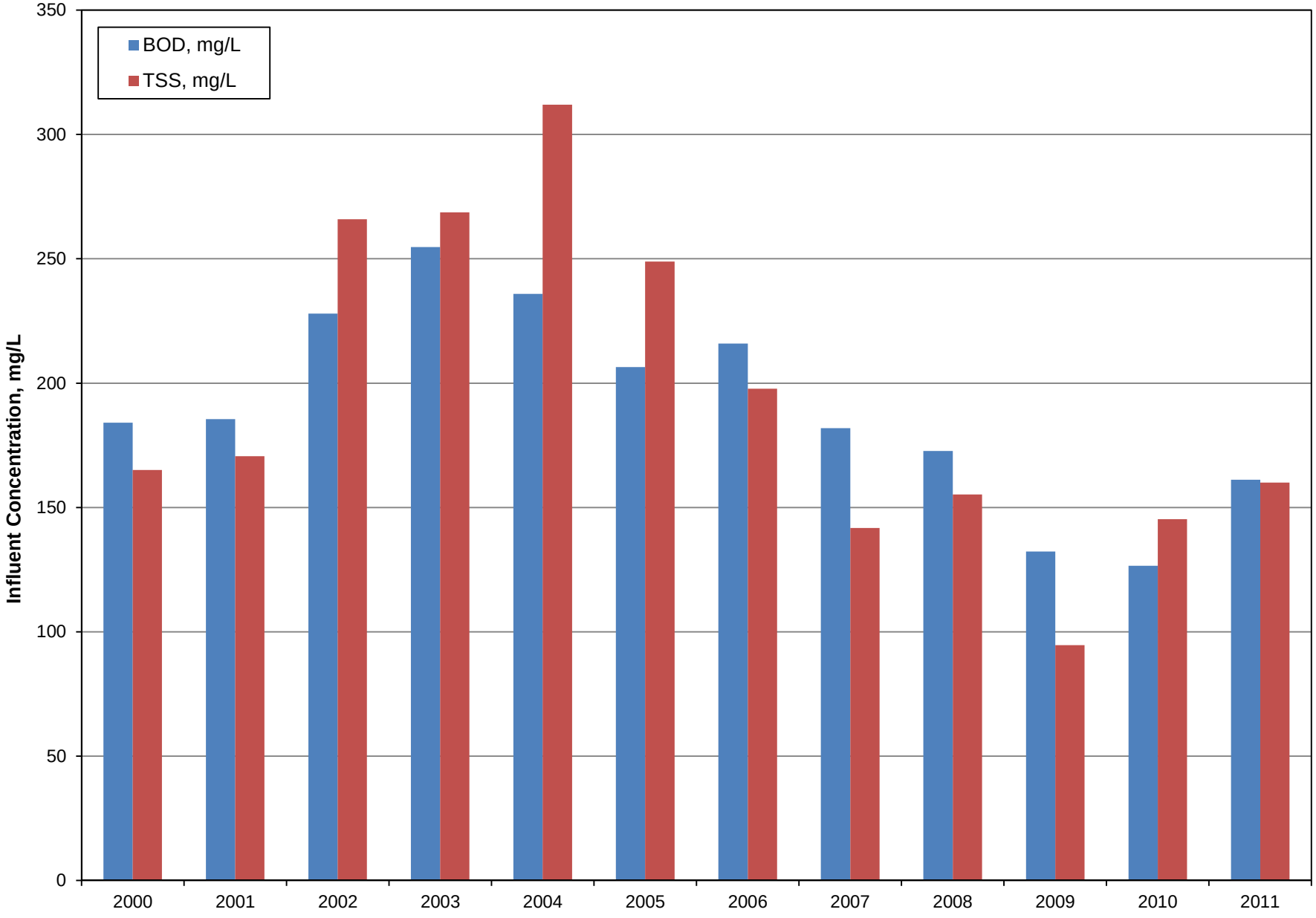


Figure 2-3. Monthly Influent BOD Concentrations, 2000–2011

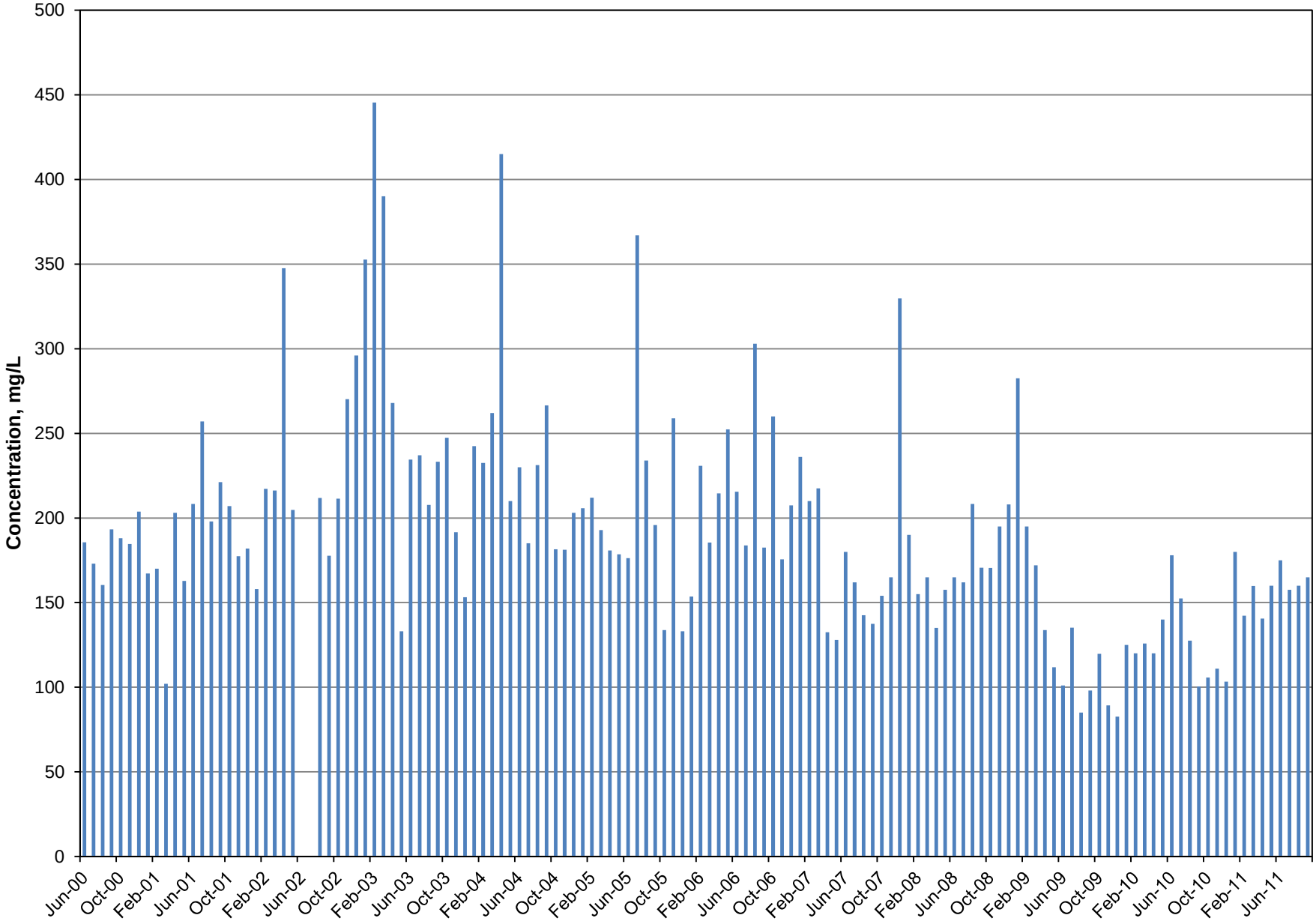


Figure 2-4. Monthly Influent TSS Concentrations, 2000–2011

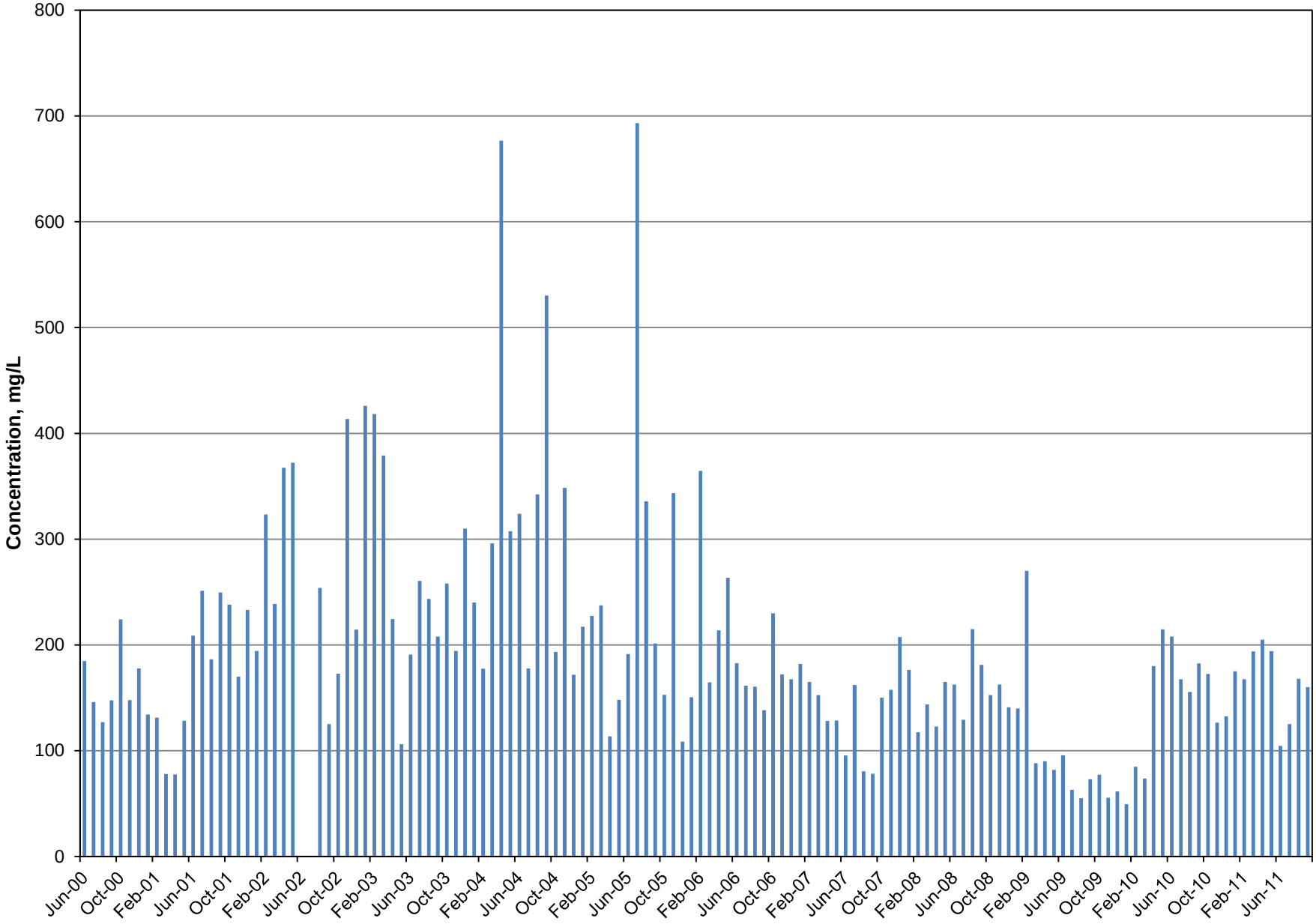
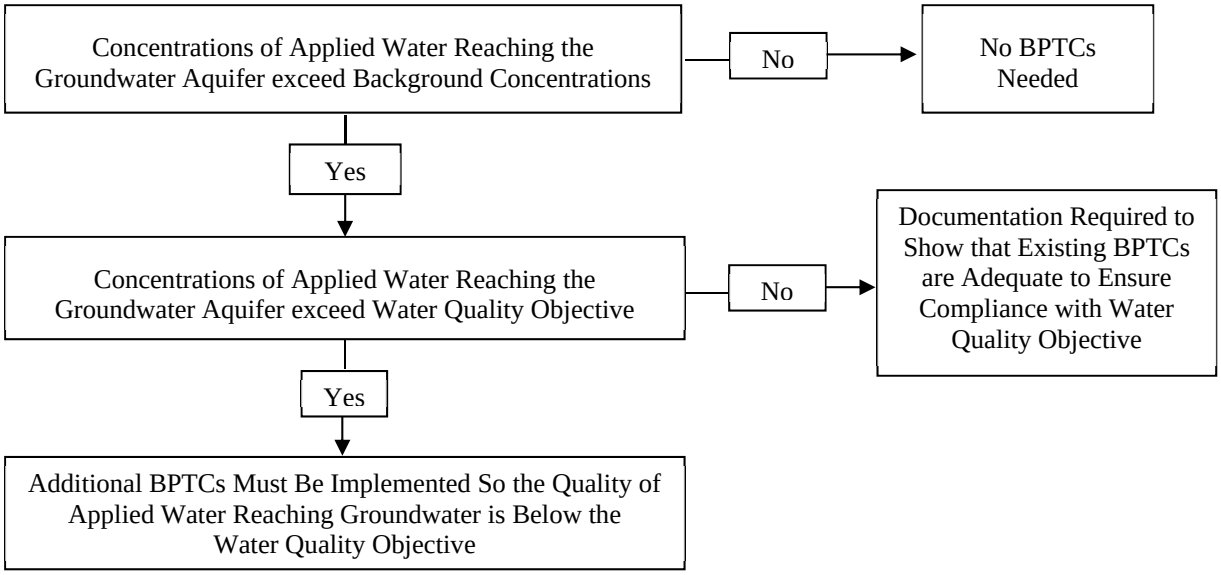


Figure 2-5. Basin Plan Groundwater Protection Requirements



CHAPTER 3
Existing Treatment Plant Performance and Condition

This chapter summarizes the results of an evaluation of the existing facilities at the City of Galt's (City) Wastewater Treatment Plant (WWTP). This evaluation includes an assessment of the function, performance, and condition of each major unit process. The major topics discussed in this chapter are as follows:

- WWTP Construction History
- Liquid Treatment Processes
- Solids Treatment Processes
- Storage and Irrigation Systems
- Plant Water Systems
- Electrical and Control Systems
- Buildings and Storage Facilities
- Other Ancillary Facilities

This assessment presented herein was based on data and information presented in the *Phase 1 Feasibility Study* (West Yost, January 2005) and in the *NPDES Permit Compliance Action Plan* (West Yost, December 2005); available record drawings of the existing facilities; water quality data collected by the City; and information gathered from City staff during site visits and interviews that occurred in March, April, May and June of 2012. A more detailed discussion of alternatives for addressing existing and/or future capacity and expansion needs is provided in later chapters of this Facilities Master Plan.

3.1 WWTP CONSTRUCTION HISTORY

The City constructed a WWTP at the existing site in 1981. The City's population at the time was 5,514. The treatment system was designed to accommodate the flows and loads associated with a 0.84 million gallons per day (mgd) Average Dry Weather Flow (ADWF) condition. The facilities that were constructed consisted of a headworks, fitted with a comminutor and manually-cleaned bar screen, followed by a pond-based treatment system. The pond treatment system included one aerated lagoon followed by four oxidation ponds operated in series. Following the final pond, effluent was discharged through a chlorine contact chamber prior to entering the Storage Reservoir. From the Storage Reservoir the treated wastewater could either be discharged to Skunk Creek via a submerged outfall from the reservoir, or it could be used for irrigation (via Irrigation Pump Station #1) on the agricultural fields located south of the treatment system. Skunk Creek joins Laguna Creek several thousand feet downstream of the Storage Reservoir outfall.

In 1990, the City constructed the majority of the existing treatment facilities, which were designed to accommodate the flows and loads associated with a 3.0 mgd ADWF condition (*i.e.*, the Current Permitted Design Condition defined in Chapter 2 (Basis of Design)). The facilities that were constructed included:

- a new headworks, fitted with a mechanical climber screen and a manually-cleaned bar screen;
- two parallel secondary treatment trains, which consist of an oxidation ditch and clarifier;
- a return activated sludge/waste activated sludge (RAS/WAS) pumping facility;
- two sludge lagoons fitted with a pump to return supernatant to the Headworks; and
- a new chlorination/dechlorination facility.

Treated effluent was still discharged to the Storage Reservoir from where it could either be discharged to Skunk Creek via a new cascade aerator outfall, or it could be used for irrigation on the agricultural fields located south of the treatment system. The four oxidation ponds (Ponds 1 through 4) were converted to effluent storage and a pipeline connecting the Irrigation Pump Station #1 to Pond 1 was installed. In 2003, the aerated lagoon was converted to the Auxiliary Basin for raw wastewater storage and facilities were constructed to return flows from the Auxiliary Basin to the WWTP. Minor upgrades/improvements following the 2003 improvements have included:

- In 2003, during the construction of the Auxiliary Basin Pump Station to return of flow from the Auxiliary Basin to the Headworks, it is believed that a connection between the sludge lagoon decant (*i.e.*, supernatant) return line and the 24-inch influent bypass pipeline (the gravity pipeline between the Headworks and the Auxiliary Basin) was also constructed. This connection is still in service, and is illustrated in Figure 3-1.
- Irrigation Pump Station #2 was constructed and irrigation was expanded to incorporate the fields north of the WWTP (construction date unknown). Irrigation pump stations are illustrated in Figure 3-3.
- In 2005, the capacity of the Storage Reservoir was increased, Return Pump Station #2 was constructed, and improvements were made to the irrigation fields north of the WWTP.
- In 2007, the Grit Removal Facility was constructed downstream of the Headworks and the Auxiliary Basin was lined. In addition, an effluent flow meter was installed downstream of the Storage Reservoir outfall to Skunk Creek. An influent flow meter was also installed between the Headworks and Grit Removal facilities, as illustrated in Figure 3-1.

- In 2009, a pipeline was constructed that allowed for the effluent being discharged to Skunk Creek to bypass the Storage Reservoir. This project also included the construction of the Effluent Diversion Structure. Both facilities are illustrated in Figure 3-1.

Finally, in 2011 the City completed construction of the tertiary treatment and biosolids handling facilities, which were also designed to accommodate the flows and loads associated the Current Permitted Design Condition. This project included construction of a Filter Feed Pump Station, Cloth Media Disc Filters, an Ultraviolet (UV) Disinfection System and Biosolids Dewatering Facilities.

3.2 LIQUID TREATMENT PROCESSES

This section presents an evaluation of the following existing liquid treatment unit processes at the City's WWTP:

- Headworks/Screening
- Grit Removal
- Oxidation Ditches
- Secondary Clarifiers
- Polymer Storage and Dosing Facility
- Filter Feed and Utility Water Pump Station
- Cloth Media Disc Filters
- UV Disinfection
- Tertiary Plant Drain Pump Station
- Effluent Diversion Structure
- Effluent Flow Monitoring
- Chlorine Contact Chamber (Abandoned)

A schematic showing these liquid treatment unit processes is provided in Figure 3-1, and the major design parameters for each unit process are listed in Appendix D Table D-1.

The discussion below presents the following information for each process: a description of the process/facility function; an evaluation of rated capacity (or rating) and recent performance with respect to meeting the Current Permitted Design Conditions; and a summary of recommended improvements based on the conditions assessment conducted with City staff input. A detailed discussion of the condition assessment findings is presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.1 Headworks and Screening

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Headworks and Screening Facility identified through the conditions assessment.

3.2.1.1 Process Overview

The Headworks and Screening Facility has not been upgraded or significantly modified since it was constructed in 1990, although the Grit Removal Facility and influent flow meter were appended to the original Headworks structure.

Wastewater from the City enters the Headworks and Screening Facility via a 24-inch force main from the City's Live Oak sewage pump station. In addition, the California Department of Corrections and Rehabilitation (CDCR) Basic Correctional Officer Academy, which is located south of the WWTP on Twin Cities Road, discharges wastewater into the 24-inch force main before it reaches the WWTP. Other flows that periodically enter the Headworks include decant water (*i.e.*, supernatant) from the Sludge Lagoons pumped via the lagoon decant pump; and biosolids dewatering filtrate, Storage Reservoir contents, and drainage/storm water from the solids handling areas pumped via the Filtrate Pump Station. Wastewater entering the Headworks and Screening Facility may also be diverted to the Auxiliary Basin via a 24-inch bypass line, and water diverted to the basin may be returned to the Headworks and Screening Facility using the influent force main via the Auxiliary Basin Pump Station.

The Headworks and Screening Facility has three channels. The west, and primary, channel contains a mechanical climber screen with an automated rake and screenings auger, which is typically used to treat all of the flow entering the WWTP. The middle channel was designed to accommodate a future mechanical climber screen and is currently not in use. Flows are excluded from this channel with a removable stop gate. The east channel contains a manually cleaned bar screen and is also fitted with stop gates that must be manually removed. The east channel allows for the bypass of the mechanical climber screen in the event of power or mechanical failure.

Screened wastewater leaving the Headworks and Screening Facility flows by gravity through a magnetic flow meter to the Grit Removal facility. Screenings are deposited via a chute into a dumpster at grade, and are picked up every Thursday.

3.2.1.2 Rating and Performance

The mechanical climber screen is capable of treating flows up to 18 mgd (James M. Montgomery Consulting Engineers, 1990), which is more than adequate to treat the 9.0 mgd PWWF associated with the Current Permitted Design Condition. The treatment capacity of the manually cleaned bar screen is not defined. However, the bars on the manually cleaned screen are spaced 2.5 inches apart. Therefore, the screen is unlikely to cause hydraulic constraints at the design PWWF condition. If the mechanical climber screen is out of service, flow is diverted through the manually cleaned bar screen by manually removing the stop gate to the east channel. Should the mechanical climber screen fail during hours when the WWTP is not staffed, the water level in the Headworks structure would rise, triggering the high flow alarm. This alarm will

automatically open the gate to the 24-inch bypass pipeline thereby directing flows to the Auxiliary Basin. The available storage capacity in the Auxiliary Basin is approximately 5.8 MG.

The mechanical climber screen rake operates on an adjustable timer with one cycle every 15 minutes. Typically performance is adequate, and rags, plastics and other debris can be observed in the screenings. Staff has reported that some plastics and rags can pass through the screen openings and reach downstream processes (*e.g.*, the tertiary disc filters). The accumulation of small plastics in the tertiary filter backwash troughs has increased the maintenance requirements for the tertiary filters, namely for additional and more time-consuming cleaning procedures. On windy days, debris and water tend to get sprayed around in the area of the screen and dumpster, and can create a mess.

The magnetic flow meter is reliable; annual calibrations have not resulted in significant fluctuations in measured flows since the instrument was installed in 2007.

A composite sampler located at grade has recently been retrofitted to collect flow-paced samples. The sampler's 3/8-inch inlet line was also recently replaced, and the location of the sample line has been modified to minimize influences from WWTP sidestreams entering the Headworks and Screening Facility. Prior to these improvements, the screen on the sampler's inlet line had not been adequately maintained and is believed to have contributed to erroneous influent constituent measurements.

3.2.1.3 Conditions Assessment Summary

The following recommended improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Replace the existing mechanical climber screen with a new screen with ¼-inch openings[1], level-based rake controls, and supervisory control and data acquisition (SCADA) monitoring and control capabilities.
- Provide a redundant mechanical climber screen and a single new screenings washer and compactor capable of handling both screens.
- Provide automatically actuated control gates for screening channels.
- Replace the bar screen control panel and locate it at grade.
- Provide an alarm to indicate auger failure.
- Secure grating to address safety concerns.

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

[1] The opening size for a new screen will be influenced by the needs of downstream processes, which are discussed in later chapters of this Facilities Master Plan.

3.2.2 Grit Removal

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Grit Removal Facility identified through the conditions assessment.

3.2.2.1 Process Overview

The Grit Removal Facility has not been modified since it was constructed in 2007. This facility receives screened wastewater from the Headworks and Screening Facility. Wastewater enters the vortex grit chamber tangentially, where a decrease in fluid velocity causes grit and other heavier solids to settle into a grit tank. An impeller above the grit tank acts on lighter solids to cause an upward spiral out of the grit chamber. De-gritted wastewater leaves the grit chamber and flows by gravity to the Headworks Splitter Box.

Grit slurry in the grit tank is periodically drained by gravity to a grit classifier, where it is dewatered and discharged to a dumpster. A W3 utility water line was hot-tapped into the grit slurry line feeding the classifier to assist with cleaning and this water is used to agitate or scour the grit tank during grit slurry draining operations and help loosen settled grit. A motorized plug valve opens to allow agitated grit slurry to drain to the grit classifier. A screw conveyor in the grit classifier dewaters and discharges grit into a dumpster for removal.

3.2.2.2 Rating and Performance

The Grit Removal Facility is capable of treating flows up to 10 mgd (Boyle Engineering Corporation, 2007), which is adequate to handle the 9.0 mgd PWWF at the Current Permitted Design Condition. The WWTP is not equipped with a back-up (or redundant) grit removal system. Therefore, if the Grit Chamber is offline for maintenance, grit entering the WWTP will flow into the Oxidation Ditches, where it will fall out and accumulate. Prior to the installation of the Grit Chamber in 2007, this was the typical operating mode. Although not ideal, the treatment system should be able to operate in this mode for an intermediate period of time. However, the Oxidation Ditches may need to be cleaned after such occurrences. The grit facility was designed to be expanded to the west in a mirror image of the existing facilities. Such an expansion should increase process capacity to 20 mgd.

The Grit Chamber drive motor runs continuously. Grit slurry removal occurs periodically based on an adjustable timer, operating for 15 minutes every 2-3 hours. The Grit Removal Facility effectively removes grit that is rice grain size and larger; approximately 1-2 dumpsters of grit are removed weekly. Neither of the Oxidation Ditches have been taken out of service for inspection or maintenance since the Grit Removal Facility was installed, so it is not known how much grit may still be reaching the Oxidation Ditches. Under ideal circumstances, oxidation ditches should normally be taken offline and inspected once per year.

Staff has also reported that agitation and scouring at the bottom of the grit tank is inadequate and does not effectively suspend grit for removal, and settled grit tends to compact. According to City staff, the original design was to include four (4) scouring nozzles along a ring pipe, but only a single, open-ended pipe was installed inside the grit tank. Weekly maintenance of the grit tank drain line and plug valve is necessary to avoid excessive grit compaction. The grit classifier and

screw conveyor effectively dewater grit as long as weekly maintenance of the grit tank drain line and plug valve is performed.

3.2.2.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Provide two easily accessible cleanouts for the grit tank drain line, one in each direction
- Modify grit chamber agitation piping to improve effectiveness
- Remove and replace damaged grit chamber lining
- Miscellaneous coating touchup

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.3 Oxidation Ditches

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Oxidation Ditches identified through the conditions assessment.

3.2.3.1 Process Overview

The Oxidation Ditches have not been modified since they were constructed in 1990.

Screened and de-gritted wastewater from the Grit Removal facility enters the Oxidation Ditches via the Headworks Splitter Box, which splits the flow evenly among the two ditches. RAS is combined with the wastewater entering the Oxidation Ditches upstream of the Headworks Splitter Box.

Wastewater enters each ditch near one of two surface aerators. Partially submerged rotating paddles on the aerators simultaneously impart oxygen and rotation to the mixed liquor in the ditch, causing the oxygenated mixed liquor to follow a serpentine path through the ditch. The Oxidation Ditch aerators are capable of running at two speeds (high and low), and are ordinarily run continuously at high speed. Aerobic and anaerobic microbial populations reduce the biochemical oxygen demand (BOD) and provide both nitrification (*i.e.* ammonia removal) and denitrification (*i.e.* nitrate removal) of the wastewater. W3 utility water sprayers are installed upstream of the aerators to control foam. Mixed liquor leaving the Oxidation Ditches flows by gravity to the Secondary Splitter Box before reaching the Secondary Clarifiers.

Over the last two years, the average mixed liquor suspended solids (MLSS) concentration in the Oxidation Ditches has been approximately 2,900 milligrams per liter (mg/L); and concentrations have ranged from approximately 900 to 5,400 mg/L. Typical sludge volume index (SVI) values have ranged from approximately 80 to 400 mL/g, with an average of 190. Normal design values

for MLSS concentration in extended aeration oxidation ditches range from 1,500 to 5,000 mg/L (Metcalf & Eddy, Inc., 1991). Normal SVI values for extended aeration WWTPs are on the order of 50 to 150 mL/g.

3.2.3.2 Rating and Performance

The Oxidation Ditches were designed to accommodate the BOD loads anticipated at the Current Permitted Design Condition. However, at the time of construction, the City was not required to meet effluent ammonia or nitrate limitations. Therefore, the Oxidation Ditch design did not include features needed to provide reliable nitrification/denitrification. Nevertheless, oxidation ditches by their nature operate with high mean cell residence times (MCRT) and a significant amount of mixed liquor recycle, and they typically contain both aerobic and anoxic regions. Therefore, oxidation ditches typically provide some level of both nitrification and denitrification – even if they were not designed to do so. The City's effluent data confirms that the Oxidation Ditches at the WWTP currently provide some nitrification and denitrification.

An initial capacity evaluation presented in the Phase 1 Feasibility Study (West Yost, January 2005) concluded that the Oxidation Ditches can only reliably nitrify and denitrify the wastewater up to a loading condition associated with an ADWF of approximately 2.8 mgd. Dissolved oxygen (DO) is measured in both ditches at a location approximately 1.5 feet below water surface, just upstream of the mixed liquor effluent weir on the outside track of the ditch. The ability to maintain adequate DO concentrations is one aspect of process capacity. Historical data indicates that DO concentration can decrease significantly during high loading periods, apparently below 1 mg/L for several hours each day2. Recent data is subject to further evaluation as part of this Facilities Master Plan; however, low DO levels tend to support the suspicion that continuing to adequately nitrify will be difficult as loads increase. City staff has also reported inadequate nitrification when large loads (i.e. return from the Auxiliary Basin or solids overflow from the Sludge Lagoons) are sent to the WWTP headworks. This information is evidence that an upgrade and/or expansion of the Oxidation Ditches (or a reduction in the biological loading rate per gallon) is needed to ensure reliable compliance with the City's ammonia and nitrate limitations in the current permit, especially as flows increase. Process modeling has been performed, as described in later chapters of this Facilities Master Plan, to further assess the treatment capacity.

The WWTP should also be designed such that one Oxidation Ditch can be taken offline during typical dry weather conditions for maintenance, as often as annually, while still meeting the treatment requirements of the WWTP. The ditches were last drained and cleaned prior to the installation of the Grit Removal Facility. Although it is expected that the Grit Removal Facility will significantly reduce the potential for grit accumulations in the ditches, the facility will not completely eliminate grit accumulation. Consequently, periodic removal of grit accumulations from the floor of the ditches will still be required, and the design of the facilities should be adequate to enable this (as well as other maintenance activities) to happen. Given the information presented above, effluent limitations for surface water discharge are not likely to be met with one of the ditches offline.

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[2] During the two-week sampling period conducted in conjunction with this Facilities Planning effort, the DO probes were discovered to be improperly calibrated and were recording DO levels lower than actual values (by approximately 1.5 mg/L during peak DO demands). This calibration issue has since been remedied.

As an alternative to surface water discharge, effluent that has not been adequately nitrified/denitrified may be discharged during emergencies to the Storage Reservoir and used for irrigation as long as the final effluent quality meets the BOD and TSS limitations required for land disposal (30 and 30 mg/L, respectively, on an average monthly basis).

The W3 sprayers are moderately effective at controlling foam. However, the foam can get 2-3 inch thick, and extend roughly 12 feet towards the mixed liquor effluent weirs. Therefore, additional foam control is needed.

Additional evaluation of the existing Oxidation Ditch treatment capacity and alternatives for expansion/upgrades is provided in later chapters of this Facilities Master Plan. The evaluation takes into account the need to treat anticipated loads from return flows from the WWTP's solids handling systems, the provisions for increased monitoring and control of the ditches identified by City staff, and the other performance issues and considerations discussed above.

3.2.3.3 Conditions Assessment Summary

The following recommended improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Implement SCADA controls to automatically restart aerator motors in the event of a power outage
- Address foam control and other minor issues as part of the overall secondary process upgrade.

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.4 Secondary Clarifiers

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Secondary Clarifiers identified through the conditions assessment.

3.2.4.1 Process Overview

The Secondary Clarifiers have not been modified since they were constructed in 1990.

Mixed liquor leaving the Oxidation Ditches enters each Secondary Clarifier after being divided equally at the Secondary Splitter Box. Inner and outer influent baffles located in the middle of the clarifier reduce the velocity of influent mixed liquor to promote settling. Flocs in the mixed liquor settle in the clarifier tank and are collected at the bottom of the tank using plows attached to the mechanical clarifier rake arm that push the solids toward separate RAS and WAS hoppers located in the center of each clarifier. W3 water sprayers are located along the underside of the clarifier access bridge and are used continuously to control foam and floating material buildup at the clarifier water surface.

RAS and WAS are withdrawn from the clarifier by RAS and WAS pumps located in the Secondary Sludge Pump Station. Floating materials (scum) are collected at the surface of the tank using a mechanical skimmer arm and pushed into a scum hopper, where the scum flows by gravity to a sump located at the Secondary Sludge Pump Station.

Clarified secondary effluent leaves the clarifier via perimeter V-notch weirs, and flows by gravity to the Filter Feed and Utility Water Pump Station.

3.2.4.2 Rating and Performance

The Secondary Clarifier design overflow rates that occur at 3.0 mgd (sustained flow) and 9.0 mgd (peak flow) conditions, combined with the corresponding solids loading rates at a RAS flow rate of 100 percent of pumping capacity (shown in Appendix D Table D-1), should be adequate to provide reliable solids removal given the current clarifier design.

With both clarifiers operating under normal conditions, the current performance of the clarifiers is good. Average clarifier effluent TSS concentrations are approximately 1.3 mg/L and settled sludge (RAS and WAS) has a typical solids concentration of 0.8 percent. Approximately 1,500 gpd of scum are produced by the clarifiers. Turbulence (normally undesirable) can be observed at the openings along the outer baffle of each clarifier inlet; however this has not appeared to affect performance.

Similar to the Oxidation Ditches, the WWTP's design should allow for one clarifier to be taken off line during typical dry weather conditions for maintenance. Based on the design conditions listed in Appendix D Table D-1, the sustained overflow rate at the Current Design Condition ADWF would be approximately 600 gpd/ft^2, and the sustained solids loading rate would be 40 lb/ft^2/day, if one of the clarifiers were offline. Further analysis is performed in later chapters of this Facilities Master Plan to more fully evaluate clarifier capacity; however, this relatively high solids loading rate could result in the inability to remove solids quickly enough from the clarifier. The plow-type sludge collectors are less efficient than the suction-type collectors found in more modern clarifiers. Performance may thus be marginal at the sustained loading rates anticipated with one clarifier offline.

In October 2012, each clarifier was drained and inspected. Some corrosion was observed, and City staff determined that recoating of all metal components is necessary. Repair and replacement of the skimmer arms and the energy dissipating inlets was also deemed necessary by City staff. Additional deferred maintenance improvements are identified in the Existing Facilities Conditions Assessment TM (Appendix E).

The *Phase 1 Feasibility Study* (West Yost, January 2005) concluded that a third clarifier is needed for redundancy at the Current Permitted Design Condition[3]. During the October 2012 inspections mentioned above, the City took each of the clarifiers offline (at separate times) for a period of

[3] This conclusion was based, in part, on data indicating the WWTP experiences regular sustained influent BOD and TSS in excess of 300 mg/L. The influent characterization completed in conjunction with this Facilities Master Plan effort supports a conclusion that influent BOD and TSS loadings are actually lower than the older data indicated.

several days. During those periods, the remaining clarifier was able to adequately remove solids. According to City staff, no other unusual or adverse conditions were present during the same period, and the WWTP was able to meet effluent turbidity limits. Several days after both clarifiers had been returned to normal service, City staff observed a gradual increase in effluent turbidity which then returned to normal conditions within another week. Based on the results of the October 2012 inspections, it appears the WWTP can operate with one clarifier offline under certain current low flow conditions. As mentioned above, later chapters of this Facilities Master Plan present additional analysis of clarifier capacity and the need for additional units.

Given the rating information presented above, the City would not likely be capable of taking one of the clarifiers offline for maintenance and still achieve the effluent limitations required for surface water discharge. Having a third clarifier would address this issue, and would also better equip the WWTP to accommodate takedown of an oxidation ditch since, during that time, higher MLSS concentrations would be needed to maintain treatment. Additional evaluation of the clarifier capacities is presented in later chapters of this Facilities Plan. Similar to the Oxidation Ditch situation, the use of pond storage of water for irrigation use rather than surface water discharge must also be considered as an alternative to a third clarifier.

3.2.4.3 Conditions Assessment Summary

The following recommended improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Drain and inspect each clarifier, if possible without overloading either clarifier
- Repair skimmer arm
- Replace skimmer arm wipers
- Repair out of service utility station(s)
- Overhaul drive mechanism torque gauges
- Replace spray header globe valves with ball valves

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E). It should be noted that the above-listed improvements may not be feasible until clarifier improvements and or a new (redundant) clarifier is constructed.

3.2.5 Polymer Storage and Dosing Facility

This section presents a process overview and a capacity/performance evaluation for the Polymer Storage and Dosing Facility. No improvements were identified for this facility as part of the conditions assessment analysis.

3.2.5.1 Process Overview

The Polymer Storage and Dosing Facility, which was constructed as part of the tertiary upgrade project completed in 2010, is located inside the Operations and Electrical Building. Bulk

polymer, stored either in a tank or in totes, is withdrawn and metered using skid-mounted polymer metering pumps, and combined with dilution water (W2) to create a polymer solution. The polymer solution can be injected at two locations: the Secondary Splitter Box, to promote better flocculation and settling in the Secondary Clarifiers, or just downstream of the Filter Feed pumps, to increase flocculation ahead of the Cloth Disc Filters.

3.2.5.2 Rating and Performance

Each of the polymer metering pumps is capable of providing up to a 20 mg/L polymer dose to facilitate coagulation for a peak flow of up to 11.5 mgd, which exceeds the PWWF associated with the Current Permitted Design Condition. At moderate to high flows and turbidity conditions, it is estimated that tank fillings would occur no more than 4-5 times per year. At lower flows and turbidity, polymer will be required less often, and the storage tank will be filled infrequently. The storage room and the tank foundation are large enough to accommodate a larger tank in the future, if required.

The polymer storage tank has not been filled since installation, and smaller totes have been used instead. Polymer solution has not been fed at the Filter Feed Pump Station, but has been added at the Secondary Splitter Box during an upset in the clarifier. Available dilution water pressure for polymer dilution is adequate. City staff are hesitant to add polymer solution at the Filter Feed Pump Station for fear of blinding the disc filter panels.

3.2.6 Filter Feed and Utility Water Pump Station

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Filter Feed and Utility Water Pump Station identified through the Conditions Assessment efforts.

3.2.6.1 Process Overview

The Filter Feed and Utility Water Pump Station was constructed as part of the tertiary upgrade project completed in 2010. This facility receives secondary effluent from the Secondary Clarifiers. Flow leaving the clarifiers enters a bypass valve vault, where the positions of two knife gate valves allow the secondary effluent to either enter the pump station wet well or be diverted around the tertiary system and sent directly to the Effluent Diversion Structure. During the irrigation season, secondary effluent can bypass the tertiary system in this manner and be stored in the Storage Reservoir. As noted later in this chapter, undisinfected secondary effluent in the Storage Reservoir can be applied directly to the City's land application areas.

A total of eight (8) pumps are located at the pump station: three (3) variable speed Filter Feed vertical turbine pumps; three (3) variable speed high-pressure vertical turbine utility water pumps; and two (2) constant speed low-pressure vertical turbine utility water pumps. The Filter Feed pumps send secondary effluent to the Cloth Disc Filters. The high-pressure utility water pumps supply the WWTP's primary W3 distribution system. The low-pressure utility water pumps supply the Biosolids Dewatering Beds with pre-saturation water. An in-line mechanical mixer is situated on the tertiary filter influent header, and can be used to disperse polymer solution from the Polymer Storage and Dosing Area.

An overflow weir in the pump station wet well permits wastewater to passively overflow the structure and flow by gravity to the Effluent Diversion Structure. Therefore, if peak flows exceed the Filter Feed pumping capacity the excess flows will be automatically diverted to the Storage Reservoir.

3.2.6.2 Rating and Performance

With all three pumps online, the Filter Feed Pumps have a PWWF capacity of 9.0 mgd, which is adequate for the Current Permitted Design Condition. However, typical wastewater treatment plant design standards dictate that pump station firm capacity (*i.e.*, the capacity with the largest unit out of service) should be adequate to convey the PWWF. As mentioned above, if peak flows exceed the capacity of Filter Feed Pumps they will be automatically diverted to the Storage Reservoir where they can be held until they are land applied. Because the WWTP includes this reliability feature, it was determined that the Filter Feed Pump Station did not require a firm capacity of 9.0 mgd. The pump station has two spare pump bays to accommodate additional filter feed pumps when they are needed in the future. All of the five filter feed pump bays were designed to accommodate pumps in the future that are larger than those currently installed.

The firm capacity of the high-pressure W3 pumps was selected to accommodate a doubling of the number of sprayers at oxidation ditches and secondary clarifiers. As such, there is likely significant capacity under current conditions. The firm capacity of the low-pressure W3 pumps is adequate to pre-saturate one Biosolids Dewater Bed at a time, which is typically how the dewatering beds will be operated. In the event that operations wishes to decrease the pre-saturation time and/or pre-saturate multiple beds simultaneously, the pump controls could be modified to permit both pumps to run simultaneously.

The Filter Feed pumps are controlled automatically to maintain a constant level in the wet well. The high-pressure W3 pumps are controlled automatically to maintain discharge pressure within a pre-set range. The low-pressure W3 pumps are manually started and stopped according to Biosolids Dewatering Bed pre-saturation needs.

Wet well level, as controlled by the Filter Feed pumps, is steady. The pump station has overflowed its internal weir once since startup, allowing overflow to reach the Effluent Diversion Structure, with no adverse hydraulic issues observed. The bypass valve vault sump does not have a permanent pump installed. A portable diesel sump pump is used to periodically drain water from this structure. The in-line mixer has not been used to apply polymer solution.

In the event of a solids overflow from the secondary clarifiers, solids will enter the pump station and get into the W3 systems. During a clarifier upset experienced in April 2011, solids clogged the sludge feed pump seal water panels and plugged many of the W3-fed utility stations throughout the WWTP. City staff has indicated a desire to relocate the utility water systems to downstream of the filtration/disinfection system, thereby changing the source of the utility water to disinfected final effluent and reducing health risks to staff.

3.2.6.3 Conditions Assessment Summary

The following recommended improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Provide a dedicated bypass vault sump pump and hard piping, with level control by float switch
- Change the source of W3 to UV effluent by relocating the high-pressure W3 pumps, or provide strainers on the utility water pump discharge headers to protect both W3 systems from solids overflow until such time as the source of W3 is changed to UV effluent
- Provide a portable motorized valve operator to facilitate exercising of bypass knife gate valves (and other valves throughout the WWTP).
- Modify location or installation of tertiary filter influent turbidimeter if warranted to resolve undiagnosed periodic high turbidity readings.

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.7 Cloth Disc Filters

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Cloth Disc Filters identified through the conditions assessment.

3.2.7.1 Process Overview

The Cloth Disc Filters, which were constructed in 2010, receive tertiary filter influent (*i.e.*, secondary effluent) pumped from the Filter Feed and Utility Water Pump Station. Three (3) disc filter units treat the incoming flow, which is divided among the operating units equally by weirs set inside each filter unit. In each filter unit, influent travels through a center tube to which disc filter cartridges are attached. Each cartridge contains a number of individual filter panels. Influent enters the filter cartridges through openings in the center tube, and must pass through the 10-micron filter panels before reaching the filtered effluent sump inside the filter unit. Tertiary filter effluent flows by gravity to the UV Disinfection facility.

Accumulated (filtered) solids are periodically removed from the inside (influent side) faces of the filter panels by high pressure sprayers positioned along the outside (effluent side) faces of the panels. The resulting backwash is captured in a trough situated along the centerline of the feed tube, and flows by gravity out of the filter unit and to the Tertiary Plant Drain Pump Station. Backwash spray is provided by an integral pump that withdraws filtered effluent from inside the filter.

In the event of filter failure or excessive head losses inside the filter unit, influent may passively overflow the filter unit and drain by gravity towards the Effluent Diversion Structure for discharge to the Storage Reservoir (and eventual land application).

3.2.7.2 Rating and Performance

The filters were designed in accordance with regulatory guidelines for unrestricted recycled water irrigation that have been established by the California Department of Public Health (DPH). This guidance dictates that the maximum allowable filtration rate in this type of filtration unit is

6 gpm/ft^2. As noted above, based on Appendix D Table D-1, the loading rate on each filter unit is less than the DPH standard when flows to the unit are 3.0 mgd. Therefore, with all three units online, the filtration system has the capacity to treat the peak flows expected at the Current Permitted Design Condition. However, with one unit out of service, the system will not be capable of treating the peak design flows. Nevertheless, as mentioned above, the intent of the tertiary system design was to allow flows exceeding the treatment capacity of the operating filtration and disinfection facilities to passively overflow to the Storage Reservoir. Although the filters were designed to operate with two duty units and one standby unit, all three units are typically operated simultaneously.

The piping serving the filter facility (including filter feed, filter effluent, overflow and backwash piping) was designed to accommodate an expansion of the facility, either to the north or west of the existing facility. As such, the filters can be expanded with minimal disturbance to the existing piping and equipment.

Backwash cycles are typically initiated based on the measured water level on the influent side of the filter cartridges, but will automatically commence after a maximum of 2 hours of operation without backwash. The 2-hour cycle may be user-adjustable, in which case backwashing frequency could be reduced under favorable influent turbidity conditions.

The filters effectively reduce turbidity. Typical filter influent turbidity is 1.2 NTU, and typical effluent turbidity is 0.4 NTU (the City's permit limits turbidity to 2 NTU on a daily average basis). As noted above, City staff have observed periodic spikes in influent turbidity (via the turbidimeter probe located in the Filter Feed pump discharge header) but have been unable to diagnose the cause. The spike typically occurs between 3 and 5 am, and may be due to bubbles adhering to the probe surface. If the periodic spikes continue, it is recommended that City staff coordinate with the probe manufacturer to determine the likely cause and remedy.

3.2.7.3 Conditions Assessment Summary

The following recommended improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Provide mesh screen in filter influent chambers to capture plastics and other debris
- Repair or replace backwash piping pressure gauges
- Replace damaged filter panels

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.8 Tertiary Plant Drain Pump Station

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Tertiary Plant Drain Pump Station identified through the conditions assessment.

3.2.8.1 Process Overview

The Tertiary Plant Drain Pump Station was constructed as part of the tertiary upgrade project completed in 2010. This facility receives drainage flows from the Cloth Disc Filters (filter backwash), the UV Disinfection Facility (chemical drainage and UV channel drainage), and the Operations and Electrical Building (chemical drainage and sanitary waste). Two (2) submersible pumps are operated with level controls. The station discharges to the Headworks Splitter Box. Integral flush/mix valves immediately downstream of each pump's volute agitate the wet well contents to re-suspend settled solids; the valves only operate during pump operation.

3.2.8.2 Rating and Performance

Each of the two pumps, combined with the available storage capacity in the pump station wet well and the UV channel being drained, is capable of handling the total combined flow associated with two simultaneously backwashing filters, drainage from one UV channel, plus a variety of sanitary and maintenance drainage flows from the Operations & Electrical Building. The control philosophy programmed into the filter units dictates that only one filter should be backwashing at any given time. Therefore, the firm capacity of Tertiary Plant Drain Pump Station is adequate to handle all anticipated flows that would occur under the Current Permitted Design Condition. In the rare event of high wet well level, the standby pump is also automatically called into service by the SCADA system.

The pump station has performed adequately since startup and the pump station has kept up with influent during UV channel drainage events. The flush/mix valves provide more than enough re-suspension of solids.

3.2.8.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Replace submersible level transmitter and LOW-LOW, LOW and HIGH float switches with an ultrasonic level sensor; modify control logic such that a HIGH-HIGH signal from the remaining float switch would trigger a timer-controlled wet well draw-down
- Provide air release mechanism for pressure cleanout lids on UV facility drain line

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.9 UV Disinfection

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the UV Disinfection Facility identified through the conditions assessment.

3.2.9.1 Process Overview

The UV Disinfection Facility, which was constructed in 2010, receives tertiary filter effluent from the filter units. Electrically actuated slide gates separate each of three UV channels from a common influent channel. Flow passing through each channel is measured using a Parshall Flume. Before reaching the first UV bank, water passes through a perforated baffle plate. Each channel contains a total of four active banks and one redundant bank. Water passing illuminated banks is treated with a UV dose of 100 mJ/cm^2 (*i.e.* the required minimum dose dictated by DPH for unrestricted irrigation of recycled water). The lamp intensity, number of operating banks, and number of operating channels varies with the incoming flow and transmittance to ensure the minimum required dose is provided in accordance with the manufacturer's (Trojan UV) dose pacing program. Disinfected water leaving each channel passes over a serpentine weir and into a common effluent channel. The resulting final effluent flows by gravity to the Effluent Diversion Structure.

In the event of a critical alarm at the UV facility (*e.g.*, power failure or high turbidity), a hydraulically actuated sluice gate inside the Effluent Diversion Structure will close, automatically diverting UV effluent to the Storage Reservoir.

3.2.9.2 Rating and Performance

Each UV channel can provide the target dose of 100 mJ/cm^2 for a flow up to 3.0 mgd when the UVT of that flow is as low as 55%. Typical UV transmittance is 70-75%. Therefore, each UV channel is capable of treating more than 3.0 mgd under most conditions. As noted above, the maximum flow through the filter and UV system is 9.0 mgd (when all three filter pumps are operating at maximum speed), and flows in excess of this amount are automatically diverted to the Storage Reservoir. Therefore, the UV system (like the filtration system) is capable of treating flows at the Current Permitted Design Condition.

The UV facility was designed with knockout walls in the influent and effluent channels to permit expansion of the facility and the addition of UV channels to the south. In addition, the piping serving the UV facility was designed to accommodate an expansion of the facility. As such, the UV system can be expanded with minimal disturbance to the existing piping and equipment.

The UV facility was shut down for approximately five months in 2011 (during irrigation season), but was brought back online in October 2011, and has not been shut down since. UV effluent grab samples were originally taken from a 36" square access hatch above the UV effluent channel, but City staff have been recently been taking grab samples between the end of the last bank in each channel and the serpentine weir. No water quality violations have occurred since sampling at this location. Sediment accumulation in channels has been minor, and is easily removed during cleaning. Minimal scale has been observed on UV lamps, and staff reports that chemical cleaning of the UV lamps in the cleaning tank appears to be effective.

3.2.9.3 Conditions Assessment Summary

The following recommended actions and improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Evaluate enclosing the UV structure to protect against weather, dust and debris

- Address safety issues by removing eye-level sunshades at instrument panels
- Add a safety grate to the UV effluent channel access hatch

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.10 Effluent Diversion Structure

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Effluent Diversion Structure identified through the conditions assessment.

3.2.10.1 Process Overview

The Effluent Diversion Structure was constructed in 2009. The structure is split into two chambers separated by a concrete wall that acts as a weir. A manually operated sluice gate is located in the concrete wall and is normally closed. The southern chamber receives final effluent from the UV Disinfection Facility and the northern chamber receives overflow and diversions from upstream of the tertiary processes. Normally, final effluent entering the south side of the structure exits through a hydraulically actuated sluice gate into an effluent pipeline that conveys flow to the Effluent Flow Metering facility and to Skunk Creek. Overflow/diversions from upstream of the tertiary processes normally enters the north side of the structure and exits through a gravity pipeline to the Storage Reservoir. In the event that a critical alarm is raised at the UV facility (*i.e.*, loss of power or high turbidity), the hydraulically actuated sluice gate automatically closes, and effluent from the UV facility will overflow the weir wall and spill into the north side of the structure and then flow by gravity to the Storage Reservoir.

3.2.10.2 Rating and Performance

The pipelines leading to and from the Effluent Diversion Structure, as well as the overflow weir wall in the center of the structure, were designed to accommodate a flow up to 21.0 mgd, which exceeds the PWWF associated with the Current Permitted Design Condition.

During start-up of the facility, one out-of-compliance discharge to Skunk Creek occurred when water from the Effluent Storage Reservoir backed up into the structure. The manually operated sluice gate opening in the weir wall separating the two sides of the structure had been temporarily (and inadequately) blocked with sandbags. Reservoir water exited through the open hydraulically actuated sluice gate and entered the treated effluent pipeline. This event occurred during startup of the tertiary facilities and has not been repeated since.

3.2.10.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Install whisker switch on hydraulically actuated sluice gate

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.11 Effluent Flow Metering

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Effluent Flow Metering Facility identified through the conditions assessment.

3.2.11.1 Process Overview

The Effluent Flow Metering Facility was constructed in 2007 and has not been upgraded other than to replace the magnetic flow meter in early 2012. The facility is located near the Storage Reservoir dam, and measures the final effluent flow discharged from the Effluent Diversion Structure. Final effluent flows from the effluent pipeline first into a vault, and then into a short reach of gravity pipeline that passes through the flow metering vault. Flow is measured using a magnetic flow meter located inside this second vault.

Prior to construction of the Effluent Diversion Structure and associated reservoir bypass (*i.e.*, final effluent) pipeline, the Effluent Flow Metering facility measured flow leaving the Storage Reservoir via a cascade aerator. This point of discharge is no longer utilized by the City.

3.2.11.2 Rating and Performance

The magnetic flow meter currently performs adequately, but was not operational for some time prior to being replaced in 2012.

3.2.11.3 Conditions Assessment Summary

The following recommended improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Enlarge the mag meter vault for improved instrument access and maintenance
- Provide a hard-wired or solar-powered drainage pump in the mag meter vault
- In lieu of the above improvements, the existing metering vault could be replaced by a new vault located nearer to the Effluent Diversion Structure.

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.2.12 Chlorine Contact Chamber (Abandoned)

Prior to the construction of tertiary treatment facilities in 2010, the Chlorine Contact Chamber (originally constructed in 1990) provided disinfection to secondary effluent before discharging to the Storage Reservoir, or (more recently) the Effluent Diversion Structure. Disinfected and

dechlorinated effluent was also previously used to supply the facility-wide W3 system, with two pumps and a hydropneumatic tank stationed at the end of the contact chamber.

The Chlorine Contact Chamber is no longer in use. The 36-inch pipelines that once connected the structure to the Secondary Clarifiers and the Effluent Diversion Structure have been blocked, so wastewater can no longer enter the structure.

At present, the City has not identified plans to demolish or re-use the structure. However, the structure would be removed as part of a construction project if new facilities were required in the same location.

3.3 SOLIDS TREATMENT PROCESSES

This section presents an evaluation of the following existing solids treatment unit processes at the City's WWTP:

- Secondary Sludge Pump Station
- Sludge Lagoons
- Sludge Feed Pump Station
- Biosolids Polymer Facility
- Biosolids Dewatering Beds
- Filtrate Pump Station

A schematic showing these solids treatment unit processes is provided in Figure 3-2, and the major design parameters for each unit process are listed in Appendix D Table D-2.

The discussion below presents the following information for each process: a description of the process/facility function; an evaluation of rated capacity (or rating) and recent performance with respect to meeting the Current Permitted Design Condition; and a summary of recommended improvements based on the conditions assessment conducted with City staff input. A detailed discussion of the condition assessment findings is presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.3.1 Secondary Sludge Pump Station

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Secondary Sludge Pump Station identified through the conditions assessment.

3.3.1.1 Process Overview

The Secondary Sludge Pump Station was constructed in 1990. A total of seven (7) pumps are located at the pump station: three (3) RAS pumps (two of the original screw centrifugal pumps and one recently installed horizontally mounted Flygt submersible pump); two (2) WAS diaphragm pumps; and two (2) scum diaphragm pumps. The RAS and WAS pumps draw from a

single hopper in each of the two secondary clarifiers. The RAS pumps discharge to the wastewater line upstream of the Headworks Splitter Box to enable the RAS and influent wastewater to mix before being divided between the two oxidation ditches. The WAS pumps discharge to the Sludge Lagoons. The scum from the secondary clarifiers flows by gravity to a small scum wet well at the Secondary Sludge Pump Station, which feeds the scum pumps. Discharge from the scum pumps can either be directed to the WAS pump discharge header (and therefore to the Sludge Lagoons) or to the RAS pump discharge header (and therefore back through the WWTP).

The City plans to overhaul the Secondary Sludge Pump Station during 2012 and 2013. Improvements will include new VFD-driven RAS and WAS pumps, new constant speed scum pumps, and additional flow metering and isolation valves. These new RAS pumps will all be horizontally mounted submersible pumps, and that the new WAS and scum pumps will all be progressing cavity pumps. Additionally, the improvements will enable both the RAS and WAS pumps to be paced with the influent flow.

3.3.1.2 Rating and Performance

Each RAS pump is capable of conveying approximately 150 percent of a 1.5 mgd influent flow to one clarifier (half of the ADWF under the Current Design Condition), and approximately 50 percent of the 4.5 mgd influent flow to one clarifier (half of the PWWF under the Current Design Condition). Typically, RAS pumping systems are designed to accommodate 100 percent RAS return from each clarifier with one unit out of service. However, even with all three pumps in service, the RAS pumps can only accommodate approximately 75 percent RAS return at a 9 mgd flow condition. Therefore, the capacity of the RAS pumps may need to be increased. The *Phase 1 Feasibility Study* (West Yost, January 2005) recommended that a fourth RAS pump be installed with the construction of a third clarifier, to continue to allow for a dedicated RAS pump for each clarifier. There is available space in the pump station to accommodate an additional RAS pump. However, as noted above, the current clarifier sludge collection mechanism design limits sludge removal capabilities, as evidenced by low sludge solids concentrations. Therefore, increasing the RAS capacity without improving the clarifier would likely have limited benefit. The need for additional RAS pumping capacity (as well as clarifier improvements) is evaluated and confirmed in later chapters of this Facilities Master Plan.

The required pumping capacity for the WAS pumps is a function of anticipated mean cell residence time (MCRT) and MLSS concentrations. These parameters will likely change when the Oxidation Ditches are upgraded/expanded to accommodate the nitrification/denitrification requirements. The *Phase 1 Feasibility Study* (West Yost, January 2005) recommended that a third WAS pump be installed with the construction of a third clarifier, and that the new and existing pumps be increased in capacity to overcome frictional losses in the considerable length of piping to the sludge lagoons. There is available space in the pump station to accommodate an additional WAS pump. The need for additional WAS pumping capacity is evaluated and confirmed in later chapters of the Facilities Master Plan.

3.3.1.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Implement planned near-term overhaul (replace pumps and motor controls, provide additional valving, provide additional flow metering, and provide additional SCADA monitoring and flow-pacing control capabilities)

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.3.2 Sludge Lagoons

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Sludge Lagoons identified through the conditions assessment.

3.3.2.1 Process Overview

The lined sludge lagoons, which were constructed in 1990, receive combined sludge (WAS and scum) from the Secondary Sludge Pump Station. Sludge is directed to one lagoon at a time. As the lagoons are filling, supernatant is decanted from the sludge using a decant pump station that sits between the two lagoons and is directed to the Headworks. Once full, the lagoons are blended using the mixers installed in each basin to create a uniformly mixed sludge that is sent to the Biosolids Dewatering Beds via the Sludge Feed Pump station. The system should allow the City to operate one lagoon in filling/decant mode, while operating the second lagoon in mixing/discharge mode.

3.3.2.2 Rating and Performance

The detention time in the lagoons is a function of the sludge volumes delivered and amount of thickening that can be achieved within the lagoons. The design of the biosolids dewatering system was based on an estimated 1,000 dry tons of solids produced by the WWTP annually. This rough approximation was the assumed production rate for the Current Permitted Design Condition assuming the WWTP's secondary process was providing full nitrification and denitrification. A more refined analysis of WAS loads is developed later in this Facilities Master Plan.

Currently, the City operates the lagoons in a manner where one lagoon receives and decants the fresh sludge. Thickened sludge is then transferred to the second lagoon in batches where it is mixed to create a uniform solids concentration prior to being directed to the Biosolids Dewatering Beds. Using this method, the City is reportedly thickening the sludge to approximately 2 percent solids; however, the City continues working to define an optimal operating strategy for the sludge lagoons and the drying beds.

Some dewatering will occur on the beds during rainfall periods, but significant drying (*i.e.*, dry enough to lift and remove) is not likely to occur in the beds during such periods. Therefore, the lagoons should have enough storage capacity to ensure solids can be stored during periods when dewatering is hampered by rainfall and cooler temperatures.

The *Phase 1 Feasibility Study* (West Yost, January 2005) suggested that two additional sludge lagoons would be necessary to resolve issues related to sludge settling and volatile suspended solids (VSS) loadings to the lagoons. With the construction of the biosolids dewatering facilities, the need to increase settling time and settled sludge concentrations has been partially alleviated. Estimated VSS loadings to the lagoons, which were based in 2005 on an assumed percentage of volatile solids in the WWTP WAS, are roughly twice the recommended loading rate. Therefore, according to these calculations there is a potential for odor issues. Anecdotal evidence, however, suggests that VSS loading may not be problematic, due to a lack of odor issues.

3.3.2.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Provide an additional vehicle access ramp to the lagoon berm
- Provide pedestrian stairway to the top of the lagoon berm

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.3.3 Sludge Feed Pump Station

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Sludge Feed Pump Station identified through the conditions assessment.

3.3.3.1 Process Overview

The Sludge Feed Pump Station was constructed as part of the tertiary and biosolids dewatering facilities project that was completed in late 2010. The Sludge Feed Pump Station is fitted with two (2) progressing cavity sludge pumps. These pumps draw thickened biosolids from the Sludge Lagoons, and discharge them on the Biosolids Dewatering Beds via the Biosolids Polymer Facility.

3.3.3.2 Rating and Performance

The firm capacity of the sludge feed pumps is adequate to load one Biosolids Dewatering Bed at a time, which is typically how the dewatering beds are operated. The pumps have a hardwire lockout in the Motor Control Center (MCC) to prevent the pumps from operating simultaneously. However, in the event that operations staff wishes to increase the sludge feed flow rate in order to decrease the loading time and/or load multiple beds simultaneously, the

pump controls could be modified to permit both pumps to run simultaneously; there is adequate electrical capacity to permit simultaneous operation of the pumps.

The sludge feed pumps are typically operated manually, and are started and stopped according to desired loading of the Biosolids Dewatering Beds. A mag meter measures the flow rate reaching the dewatering beds, and performs well. The pumps typically produce about 225 gpm, but will produce less (185-200 gpm) when thickened sludge concentrations are high.

During the brief period when one clarifier was out of service in April 2011 and solids entered the W3 distribution system, as discussed above, the sludge feed pump seal water panels became clogged with solids. Additionally, some biological growth has been observed in the seal water panels, but has not been problematic for City staff. Addition of strainers in the existing utility water pump discharge piping should reduce the incidence of solids in the panels.

3.3.3.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Restore temperature and pressure switches to service to protect sludge feed pumps from over pressurization and run dry conditions
- Provide a dedicated sample port on sludge feed line, either at the Sludge Feed Pump Station, or at the Biosolids Polymer Facility

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.3.4 Biosolids Polymer Facility

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Biosolids Polymer Feed Facility identified through the conditions assessment.

3.3.4.1 Process Overview

The Biosolids Polymer Facility was constructed in 2010 and conditions sludge prior to reaching the Biosolids Dewatering Beds. Bulk polymer, stored either in totes or drums, is withdrawn and metered using a skid-mounted polymer blending system, and combined with dilution water (W2) to create a polymer solution. Sludge pumped from the Sludge Feed Pump Station is then injected with the polymer solution upstream of a non-mechanical, in-pipe flocculating-mixing device, which promotes the development and aggregation of sludge flocs.

3.3.4.2 Rating and Performance

Original calculations by the Biosolids Dewatering Bed proprietary equipment manufacturer (Deskins) based on lab testing of biosolids characteristics called for a dilution water flow rate of approximately 4 gpm and a polymer feed rate of approximately 1.2 gph. At an observed

maximum 225 gpm biosolids pumping rate and a design solids concentration of 2.5 percent, this polymer system capacity is equivalent to a dose of approximately 7 pounds per ton of dry solids. However, the polymer doses required for adequate dewatering have been higher than the dewatering system manufacturer anticipated. Despite the higher polymer doses (up to 160 pounds per ton of dry solids during startup), the metering pump and blending unit have performed adequately, and dilution water pressure and flow have been sufficient. Specifically, polymer and dilution water flow rates of over 7 gph and 17 gpm, respectively, have been attained without incident.

There is currently no redundant equipment at the Biosolids Polymer Facility. If repairs or replacement are necessary, the City would need to temporarily halt the loading of the Biosolids Dewatering Beds. Because the Biosolids Polymer Facility is used at most once per day, and because solids may be temporarily stored in the Sludge Lagoons, it is likely that repair and/or replacement of the polymer blending unit equipment could be made quickly enough to avoid significant impacts to the City's solids handling process.

3.3.4.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Provide a spill containment unit for polymer totes/drums
- Provide a gantry crane or similar for maneuvering totes
- Provide a concrete landing outside building
- Provide a pressure reducing valve on dilution water line
- Provide an alternative sludge sampling port

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.3.5 Biosolids Dewatering Beds

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Biosolids Dewatering Beds identified through the conditions assessment.

3.3.5.1 Process Overview

The four Biosolids Dewatering Beds were also constructed in 2010. Each time a bed is loaded, it is first pre-saturated with W3 water prior to receiving conditioned biosolids. The bed is then loaded with sludge that has been conditioned with polymer. The amount of solids sent to a bed is dependent on the sludge feed solids concentration. The higher the percent solids the less the beds are filled. The beds can accommodate a maximum of approximately 54,000 gallons of sludge, depending on the level of pre-saturation water and the solids concentration of the sludge.

Once a bed is loaded, valves in the drainage piping beneath the bed are opened and the pre-saturation water and free water in the conditioned sludge is removed by gravity and natural vacuum action. Further dewatering of the sludge left on the surface of the beds is achieved through air and solar drying. The sludge feed, pre-saturation and drainage valving for the beds are all manually operated.

When sludge is dry enough to be handled effectively, the solids are harvested using a special harvesting tractor and are stockpiled on the Biosolids Storage Facility. Filtrate from the dewatering process drains to the Filtrate Pump Station before being pumped back to the Headworks.

3.3.5.2 Rating and Performance

The dewatering beds are rated to handle 2 dry pounds per square foot (or 9,600 dry pounds per bed per cycle). Upon removal from a bed, a solids content of 20 to 60 percent is expected based on the manufacturer's proposal; a minimum concentration of 15 percent solids was required in the specifications.

During the first year of operation, it was difficult to evenly distribute sludge over a bed such that it dried evenly and quickly for easy removal using the sludge harvesting tractor. Sludge has tended to mound near the vertical risers feeding the beds, resulting in thickness and moisture gradients (*i.e.*, thicker, wetter sludge in the middle of a bed, and thin, very dry sludge at the perimeter). As a result, the City initially had difficulty achieving even a ten percent solids content after 9 days of drying.

Recently, the City has determined that a lower pre-saturation water level that leaves the water surface at or below the surface of the top sand layer has yielded better results than pre-saturation that leaves the water surface several inches above the top of the sand. This change in operation has helped to remedy the sludge mounding issue. Drying time may be reduced as a result of the changes, but more operating experience is necessary before drawing such a conclusion. The City also continues to experiment with different solids contents of the sludge being directed to the drying beds.

Polymer use has historically been much higher than anticipated, but has recently been reduced to approximately 10-30 pounds per ton of dry solids[4] with satisfactory flocculation results. Reductions in polymer doses have coincided with revised pre-saturation procedures, resulting in more uniform sludge distribution across the beds.

The harvester has required repairs to a bucket vibrator system twice. When the harvester is fully functional, performance is adequate when dewatered sludge concentrations reach 15-20 percent solids.

[4] Based on limited data from February and March 2012. City staff continues to adjust and optimize the dewatering process to reduce polymer use and improve drying times.

The secondary process evaluation completed in conjunction with this Facilities Planning effort includes developing a revised analysis of the predicted sludge production rates. This information above has been used in conjunction with the refined operational parameters developed from the City's current tests on the drying beds to determine if additional dewatering capacity is necessary, as discussed later in this Facilities Master Plan.

3.3.5.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Provide throttling capability on the sludge feed risers
- Provide modifications to the sludge harvester or provide a more robust harvester

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.3.6 Filtrate Pump Station

The Filtrate Pump Station was constructed in 2010. The pump station is fitted with three (3) submersible pumps that are operated automatically based on wet well level settings. The pump station receives drainage flows from the Biosolids Dewatering Beds, the Biosolids Storage Area, and small area drains at the Sludge Feed Pump Station and the Biosolids Polymer Facility. In addition, the Filtrate Pump Station is connected to the Storage Reservoir, and may be used to partially drain the reservoir when desired by City staff. Wastewater from the pump station is discharged to the Headworks.

3.3.6.1 Rating and Performance

The pump station incorporates three pumps to provide the City with flexibility to select from a wide range of flows when returning flow back to the Headworks. In the event of high wet well level, all three pumps can be called into service. With two pumps operating, the pump station has a rated (firm) capacity of 560 gpm.

The maximum anticipated flow from a single Biosolids Dewatering Bed is 900 gpm (F.D. Deskins Co, Inc., 2009). The pump station was designed to allow one dewatering bed to drain entirely without requiring the pumps to operate, by utilizing storage in the wet well and in the 36-inch and 24-inch drain pipes that feed the wet well. Additionally, the system was designed to accommodate runoff from a 10-year, 3-day design storm whose peak coincides with the peak filtrate flow from one Biosolids Dewatering Bed. Under this wet weather condition, system storage will not be exceeded with the firm pumping capacity of the pump station available (2 pumps operating).

The pumps have generally operated well. The pump station's reservoir return capability has been used and has worked well. The magnetic flow meter works well.

3.3.6.2 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Replace submersible level transmitter and LOW-LOW, LOW and HIGH float switches with an ultrasonic level sensor; modify HIGH-HIGH float switch control logic to trigger a timer-controlled draw down of the wet well

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.3.7 Biosolids Storage Area

This section presents a process overview and a rating and performance evaluation for the Biosolids Storage Area. No improvements were identified for these facilities as part of the conditions assessment analysis.

3.3.7.1 Process Overview

The Biosolids Storage Area consists of a large concrete slab (approximately 18,000 square feet) located near the Biosolids Dewatering Beds. The storage area is intended to contain dewatered biosolids prior to land application on the City-owned fields surrounding the WWTP. Solids are piled and/or windrowed throughout the slab. Drainage from the stockpiled solids or runoff during storm events is captured in valley gutters that surround three sides of the slab, and conveyed toward a catch basin that leads to the Filtrate Pump Station.

3.3.7.2 Rating and Performance

The Biosolids Storage Area was designed to store dewatered biosolids produced over a six-month period, given a solids production rate of 1,000 dry tons of solids per year at 30 percent solids. Furthermore, the storage area was designed assuming that dewatered solids would be placed in 5 foot high windrows with 1:1 side slopes and no separation between adjacent windrows.

- In June 2012, City staff conducted the first land application of stored biosolids since startup of the Biosolids Dewatering Facilities in the spring of 2011. As such, there was well over six months' worth of dewatered biosolids located on the storage area. Based on the fact that the stored biosolids did not take up the entire storage area, some combination of the following conclusions can be reached:
  - Current biosolids production is significantly below estimated values used for design
  - Stockpiled dewatered biosolids solids concentrations are significantly higher than assumed during design
  - Stockpile windrows are taller than assumed during design

3.4 STORAGE AND IRRIGATION SYSTEMS

This section presents an evaluation of the following storage and land disposal system components at the City's WWTP:

- Storage Facilities
- Auxiliary Basin Pump Station
- Irrigation Facilities
- Irrigation Pump Stations

The major design parameters for the storage facilities and irrigation pump stations are shown in Appendix D Table D-3.

The discussion below presents the following information for each process: a description of the process/facility function; an evaluation of rated capacity (or rating) and recent performance with respect to meeting the Current Permitted Design Condition; and a summary of recommended improvements based on the conditions assessment conducted with City staff input. A detailed discussion of the condition assessment findings is presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.4.1 Storage Facilities

This section presents a process overview and a rating and performance evaluation for the Storage Facilities. No improvements were identified for these facilities as part of the conditions assessment analysis.

3.4.1.1 Process Overview

A schematic of the existing storage facilities and associated conveyances is provided in Figure 3-3. As shown, the WWTP includes five storage basins: the Auxiliary Basin, three usable storage ponds (Storage Ponds 1, 3 and 4) and the Storage Reservoir. Storage Pond 2 has been filled and converted to use as a photovoltaic (*i.e.*, solar) array, and can no longer be used to store water.

Storage Ponds 1 through 4 and the Storage Reservoir were constructed in 1980 and, as noted above, served as oxidation ponds and as a storage pond (respectively) for the original treatment plant constructed at the existing WWTP site. With the exception of converting their use to storage in 1990 when the existing treatment processes came online and the conversion of Pond 2 to a solar array site, the only major modification to the Storage Ponds and Storage Reservoir since they were constructed is the removal of material from the Storage Reservoir that occurred in 2005 to increase the available storage capacity and provide fill material for leveling Field 18 of the land application area. Figure 3-4 is a stage versus volume and surface area curve that has been developed for the Storage Reservoir, reflecting modifications to the grading of the reservoir implemented in 2005.

The Storage Reservoir and the four Storage Ponds have historically been used for storage of treated effluent prior to its use for irrigation on the City-owned agricultural properties surrounding the WWTP. Treated effluent is pumped to the agricultural fields by the two irrigation pump stations located in the Storage Reservoir. The irrigation pumps can also direct Storage Reservoir contents to Pond 1 from where it can flow by gravity into Ponds 3 and 4 or back into the Storage Reservoir. (It is also possible to direct some water from Pond 1 to the Auxiliary Basin from where it could be pumped to the WWTP Headworks.) The Filtrate Pump Station can also be used to partially drain the Storage Reservoir by pumping contents back to the Headworks. As noted on Figure 3-3, the Filtrate Pump Station can be used to drain the reservoir to elevation 28 feet, while the irrigation pump stations can be used to drain the reservoir to approximately 24 feet.

The Auxiliary Basin was originally constructed in 1980, and (as mentioned above) served as an aeration basin for the original treatment plant that was constructed at the existing WWTP site. In 1990, when the existing WWTP was constructed, the Auxiliary Basin was converted to an influent bypass storage facility. In 2003, the Auxiliary Basin Pump Station was constructed, allowing for return flows from the Auxiliary Basin to the Headworks. A connection between the biosolids lagoon supernatant return line and the influent bypass pipeline is suspected of having been constructed at that time, thereby allowing for the diversion of supernatant and/or biosolids to the Auxiliary Basin. Finally, in 2007 the Auxiliary Basin was lined with membrane material.

Since 1990 the Auxiliary Basin has been used almost exclusively to receive and store influent during either planned diversion periods (*e.g.*, during construction) or unplanned diversion periods (*e.g.*, during equipment failure or peak wet weather flows exceeding WWTP capacity). Once the diversion condition has subsided, the City returns the diverted wastewater to the treatment plant headworks via the Auxiliary Basin Pump Station. During the recent short-term period before the biosolids dewatering system was brought online, the City directed biosolids that exceeded the available storage capacity of the two sludge lagoons to the Auxiliary Basin via the connection between the biosolids lagoon supernatant return line and the influent bypass pipeline.

City staff has also indicated a desire to install aerators in the Auxiliary Basin. These aerators would help to reduce the potential for nuisance odor conditions to develop during a diversion period. In addition, if aeration were added to the Auxiliary Basin the City could conceivably use the basin as a supernatant treatment system, thereby minimizing the impact of elevated ammonia loads in the supernatant on the Oxidation Ditches. However, if floating aerators were added to the basin, a minimum basin depth of 3 feet would be required, reducing the usable storage capacity by approximately 30 percent (to 4.1 MG).

3.4.1.2 Rating and Performance

Prior to October 2011 the City was not permitted to discharge between May 1 and October 31 of each year. Consequently, the entire storage capacity in the Storage Reservoir and Ponds 1 through 4 was needed during the late fall months (when irrigation demands are low, but discharge to Skunk Creek was not allowed) to ensure an unpermitted discharge did not occur. Since October 2011, the City has been permitted to discharge effluent to Skunk Creek year-round. However, the City is also still required to provide recycled water for irrigation of the

City-owned agricultural area surrounding the WWTP. Therefore, some storage of effluent is required to accommodate this use. Nevertheless, storage required for agricultural operations is significantly less than was previously required. Therefore, the excess storage available could potentially be used for other purposes. A detailed discussion of options for use of this available storage capacity is provided elsewhere in the Facilities Master Plan.

3.4.2 Auxiliary Basin Pump Station

This section presents a process overview and a rating and performance evaluation for the Auxiliary Basin Pump Station. No improvements were identified for these facilities as part of the conditions assessment analysis.

3.4.2.1 Process Overview

The Auxiliary Basin Pump Station was constructed in 2003 and has not been modified since. The pump station wet well receives gravity drainage from the Auxiliary Basin, the nearby Vactor Waste Receiving Station, and sanitary wastewater from the Control Building. Two submersible pumps in the wet well pump effluent in parallel 3- and 8-inch force mains, which manifold into the original 16-inch force main to the WWTP, just upstream an expansion of the 16-inch pipe to the newer 24-inch pipe that ultimately discharges in the Headworks.

3.4.2.2 Rating and Performance

City staff have not indicated any issues with the pump station or its controls. At the pump station's firm capacity of 100 gpm, the Auxiliary Basin could be completely emptied in approximately 40 days. With only the larger of the two pumps in operation, with a capacity of 650 gpm, the basin can be emptied in less than 7 days.

3.4.3 Irrigation Facilities

This section presents a process overview and a rating and performance evaluation for the Irrigation Facilities. No improvements were identified for these facilities as part of the conditions assessment analysis.

3.4.3.1 Process Overview

The irrigation facilities consist of approximately 175 acres of the irrigation area (divided into 15 different field areas), the buried pipeline transmission main and laterals, the earthen surface irrigation ditches, and the earthen runoff control ditches used to convey recycled water between the fields and the Storage Reservoir. The layout of these facilities is shown on Figure 3-5, and a summary of the acreages for each field area is listed in Appendix D Table D-4.

The original WWTP constructed in 1981 included eighteen fields, totaling 148 acres, all of which were located south of the treatment ponds. Two of the original eighteen fields were converted to the site of the existing main treatment facilities that were constructed in 1990. In 1991, the City began land applying biosolids on a 20-acre portion of the irrigated areas located to north of the main treatment facilities (referred to at one time as Field 18). Sometime around this same period, the City also converted the remaining available City-owned land located north of the main WWTP

site to a 30-acre irrigation field (referred to at the time as Field 19). In 2004, Fields 11, 12, and 13 were rehabilitated and combined, resulting in one, 24.6 acre field (currently referred to as Field 11). Finally, in 2005 the City combined the historic 20-acre biosolids disposal field with the adjacent 30-acre field to create one 50-acre field (currently referred to as Field 18).

The City also historically has irrigated an additional 160-acre irrigation area located adjacent to, and south of, the City-owned agricultural fields. This offsite property was leased to the City by the Roman Catholic Bishop of Sacramento (RCB Property) and was used for land application of effluent between May and October of each year. With the recently permitted year-round discharge, the City has allowed this lease with the RCB to expire and future irrigation on this site is not anticipated.

The City has historically land applied biosolids to several of the City-owned irrigation fields, with the majority of applications occurring on the historic 20-acre biosolids disposal field. Prior to the construction of the dewatering facilities, all applications were made using a sludge injection vehicle. With the recent construction of the dewatering system, the City has begun applying solids via surface spreading, followed by tilling to incorporate the biosolids into the soil of the application fields. The application occurs twice per year: once in the spring before planting and again in the fall after the crops are harvested. The biosolids storage area, which provides approximately six months of biosolids storage at the current permitted design flow condition, is used to accommodate the land application schedule.

The City leases the irrigation areas to local farmers, and day to day operations of the land application areas is maintained by the farmers. In addition, the City contracts with these farmers to apply the dewatered biosolids. To date, only one application of dewatered biosolids has occurred.

3.4.3.2 Rating and Performance

As discussed above, the City was not permitted to discharge between May 1 and October 31 of each year prior to October 2011, and the City had to rely on the irrigation fields for wastewater disposal. In addition, the City has historically used portions of the City-owned fields for biosolids land application. Now that the City has the ability to discharge treated effluent to Skunk Creek year-round, the primary function of the of the irrigation fields with respect to the WWTP is the land application of biosolids. However, the City also irrigates the fields with treated (recycled) wastewater as part of the WWTP's effluent disposal and reuse practices.

The capacity of the land application area for biosolids applications is limited by the total nitrogen loading to any individual field in a given year (accounting for nitrogen loadings from the applied treated wastewater) and the cumulative loadings for metals of concern (as defined by the United States Environmental Protection Agency 503 Biosolids Regulations). The recently constructed biosolids dewatering process has the potential to change the nitrogen and metal concentrations in the land applied biosolids. Therefore, previous analyses of allowable biosolids loadings rates and field disposal capacity must be revisited. This revised analysis is presented later in the Facilities Master Plan.

3.4.4 Irrigation Pump Stations

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Irrigation Pump Stations identified through the conditions assessment.

3.4.4.1 Process Overview

A total of four irrigation pump stations serve the Storage Reservoir and the agricultural fields surrounding the WWTP. Two of the pump stations are used to convey recycled water from the Storage Reservoir to the fields for irrigation; these pump stations are referred to Irrigation Pump Station No. 1 (or the Dam Pump Station) and Irrigation Pump Station No. 2 (or the "50-Acre Pump Station"). The remaining two pump stations collect and pump tailwater from the fields back to the Storage Reservoir; these pump stations are referred to Return Pump Stations Nos. 1 and 2.

3.4.4.2 Process Operation and Performance

The farmers that cultivate the land surrounding the WWTP operate the pump stations according to irrigation and tailwater return needs, typically between the months of May and September. The farmers operate irrigation valves and record flow meter data. City staff are responsible for maintaining the pump stations and recording operating pump hours.

3.4.4.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Provide fish screens for Irrigation Pump Station #1
- Provide grating for Irrigation Pump Station #2
- Replace pressure transducer in Irrigation Pump Station #2 wet well with ultrasonic level sensor
- Provide a longer service truck boom to facilitate pump removal
- Provide new simplex control panels with local power distribution for Irrigation Pump Station #1 and Return Pump Station #1

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.4.5 WWTP Water Systems

This section presents an evaluation of the following two water systems at the City's WWTP:

- High Pressure Utility Water (W3)
- Non-Potable Water (W2)

The major design parameters for the WWTP water systems are shown in Appendix D Table D-5.

The discussion below presents the following information for each system: a description of the process/facility function; an evaluation of rated capacity (or rating) and recent performance with respect to meeting the Current Permitted Design Conditions; and a summary of recommended improvements based on the conditions assessment conducted with City staff input. A detailed discussion of the condition assessment findings is presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.4.6 High Pressure Utility Water (W3)

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the High Pressure W3 Water System identified through the conditions assessment.

3.4.6.1 Process Overview

The high pressure W3 system serves utility stations, sprayers and seal water systems located throughout the WWTP. The system is supplied by undisinfected secondary effluent from the high-pressure W3 pumps located at the Filter Feed and Utility Water Pump Station.

3.4.6.2 Operation, Performance and Issues

City staff has reported that the number of utility stations is adequate. As discussed above, many of the utility stations and seal water systems supplied by W3 became clogged during a temporary shutdown of one of the secondary clarifiers in April 2011. Refer to the section on the Filter Feed and Utility Water Pump Station for recommended activities and improvements for the High-Pressure W3 System.

3.4.6.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Replace PVC nipples with threaded female connections with stainless steel sleeves for additional strength

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.4.7 Non-potable Water (W2) System

This section presents a process overview, a rating and performance evaluation, and a summary of recommended improvements for the Non-potable W2 Water System identified through the conditions assessment.

3.4.7.1 Process Overview

The W2 water system is fed by two non-potable water wells and hydropneumatic tanks serving the WWTP. One well and tank, the smaller pair of the two, was part of the original WWTP construction in 1980. A second, larger well and tank were installed in 2010 as part of the tertiary improvements, and now supply the majority of the WWTP's non-potable water needs. The hydropneumatic tank stores water and maintains pressure within a pre-set range. When the pressure drops below a pre-set value, a compressor operates to add air and increase pressure.

The W2 system serves restrooms, sinks, chemical makeup and some of the washwater stations (*i.e.*, hose bibs) throughout the WWTP. The W2-supplied utility stations are concentrated near the tertiary facilities and the Operations & Electrical Building. Although the WWTP currently purchases drinking water, City staff is unclear as to whether the well water has ever been tested to determine suitability for direct potable use.

3.4.7.2 Process Operation and Performance

The W2 system does not receive heavy use, and the new well and hydropneumatic tank do not operate frequently. No performance issues are currently identified.

3.4.7.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Repair or replace new hydropneumatic tank sight glass

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.5 ELECTRICAL AND CONTROL SYSTEMS

This section presents an evaluation of the following major electrical and controls systems at the City's WWTP:

- Utility Service
- Electrical Switchgear
- Standby Generators
- Motor Control Centers
- SCADA Systems
- Solar Facility

The major design parameters for the major electrical and controls systems are shown in Appendix D Table D-6.

3.5.1 Utility Service

This section presents a system overview and a capacity/performance evaluation for the WWTP's Utility Service. No improvements were identified for this system as part of the conditions assessment analysis.

3.5.1.1 System Overview

The Sacramento Municipal Utility District (SMUD) provides electrical service to the WWTP. The WWTP's original electrical service was provided in 1980, and was upgraded in 1990 and again in 2010. The utility transformer included with the WWTP's original service was replaced in 1990. As part of the tertiary and biosolids dewatering facilities construction project in 2010, the 1990s-era SMUD service drop and 750 kilovolt-amp (kVA) transformer were abandoned and the existing WWTP electrical loads were transferred to a new, larger utility service and transformer.

The new utility transformer steps 12 kilovolt (kV) power from overhead power lines down to 480 volt (V). The new service is rated at 2000 amps at 480 volt (V).

3.5.1.2 Rating and Performance

Following startup of the 2010 upgrades, the new SMUD service experienced problems. It was determined that 12 kV fuses located on SMUD's overhead lines feeding the WWTP were blowing prematurely. Following replacement of the fuses by SMUD in 2011, service has been reliable.

The existing 2000-amp service can accommodate some expansion of the WWTP. The service and subsequent distribution equipment has approximately 350-400 hp of remaining capacity. Of this capacity, 40 hp was envisioned for two future filter feed pumps. WWTP expansion requiring additional electrical loads greater than 350-400 hp will require a larger utility service. A second service drop and/or changes in power distribution would also be required.

3.5.2 Electrical Switchgear

This section presents a system overview and a capacity/performance evaluation for the WWTP's Electrical Switchgear. No improvements were identified for this system as part of the conditions assessment analysis.

3.5.2.1 System Overview

Two switchboards serve the WWTP. The older of the two switchboards was installed in 1990 and is located near the Control Building. The newer equipment was installed in 2010, and aside from the connection to a new photovoltaic system at the WWTP in 2011, no upgrades to the switchboard have occurred since installation.

The new switchboard receives 480 V power from the SMUD utility transformer. Power is currently distributed to the WWTP through four main feeder circuits: two feeders to serve WWTP loads from facilities installed prior to the 2010 upgrade project, and two feeders to new loads from the 2010 upgrade. In the event of a utility service disruption, a 600 kilowatt (kW) standby generator installed in 2010 feeds facilities constructed in 2010 via an automatic transfer

switch (ATS). A 300 kW generator feeds loads to motor control centers MCE and MCA installed in 1990 via an ATS located in the MCC room in the Chemical / Maintenance Building. A 1,000 kilowatt (kW) photovoltaic array installed in 2011 feeds the main switchboard "upstream" of the branch feeders to the pre-2010 WWTP.

3.5.2.2 Rating and Performance

The new switchboard has performed without issue since installation. The switchboard is rated for 2,000 amps at 480 volts, and the main breaker installed in the switchboard will allow for connected loads up to 2,000 Amps. Currently 1,400 Amps are utilized for existing WWTP loads, leaving approximately 600 Amps spare capacity before reaching the switchboard's safe operating capacity.

Space is available for future connections. The 600 kW generator is connected to the switchboard and power is switched between SMUD service and generator service via a 2,000 Amp ATS. Up to two additional generators may be added in parallel with the existing 600 kW generator if additional standby loads are required. In addition, the Solar Facility is connected directly to the Utility Electrical Bus within the Switchboard via a 1600 Amp breaker.

3.5.3 Standby Generators

This section presents a system overview and a capacity/performance evaluation for the WWTP's Standby Generators. No improvements were identified for this system as part of the conditions assessment analysis.

3.5.3.1 System Overview

The WWTP has two operating generators. A 300 kW generator was installed in 1990, and provides backup power to loads associated with the primary and secondary treatment processes. A new 600 kW generator was installed in 2010 and provides backup power primarily to the WWTP's tertiary and solids dewatering processes.

Both generators require diesel fuel. The 300 kW generator uses fuel stored in a separate, outdoor concrete vault. The 600 kW generator uses fuel stored in a steel vessel located beneath the generator unit. The 600 kW sits outdoors within a sound-attenuated enclosure. The 300 kW generator sits indoors in the Chemical / Maintenance Building.

3.5.3.2 Rating and Performance

Both generators are in good working condition. With a combined capacity of 900 kW, the generators can accommodate approximately 1,200 horsepower. This capacity is sufficient for currently installed equipment required in emergencies. As WWTP loads requiring standby backup increase, additional generators can be installed. As discussed above, the WWTP's new switchboard can accommodate two additional 600 kW generators for this purpose. Additional generators beyond this amount will require additional switchgear.

3.5.4 Motor Control Centers

This section presents a system overview and a capacity/performance evaluation for the WWTP's MCCs. No improvements were identified for this system as part of the conditions assessment analysis.

3.5.4.1 System Overview

The WWTP features four MCCs. The original MCC, which is located in the Control Building, was installed in 1980 and was upgraded in 1990. The second and third MCCs (MCA and MCE) were installed in 1990 and are located in the Chemical / Maintenance Building. The third MCC (MCC-TF) was installed in 2010 and is located in the Operations & Electrical Building.

The original MCC receives power from the WWTP's old switchboard, located near the Control Building. The original MCC was designed to house controls for the WWTP's original equipment, much of which has been abandoned. As a result, there is available capacity in the MCC for future equipment.

MCE houses controls for the majority of the WWTP's primary and secondary treatment processes. Prior to the 2010 upgrade, MCA housed controls for the WWTP's chlorine disinfection system, but most of those motors have been removed. Motor controls and a stepdown transformer for the Grit Removal facilities were added to MCA in 2007. There is space in the MCC room in the Chemical/Maintenance Building for a future MCC, which would include controls for expansion of the WWTP's primary and secondary facilities.

MCC-TF houses controls for the WWTP's tertiary and biosolids dewatering facilities, as well as several stepdown transformers. There are a total of 10 spare buckets in the MCC for the addition of future tertiary and dewatering facilities. MCC-TF does not house controls for the UV and filter facilities; these facilities are instead fed power directly from the Power Panel PP-TF1, and motor controls and transformers are located near the equipment.

3.5.4.2 Rating and Performance

The equipment located in the original MCC and the WWTP's old switchboard is operational, however many of the motor controls located in the original MCC are not connected to any motors. Receptacles located around the Auxiliary Basin are powered from this MCC. The original MCC and old switchboard can accommodate future expansion. The old switchboard has three spare breakers and five spare breaker locations. However, the age of this equipment may limit its usefulness. The original MCC has seven spare motor control buckets, and five spare 2x[5] spaces for future expansion.

MCC-MCA could accommodate limited future expansion. The MCC has three spare 1x spaces, one spare 2x space and one spare 3x space.

[5] Designation of MCC spaces indicates the vertical dimension of the available space, where "x" is equal to 6 vertical inches (*e.g.*, 2x = 12-inch vertical dimension).

MCC-MCE could also accommodate limited future expansion. The MCC has one spare 1x space, two spare 2x spaces and one 3x spare starter that was used for the old utility water pump P-16. The power factor correction unit is not functional, is not being used for the current VFD-driven RAS pumps, and may be removed in the future. The bucket used to power the power factor correction unit may be considered a spare bucket/breaker; the power factor correction unit was housed in a 2x space and has a 200 amp three-phase breaker.

MCC-TF can accommodate some future expansion, and has a single spare 1x space, two spare 2x spaces, four spare 3x spaces and two spare 7x spaces.

3.5.5 SCADA System

This section presents a system overview, a rating and performance evaluation, and a summary of recommended improvements for the WWTP's Supervisory Control and Data Acquisition (SCADA) System.

3.5.5.1 System Overview

The WWTP's SCADA system was developed and installed in 2010. The system includes a Wonderware Historian 10.1 database which supports up to 5,000 tags; Historian Clients that provide trending and data querying capabilities; Wonderware's HMI Reports, an alarm and event notification (SCADAlarm) software; Wonderware's InTouch (10.1) development studio; Wonderware's InTouch terminal service runtimes with inputs/outputs capable of monitoring 60,000 tags; Microsoft SQL Server, and Microsoft's basic Client Access License (CALs).

Two main WWTP SCADA server computers are located at the WWTP which poll local WWTP Programmable Logic Controller (PLC) data. In addition, the WWTP SCADA system polls data from the City's Municipal Service Center (MSC) SCADA system by way of high speed data radio. The MSC SCADA system polls data from remote sewer lift stations, water treatment plants, and wells.

The WWTP and MSC SCADA systems utilize 119 screen windows and monitor a total of 9,935 system tags (5,660 local and 4,275 remote).

3.5.5.2 Rating and Performance

For some time following startup of the facilities constructed in 2010, the City experienced issues transmitting SCADA information and internet signal beyond the WWTP site using the high speed radio system. If the radio link to the City's MSC is not working, water and wastewater section Utilities Division staff stationed at the WWTP cannot monitor the remote facilities that communicate remotely to the MSC. Connectivity issues can also limit the WWTP's access to the City's network and/or internet signal. However, recent performance of the radio link has been good, and City staff have experienced no connectivity issues.

3.5.5.3 Conditions Assessment Summary

The following improvements were identified through the WWTP conditions assessment process that was completed between March and June 2012:

- Provide SCADA computer and related software devoted to water section Utilities Division staff.
- Migrate the MSC SCADA system for monitoring remote facilities to the WWTP.
- Establish functional reports from both wastewater and water SCADA system data.

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.5.6 Solar Facility

This section presents a system overview and a capacity/performance evaluation for the WWTP's Solar Facility. No improvements were identified for this system as part of the conditions assessment analysis.

3.5.6.1 System Overview

The 1 megawatt (MW) solar generation facility was installed in 2011. The facility generates power from photovoltaic (PV) panels installed in the former Storage Pond #2, and backfeeds power to SMUD at the WWTP's main (*i.e.*, new) switchboard. The facility is maintained and operated by Solar Star California XXII LLC (Solar Star). The City entered a power purchase agreement with Solar Star in March 2011.

3.5.6.2 Rating and Performance

The equipment is new and in good working condition. The 1 MW system was designed to provide up to 2,000,000 kilowatt-hours (kWh) of electricity annually. The equipment occasionally trips offline and City maintenance personnel reset the trip to re-energize the equipment. The expansion capacity of the system is limited by the WWTP's new switchboard, which has a capacity of 2,000 Amps at 480 Volts. With the current switchboard, the Solar Facility could be expanded to 1.3 MW.

3.6 BUILDINGS AND STORAGE FACILITIES

This section presents an overview of, and (as appropriate) recommended improvements for the following building and storage facilities at the City's WWTP:

- Operations and Electrical (O&E) Building
- Control Building
- Chemical/Maintenance Building
- Vehicle Storage Building
- Covered Equipment Storage

3.6.1 Operations & Electrical (O&E) Building

The O&E Building was constructed in 2010 and serves as the hub of the WWTP's operations, and serves as the principal meeting point for City wastewater and water section Utilities Division staff. The south side of the concrete masonry unit (CMU) building includes offices, workspaces and meeting space for O&M staff, and houses all existing WWTP SCADA terminals. Filing, storage and restroom facilities are also included in the south side of the building. An electrical room and a polymer storage and dosing room are included on the north side of the building. The electrical room houses an MCC for the tertiary facilities, as well as a stepdown transformer and several power panels. Additionally, the SCADA rack for the WWTP is located in this room. The polymer storage and dosing room contains bulk polymer storage equipment, as well as metering and dosing equipment. Additional miscellaneous storage is provided in the room as well.

3.6.1.1 Existing Conditions

The control room area (consisting of offices, workspaces, meeting space and filing/storage) is used heavily by City staff. Staff report that the offices feel crowded with the existing furniture, and that larger offices would be better able to accommodate private meetings with 3-4 people. Staff like having the shared computer workstations located along the periphery of the control room.

The electrical room is adequately sized. Staff have expressed concern that the SCADA historian cabinet is not fixed to the floor or wall, and seems vulnerable to damage.

The location of the wall-mounted heater in the polymer storage and dosing room restricts the placement of the industrial shelving along the south wall of the room; the shelving cannot be placed too closely to the heater due to fire risk.

3.6.1.2 Planned Uses and Recommended Improvements

The City plans to transition all water and wastewater section Utilities Division staff to the WWTP, and the Operations & Electrical Building will need to accommodate the increase in personnel. It is estimated that 20-22 people will occupy the building within 5-10 years.

Figure 3-6 presents a conceptual sketch of the building incorporating the increase in staff and the identified existing deficiencies. The improvements consist of a 40' wide by 40' long addition, similar to the existing wing to the west. To the north of the new structure, there will be an outside patio with a sloping roof cover.

Following are the revisions and new added facilities:

- The existing men's shower and toilet room will become the women's shower and toilet room.
- The men's revisions include a separate stalled toilet, the existing shower and an increase in lockers and locker area.
- A new men's restroom adjacent and accessible to the Men's shower and restroom area.

- A new kitchen with access from the hallway and access to the outside patio.
- At the end of the hallway, a training room which can seat up to 24 people. This room can be made accessible from the kitchen during non-meeting times.
- Changes to the existing Control Room, include:
  - No more access to the Polymer room.
  - Opening in wall to new Control Room 2.
- The new Control Room 2 includes:
  - Two new offices with room in each for a table and several chairs.
  - Additional storage.
  - 4 new workstations.
  - An additional exit door to the South.

3.6.2 Control Building

The Control Building was the WWTP's first occupied building, and was constructed in 1980. The CMU building originally housed the WWTP's only MCC, the laboratory, chemical storage and feed equipment, a single restroom, and a small computer workstation. When the WWTP's secondary treatment facilities were constructed in 1990, the chemical storage and feed equipment and a number of the motor control devices were removed, and the space was reconfigured to include a break room, locker facilities and a small kitchen. Following construction of the tertiary facilities in 2010, additional electrical and control functions, as well as locker facilities, were migrated to the Operations & Electrical Building.

3.6.2.1 Existing Conditions

The laboratory is dated, and in need of rehabilitation, including additional storage space, a larger sink, more counter space and a dedicated computer workstation. The City has an annual budget of $15,000 for laboratory improvements. Similarly, the City has a one-time budget of $25,000 for rehabilitation of the break room.

The HVAC unit that services the Control Building is new, however staff report that the ventilation for the building is inadequate.

3.6.2.2 Planned Uses and Recommended Improvements

The City is planning to make the WWTP the day-to-day hub for all water and wastewater Utilities Division staff, increasing the number of staff on site to 20-22 within 5-10 years.

Figure 3-7 presents a conceptual sketch of the Control Building incorporating all of the changes within the existing building. Following are the revisions and new added facilities:

- The existing laboratory, Laboratory 1, has increased counter space and a new center island counter.
- Addition of an emergency shower/eyewash area.

- A new opening in the wall to the North providing access to the new Laboratory 4.
- Laboratory 4 revisions include:
  - A work station, storage counters and lockers, and an ice maker.
  - Double exterior doors converted to windows
  - A door added to the existing electrical room.
- The existing kitchen has been modified to Kitchen 3 with:
  - A small refrigerator.
  - A microwave.
  - Sink with counter and cabinets.
  - An opening to Laboratory 4.
- The bathroom remains the same, with a new access to the north, and the south access removed.

3.6.3 Chemical / Maintenance Building

The Chemical Building was originally constructed in 1990, and housed chlorination and dechlorination chemical storage and metering equipment, MCC equipment, and a standby generator. After completion of the tertiary treatment facilities and the abandonment of the Chlorine Contact Chamber in 2010, the chemical handling equipment and related appurtenances were removed from the building, and the building has since begun transitioning to use for maintenance purposes (thus the terminology "Chemical / Maintenance Building"). The chlorinator and sulfonator rooms have been used as temporary offices and are being repurposed as storage rooms, while the chlorine and sulfur dioxide storage bays will be used as a maintenance shop capable of housing several maintenance vehicles.

The MCCs and standby generator located in the west end of the building remain in use.

3.6.3.1 Existing Conditions

As discussed above, all chemical storage and metering equipment and related alarms and appurtenances have been removed from the building. However, the hoist structures that serve the north and south former chemical storage bays remain intact. In order to facilitate easier vehicle access to the bays, the columns supporting the hoists outside the east end of the building should be relocated.

Additional industrial shelving in the building would allow spare equipment to be stored. A door between the north and south chemical bays would permit pedestrian traffic. There is no utility sink in the building, limiting cleanup capabilities.

The HVAC unit serving the MCC room needs to be replaced.

3.6.3.2 Planned Uses and Recommended Improvements

A preliminary list of recommended improvements to the Chemical / Maintenance Building related to both the increase in staff and identified existing deficiencies is included below:

Figure 3-7 presents a conceptual sketch of the Chemical / Maintenance Building incorporating all of the changes within the existing building. Following are the revisions and new added facilities:

- The repurposed offices have been revised to storage and will have storage shelving added.
- A door has been added from Storage Room 1 to Maintenance Room 1.
- A 6 foot wide opening has been added between Maintenance Rooms 1 & 2.
- A new beam and new support system will relocate the existing overhead crane vertical supports to allow improved vehicle access to Maintenance Rooms 1 & 2.

3.6.4 Vehicle Storage Building

The Vehicle Storage Building was originally constructed in 1990 to house the WWTP's subsurface biosolids injector. The building was subsequently expanded (date unknown) to house additional vehicles.

The building is in adequate condition and continues to serve its intended purposes, and no improvements have been identified at this time.

3.6.5 Covered Equipment Storage

A variety of large, portable equipment pieces are stored at the WWTP, notably pumps and generators. This equipment is currently being stored in the open in a paved area north of the Secondary Clarifiers. The City wishes to provide a canopy structure with open ends to provide a total of eight similarly sized pieces of equipment with a measure of protection against the elements.

3.7 OTHER ANCILLARY FACILITIES

This section presents an overview of, and (as appropriate) recommended improvements for the following ancillary facilities at the City's WWTP:

- Compressed Air System
- Fire Protection
- Site Lighting
- Site Access and Security
- Site Drainage
- Parking and Ingress/Egress

- Non-Wastewater Related Site Uses
- Landscaping

3.7.1 Compressed Air System

There is no facility-wide compressed air system at the WWTP. An 18 cubic feet per minute (CFM) compressor provides air to the Chemical / Maintenance Building. According to City staff, this compressor is adequate for WWTP maintenance needs. An additional duplex compressor located in the Secondary Sludge Pump Station currently provides compressed air to the WAS and scum diaphragm pumps. These pumps are scheduled to be replaced by City staff in the near future, and the accompanying duplex compressor and related appurtenances will be removed from service.

3.7.2 Fire Protection

A sprinkler system provides fire suppression for the Operations & Electrical Building. The sprinkler system is supplied by the WWTP W2 system. The Control Building does not have a fire suppression system. Because the WWTP is not connected to the City's main water supply, no fire hydrants exist at the site.

No additional fire protection requirements have been identified at this time.

3.7.3 Site Lighting

Outdoor site lighting fixtures consist primarily of high pressure sodium (HPS) bulbs. No issues regarding lighting adequacy have been identified by City staff. In the future, light-emitting diode (LED) bulbs may be evaluated as a cost-effective alternative to HPS bulbs; according to staff, recent studies suggest that LED bulbs are not yet reliable enough to justify replacement of HPS bulbs.

3.7.4 Site Access and Security

The automatic entrance gate at the southeast corner of the WWTP and the manual chain link gate nearest the Effluent Diversion Structure (EDS) are the main entrances to the WWTP. Six additional chain link double swing gates are located along the chain link fence that surrounds the WWTP. Chain link gates are secured with chains and locks.

City staff and visitors generally use the automatic gate to enter and leave the WWTP. Commercial vehicles are required to use the gate near the EDS, which is accessed by driving east and north of the of storage ponds before turning south between Storage Pond #4 and the Storage Reservoir. Presently, only portions of the road separating Storage Pond #4 and the Storage Reservoir are paved.

A security camera system provides video surveillance of portions of the WWTP, but has not functioned reliably recently. One camera is located at the automatic entrance gate and two cameras are located on a monopole located near the Operations & Electrical building. The camera located at the automatic entrance gate communicates to the security system via radio, however the radio system works intermittently and cannot be relied on as a functional system.

The monopole-mounted cameras were designed with pan, tilt and zoom capabilities, however one of the cameras is unable to move and is pointed downwards towards the parking lot. Significant blind spots exist in the monopole-mounted camera's field of vision near the south sludge lagoon and the solar facility. Additionally, the existing security system does not protect the Chemical / Maintenance Building. A building intrusion sensors interface with the security system was installed in 1990.

The following improvements are recommended for the site access and security systems:

- Service and/or replace the security camera system
- Add a minimum of one new security camera to eliminate blind spots
- Reconfigure the building intrusion detection system to access the existing PLC control system directly to provide global arm/disarm and call-outs to personnel and/or authorities
- Provide intrusion alarms on all gates
- Pave the remainder of the road separating the Storage Reservoir and Storage Pond #4

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.7.5 Site Drainage

Drainage of the WWTP is accomplished with catch basins and storm drainage piping. Storm drains discharge to the Filtrate Pump Station, the Storage Reservoir, Storage Pond #4, and surface drainage channels. After storms, the drainage ditch in the northwest corner of the WWTP holds some water.

As part of the tertiary system upgrade in 2011, a seal coat was applied to existing pavement. Many of the cracks present in the original, underlying pavement have appeared in the seal coat.

3.7.6 Parking and Ingress/Egress

There are currently 14 non-disabled parking spots. With the addition of water Utilities Division staff, additional parking for staff, service vehicles and visitors will be required. Additional parking should be dispersed around the WWTP.

3.7.7 Non-Wastewater Related Site Uses

The WWTP is used for several non-wastewater related purposes. Waste Management, Inc. uses the WWTP as a transfer facility. Additionally, the City owns a Police Firearms Range (Pistol Range) northwest of the Storage Reservoir dam. The Pistol Range is leased to the CDCR for training of correctional officer cadets.

The City has plans to eventually expand the Pistol Range. Plans include a new building, a parking lot and a training center. The estimated capital costs for the improvements are $2 million. At this time, there is no funding available for this project. During the construction of the tertiary facilities, the new nonpotable well was expanded to provide additional capacity to serve the additional Pistol Range facilities.

No other future uses of the WWTP site are on the City's planning horizon.

3.7.8 Landscaping

Landscaping at the WWTP is minimal, consisting primarily of a small tree and planted area adjacent to the Control Building. Future landscaping needs may include perimeter screening along the edges of the WWTP property, as well as shade trees adjacent to parking areas.

The following improvements are recommended for site landscaping:

- Consider addition of perimeter landscaping
- Consider addition of shade trees adjacent to parking areas

Details regarding these recommended improvements – including the justifications, priority rankings, and estimated costs – are presented in the Existing Facilities Conditions Assessment TM (Appendix E).

3.8 REFERENCES

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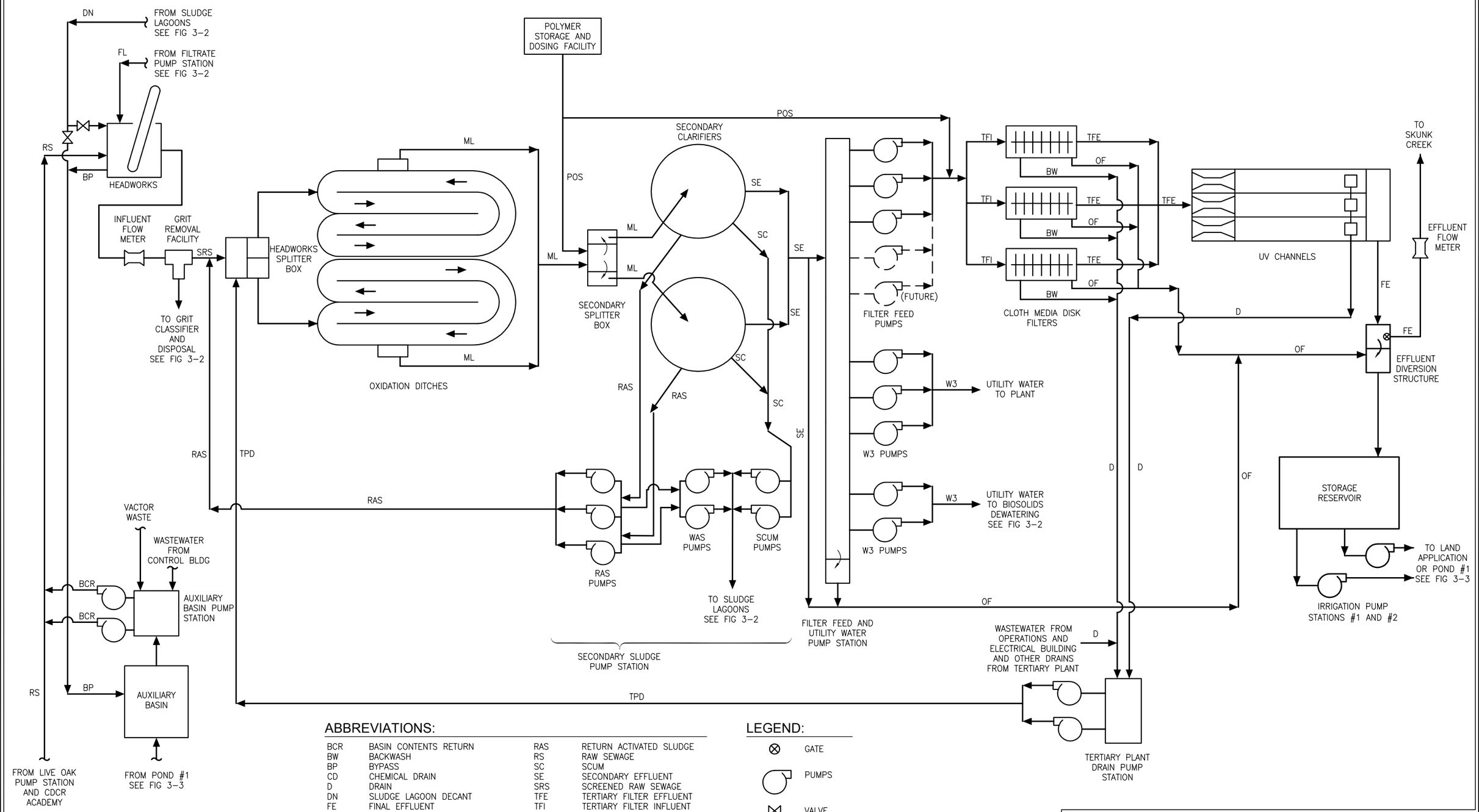


ABBREVIATIONS:

BCR	BASIN CONTENTS RETURN	RAS	RETURN ACTIVATED SLUDGE
BW	BACKWASH	RS	RAW SEWAGE
BP	BYPASS	SC	SCUM
CD	CHEMICAL DRAIN	SE	SECONDARY EFFLUENT
D	DRAIN	SRS	SCREENED RAW SEWAGE
DN	SLUDGE LAGOON DECANT	TFE	TERTIARY FILTER EFFLUENT
FE	FINAL EFFLUENT	TFI	TERTIARY FILTER INFLUENT
FL	FILTRATE	TPP	TERTIARY PLANT DRAINAGE
ML	MIXED LIQUOR	WAS	WASTE ACTIVATED SLUDGE
OF	OVERFLOW	W3	UTILITY WATER
POS	POLYMER SOLUTION		

LEGEND:

- GATE
- PUMPS
- VALVE

Figure 3-1

City Of Galt
WWTP Facilities Master Plan
LIQUID TREATMENT SCHEMATIC
(EXISTING)

WEST YOST ASSOCIATES
Consulting Engineers

N:\Clients\175 Galt\00-12-36 WWTP Facilities Master Plan\CAD\Figures\17500-1236-FIG-1-LIQUIDS.dwg 12/19/2012 11:11 AM chardy

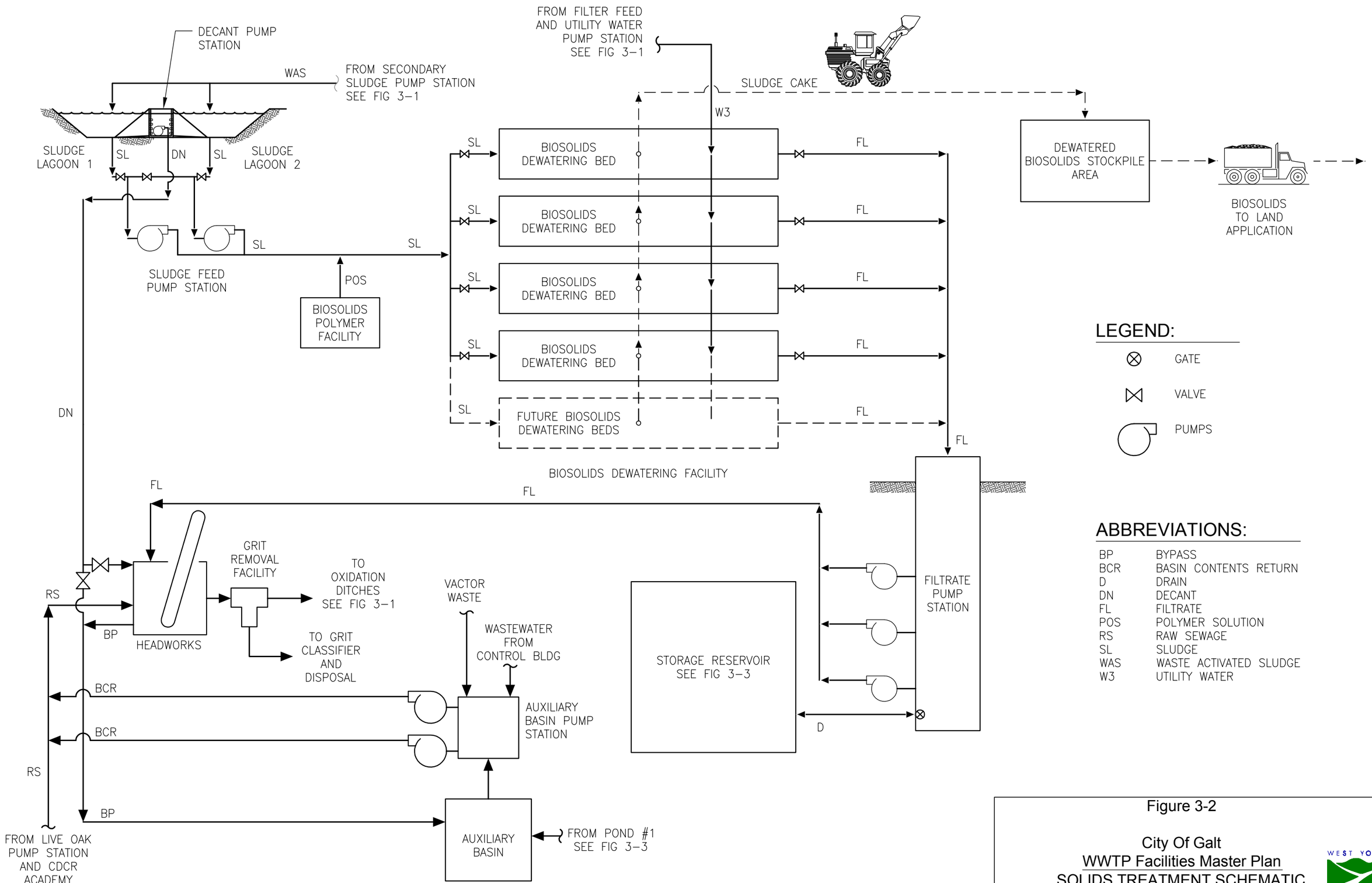


Figure 3-2

City Of Galt
WWTP Facilities Master Plan
SOLIDS TREATMENT SCHEMATIC
(EXISTING)

WEST YOST ASSOCIATES Consulting Engineers

N:\Clients\175 Galt\00-12-36 WWTP Facilities Master Plan\CAD\Figures\17500-1236-FIG-2-SOLIDS.dwg 12/19/2012 10:36 AM chardy

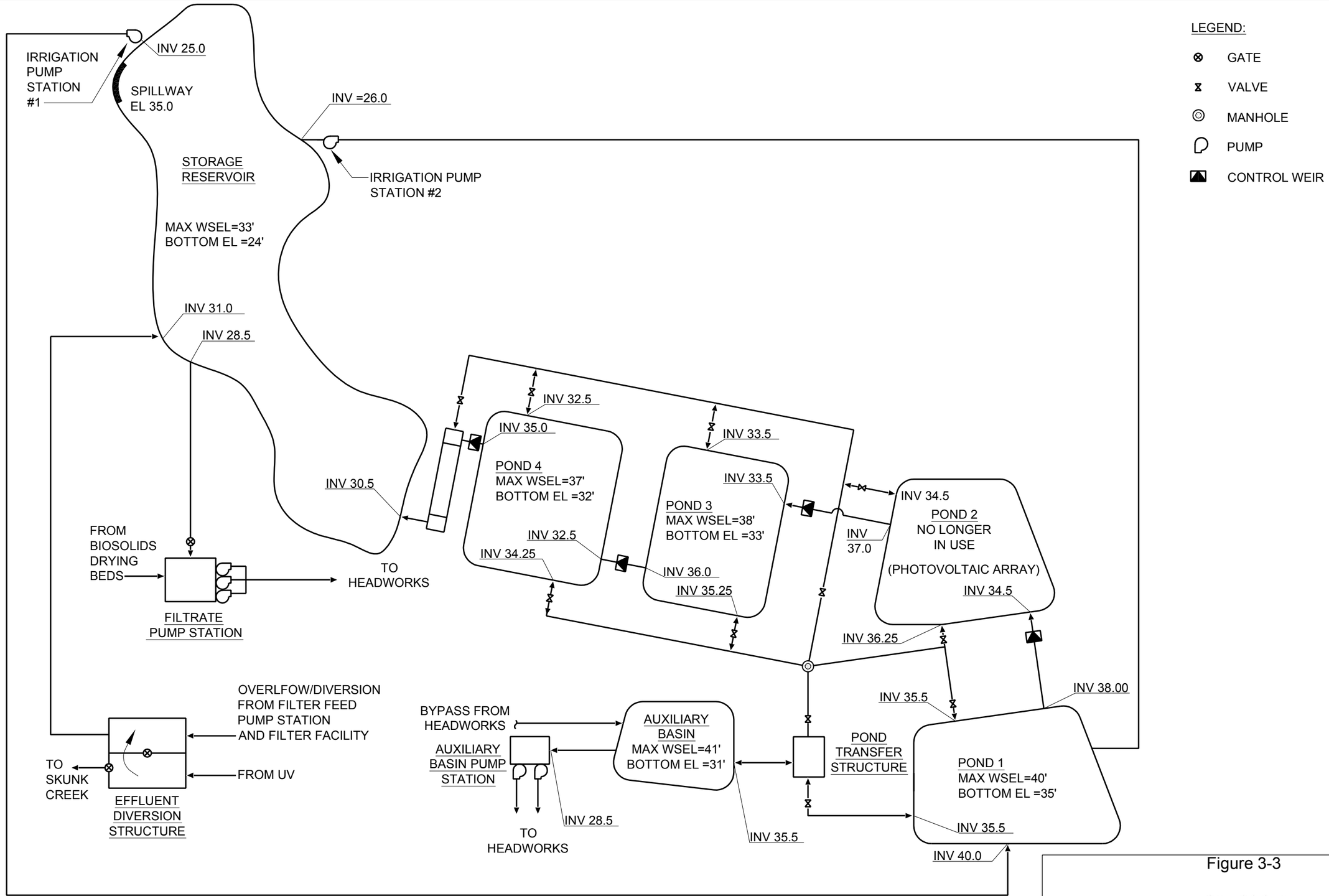


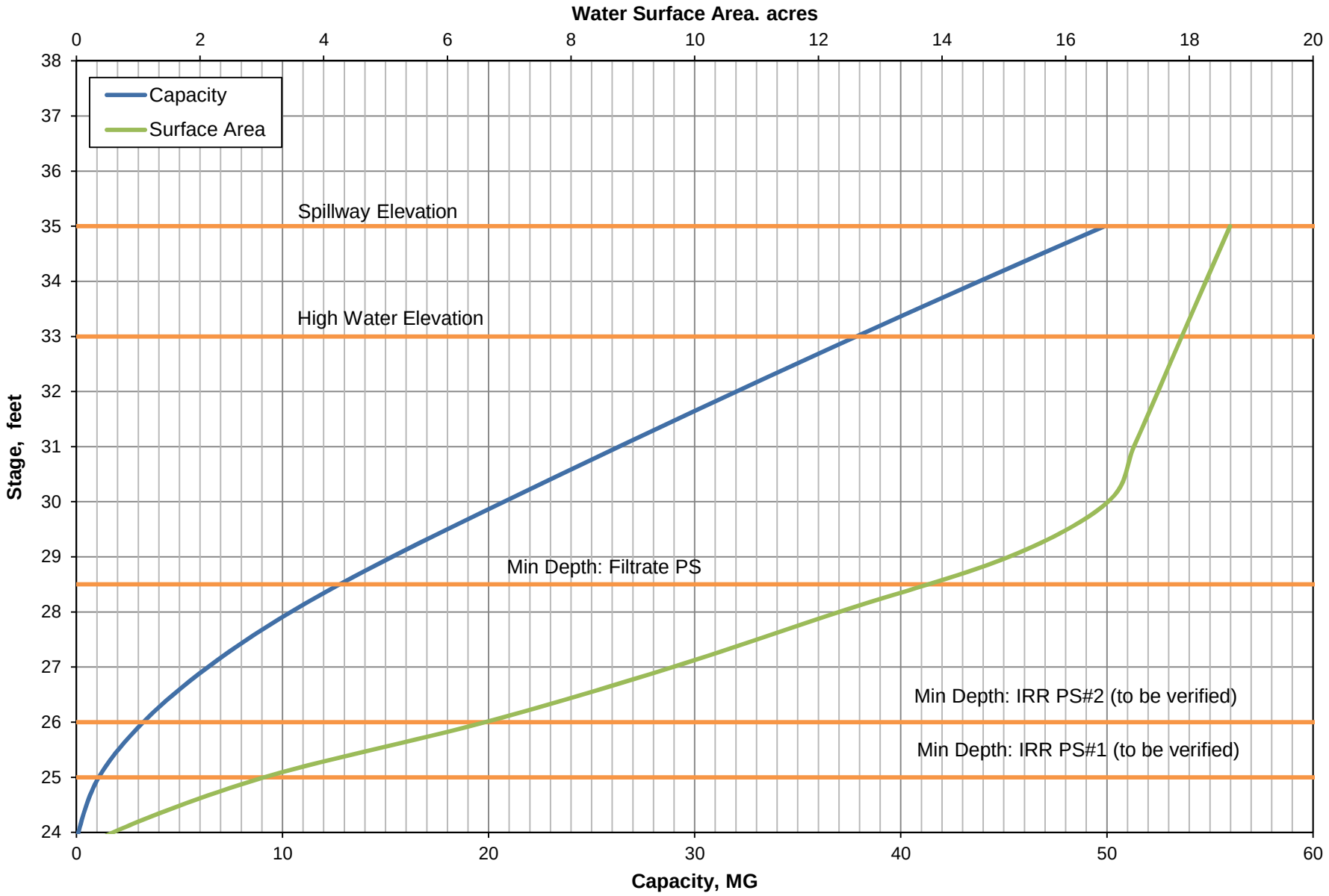
Figure 3-3

City Of Galt
WWTP Facilities Master Plan
POND SCHEMATIC

WEST YOST ASSOCIATES
Consulting Engineers

N:\Clients\175 Galt\00-12-36 WWTP Facilities Master Plan\CAD\Figures\17500-1236-Fig-3-POND SCHEMATIC.dwg 12-19-12 11:37:23 AM chardy

Figure 3-4. Storage Reservoir Capacity Curves



LEGEND

- Runoff Collection Ditches
- Irrigation Pump Stations
- Buried Distribution Pipeline
- Surface Ditches
- Monitoring Well
- City-Owned Properties
- Field Number

N

0 250 500
SCALE IN FEET

Figure 3-5

City of Galt
WWTP Facilities Master Plan
IRRIGATION FACILITIES LAYOUT

WEST YOST ASSOCIATES
Consulting Engineers

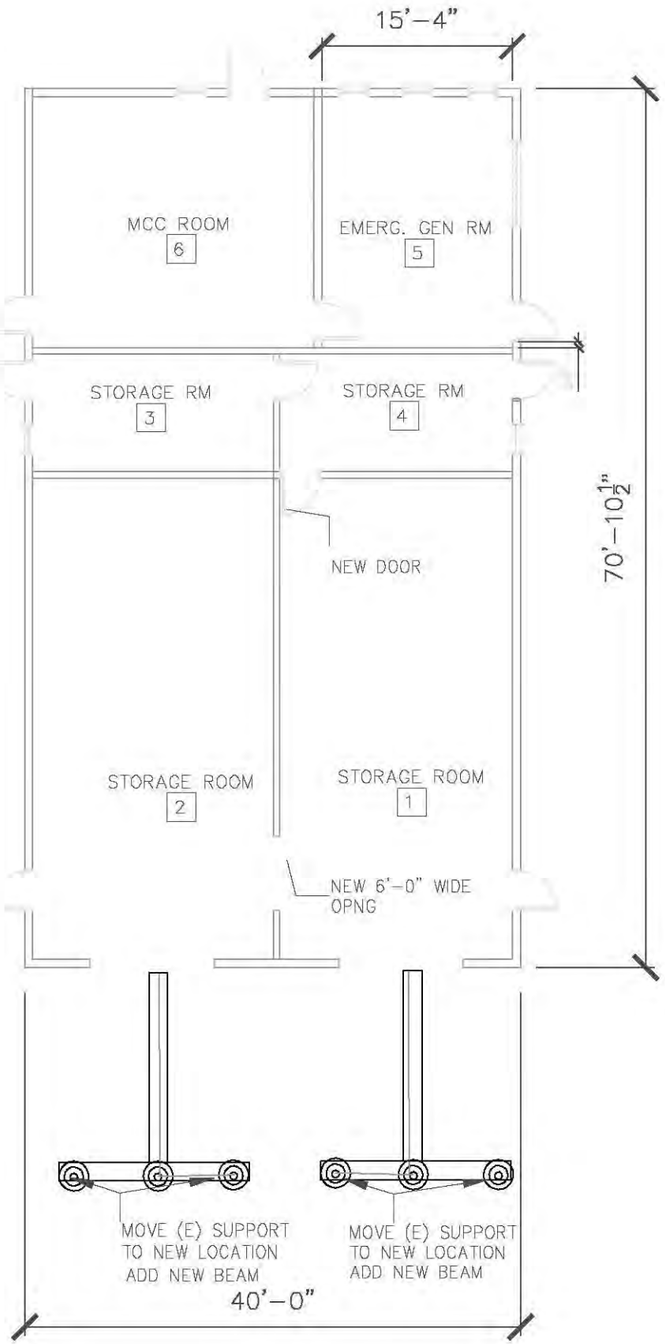
N:\Clients\175 Galt\00-12-36 WWTP Facilities Master Plan\CAD\Figures\17500-1236-FIG-5-IRRIGATION FACILITIES.dwg 12-19-12 09:23:08 AM chordy

Figure 3-6

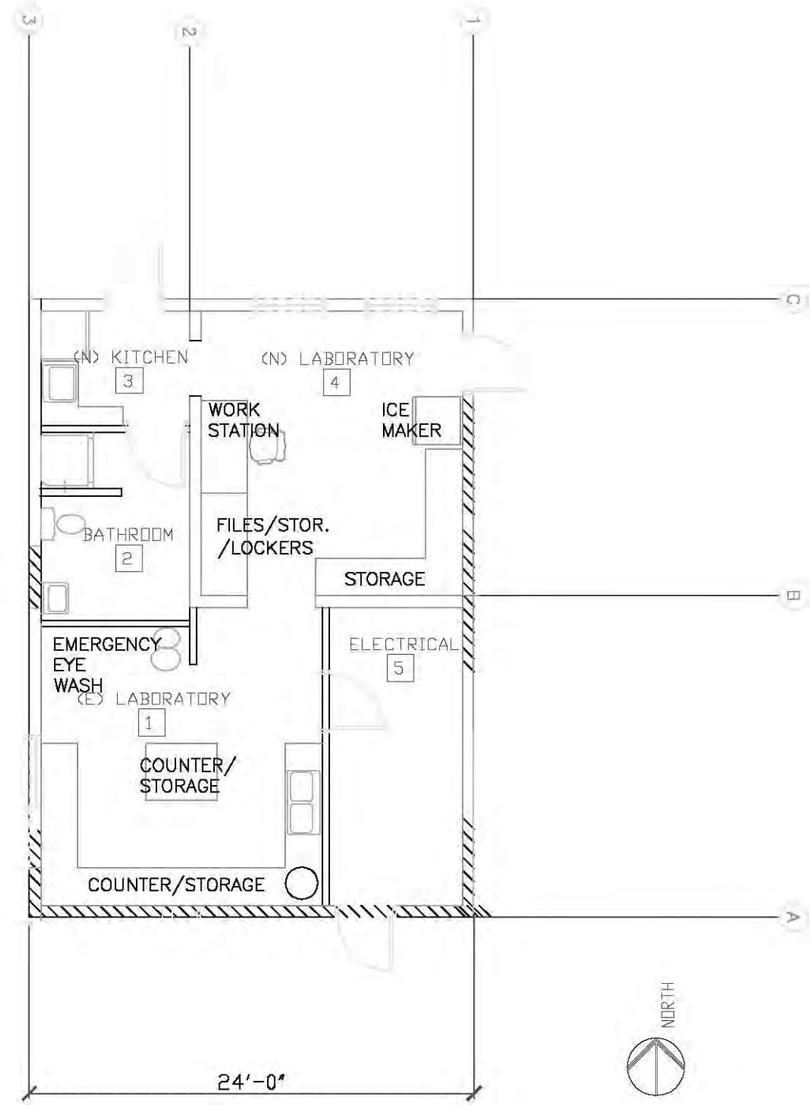
City of Galt
WWTP Facilities Master Plan
OPERATIONS & ELECTRICAL BUILDING
CONCEPTUAL DRAWING

WEST YOST ASSOCIATES
Consulting Engineers

H:\ACE MAIN\Jobs\10\1045 - WYA Galt WTP\Drawings\West Yost\17500-1236-O & E CONCEPTUAL-2007.dwg 7/2/2012 3:37 PM Cynthia



MAINTENANCE/CHEMICAL BUILDING
(PREVIOUSLY CHEMICAL BUILDING)
SCALE: 3/32"=1'-0"



CONTROL BUILDING
SCALE: 1/8"=1'-0"

Figure 3-7
City of Galt
WWTP Facilities Master Plan
MAINTENANCE/CHEMICAL BUILDING
AND CONTROL BUILDING
CONCEPTUAL DRAWING

WEST YOST ASSOCIATES
Consulting Engineers

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CHAPTER 4
Secondary Treatment Capacity Analysis and Plant Hydraulics

This chapter summarizes the results of an analysis of the current capacity of the WWTP secondary processes and the hydraulic capacities of the liquid stream processes. The major topics discussed in this chapter are as follows:

- Oxidation Ditch Treatment Capacity
- Secondary Clarifier Treatment Capacity
- Hydraulic Capacity

4.1 OXIDATION DITCH TREATMENT CAPACITY

Recent projects have demonstrated that, with proper equipment and controls in place, oxidation ditches can provide reliable ammonia and nitrate removal using simultaneous nitrification/denitrification. In a plug-flow reactor, nitrification and denitrification occur in two different basins: one catering to the autotrophic bacteria and the second to the heterotrophic bacteria. However, with strict control of dissolved oxygen levels and adequate detention time, nitrification and denitrification in one basin is possible.

The first design component for simultaneous nitrification/denitrification is to create an oxygen gradient by adding oxygen in one location in the basin. Near the injection point, a high DO concentration is maintained allowing for nitrification and oxidation of other organic compounds. The DO level in localized environments decreases with increasing distance from the injection point. In these low DO locations, the denitrification processes can occur.

The second key component for simultaneous nitrification/denitrification is aeration system control. Specifically, if too little or too much oxygen is added, the size of the nitrification or denitrification zone will be decreased, respectively, thereby reducing the treatment capacity. Recent advances in real-time ammonia and nitrate monitoring equipment have made simultaneous nitrification/denitrification more reliable. When used in combination with more typical DO monitoring, real-time ammonia and nitrate monitoring equipment allows for more accurate aeration adjustments as flows and loads change throughout the day.

Third, for simultaneous nitrification/denitrification to be successful, the extended MCRT and hydraulic detention times provided by oxidation ditches are necessary. Specifically, the high level of mixing and recycle that occurs in oxidation ditches allows the system to “absorb” the minor spikes in ammonia that can occur before the aeration system can adjust to changes in load conditions.

Finally, most oxidation ditch systems (including the City’s system) rely on the aerators to provide both forward movement and mixing of the wastewater, as well as aeration. Under a simultaneous nitrification/denitrification scenario, the aerators may be turned down during periods of low loading (to ensure denitrification will occur), resulting in adequate mixing. Therefore, oxidation ditches designed for simultaneous nitrification/denitrification should also be equipped with supporting mixing equipment that can be used during periods of low loading.

The only features discussed above that are not provided by the City's existing oxidation ditches is aeration system control and supplemental mixing. Specifically, the City is currently only able to operate the aerators at one of two speeds – resulting in some conditions where the system is either under or over aerated. Nevertheless, the City did observe simultaneous nitrification/denitrification activities in the oxidation ditches for a number of years. However, it appears that, under the current loading condition, the low aerator speed does not provide enough aeration to ensure reliable ammonia reductions, but the higher aerator speed imparts too much oxygen resulting in poor denitrification performance.

It is expected that the City could reliably achieve simultaneous nitrification/denitrification if a higher level of control is provided for the aeration system (and supplemental mixing is installed). A steady-state model, developed using the EnviroSim BioWIN® secondary process modeling program, was used to determine whether the existing oxidation ditches operating up to their maximum aeration capacity would have adequate capacity to achieve the ammonia and nitrate effluent limitations in the City's permit at the Current Permitted Design Condition with simultaneous nitrification/denitrification.

The major oxidation ditch performance assumptions and flow/load conditions that were evaluated under this effort are provided in Table 4-1. A detailed discussion of the data collection/analyses and calibration processes used to develop the BioWin® model, including the basis for the criteria presented in Table 4-1, are presented in the WWTP Secondary Process Modeling TM (Appendix B).

Note that the assumed design conditions shown in Table 4-1 take into account the current oxidation ditch and secondary clarifier conditions. As shown, the secondary process modeling is driven by the need to maintain an average and maximum Mixed Liquor Suspended Solids (MLSS) concentrations of 3,000 milligrams per liter (mg/L) and 3,500 mg/L, respectively. As discussed in the Secondary Clarifier Treatment Capacity section of this chapter, this is the highest range of MLSS concentrations that can likely be handled with the single-point solids draw-off clarifiers installed at the City's WWTP.

Also noted in Table 4-1 is that the waste activated sludge (WAS) flow rate was adjusted to ensure the MLSS does not exceed the target concentration for each scenario evaluated using the secondary process model. However, because the WWTP's oxidation ditches provide digestion of the biological solids in addition to performing biological treatment of the liquid stream, the solids should be treated in the oxidation ditches for at least 25 days, as measured by the mean cell residence time (MCRT). Therefore, the oxidation ditch evaluation also considers whether a target MCRT of 25 to 30 days can be achieved given the limitations on the MLSS concentrations.

Table 4-1. Existing Facilities Performance Criteria Assumptions for Secondary Process Modeling

Parameter	Value	Basis
Return Activated Sludge (RAS) Flow	Paced at 75 percent of influent flow – WAS Flow	Average calculated value, January 2011 to March 2012
WAS Flow, mgd (per treatment train)	0.027 – 0.060	Assume a constant flow rate. Flow value is set as needed to maintain desired MLSS concentrations. Note that data collected by the City indicates lower wasting rates than those predicted by the BioWIN model
MLSS Concentration, mg/L	3,000	Assumed average design MLSS condition (see Secondary Clarify discussion for more detail regarding the basis of this assumption)
	3,500	Assumed maximum design MLSS condition (see Secondary Clarify discussion for more detail regarding the basis of this assumption)
Mean Cell Residence Time (MCRT)	Limited to an amount that ensures the target MLSS listed above is not exceeded.	
Aeration Capacity (per surface aerator)	60 HP	The maximum aeration capacity based on information from the manufacturer
	3.5 lb O_2/HP	
Temperature, °C	21.6	The average value measured for June 2010 through March 2012
	17	Lowest monthly average temperature for June 2010 through March 2012
	14.5	Lowest daily temperature for June 2010 through March 2012
Influent Flow, mgd	3.0	Average Dry Weather Flow (ADWF) (from Chapter 2 (Basis of Design))
	3.9	Average Day Maximum Month Flow (ADMMF) (from Chapter 2)
Influent Loading Factor	1.0	Average Daily Load (ADL) (from Chapter 2) Assumed to Correlate with ADWF Conditions
	1.3 x ADL	Maximum Month Load (MML) (from Chapter 2) Assumed to Correlate with ADWF Conditions
	1.6 x ADL	Maximum Day Load (MDL) (from Chapter 2) Assumed to Correlate with ADMMF Conditions

A summary of the secondary process modeling results for the average and two "worst-case" conditions described in Chapter 2 (Basis of Design) for the Current Permitted Design Condition is provided in Table 4-2.

As shown in Table 4-2, the existing oxidation ditches appear to have adequate capacity to nitrify and denitrify the City's wastewater at the ADL and MML conditions; however, the effluent ammonia levels would likely be a concern at the maximum day load, cold temperature (i.e. 14.5°C conditions. To address this concern, additional process modeling was performed using increased aerator capacity to test the ability of the oxidation ditches to meet requirements with increased aeration. This additional modeling demonstrated that the ammonia limits could be satisfied under the worst-case maximum daily load/ADMMF condition shown in Table 4-2 if the aeration capacity is increased by 25 percent.

Finally, adjustments to the WAS flow rate to limit the MLSS concentration results in an MCRT that is much less than the recommended 25 to 30 day range needed to ensure Class B biosolids pathogen standards are satisfied. Therefore, this analysis demonstrates that, even with increased aeration capacity (and the ability to better control the aeration system), the existing oxidation ditches do not have adequate capacity to nitrify and denitrify the wastewater while still providing the MCRT needed for solids treatment.

The following alternatives for meeting the treatment needs at the 3 mgd design condition could be considered:

- Construct a third oxidation ditch;
- Construct a separate anoxic basin (with mixed liquor return) for denitrification, effectively increasing the overall basin volume;
- Reduce the organic loading to the oxidation ditches by providing primary treatment;
- Provide an alternative means of solids treatment so that the MCRT can be reduced without impacting the City's ability to meet Class B biosolids requirements; or
- Improve the solids removal capacity of the clarifiers (discussed further in the next section) so that the MLSS levels in the oxidation ditches can be increased.

In addition to the expansion needs, better aeration system control and supplemental mixing will need to be provided in the existing oxidation ditches. These process modifications are evaluated further in Chapter 6 (WWTP Treatment Train Upgrade and Expansion Alternatives). Reliability and redundancy criteria are also addressed in Chapter 6.

4.2 SECONDARY CLARIFIER TREATMENT CAPACITY

The WWTP's secondary clarifiers were modeled using a spreadsheet state point analysis tool, which provides a graphical means of predicting and evaluating clarifier dynamics based on a gravity flux curve (Daigger, 1995; Metcalf & Eddy, Inc., 1991). The state point analysis is a common method used to predict a typical clarifier's performance under a range of load (i.e., MLSS concentrations) and settleability (i.e., sludge volume index, or SVI) conditions.

Table 4-2. Secondary Process Model Results

Influent Conditions					Predicted Oxidation Ditch Conditions		Predicted Effluent Concentrations		
Load Condition	Load Factor	Flow Condition	Flow mgd	Temp. °C	MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L
MLSS Target = 3,000 mg/L									
ADL	1.0	ADWF	3.0	21.6	2,980	19	0.3	0.09	1.3
ADL	1.0	ADWF	3.0	17.0	2,970	19	0.4	9.4	1.2
MML	1.3 x ADL	ADWF	3.0	17.0	3,050	14	1.3	[a]	1.8
MDL	1.6 x ADL	ADMMF	3.9	14.5	3,030	11	9.2	[a]	2.3
MLSS Target = 3,500 mg/L									
ADL	1.0	ADWF	3.0	21.6	3,510	24	0.3	0.02	1. 3
ADL	1.0	ADWF	3.0	17.0	3,490	23	0.4	7.5	1.2
MML	1.3 x ADL	ADWF	3.0	17.0	3,510	17	1.2	[a]	1.8
MDL	1.6 x ADL	ADMMF	3.9	14.5	3,510	13	7.2	[a]	2.3

[a] The BioWIN® model predicts complete denitrification under this scenario (i.e. nitrate of 0 mg/L) due to the very low DO conditions that occur in portions of the oxidation ditch combined with the high recycle rates. In practice, it is expected that a high level of denitrification will occur, but that effluent nitrate levels will be greater than 0 mg/L.

As discussed in Chapter 3 (Existing Facilities), the WWTP clarifiers use plows to push sludge towards a single collection (or drawoff) point. In general, the maximum solids loading rate for single-point drawoff clarifiers is lower than for a multi-point drawoff clarifier for a given MLSS and/or SVI value (Water Environment Federation (WEF), 1998). Because the state point analysis method does not make a distinction relative to clarifier geometry and/or sludge drawoff, the method likely overestimates the capacity of the existing clarifiers. Therefore, the state point analysis results were compared to published curves that represent solids loading rates as a function of SVI for both single- and multi-point sludge drawoff clarifier types (Water Environment Federation (WEF), 1998). This comparison was achieved by plotting the solids loading rates calculated using the state point analysis tool at various SVI values against the published curves to determine the likelihood of failure.

The assumptions for the influent flow, RAS flow, MLSS concentration and SVI values used in this analysis are provided in Table 4-3. The SVI values shown (based on SVI data from the WWTP) are on the high end for municipal treatment plants. The high SVI values are likely a result of the high MCRTs that must be maintained in the oxidation ditches to ensure reliable pathogen reductions in the biosolids and the low food-to-microorganism ratios that result from the recirculation of the ditch contents.

Table 4-3. Bases of State Point Analysis Input Parameters

Parameter	Value	Basis
Current Influent Flows, mgd	2.1	Current ADWF
	2.6	April 2011 average flow, used for calibration to known clarifier failure event
	2.7	Current ADMMF
	4.0	Current PDWF
	6.3	Current PWWF
Permitted Influent Flows, mgd	3.0	Permitted ADWF
	3.9	Estimated ADMMF associated with permitted ADWF
	5.7	Estimated PDWF associated with permitted ADWF
	9.0	Estimated PWWF associated with permitted ADWF
RAS Flow, % of Influent Flow	75%	Average calculated value, January 2011 to March 2012
MLSS Concentration, mg/L	3,000	Target MLSS for oxidation ditches (Average recorded value from June 2010 to March 2012 is 2,900 mg/L)
	3,500	Likely maximum design MLSS for oxidation ditch improvements due to observed capabilities of existing clarifiers; (Approximately 75th percentile of recorded values, June 2010 to March 2012)
SVI, mL/g	190	Average recorded value, June 2010 to March 2012
	250	Approximately 85th percentile of recorded values, June 2010 to March 2012

Figure 4-1 and Figure 4-2 contain plots of the calculated solids loading rates for the range of loading conditions versus assumed SVI values, for current and permitted flows, respectively. The combined SVI and solids loading rate conditions that plot to the right of the "Maximum Solids Loading Rate, Single-Point Drawoff" and "Maximum Solids Loading Rate, Multi-Point Drawoff" curves on the figures represent conditions of excessive solids loading on the clarifiers – and therefore, a potential for either a clarifier thickening or clarification failure.

A thickening failure occurs when the required solids flux (i.e., solids removal rate) cannot be achieved due to inadequate thickening, and will result in a gradual increase in the sludge blanket until solids wash over the effluent weirs. Thickening failure is typically corrected by increasing the WAS flow rates (thereby reducing the solids inventory, or the MLSS concentration, in the secondary treatment process). Alternatively, the recycle rate can be increased for a corresponding lower underflow concentration to remove solids from the clarifier more quickly.

A clarification failure occurs when the solids loading to a clarifier exceeds the rate at which the clarifier can settle the solids for removal, and is typically the result of higher flows and/or MLSS concentrations than a clarifier was designed for. The state point analysis tool indicates that clarification failures are most likely to occur at the City's WWTP given the range of conditions evaluated.

Summaries of evaluated load and SVI conditions and the resulting predictions of clarifier performance based on the solids loading rate curves shown on Figure 4-1 and Figure 4-2 are provided in Table 4-4 and Table 4-5. As the tables and figures indicate, the existing clarifiers appear to have adequate capacity under certain flow, loading, and SVI conditions, while a number of conditions are predicted to result in excessive solids loading and inadequate performance.

Based on Figure 4-1 and Figure 4-2, the maximum solids loading rate to either clarifier should be limited to approximately 16 pounds per square foot per day (lb/ft2-d) when the SVI is 250 milliliters per gram (mL/g). This loading rate is well below the 3.0 mgd ADWF design loading rates from the 1990 WWTP expansion of 20 lb/ft2-d (sustained) and 40 lb/ft2-d (peak). Since future periods with sustained SVI values of 250 seem possible[1], it is recommended that determination of clarifier needs be based on an SVI of 250 and a maximum solids loading rate to the existing clarifiers of 26 lb/ft2-d. Improvements to the existing clarifiers to increase the maximum solids loading rate are discussed in Chapter 6 (WWTP Treatment Train Upgrade and Expansion Alternatives).

[1] The average SVI value was near 250 for a 9-month period from December 2010 through September 2011.

Table 4-4. Secondary Clarifier State Point Analysis Summary - Current Flows

Flow Conditions	Influent Flow, mgd	Overflow Rate, gpd/ft^2	Underflow Rate[(d)], gpd/ft^2	MLSS, mg/L	SVI[(a)], mL/g	Solids Loading Rate, lb/ft^2-d	Underflow Concentration, mg/L	Predicted Performance[(c)]	Failure Mode Predicted by State Point Analysis
One Unit in Service									
April 2011[(b)]	2.6	511	394	3,506	251	27	8,100	Probable overload	Near thickening and clarification failure
ADWF	2.1	420	315	3,000	190	18	7,000	Adequate	Adequate
				3,500	250	22	8,200	Probable overload	Adequate, but near capacity
ADMMF	2.7	537	403	3,000	190	24	7,000	Adequate	Adequate
				3,500	250	28	8,200	Probable overload	Thickening failure
PDWF	4.0	796	597	3,000	190	35	7,000		Adequate, but near capacity
				3,500	250	41	8,200		Clarification failure
Two Units in Service									
ADWF	2.1	210	157	3,000	190	9	7,000	Adequate	Adequate
				3,500	250	11	8,200		
ADMMF	2.7	269	201	3,000	190	12	7,000		
				3,500	250	14	8,200		
PDWF	4.0	398	298	3,000	190	17	7,000		
				3,500	250	20	8,200	Probable overload	Adequate, but near capacity
PWWF	6.3	630	472	3,000	190	28	7,000	Marginal	Adequate
				3,500	250	32	8,200	Probable overload	Clarification failure

(a) Assumes SVI results based on unstirred mixed liquor samples.
(b) The WWTP experienced a clarifier failure (solids washout) in April 2011, when one clarifier was briefly taken offline. Influent flow, RAS flow, MLSS and SVI values for this condition are based on averages of data recorded by plant operators during the month of April 2011.
(c) Performance predictions based on more conservative of state point analysis diagrams and design solids loading rates for single-point sludge drawoff clarifiers as a function of SVI (WEF, Manual of Practice 8 – Design of Municipal Wastewater Treatment Plants, 4th ed., 1998, pp. 586).
(d) Based on RAS flow of 75 percent of influent flow.

Table 4-5. Secondary Clarifier State Point Analysis Summary - Permitted Flows

Flow Conditions	Influent Flow, mgd	Overflow Rate, gpd/ft^2	Underflow Rate[(a)], gpd/ft^2	MLSS, mg/L	SVI[(b)], mL/g	Solids Loading Rate, lb/ft^2-d	Underflow Concentration, mg/L	Predicted Performance[(c)]	Failure Mode Predicted by State Point Analysis
One Unit in Service									
ADWF	3.0	597	448	3,000	190	26	7,000	Adequate	Adequate
				3,500	250	31	8,200	Probable overload	Clarification failure
ADMMF	3.9	776	582	3,000	190	34	7,000		Adequate
				3,500	250	40	8,200		Clarification failure
PDWF	5.7	1,134	850	3,000	190	50	7,000		Clarification failure
				3,500	250	58	8,200		Clarification failure
Two Units in Service									
ADWF	3.0	298	224	3,000	190	13	7,000	Adequate	Adequate
				3,500	250	15	8,200		
ADMMF	3.9	388	291	3,000	190	17	7,000		
				3,500	250	20	8,200	Probable overload	Adequate
PDWF	5.7	567	425	3,000	190	25	7,000	Adequate	Adequate
				3,500	250	29	8,200	Probable overload	Clarification failure
PWWF	9.0	895	671	3,000	190	39	7,000	Probable overload	Near clarification failure
				3,500	250	46	8,200	Probable overload	Clarification failure

(a) Based on RAS flow of 75% of influent flow.
(b) Assumes SVI results based on unstirred mixed liquor samples.
(c) Performance predictions based on more conservative of state point analysis diagrams and design solids loading rates for single-point sludge drawoff clarifiers as a function of SVI (WEF, Manual of Practice 8 – Design of Municipal Wastewater Treatment Plants, 4th ed., 1998, pp. 586).

Under current flow conditions (i.e., flows associated with an ADWF of 2.11 mgd), the clarifiers appear to have adequate capacity when historical average MLSS concentrations and sludge settling characteristics are present. Moreover, the analysis suggests that the WWTP may be able to operate with one clarifier out of service during dry weather, provided that MLSS and SVI values are kept near the historical averages and that the peak dry weather flow is not approached. Conversely, this capability is diminished as MLSS and SVI values increase, as demonstrated by the modeled results for the April 2011 flow conditions, and as corroborated by the City's experience during the same month[2]. It is noteworthy that under April 2011 average conditions with one clarifier out of service, the curves on Figure 4-1 indicate that a single clarifier with a multi-point sludge drawoff system would be just beyond its capacity, while a clarifier with a single-point drawoff system would be well beyond its capacity.

As the plant nears an ADWF of 3.0 mgd, the existing clarifiers are predicted to experience problems under certain peak flow conditions and are predicted to operate inadequately when faced with high MLSS and SVI values. The analysis indicates that the WWTP could not be operated with a single clarifier in service during peak dry weather flow conditions, impacting the City's ability to perform necessary repairs or maintenance on either unit.

The deficiencies described above can be addressed by constructing additional clarifiers and potentially retrofitting the two existing clarifiers to provide multi-point draw-off. Clarifier capacity is limited by the SVI, so process changes that improve the SVI will improve the performance of each clarifier in operation.

4.3 HYDRAULIC CAPACITY

The hydraulic capacity of the WWTP is affected by the sizes of pipelines, flumes, weirs, and other hydraulic structures. As WWTP flows increase, the head losses through each unit process increase, resulting in higher water levels. Maximum allowable flows are those flows that can be safely contained (i.e., that do not create overflows or unacceptable operating conditions). Development of a hydraulic model and accompanying hydraulic profile schematics facilitates the identification of the acceptable hydraulic capacity of individual unit processes and the WWTP as a whole.

4.3.1 Spreadsheet Model Development

A spreadsheet hydraulic model was developed for the WWTP as part of the Tertiary Filtration and Biosolids Dewatering Improvements Project in 2009. The model was updated and expanded for the Facilities Master Plan work.

The hydraulic model follows a downstream-to-upstream convention. Boundary conditions (i.e., water surface elevations) at the Skunk Creek outfall and in the Storage Reservoir are the starting points for the model, depending on the discharge condition. Head losses from sequential upstream processes are added to the boundary conditions to develop the hydraulic profile of the WWTP.

[2] One clarifier was briefly taken out of service, and the remaining clarifier quickly became overloaded, resulting in a solids washout to the downstream tertiary processes.

4.3.2 Available Data and Assumptions

Inputs for the hydraulic profile are based on available data and a limited number of assumptions. Record drawings from construction projects in 1990 (primary and secondary treatment expansion), 2007 (grit removal and flow metering), 2008 (effluent bypass), and 2010 (tertiary improvements) are used as the basis for the types, dimensions and elevations of pipelines, fittings, channels, weirs, flumes, and other related hydraulic features of the WWTP. In addition, hydraulic performance data from manufacturers are used where typical hydraulic calculations cannot be readily applied to determine head losses (e.g., losses across UV banks, tertiary disc filter cartridges, vortex grit chambers and bar screens). Estimates of typical hydraulic parameters for pipelines, fittings, and structures (i.e., friction and minor loss coefficients) have been made based on standard industry practice.

The assumed numbers of units in service, as well as the sidestream and return activated sludge (RAS) flow assumptions, for each of the scenarios developed in the hydraulic model are presented in Table 4-6. For sidestream and RAS flow rates, the hydraulic calculations are based on maximum instantaneous rates. For processes in which more than one unit currently exists, the hydraulic calculations are based on the head losses to and from the most remote unit.

Table 4-6. WWTP Flow vs. Process Units Assumed to be in Service

WWTP Influent Flow, mgd	Instant-aneous Side-stream Flow[(a)], mgd	RAS Flow[(b)], mgd	Headworks Channels	Vortex Grit Chambers	Oxidation Ditches	Clarifiers	Filter Units	UV Channels	Comments
2.11	1.40	3.17	1	1	2	2	2	2	Current ADWF
3.00	1.40	4.50	1	1	2	2	2	2	Current Permitted ADWF
9.00	1.40	4.50	1	1	2	2	3	3	Current PWWF
4.50[(c)]	1.40	6.75	1	1	3	3	3	3	Future ADWF
6.00[(c)]	1.40	9.08	2	2	4	4	5	5	Buildout ADWF
13.50[(c)]	1.40	6.75	1	1	3	3	5	5	Future PWWF
18.00[(c)]	1.40	9.08	2	2	4	4	7	7	Buildout PWWF

(a) Instantaneous sidestream flow assumed to be the sum of the Sludge Lagoon Decant pump and the firm capacity of the Filtrate Pump Station (450 and 520 gpm, respectively). According to City staff, typical daily volumes returned from Sludge Lagoons and Filtrate Pump Station are 25,000 and 100,000 gallons, respectively.
(b) Maximum RAS flow assumed to be 150 percent of ADWF; typical RAS flow is approximately 75 percent of ADWF.
(c) From Chapter 2 (Basis of Design).

For examining peak flow conditions of the existing WWTP as part of developing this chapter, the existing filters and ultra violet (UV) disinfection systems are assumed to operate without a standby unit or channel when WWTP influent flow exceeds 6.0 mgd, allowing each of the three installed filters and UV channels, respectively, to treat a peak of 3.0 mgd. During periods of time when a filtration unit or disinfecting channel is offline, a portion of flow may be diverted to either the Auxiliary Basin (raw influent) or to the Storage Reservoir (secondary effluent).

For future scenarios (based on ADWFs of 4.5 and 6.0 mgd), an adequate number of filters and UV channels are assumed to be available for limiting the flow per filter or channel to 1.5 mgd and 3.0 mgd under ADWF and PWWF scenarios, respectively.

4.3.3 WWTP Hydraulic Capacity Results

The hydraulic profile for the existing WWTP is shown on Figure 4-3 and Figure 4-4 for influent flows of 2.1, 3.0, and 9.0 mgd. For the 9.0 mgd PWWF scenario, 100-year flood conditions are assumed, resulting in a high water elevation (estimated to be 35.0 feet) in both Skunk Creek and the Storage Reservoir and the worst-case hydraulic conditions throughout the WWTP. For non-PWWF scenarios, a water surface elevation of 25.0 feet is assumed in Skunk Creek and the Storage Reservoir.

As indicated on Figure 4-3 and Figure 4-4, there are no overflow conditions for the flows listed in Table 4-6, with the exception of the weir inside the Effluent Diversion Structure at the assumed 100-year flood elevation of 35.0 feet. In this instance, a portion of the treated effluent would overflow the weir inside the Effluent Diversion Structure and discharge to the Storage Reservoir. The elevated hydraulic grade line and resulting weir overflow is a result of excessive head losses in the terminal section of the final effluent pipeline, just downstream of the Outfall Junction Structure. Since the Storage Reservoir spillway elevation is 35.0 feet, any effluent discharged to the reservoir via the Effluent Diversion Structure under this condition will result in the reservoir discharging over the spillway, which is not currently allowed per the City's discharge permit (the WDRs).

As indicated on Figure 4-3, there is minimal free fall (approximately 3 inches) over the Headworks Splitter Box weirs under the 9.0 mgd WWTP influent flow condition, indicating that the existing structure cannot handle much more than 4.5 mgd per operating clarifier.

The hydraulic profile at future flows is presented on Figure 4-5 and Figure 4-6 for influent flows of 4.5, 6.00, 13.5, and 18.00 mgd. For the 13.50 and 18.00 mgd PWWF scenarios, 100-year flood elevations are assumed.

As indicated on Figure 4-5 and Figure 4-6, several overflow conditions may result at future flows without hydraulic improvements to the WWTP. A discussion of the predicted hydraulic deficiencies under future conditions is provided below. The facilities with predicted deficiencies are listed in Table 4-7, along with recommended improvements to correct the deficiencies.

Table 4-7. Predicted Future Hydraulic Deficiencies and Recommended Improvements

Location	Predicted Deficiency	Recommended Improvement	Trigger for Improvements
Pipe between Grit Removal Facility and Headworks Splitter Box	Excessive head loss during peak flows resulting in overflows at Grit Removal Facility and Headworks	· Increase above grade piping to 30” · Increase buried piping between RAS junction and splitter box to 42”	Prior to: · PWWF = 13.5 mgd[(b)]
Pipe between Headworks Splitter Box and Oxidation Ditches	Excessive head loss during peak flows resulting in weir submergence at Headworks Splitter Box	· Install 36” pipe between splitter box and future oxidation ditches	Prior to: · PWWF = 13.5 mgd[(b)]
Outfall Pipe between Outfall Junction Structure and Skunk Creek	Excessive head loss during peak flows resulting in overflow of Effluent Diversion Structure weir	· Increase pipe to 36” (or eliminate 16” piping after constructing new effluent flow monitoring station[(a)])	Prior to: · PWWF = 13.5 mgd[(b)]
Overflow Pipe between Filter Feed Pump Station and Effluent Diversion Structure	Excessive head loss during peak flows resulting in potential overflow of Filter Feed Pump Station weir	· Construct extension of 30” overflow pipe to Pond 4	Prior to: · PWWF = 13.2 mgd

(a) Chapter 3 (Existing Treatment Plant Performance and Condition) includes a discussion of the potential replacement of the existing effluent flow monitoring facilities with a new structure closer to the Effluent Diversion Structure.
(b) Equivalent to an ADWF of 4.5 mgd (Future Permitted Design Condition).

4.3.3.1 Pipe between Grit Removal Facility and Headworks Splitter Box

The pipe connecting the Grit Removal Facility to the Headworks Splitter Box ranges in diameter from 20 to 30 inches. Above-grade piping at the Grit Removal Facility is 20 inches in diameter, before enlarging to 30 inches and joining a tee that will eventually be connected to effluent from a second vortex grit system. The remainder of the 30-inch pipe is buried. Approximately 20 feet upstream of the splitter box entrance, a 12-inch RAS pipe connects into the 30-inch pipe.

Due to its location in the process, the existing pipe must convey the largest flows the WWTP may experience: WWTP influent, RAS, and sidestream flows discharged to the Headworks. Under the 9.0 mgd scenario, head losses in this section of pipe are 2.12 feet, enough to submerge the vortex grit chamber effluent shelf as depicted on Figure 4-3, but not enough to overtop the grit facility walls. At an influent flow of 13.5 mgd (the PWWF associated with an ADWF of 4.5 mgd), the losses in this section of pipe will increase to 4.33 feet (assuming that a second vortex grit chamber has not been constructed). The resulting hydraulic grade line (HGL) will exceed the tops of the grit facility structure and the Headworks, resulting in one or more overflows. The outcome is exaggerated at an influent flow of 18.00 mgd (total head loss of

4.66 feet), even with an assumption that a second vortex grit chamber will have been constructed by the time WWTP influent flows reach this level.

Addressing this deficiency will likely require modification to the existing above-grade and buried piping. Increasing the above-grade pipes to 30 inches will lower the HGL to just above the top of the grit facility structure. Increasing the buried portion of the pipe between the RAS junction and the splitter box to 42 inches will lower the HGL to approximately 1.3 feet below the top of the grit facility structure, eliminating the risk of overflow. However, the vortex grit chamber effluent shelf would remain submerged under this condition, although this should not significantly impact grit removal performance.

4.3.3.2 Pipe between Headworks Splitter Box and Oxidation Ditches

The pipes leaving the Headworks Splitter Box to the Oxidation Ditches are currently 30 inches in diameter, and only two pipes are installed. Two 30-inch pipe stubs are included for connections to future oxidation ditches. As indicated on Figure 4-5, the Headworks Splitter Box weirs will be slightly submerged under the 13.5 and 18.0 mgd scenarios, which will impact the flow splits between the existing and future oxidation ditches. The hydraulic profile includes rough assumptions about the distance (550 feet) and number of fittings (six 90-degree bends) required to connect future ditches to the splitter box.

Because future oxidation ditches cannot likely be placed near enough to the Headworks Splitter Box to eliminate enough head losses to avoid weir submergence at peak flows, this deficiency can be addressed by constructing connecting pipes that are larger than 30 inches in diameter. The hydraulic profile model indicates that 36-inch pipes between the splitter box and any future oxidation ditches would provide a minimum weir free fall of approximately 4 inches; the existing 30-inch pipes connecting the existing oxidation ditches to the splitter box can remain.

4.3.3.3 Final Effluent Pipe Leaving the Outfall Junction Structure

The pipe leaving the Outfall Junction Structure to Skunk Creek is 16 inches in diameter and is approximately 24 feet long. The relatively new effluent pipeline upstream of the Outfall Junction Structure is 36-inch diameter. Assuming a Skunk Creek water surface elevation of 25.0 feet, the smaller pipeline downstream of the junction structure will result in overflows at the Outfall Junction Structure under several flow scenarios. (Under estimated 100-year flood conditions, the structure would already be under water from the creek).

At 13.5 mgd, losses through the 16-inch section of pipe exceed 8 feet[3], resulting in an HGL of over 33 feet at the Outfall Junction Structure. The top of the Outfall Junction Structure is at 30.0 feet, meaning that some effluent will escape through the access hatch in the structure. Depending on the amount of water that escapes through the Outfall Junction Structure access hatch, the upstream 36-inch effluent pipe could remain full under this condition, and an overflow at the Effluent Diversion Structure weir is possible.

[3] Assumes that all effluent leaves through the 36” and 16” outfall to Skunk Creek. In reality, effluent will begin overflowing the Effluent Diversion Structure weir at 35.0 feet, and the overflow will balance such that the head losses between the two outfall pipes (one to Skunk Creek, one to the reservoir) are equal.

Under the 18.00 mgd scenario, losses through the 16-inch section of pipe exceed 13 feet, resulting in an HGL of over 38 feet at the Outfall Junction Structure and nearly 43 feet inside the Effluent Diversion Structure (which will result in tertiary effluent flowing over the weir inside the structure and being sent to the Storage Reservoir).

This deficiency can be addressed by replacing the 16-inch section of pipe with a larger diameter pipe. Increasing the diameter to 36 inches will keep the HGL below the top of the Outfall Junction Structure under the 18.00 mgd scenario (assuming a Skunk Creek water surface elevation of 25.0 feet), and will also keep the HGL within the 36-inch effluent pipe at the Effluent Diversion Structure, eliminating any risk of overflow to the Storage Reservoir. Raising the top of the Outfall Junction Structure above 35.0 feet would ensure that effluent does not overflow at the structure even under 100-year flood conditions.

4.3.3.4 Overflow Pipe Upstream of Effluent Diversion Structure

The overflow pipe between the Effluent Diversion Structure and the Filter Feed Pump Station, which is intended to convey overflow from the Filter Feed Pump Station and/or the Filter Facility, varies in diameter from 36 to 42 inches. The section of pipe between the Effluent Diversion Structure and the Filter Feed Pump Station was originally designed to be entirely 42 inches, but during construction of the tertiary facilities in 2010, the City elected to reuse approximately 240 feet of existing 36-inch pipe that previously connected the Chlorine Contact Basin with the Effluent Diversion Structure. In order to compensate for the lost hydraulic capacity, a 30-inch stub was added to the manhole that was constructed at the junction of the old 36-inch and new 42-inch overflow pipes. The intent of the 30-inch pipe stub was to allow for the construction of a 30-inch overflow pipe to the adjacent Pond 4, which would provide a passive overflow mechanism if the HGL inside the 36-inch pipe becomes too high to prevent upstream hydraulic deficiencies. The invert of the 30-inch stub was located to prevent the HGL inside the manhole from exceeding 35.9 feet, which will result in a free fall over the Filter Feed Pump Station overflow weir (assuming the cause of the overflow is a shutdown of the filter feed pumps) of approximately 6 inches.

The extension of the 30-inch pipe to Pond 4 should be constructed before the WWTP's PWWF would result in an HGL in the manhole of greater than 35.9 feet. Under a 100-year flooding scenario (water surface elevations of 35.0 feet in the Storage Reservoir and Pond 4), the existing 36-inch overflow pipe is predicted to accommodate approximately 13.2 mgd before the 30-inch overflow is required. If Pond 4 is empty and the water surface elevation in the Storage Reservoir is less than 31.0 (the invert of the outfall from the Effluent Diversion Structure), the existing 36-inch overflow pipe is predicted to accommodate approximately 19.4 mgd before the 30-inch overflow is required.

4.4 REFERENCES

Daigger, G.T., "Development of Refined Clarifier Operating Diagrams Using an Updated Settling Characteristics Database." Water Environment Research, Vol. 67, No. 1, p. 95, January/February 1995.

Metcalf & Eddy, Inc., G. Tchobanoglous, F.L. Burton, *Wastewater Engineering: Treatment, Disposal, and Reuse, Third Edition*. McGraw Hill, Inc. San Francisco, CA. 1991.

Water Environment Federation, *Design of Municipal Wastewater Treatment Plants: WEF Manual of Practice No. 8; ASCE Manual and Report on Engineering Practice, No. 76, Fourth Edition.* American Society of Civil Engineers. 1998.

Figure 4-1. Clarifier Solids Loading Rates vs. SVI, Current Flows

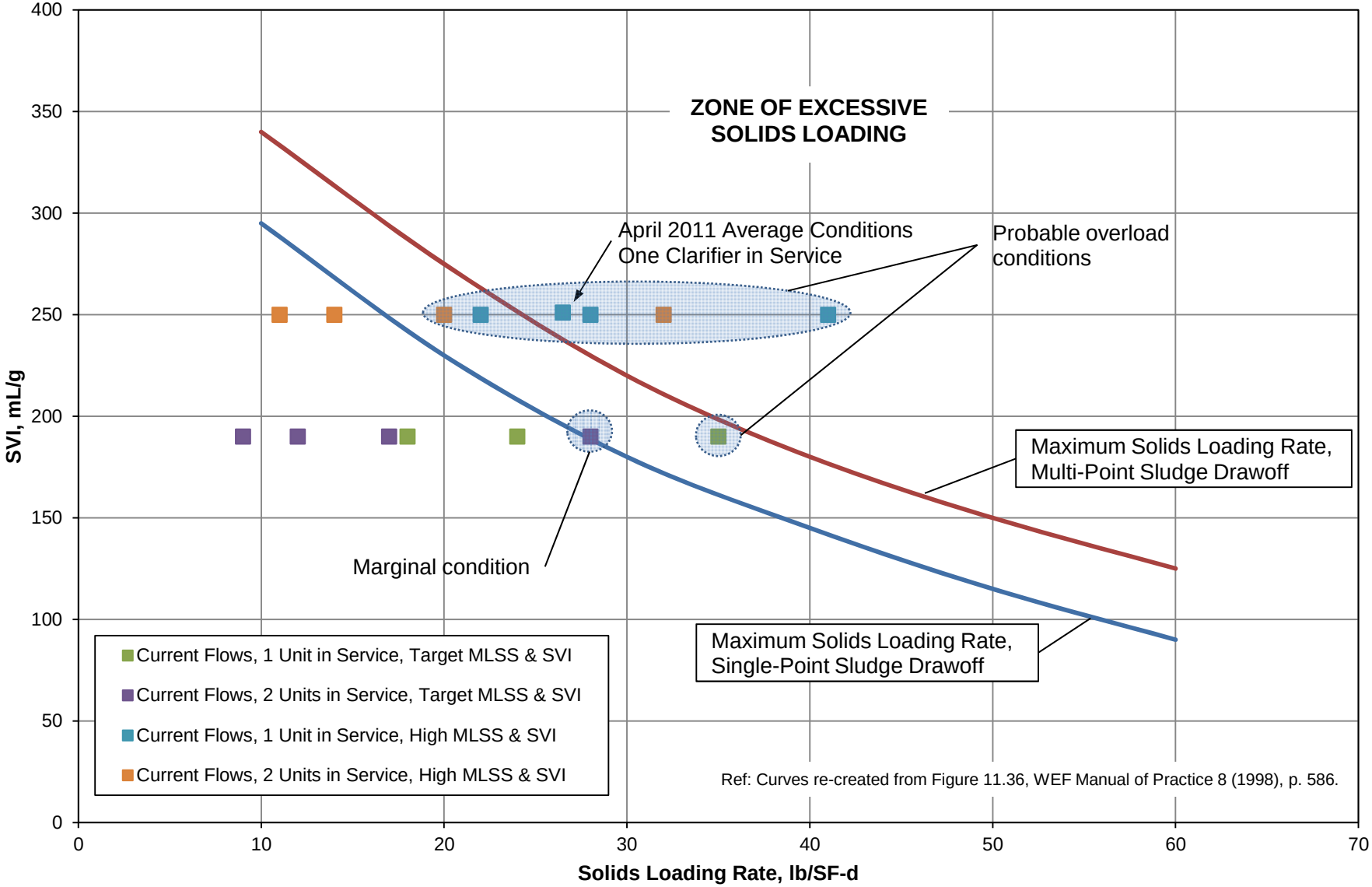
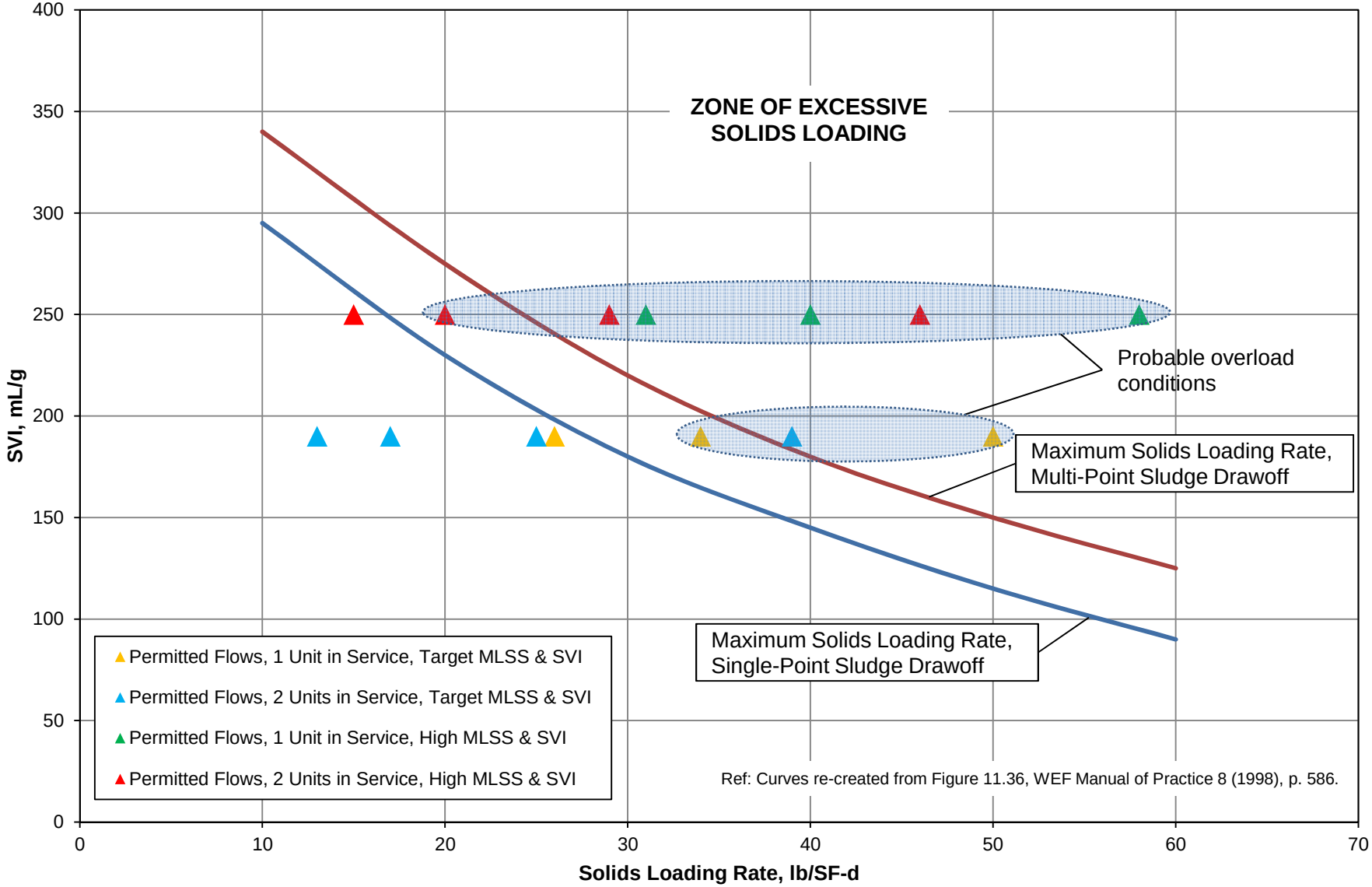
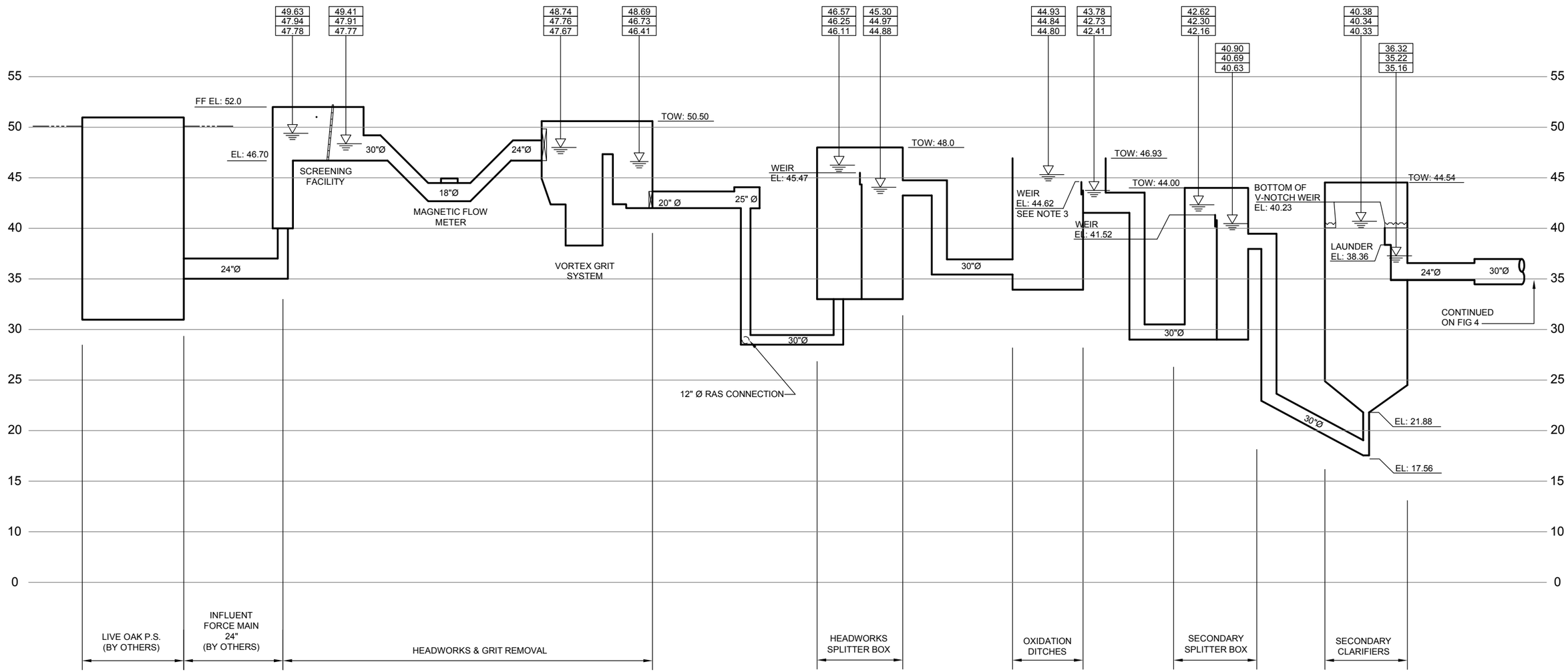


Figure 4-2. Clarifier Solids Loading Rates vs. SVI, Permitted Flows





LEGEND

****	WATER SURFACE ELEVATION, PWWF (9.0 MGD), 100-YR FLOOD CONDITIONS
****	WATER SURFACE ELEVATION, CURRENT PERMITTED ADWF (3.0 MGD)
****	WATER SURFACE ELEVATION, CURRENT ADWF (2.1 MGD)

Figure 4-3

City Of Galt
WWTP Facilities Master Plan
EXISTING HYDRAULIC PROFILE 1

WEST YOST ASSOCIATES
Consulting Engineers

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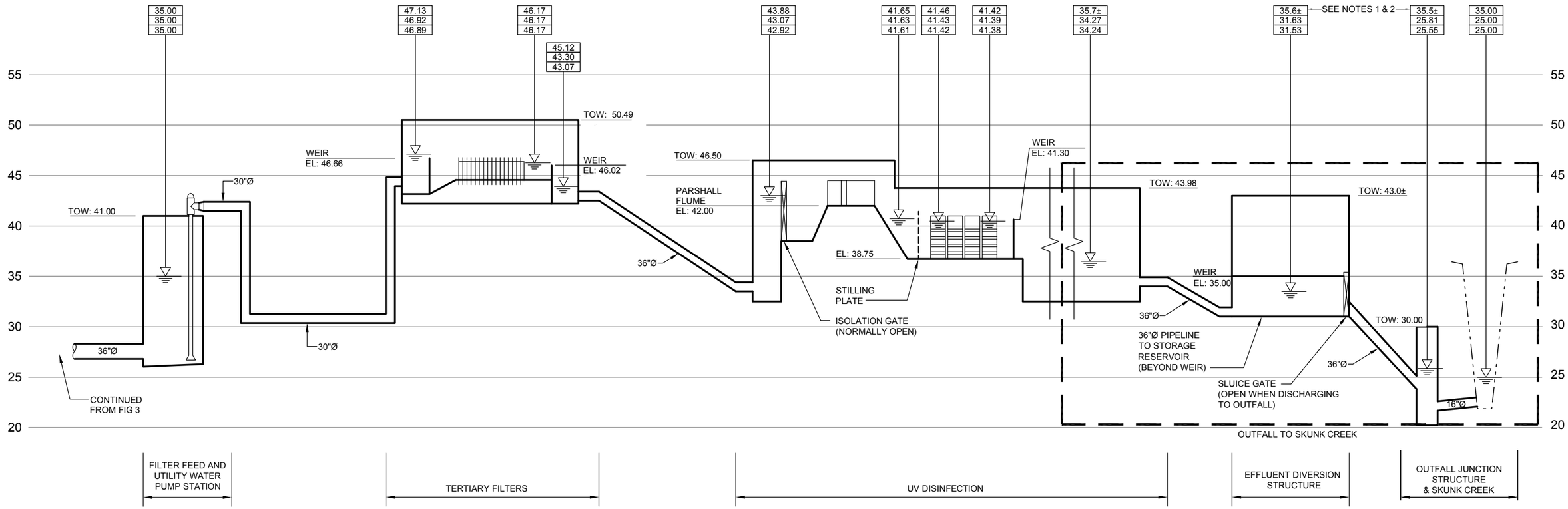

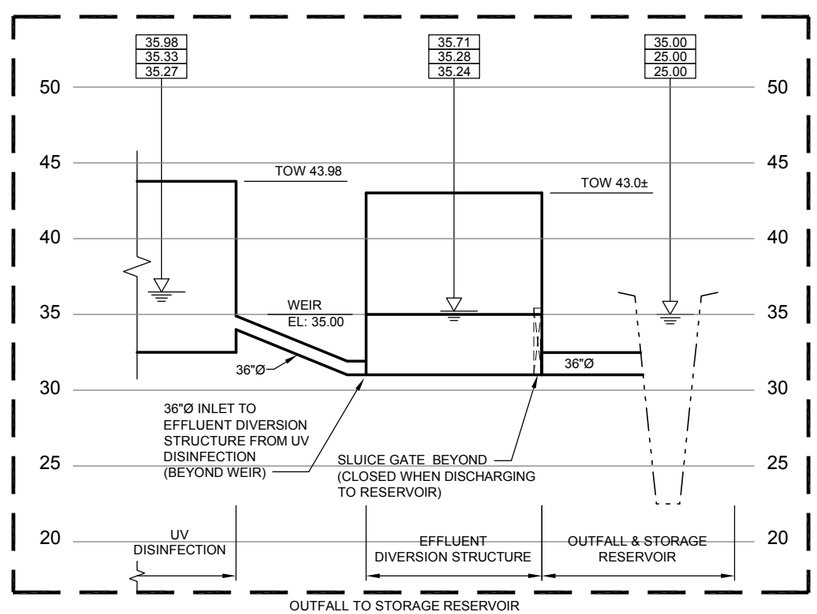

NOTES:

1. UNDER 100-YR FLOOD CONDITIONS, THE HGL AT THE EFFLUENT DIVERSION STRUCTURE WILL EXCEED THE WEIR HEIGHT OF 35.0 FT, RESULTING IN EFFLUENT FLOWING OVER THE WEIR AND REACHING THE STORAGE RESERVOIR. AS SUCH, IT IS ASSUMED THAT FLOWS TO THE SKUNK CREEK OUTFALL AND THE STORAGE RESERVOIR WILL ADJUST UNTIL HEAD LOSSES IN THE TWO PIPELINES ARE EQUAL.
2. ASSUMES THAT NO EFFLUENT IS LOST THROUGH THE ACCESS HATCH OF THE OUTFALL JUNCTION STRUCTURE.

LEGEND

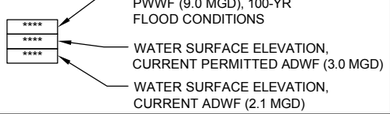

Figure 4-4

City Of Galt
WWTP Facilities Master Plan
EXISTING HYDRAULIC PROFILE 2

WEST YOST ASSOCIATES
Consulting Engineers

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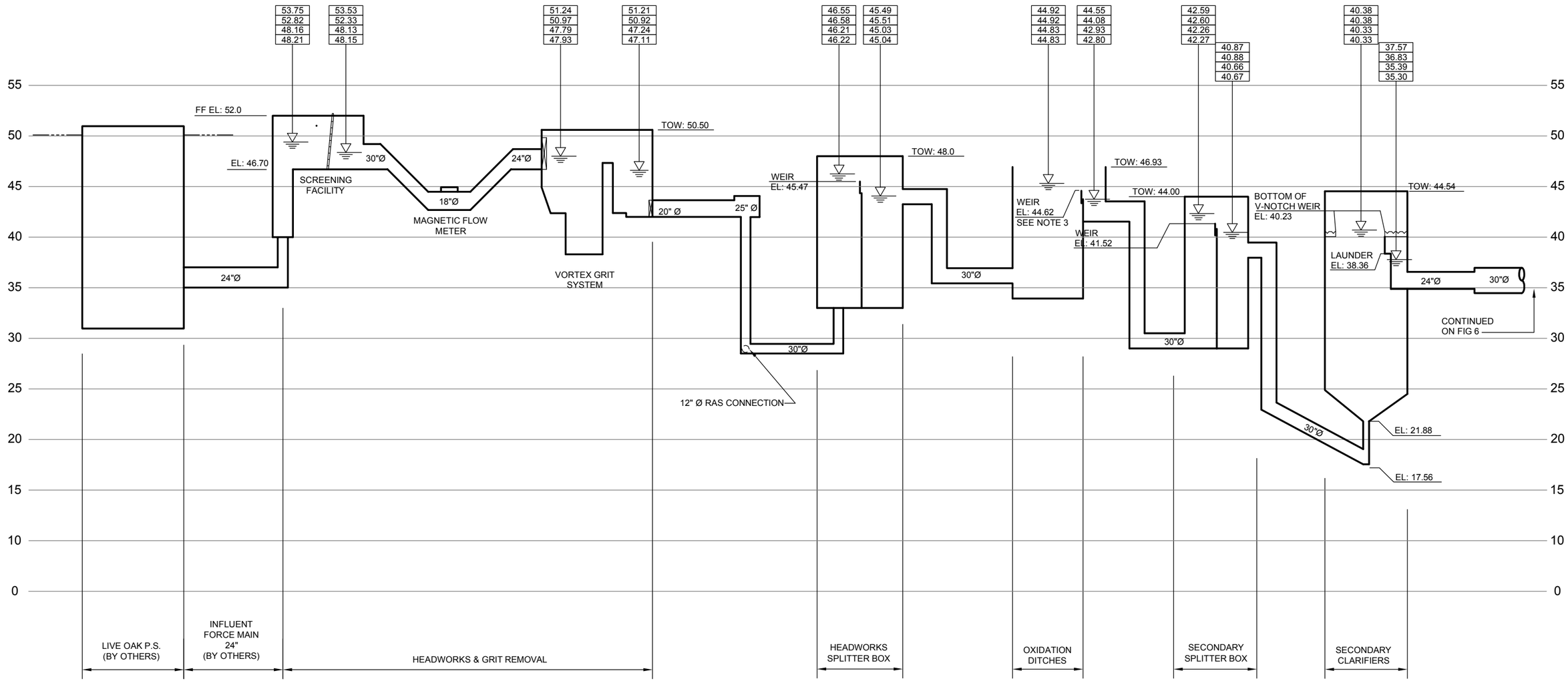

LEGEND

****	WATER SURFACE ELEVATION, BUILDOUT PWWF (18.00 MGD), 100-YR FLOOD CONDITIONS
****	WATER SURFACE ELEVATION, FUTURE PWWF (13.5 MGD), 100-YR FLOOD CONDITIONS
****	WATER SURFACE ELEVATION, BUILDOUT ADWF (6.00 MGD)
****	WATER SURFACE ELEVATION, FUTURE ADWF (4.5 MGD)

Figure 4-5

City Of Galt
WWTP Facilities Master Plan
FUTURE HYDRAULIC PROFILE 1

WEST YOST ASSOCIATES
Consulting Engineers

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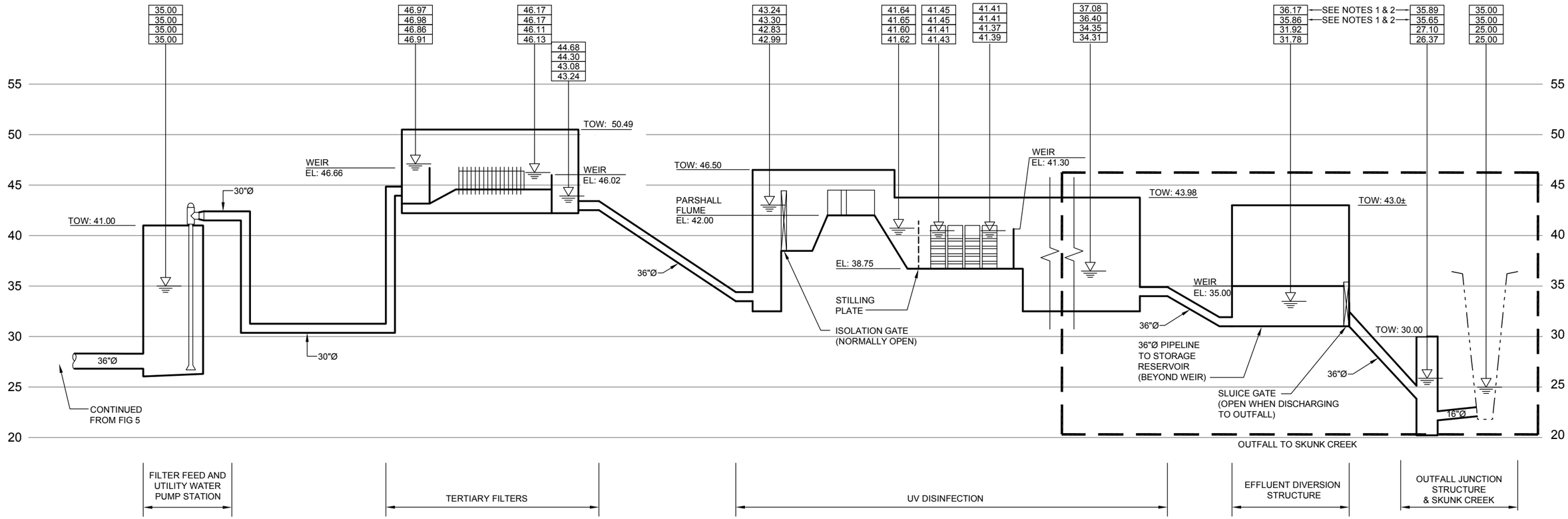

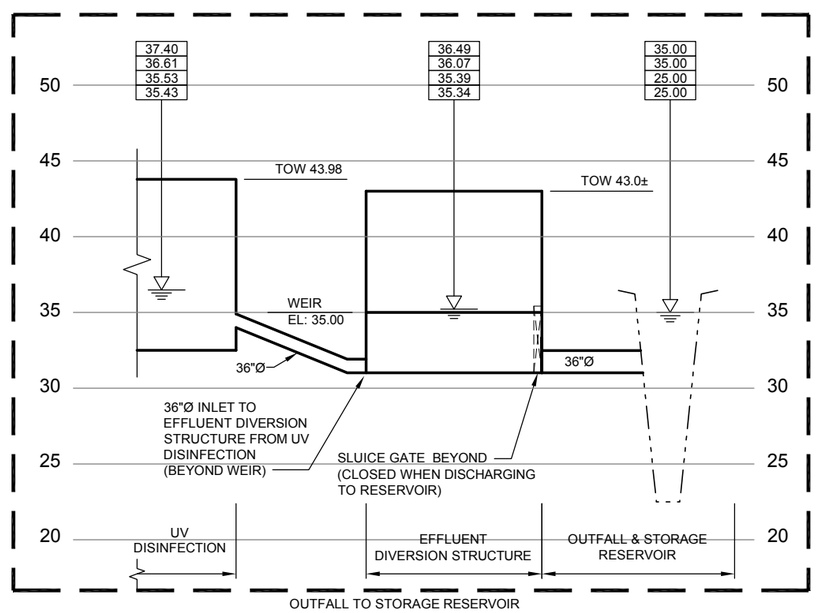

NOTES:

1. UNDER 100-YR FLOOD CONDITIONS, THE HGL AT THE EFFLUENT DIVERSION STRUCTURE WILL EXCEED THE WEIR HEIGHT OF 35.0 FT, RESULTING IN EFFLUENT FLOWING OVER THE WEIR AND REACHING THE STORAGE RESERVOIR. AS SUCH, IT IS ASSUMED THAT FLOWS TO THE SKUNK CREEK OUTFALL AND THE STORAGE RESERVOIR WILL ADJUST UNTIL HEAD LOSSES IN THE TWO PIPELINES ARE EQUAL.
2. ASSUMES THAT NO EFFLUENT IS LOST THROUGH THE ACCESS HATCH OF THE OUTFALL JUNCTION STRUCTURE.

LEGEND

****
****
****
****

- WATER SURFACE ELEVATION, BUILDOUT PWWF (18.00 MGD), 100-YR FLOOD CONDITIONS
- WATER SURFACE ELEVATION, FUTURE PWWF (13.5 MGD), 100-YR FLOOD CONDITIONS
- WATER SURFACE ELEVATION, BUILDOUT ADWF (6.00 MGD)
- WATER SURFACE ELEVATION, FUTURE ADWF (4.5 MGD)

Figure 4-6

City Of Galt
WWTP Facilities Master Plan
FUTURE HYDRAULIC PROFILE 2

WEST YOST ASSOCIATES
Consulting Engineers

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This chapter summarizes the results of an evaluation of biosolids dewatering and land application facility needs of the WWTP. The major topics discussed in this chapter are as follows:

- Projected Biosolids Production
- Biosolids Dewatering Facilities Alternatives Evaluation
- Biosolids Disposal Facilities Alternatives Evaluation
- Summary and Conclusions

The information discussed in this chapter builds off the conclusions and findings presented in the *Biosolids Process Options Alternative Evaluation Phase Report* (West Yost, 2009). Specifically, the City previously concluded that onsite dewatering and land application of Class B biosolids is the preferred solids handling approach for the foreseeable future.

5.1 PROJECTED BIOSOLIDS PRODUCTION

This section presents the estimated solids production of the WWTP, which will be is used to determine the capacity requirements for solids handling facilities, including the WWTP sludge storage lagoons and solids dewatering equipment. The following specific topics are addressed:

- Current Solids Balance
- Projected Production

5.1.1 Current Solids Balance

Solids balances for the WWTP have been developed using two methods: a spreadsheet model and the BioWin model. The input parameters for both models are shown in Table 5-1. The process characteristics used in the solids balance spreadsheet model to obtain a similar output as the BioWin model are given in Table 5-2. A comparison of solids balance results from the spreadsheet model and the BioWin model is presented in Table 5-3. For estimating the timing for expansion and/or improvement of the existing solids handling facilities, a production value of 730 dry tons per year at 3.0 mgd ADWF will be assumed.

Table 5-1. Influent and Effluent Wastewater Characteristics

Parameter	Value[(a)]
Current Influent Flow, mgd	2.1
Design Influent Flow, mgd	3.0
Influent BOD, mg/L	206
Influent TSS, mg/L	207
Influent TKN, mg/L	36
Secondary Effluent TSS, mg/L	2

[(a)] Based on 2-week sampling program in April and May of 2012.

Table 5-2. Process Characteristics

Parameter	Value
MLVSS/MLSS, ratio	0.8[a]
Biological solids yield coefficient, lb MLVSS/lb BOD	0.75[b]
Biological solids decay coefficient, d^{-1}	0.03[c]
Fraction of non-volatile solids in influent TSS, %	12[a]
Coefficient of volatile suspended solids (VSS) yield due to nitrification, g VSS/g NH_4-N	0.25[d]

(a) MLVSS = Mixed liquor volatile suspended solids. Average value measured during two-week monitoring program.
(b) Typical yield coefficient values range from 0.4 to 0.8 per "Wastewater Engineering" (Metcalf & Eddy, Inc., 1991).
(c) Typical decay coefficient values range from 0.025 to 0.075 per "Wastewater Engineering" (Metcalf & Eddy, Inc., 1991).
(d) Typical yield coefficient values range from 0.04 to 0.29 per the WEF Manual of Practice 8 – Design of Municipal Wastewater Treatment Plants (WEF, 1998).

Table 5-3. Comparison of Solids Balance Results

ADWF, mgd	Minimum MCRT[a]	Spreadsheet Model		BioWin Model		
		Dry pounds per day	Dry tons per year	MLSS, mg/L	Dry pounds per day	Dry tons per year
2.1	29	2,407	439	2,850	2,420	442
3.0	25	3,890	710	3,500	4,000	730

(a) MCRTs based on solids stored in oxidation ditches only.

5.1.2 Projected Production

Biosolids production rates are essential for planning solids handling facilities at the WWTP. Biosolids production rates predicted with the secondary process models for each of the three design ADWF conditions to be evaluated in this Facilities Master Plan are presented in Table 5-4. These solids production rates are based on the following assumptions regarding the secondary treatment process:

- Raw wastewater will be fed to the oxidation ditches (i.e., no primary treatment).
- The oxidation ditches will be operated with a minimum (aerated) mean cell residence time (MCRT) of at least 25 days to ensure the Class B pathogen reduction treatment requirements for the biosolids are satisfied.

Table 5-4. Biosolids Production Rates

Design Condition	Average Dry Weather Flow, mgd	Average Biosolids Production Rate, dry tons per year	Maximum Month Production Rate, dry tons per year
Current Permitted Design Condition	3.0	730	940
Future Permitted Design Condition	4.5	1,100	1,420
Ultimate Buildout Design Condition	6.0	1,500	1,900

As discussed in Chapter 6 (Treatment Train Upgrade and Expansion Alternatives), the MCRT in the oxidation ditches may be reduced to 12 to 15 days if the City elects to construct a separate solids digestion treatment process. This reduction in MCRT would result in biosolids production rates higher than those presented in Table 5-4. However, for purposes of this Facilities Master Plan, the volatile solids reduction provided by the aerobic digestion process is assumed to be equivalent to the reduction provided in the oxidation ditches operated at a 25-day MCRT. Therefore, under the aerobic solids digestion treatment process alternative discussed in Chapter 6, the biosolids production rates are assumed to be the same as what is presented in Table 5-4.

Also discussed in Chapter 6 is the concept that there will be a reduction in secondary solids generated if the City elects to construct primary microscreening facilities to reduce the loading to the oxidation ditches. (Primary solids removed using the microscreening process would be hauled directly to a landfill.) This reduction in production rates would result in a corresponding decrease to the capacity requirements for the solids handling facilities. While such a decrease would impact the cost analysis presented in the Facilities Master Plan (an analysis of the cost impacts is discussed in Chapter 6, the decrease is not expected to affect the conclusions discussed in this Chapter.

5.2 BIOSOLIDS DEWATERING FACILITIES ALTERNATIVES EVALUATION

This section presents an evaluation of alternatives for meeting the City's solids dewatering needs at the Current Permitted Design Condition. The purpose of this evaluation is to identify the preferred approach for meeting the City's near-term biosolids dewatering needs. The facilities identified for the preferred dewatering alternative will later be expanded, in kind, to meet the capacity needs at the Future Permitted Design Condition and the Ultimate Buildout Design Condition. Costs associated with expanding the dewatering capacity beyond the Current Permitted Design Condition are presented in Chapter 6 (Expansion Alternatives).

The following topics are addressed:

- Existing Capacity Evaluation
- Design Dewatering Bed Operating Assumptions
- Evaluation of Alternatives

- Comparison of Estimated Costs
- Recommended Alternative

5.2.1 Existing Capacity Evaluation

As described in Chapter 3 (Existing Facilities), the WWTP dewatering facilities consist of sludge storage lagoons, biosolids dewatering beds, and a biosolids storage/drying area. The WWTP's solids production rates, secondary clarifier thickening performance, solids storage lagoon thickening performance, and biosolids dewatering bed performance all have an influence on the capacity of the existing biosolids dewatering facilities. The relationship between these parameters can generally be described as follows:

- As the sludge production rate increases *or* the WAS solids concentration decreases, the volume of the solids that need to be stored prior to dewatering increases, thereby decreasing the amount of time that solids may be stored in the existing lagoons prior to dewatering.
- As solids storage time decreases, the thickening and decanting capabilities of the existing storage lagoons are reduced, resulting in lower overall lagoon solids concentrations. This in turn lowers the concentrations of solids fed to the biosolids dewatering beds. Because the dewatering beds have both solids and hydraulic loading limitations, the hydraulic loading rate to the beds will become limiting if the concentration of the solids decreases below 1.1 percent.
- As the dewatering bed solids loading rate increases, the drying times necessary to achieve a desired percent solids content in the final product generally increases. As the drying time increases, the turnover time (which includes bed preparation, loading and unloading) required for each bed increases.
- As the dewatering bed solids loading rate decreases, the number loads required to process the WWTP's solids will increase.

As discussed in Chapter 3 (Existing Facilities), WWTP staff is continuing to experiment with operating variables to achieve optimal performance of the biosolids dewatering beds. Table 5-5 provides typical lagoon and dewatering bed operating and performance variables that have been used to assess the ability of the lagoons and dewatering beds to handle the projected solids production rate at the Current Permitted Design Condition. The original design rating of the drying beds is also shown in Table 5-5 as a reference. As shown, the City has not been able to reliably achieve the rated solids loadings for the dewatering beds, and the lagoon settled sludge concentrations are significantly lower than what was anticipated when the drying beds were designed.

Table 5-5. Typical Lagoon and Dewatering Bed Operation & Performance Variables[(a)]

Operating/Performance Variable	Typical Value	Design Rating
Dewatering Bed Loading Rate	1.0 lb/ft^2	2.0 lb/ft^2
Dewatering Bed Turnover Time	8 days (warm weather) 15 days (cool weather)	7 days (year-round)
WAS Concentration	0.8%	N/A
Lagoon Settled Sludge Concentration	1.25%[(b)]	2.5%
Usable Lagoon Volume[(c)]	453,000	647,000

(a) Based on feedback provided by City staff in July 2012.
(b) Based on a reported target concentration range of 1-1.5%.
(c) According to City staff, the lagoons produce a more consistent settled sludge concentration prior to dewatering when the lagoons start out at a depth of no less than 4 feet (which is approximately the level at which a WAS pipe enters each lagoon). Furthermore, it is assumed that lagoon mixers cannot be effectively operated at depths much less than 4 feet.

Based on the typical variables listed in Table 5-5, it would take approximately nine days to fill one lagoon at the Current Permitted Design Condition, provided the sludge is gradually thickened to 1.25 percent solids[1] during the lagoon filling and decanting operation. Once one lagoon is full, the other lagoon will begin to be filled; nine days would therefore remain for sending the contents of the first lagoon to the dewatering facilities.

The amount of dry solids that will be produced in nine days by the WWTP at the Current Permitted Design Condition is estimated to be approximately 36,000 lb. At a dewatering bed loading rate of 1.0 lb/ft^2, each dewatering bed can process 4,800 dry lb of solids in each pour, meaning that eight pours would be needed within nine days to empty a lagoon filled as described in the previous paragraph. Based on the typical dewatering bed turnover time variables listed in Table 5-5, eight dewatering beds can be turned over (filled, dewatered, emptied and cleaned) in 17 days during warm weather. During cool weather, eight beds can be turned over in 31 days. Both durations exceed the allowable nine days of time during which a lagoon must be emptied. Thus, the WWTP does not have adequate solids handling capacity for the Current Permitted Design Condition given the current typical operating conditions presented in Table 5-5. An evaluation of potential alternatives for providing additional capacity is presented later in this chapter.

[1] City staff has not kept detailed records on the thickening performance of the lagoons (particularly during the incremental decanting and refilling operation typically used at the WWTP), so it is not known whether nine days is sufficiently long to achieve the target settled sludge concentration. Gravity thickening of WAS is typically marked by sporadic performance due to the slow-settling and compaction-resistant characteristics of WAS, as well as the tendency of WAS to stratify due to gas produced during continued biological activity.

5.2.2 Design Dewatering Bed Operating Assumptions

The design dewatering bed operating assumptions common to the alternatives are listed in Table 5-6. In addition to the values presented in Table 5-6, assumptions regarding the loading and unloading operations for the dewatering beds are as follows:

- Beds may only be loaded/unloaded on weekdays.
- The day that a dewatering bed is loaded is considered one of the drying days comprising the overall bed turnover period.
- The day that a dewatering bed is unloaded and prepared for the next loading is not considered part of the drying period.
- Current staffing levels allow for one dewatering bed being loaded and a second dewatering bed being unloaded and prepared for the next load each day.

Table 5-6. Dewatering Bed Operation Assumptions Common to All Alternatives

Operating Parameter	Value
Cool Weather Solids Production[(a)], dry tons	426
Warm Weather Solids Production[(b)], dry tons	304
Dewatering Beds Operations	
Usable Sludge Lagoon Volume[(c)]	453,000 gal (each lagoon)
WAS Solids Concentration[(c)]	0.8%
Minimum Sludge Feed Concentration[(c)]	1.25%
Maximum Liquid Depth in Dewatering Beds	1.5 ft
Maximum Dewatering Bed Solids Loading Rate	2.0 lb/ft^2
Dewatering Bed Turnover, Cool Weather[(a)]	15 days (14 days drying + 1 day unload/preparation)
Dewatering Bed Turnover, Warm Weather[(b)]	8 days (7 days drying + 1 day unload/preparation)
Supplemental Mechanical Dewatering Operations	
Months per Year, Minimum[(d)]	7
Days per Week	5
Hours per Day (excluding start-up and shut-down)	6

(a) Seven months (October through April) production at the Current Permitted Design Condition.
(b) Five months (May through September) production at the Current Permitted Design Condition.
(c) From Table 5-5.
(d) At a minimum, mechanical dewatering would occur throughout cool weather, and may extend into warm weather if a processing deficit remains.

5.2.3 Evaluation of Alternatives

The following options were identified for addressing the solids processing capacity needs:

- Further optimize sludge lagoon and dewatering bed operations.
- Increase the number of drying beds and/or sludge lagoons to provide needed additional capacity.
- Install mechanical dewatering equipment to supplement/replace the dewatering beds during cool weather.

5.2.3.1 Optimization of Existing Solids Handling Facilities

As discussed previously in this chapter, the City does not appear to have adequate dewatering capacity to process the biosolids generated at the Current Permitted Design Condition if the drying beds are operated at the turnover times and loading rates that have been observed since the dewatering beds have been online. However, the City may able to further optimize operations of the existing facilities, thereby avoiding the need to construct additional facilities.

To avoid construction of new storage lagoons or dewatering beds, a maximum annual average dewatering bed turnover time of eight days must be achieved. At this turnover rate, the dewatering beds solids loading rate would be 1.42 lb/ft^2. However, at the assumed sludge feed concentration of 1.25 percent, the maximum liquid volume for a given pour is reached when the solids loading rate is at 1.40 lb/ft^2. Therefore, a year-round turnover rate of eight days is not feasible at the sludge feed concentration adopted as a design criteria.

At an assumed maximum annual average dewatering bed turnover time of seven days, the dewatering beds would be loaded at a rate of at least 1.33 lb/ft^2, which is less than the achievable solids loading rate based on the assumed sludge feed concentration of 1.25 percent. However, experience with the dewatering beds to date demonstrates that achieving decreased bed turnover times is unlikely – particularly in the cool weather months. As a result, optimization of the existing solids processing facilities is not considered to be a realistic alternative for meeting the WWTP's dewatering needs at the Current Permitted Design Condition, and costs for this alternative have not been prepared.

5.2.3.2 Additional Dewatering Beds

Additional dewatering capacity could be provided by constructing additional dewatering beds. Table 5-7 presents a summary of the solids drying bed capacities and deficits if additional drying beds are added and assuming the solids loading rate is limited to 1 lb/ft^2. Because the dewatering bed turnover time is significantly greater during the cool weather season, the processing needs during the cool-weather seasons would dictate the number of drying beds needed unless storage is provided. However, to minimize the number of beds required, it has been assumed that the "Cool Weather Deficit" solids would be processed during the warm weather season – where the amount processed would be limited by the excess warm season capacity. As shown in Table 5-7, the addition of five dewatering beds - for a total of nine beds - will provide the required processing capacity. However, to meet the redundancy requirements outlined in Chapter 2 (Basis of Design), a total of six dewatering beds would be required.

Table 5-7. Solids Drying Bed Capacities and Deficits with Additional Dewatering Beds at the Current Permitted Design Condition

Operating Parameter	Number of Dewatering Beds (Including 4 Existing)					
	4	5	6	7	8	9
Cool Weather Processing Capacity, dry tons	135	168	202	235	269	302
Cool Weather Deficit, dry tons	292	258	225	191	158	124
Warm Weather Processing Capacity, dry tons	192	240	288	336	384	432
Warm Weather Deficit, dry tons	112	64	16	—	—	—
Warm Weather Excess Capacity, dry tons	—	—	—	32	80	128
Annual Deficit, dry tons	404	322	241	159	78	—

This alternative would also require additional biosolids storage volume equal to six times the usable volume of one of the existing lagoons to hold the biosolids generated during the cool-weather season that cannot be processed until the drying bed turnover time increases in warm weather months.

Finally, with a total of nine beds (firm capacity), it will not be possible for one City staff person to manage the biosolids dewatering operations. Therefore, additional staff would need to be hired so that two beds can be loaded, unload, and prepared per weekday.

5.2.3.3 Mechanical Dewatering to Supplement Dewatering Beds

As an alternative to constructing additional dewatering beds, the City could increase solids processing capacity at the WWTP by installing mechanical dewatering equipment. One option would be to size the mechanical equipment to supplement the dewatering beds by handling the solids processing deficit that occurs with only four dewatering beds in service. However, simultaneous operation of mechanical dewatering equipment and the dewatering beds may not be desirable from a WWTP staffing perspective. Therefore, a second option would be to install a larger facility with greater capacity to "replace" the use of dewatering beds during the cooler weather months. Table 5-8 presents the preliminary sizing criteria for these two options.

Table 5-8. Preliminary Mechanical Dewatering Sizing for the Current Permitted Design Condition

Operating Parameter	Option A. Supplemental Cool Weather Capacity	Option B. Replacement Cool Weather Capacity
Cool Weather Solids Processing Deficit, dry tons	292	426
Sludge Pumped to Dewatering Machine(s), dry lb/day	3,847	5,622
Minimum Solids Loading Capacity, dry lb/hr	641	937
Minimum Hydraulic Loading Capacity, gpm	103	150

Three types of mechanical dewatering equipment were previously evaluated for use at the WWTP – screw presses, rotary fan presses and centrifuges – and the results of the evaluation were presented in the *Biosolids Processing Options Alternative Evaluation Phase Report* (West Yost, April 2009). Of these three technologies, screw presses and rotary fan presses were determined to be the preferred options. Descriptions of the technologies and their ancillary facilities, as well as the relative advantages and disadvantages of each are provided in the *Biosolids Process Options Alternative Evaluation Phase Report*. The equipment costs, performance information, and layout requirements for each of these technologies are summarized in Table 5-9, based on the sizing requirements outlined in Table 5-8.

Table 5-9. Mechanical Dewatering Equipment Costs and Expected Performance for the Current Permitted Design Condition

Parameter		Rotary Press	Screw Press
Representative Manufacturer		Fournier	Huber
Rated Capacity of Machine (each)		110 dry lb/hr per channel	355-415 dry lb/hr[a]
Capacity Adjustment Factor[b]		80%	
Adjusted Capacity (each)		88 dry lb/hr per channel	284-332 dry lb/hr[a]
Number of Machines Needed without Redundancy	Option A: Supplemental Capacity	Two (2) 4-channel machines	Two (2) machines[c]
	Option B: Replacement Capacity	Two (2) 6-channel machines	Three (3) machines[c]
Total Installed Capacity	Option A: Supplemental Capacity	704 dry lb/hr	568 – 664 dry lb/hr[a]
	Option B: Replacement Capacity	1,056 dry lb/hr	852 – 996 dry lb/hr[a]
Estimated Polymer Consumption		25 lb/dry tons	25 lb/dry tons
Expected Cake Solids Concentration		15-18 % TS	18-25 % TS
Total Operating Power Required[d]	Option A: Supplemental Capacity	29.5 HP[e] 22 kW	18.5 HP 13.8 kW
	Option B: Replacement Capacity		27 HP 20.1 kW
Approximate Footprint of Each Dewatering Unit		9 ft x 13 ft	6 ft x 16 ft
Machine Footprint[f]	Option A: Supplemental Capacity	18 ft x 13 ft	12 ft x 16 ft
	Option B: Replacement Capacity	18 ft x 13 ft	18 ft x 16 ft
Expandability		Each press can be expanded to 6 channels, at a cost of $50k per channel.	The total installed capacity would provide some ability to meet future capacity needs without adding more machines.
Estimated Cost of Equipment	Option A: Supplemental Capacity	$610,000	$365,000
	Option B: Replacement Capacity	$805,000	$705,000

(a) Capacity varies based on incoming sludge characteristics.
(b) Conservative estimate to account for impact of varying sludge dewaterability on machine performance.
(c) Based on upper end of adjusted capacity range.
(d) Includes presses, flocculators, polymer blending equipment and sludge feed pumps.
(e) Assumes each 20 hp press motor is operated at 40% draw.
(f) Does not include space for ancillary equipment.

5.2.4 Comparison of Estimated Costs

This section presents a discussion of the estimated costs of the solids processing alternatives at the Current Permitted Design Condition. The major topics discussed are as follows:

- Capital Costs
- Net Present Value Costs

5.2.4.1 Capital Costs

Planning level cost estimates were developed for the potential solids handling alternatives at the Current Permitted Design Condition. These costs are presented in Table 5-10.

Table 5-10. Estimated Project Costs for Solids Processing Alternatives for the Current Permitted Design Condition, Dollars[a]

Parameter	Additional Dewatering Beds	Mechanical Dewatering			
		Supplemental		Replacement	
		Rotary Press	Screw Press	Rotary Press	Screw Press
Sludge Lagoons	2,660,000	—	—	—	—
Biosolids Dewatering Beds	1,576,500	—	—	—	—
Mechanical Dewatering Facility	—	1,192,000	947,000	1,663,000	1,393,000
Distributed Costs[b]	1,440,000	410,000	320,000	570,000	470,000
Subtotal	5,676,500	1,602,000	1,267,000	2,233,000	1,863,000
Estimating Contingencies[c]	1,700,000	480,000	380,000	670,000	560,000
Estimated Construction Bid Cost	7,376,500	2,082,000	1,647,000	2,903,000	2,423,000
Adjustment for Inflation to Construction Midpoint[d]	440,000	120,000	100,000	170,000	150,000
Allowances[e,f]	3,320,000	930,000	740,000	1,310,000	1,080,000
Estimated Total Project Cost[f]	**11,136,500**	**3,132,000**	**2,487,000**	**4,383,000**	**3,653,000**

(a) Estimated in current (November 2012) dollars unless otherwise indicated.
(b) Distributed costs estimated as a percentage of the component costs according to the following allowances: mobilization and demobilization (3%); demolition, bypassing, and shutdowns (1%); sitework, including general earthwork and paving (5%); site piping for process flows and utilities (10%); and electrical and instrumentation (15%).
(c) Based on an estimating and bidding contingency allowance of 30%.
(d) Adjusted to mid-point of construction at 3% per year. Anticipated date of construction contract award is December 2013, and a 22-month construction period is anticipated.
(e) Allowances include: construction contingencies (5%); City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (20%); construction management, including O&M manuals and record drawings (14%); and other City materials and services, including environmental (0.5%).
(f) Presented in nominal dollars (i.e., includes an allowance for inflation to the time of the expense).

The cost information presented in Table 5-10 was developed in accordance with the following factors:

- **Component Costs:** To the extent practicable, schedules of values from recently completed wastewater treatment plant biosolids dewatering projects (including the City's) were used to estimate the component costs, as many of the items would consist of similar or identical facilities to those that were constructed for the reference projects. Other cost sources were used where this was not possible, particularly for those components associated with modifications to existing facilities. All costs were adjusted to current (late 2012) dollars. For discussion purposes, the component costs are described in the following groups:
  - Sludge Lagoons
  - Biosolids Dewatering Beds
  - Mechanical Dewatering Facility
- **Distributed Costs:** These include costs not included in the component cost estimates at the planning level. The costs were determined by applying multipliers to the total of the component costs. Distributed cost allowances were as follows: mobilization and demobilization (3%); demolition, bypassing, and shutdowns (1%); sitework, including general earthwork and paving (5%); site piping for process flows and utilities (10%); and electrical and instrumentation (15%).
- **Estimating Contingency Allowance:** This allowance was applied to sum of the component and distributed costs to account for possible costs that may not have been identified in the conceptual description of a given alternative. For purposes of this memorandum, an allowance of 30 percent was used as estimating contingency, as is typically done for planning level cost estimating. The sum of the component costs, distributed costs and the estimating contingency allowance yield the estimated construction bid cost for a project bidding in late 2012.
- **Escalation to Construction Midpoint**: The anticipated date of the construction contract award is November 1, 2013, and a 24-month construction period is anticipated. An escalation rate of 3 percent per year is used to inflate the construction costs to the midpoint of construction.
- **Project Allowances:** Additional project allowances were applied to sum of the estimated construction bid cost and the adjustment for escalation to construction midpoint to account for the engineering, administration, legal, environmental and other costs that will be incurred in addition to the bid construction price for the project various. Allowances include: construction contingencies (5%); City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (20%); construction management, including O&M manuals and record drawings (14%); and other City materials and services, including environmental (0.5%).

5.2.4.2 Net Present Value Costs

Net present value (NPV) costs, which account for both capital and operation and maintenance (O&M) components, have also been prepared. Assumptions for routine O&M requirements for each of the dewatering technologies considered are presented in Table 5-11. Additional NPV O&M cost assumptions are as follows:

- Planning period = 6 years
- Discount rate = 5%
- Electrical power = $0.095/kWh
- Inflation rate = 3% annually
- Diesel fuel cost = $4.5/gal
- Average hourly labor rate = $40 (includes benefits)

Table 5-11. Assumed Routine O&M Requirements for Dewatering Technologies

Operating Parameter	Dewatering Beds	Rotary Presses	Screw Presses
Operating Days per Week	5 days/week	5 days/week	5 days/week
Operating Hours per Day[a]	8 hours/day	8 hours/day	8 hours/day
Fuel Consumption	1.5 gal/hour[b]	N/A	N/A
Polymer Dose	50 lb/dry tons[c]	25 lb/dry tons[d]	25 lb/dry tons[d]
Daily Operator Startup/Shutdown Labor	1 hour	40 min	40 min
Daily Operating Monitoring Labor	1 hour	20 min	20 min
Daily Maintenance Worker Labor	16 hours[e]	15 min	15 min
Monthly Maintenance Worker Labor	5 hours	5 hours	5 hours

(a) Includes startup and shutdown time.
(b) For sludge harvesting tractor operation.
(c) Based on recent WWTP performance data.
(d) Based on manufacturer's proposal.
(e) Assumes two maintenance workers for eight hours each per day.

A summary of capital cost and O&M cost estimates, along with the resulting NPV of each alternative, is presented in Table 5-12.

Table 5-12. Estimated NPV for Solids Processing Alternatives for the Current Permitted Design Condition, Million Dollars[a]

Alternative	Additional Dewatering Beds	Mechanical Dewatering			
		Supplemental		Replacement	
		Rotary Press	Screw Press	Rotary Press	Screw Press
Total Project Costs[b]	11.14	3.13	2.49	4.38	3.65
Present Worth Total Project Costs	10.10	2.84	2.26	3.98	3.31
Present Worth O&M Costs	1.96	1.04	1.03	0.75	0.74
NPV[c]	12.06	3.88	3.28	4.73	4.05

(a) All costs in November 2012 dollars unless otherwise indicated.
(b) From Table 5-10, presented in nominal dollars.
(c) Due to individual rounding, total may not exactly equal sum of the present worth components.

5.2.5 Recommended Alternative

As Table 5-12 indicates, the supplemental mechanical dewatering results in the lowest NPV of the alternatives considered. Moreover, because construction of supplemental mechanical dewatering capacity does not preclude the City from installing additional capacity in the future to offset capacity currently met by the dewatering beds, the supplemental dewatering alternative is the preferred approach.

In addition, screw press technology appears to result in the lowest NPV of the alternatives considered under the Current Permitted Design Condition. However, supplemental mechanical dewatering alternative using rotary fan presses is a relatively close second; and at the level of estimating accuracy associated with this analysis, this alternative should be considered a near equal to the screw press alternative. Therefore, both technologies would be acceptable, and the City should further evaluate a preferred technology at later design phases by discussing operational experiences with current owners of the two technologies, and possibly visiting operating installations.

5.3 BIOSOLIDS DISPOSAL FACILITIES ALTERNATIVES EVALUATION

This section presents an evaluation of the City's biosolids disposal facilities. The following topics are addressed:

- Existing Capacity Evaluation
- Alternatives for Expansion

5.3.1 Existing Capacity Evaluation

As discussed in Chapter 3 (Existing Facilities), the land application facilities consist of the 175 acres of agricultural fields surrounding the WWTP. The capacity of the City's existing land application facilities is limited by the total allowable nitrogen loads that may be applied in a given year and by the cumulative loading of certain metals of concern.

5.3.1.1 Nitrogen Loadings

As discussed in Chapter 2 (Basis of Design), the amount of nitrogen that can be applied to the City's land application area in a given year is limited by the nitrogen uptake rate of the crops being grown. In recent years, the farmer of the City property has grown a summer crop of corn or clover. In addition, the farmer has historically planted a winter crop of wheat or other forage crop on fields that have been harvested of their summer crop. A summary of nitrogen demands for these crops, based on the most recent edition of the Western Fertilizer Handbook (California Plant Health Association, 2002), is provided in Table 5-13, which also includes an assumed crop yield per acre and the nitrogen requirement per acre, based on multiplying the other two values.

Table 5-13. Agronomic Nitrogen Requirements for City-Owned Fields

Type of Crop	Nitrogen Requirement per Mass of Crop Yield,[a] lb/ton	Assumed Crop Yield per Field Area,[b] ton/ac	Nitrogen Requirement per Field Area, lb/acre
Corn (silage)	8.3	30	250
Corn (grain)	26	5	130
Wheat (14% protein)	50	3	150
Oats	50	1.6	80
Clover	50	6	(300[c])

(a) Values shown are for lbs of nitrogen removed per ton of harvested crop as listed in Table 4-1 of the *Western Fertilizer Handbook* (California Plant Health Association, 2002). For grain corn, wheat, and oats, the nitrogen removed is listed in the *Western Fertilizer Handbook* in units of lb/cwt, where "cwt" is a hundred weight, equal to 100 pounds. The values reported in "lb/cwt" have been converted to lb/ton in this table.
(b) Although previous versions of the *Western Fertilizer Handbook* included a yield value with the nitrogen requirements by crop, the current version (2002) does not. Yields from a previous version have thus been used in the current analysis.
(c) Clover grass obtains most of its nitrogen from the air, so the nitrogen requirement does not represent the amount of nitrogen that needs to be applied.

As indicated in Table 5-13, a (summer) corn crop has the potential for the highest nitrogen application rate of any of the crops historically grown. As noted, clover has a higher nitrogen requirement but as a legume clover obtains most of its nitrogen from the air. The amount of applied nitrogen needed by clover is much lower than clover's total nitrogen requirements. If the City were to have a corn crop in the summer months (with a nitrogen uptake rate of 250 lb/ac) and a wheat crop in the winter months (with a nitrogen uptake rate of 150 lb/ac), the allowable nitrogen loading rate could be as high as 400 lb/ac.

The land requirements at the Current Permitted Design Condition, the Future Permitted Design Condition, and the Ultimate Buildout Design Condition assuming nitrogen application rates of 250 lb/acre and 400 lb/acre, respectively, are presented on Figure 5-1 and Figure 5-2. The land requirements are based on the following assumed conditions[2]:

- Biosolids NH3-N concentration = 4,300 mg/kg
- Biosolids NO3-N concentration = 13 mg/kg
- Biosolids Org-N concentration = 56,700 mg/kg
- Future applied irrigation water total nitrogen concentration = 8 mg/L
- Irrigation water annual application rate = 3 acre-feet per acre
- The mass of applied biosolids for any given year is proportional to the flow projected as shown on Figure 2-1 (Chapter 2).
- Biosolids are distributed evenly over the existing 175-acre City-owned land application area.

As indicated on Figure 5-1, the existing land application area is not adequately sized to handle the nitrogen loadings that would occur at the Current Permitted Design Condition if the per acre nitrogen update is limited to 250 pounds per acre (i.e., if only one summer corn crop is grown). However, as shown on Figure 5-2, the existing land application area would be adequate to handle the nitrogen loadings that would occur at the Current Permitted Design Condition if uptake were as high as 400 pounds per acre (i.e., a summer corn crop and a winter wheat crop are grown). Given the significant benefits of a double-cropping scheme, it has been assumed for purposes of this analysis that the City will employ double cropping to maximize the potential for nitrogen uptake on the field areas.

Nevertheless, even with a double-cropping scenario, the City will not have adequate land disposal capacity to handle all biosolids generated at the Future Permitted Design Condition (the land disposal capacity is expected to be reached as the average dry weather flow reaches 4.0 mgd). An evaluation of potential alternatives for addressing this capacity need is presented later in this chapter.

[2] The concentrations listed are the median for the 2006 through 2008 biosolids quality data. This data does not represent dewatered solids; however, only one data set (2/23/12) for dewatered solids was available for the current analysis, so the historic data is considered more reliable. Dewatering biosolids is expected to reduce constituent concentrations, although the extent of such reductions cannot be adequately estimated at this time.

5.3.1.2 Metals Loading

The remaining time until metals loadings to the fields will likely reach the CPLR limits has been estimated based on the following:

- Biosolids are distributed evenly over the existing 175-acre City-owned land application area.
- Maximum cumulative loadings will be reached for the limiting fields indicated in Table 2-17 (Chapter 2).
- Median metals concentrations from the City's 2006-2008 biosolids monitoring data shown in Table 2-17 adequately represent biosolids quality.
- The mass of applied biosolids for any given year is proportional to flow until the Ultimate Buildout Design Condition is reached (currently estimated to be in 2041).
- After 2041, the amount of biosolids that will be field applied will be 1,500 dry tons.

The results of this evaluation, which are shown in Table 5-14, indicate that the City will not exceed the CPLR limits for any constituent within the planning horizon of this Facilities Plan. Moreover, because cumulative metals loading analysis assumes the City will continue to land apply all of the generated biosolids on the existing 175-acre land application area even though the City will likely need to expand the land application area (or identify an alternative means of biosolids disposal) prior to reaching the Ultimate Buildout Design Condition, the biosolids loadings to the fields (on a per acre basis) would likely be significantly less than the amounts assumed in developing Table 5-14.

Table 5-14. Estimate of Remaining Time for Land Application Based on Metals CPLRs

Constituent	Maximum Loading to Date[a], lb/ac	Max. Loading at Ultimate Buildout Design Condition[b], lb/ac	CPLR, lb/ac	Remaining Allowance at Ultimate Buildout Design Condition, lb/ac	Assumed Biosolids Metals Concen - tration[a], mg/kg	Metals Application Rate at Ultimate Buildout Design Condition[c], lb/ac/year,	Estimated Remaining Years to Reach Limits Beyond 2041
Arsenic	2.0	6.9	36	29.1	15.3	0.261	111
Cadmium	2.1	2.7	34	31.3	2.1	0.036	858
Copper	6.6	79	1,336	1,257	221	3.789	332
Lead	3.0	9.8	267	257.2	21.1	0.361	713
Mercury	0.3	0.8	15	14.2	1.6	0.027	533
Molybdenum	0.3	5.1	16	10.9	14.8	0.253	43
Nickel	6.7	16	374	358	28.5	0.489	733
Selenium	0.3	4.1	89	84.9	11.5	0.197	431
Zinc	13.7	186	2,494	2,308	529	9.069	255

(a) From Table 2-17 (Chapter 2).
(b) Assumes the Ultimate Buildout Design Condition is reached in 2041.
(c) Average application rate is equal to assumed concentration multiplied by the biosolids mass of 1,500 dry tons at the Ultimate Buildout Design Condition and divided by the application area of 175 acres.

The cumulative metals loading analysis presented in Table 5-14 indicates that molybdenum is the most limiting constituent, with arsenic the next most limiting. Depending on actual concentrations in the applied biosolids, one of these constituents is thus expected to limit the City's continued land application of biosolids.

As discussed in TM 9 (Appendix C), the City is evaluating potential changes to the disposal method of the backwash water from the City's wellhead treatment facilities in an effort to reduce the dissolved arsenic levels in the WWTP influent. Several of the options under consideration could also result in a change to the arsenic levels in the land applied biosolids. Specifically, the Alternatives 2 and 3 in TM 9 have the potential to increase the arsenic levels in the biosolids by as much as 2.5 times in the May through October period. Conservatively assuming a 2.5 times increase would be applied to the median 15.3 mg/kg value presented in Table 5-14, the remaining lifetime for Field 11 (the currently limited application field) is estimated to be approximately 33 years beyond the Ultimate Buildout Design Condition (for a remaining lifetime through 2074). While significantly shorter than that estimated for existing conditions, this remaining lifetime is still beyond the City's current planning horizon.

Finally, it should be mentioned that if the biosolids dewatering processes is modified such that significantly more water contained in the biosolids is returned to the WWTP, the concentration of metals in the biosolids could change over what is presented in Table 5-14. However, such changes are not expected to significantly reduce the total estimated remaining lifetime for the land application areas.

5.3.2 Alternatives for Expansion

As discussed previously, the City has adequate land application capacity to handle all of the biosolids generated through the Current Permitted Design Condition and somewhat beyond. An expansion of the biosolids land application area or another alternative option will be needed to accommodate some of the biosolids generated at higher flows.

The City will not likely need additional biosolids land application or disposal capacity for at least 15 years. Expanding the land application area is one option; however, it is not possible to develop a reliable cost estimate for expanding the biosolids land application facilities for the following reasons:

- The availability, location, and cost of land near the existing WWTP site could change significantly during the next 15-year period.
- The cost of acquiring, constructing and operating a new biosolids land application disposal area will vary significantly depending on factors such as: the site's location, the source of water used for irrigation, and whether the site is already developed for irrigation.

In addition, the City will likely need to revisit the conclusion that land application of dewatered, Class B biosolids is the preferred long term option as new facilities are required. Specifically, regulatory requirements could change with respect to the quality of biosolids needed for land application. In addition, new technologies that would allow for more efficient treatment, use, and/or disposal of biosolids may become available and should then be considered. Therefore, as the City begins to near the capacity limits of the land application site, potential expansion areas should be evaluated to develop a reliable cost for meeting the expansion needs.

For purposes of this Facilities Master Plan, costs for meeting the long-term biosolids disposal needs have been based on the cost associated with offsite hauling to a landfill. The cost of expanding the land application area has been assumed to be similar to, or less than, the cost associated with hauling. If the cost of expanding the land application area is greater than offsite disposal, the City would likely elect to haul solids that cannot be land applied on the existing properties to an offsite location for disposal. The estimated cost impact of "supplemental" offsite disposal included in the alternatives evaluation presented in Chapter 6 (Expansion Alternatives).

5.4 SUMMARY AND CONCLUSIONS

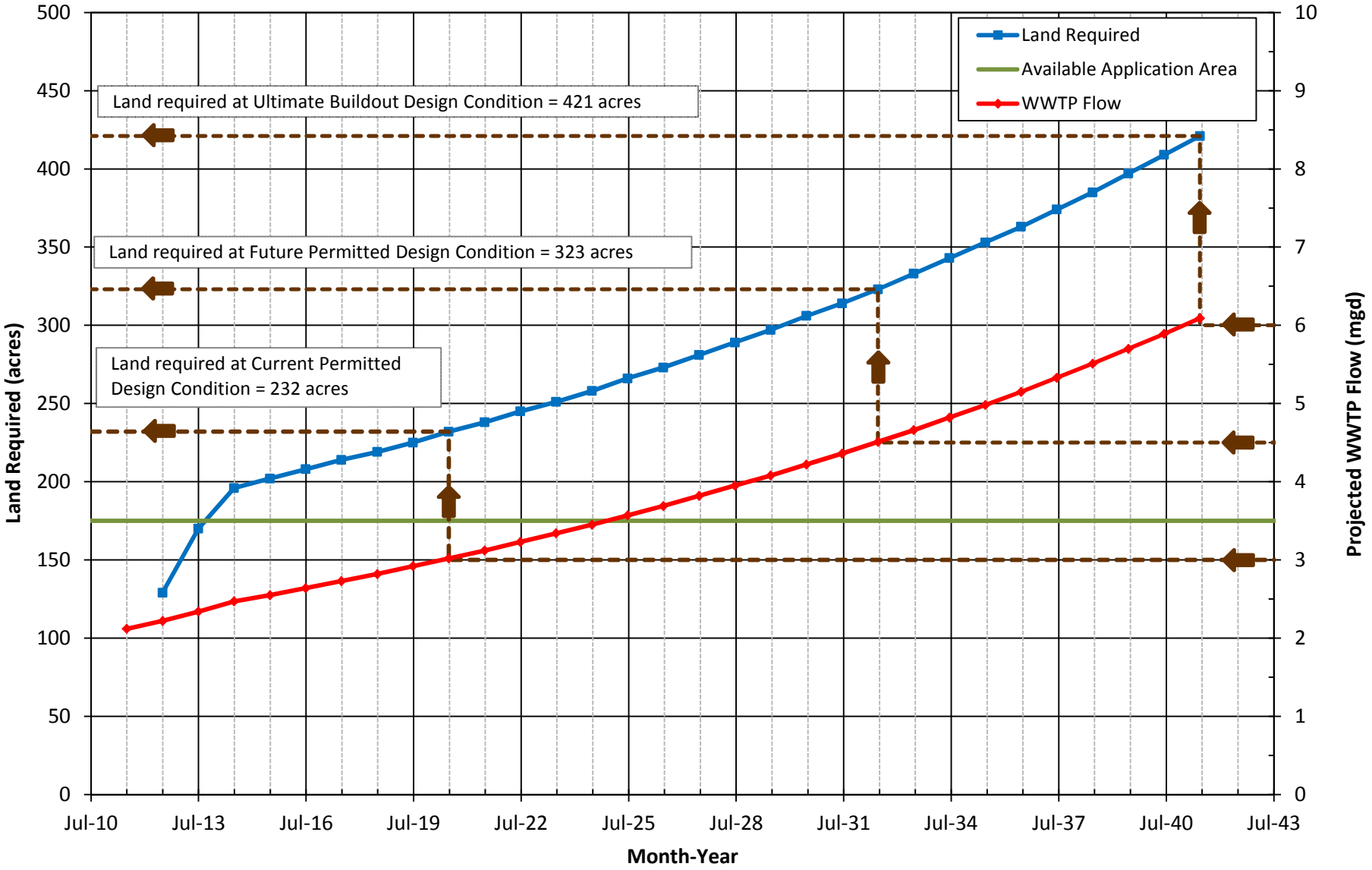
Installation of mechanical dewatering equipment is recommended to supplement the existing solids dewatering beds capacity. Either screw presses or rotary fan presses are likely to be cost effective. In addition, the City will need to expand the land application area or institute supplemental offsite disposal as the nitrogen loading capacity of the existing land application area is reached. The existing land application capacity is anticipated to be reached when WWTP influent flows approach 4 mgd, which is currently not expected for at least 15 to 20 years.

5.5 REFERENCES

California Plant Health Association, Soil Improvement Committee, *Western Fertilizer Handbook. Ninth Edition.* Sacramento, CA. 2002.

West Yost Associates. *Biosolids Process Options Alternative Evaluation Phase Report*. April 2009.

Figure 5-1. Land Required for Biosolids Application Based on Nitrogen Loadings (250 Pounds per Acre Nitrogen Application Rate)



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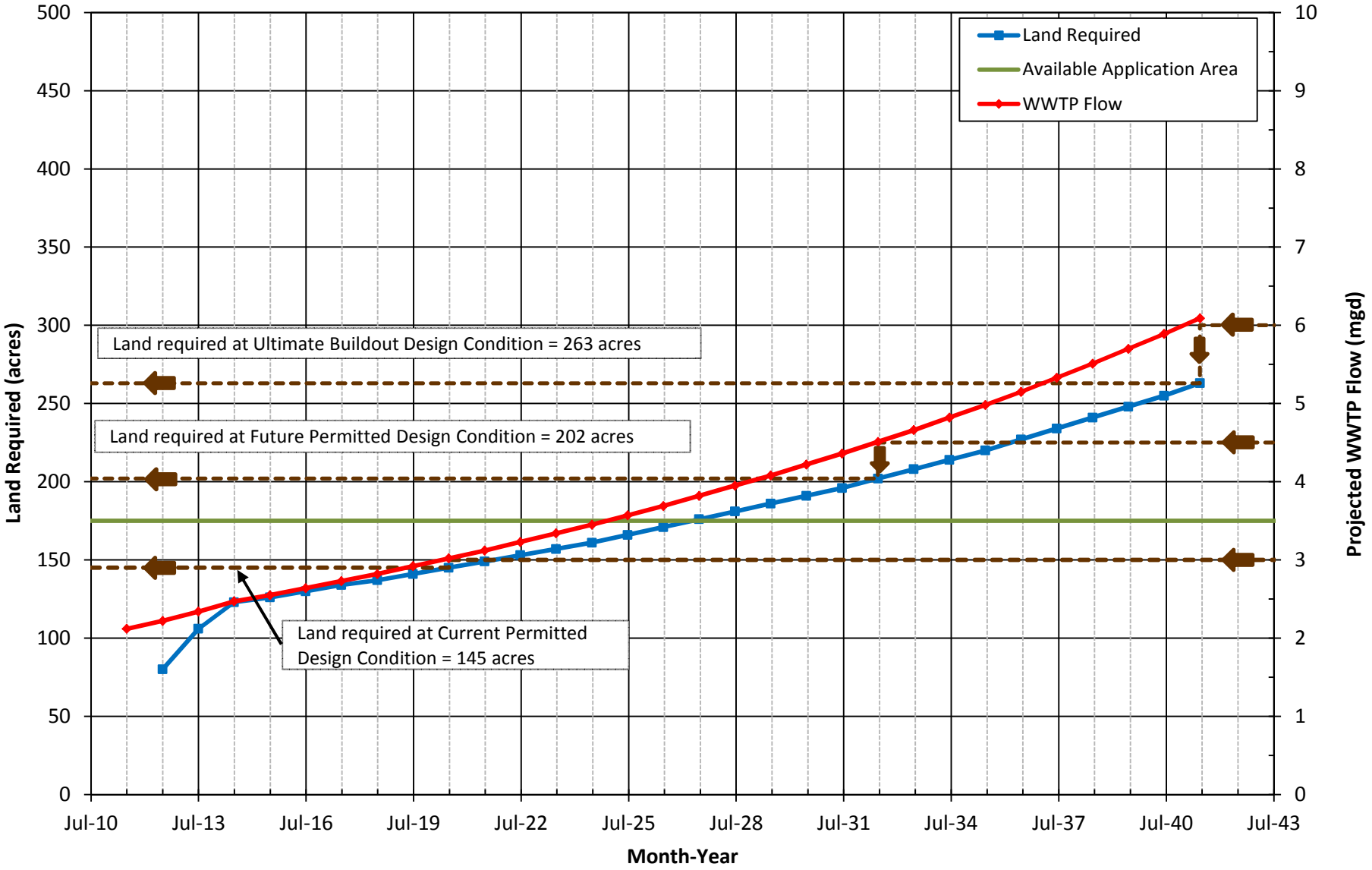
Last Revised: 1-8-13

City of Galt
WWTP Facilities Master Plan

Figure 5-2. Land Required for Biosolids Application Based on Nitrogen Loadings (400 Pounds per Acre Nitrogen Application Rate)



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Last Revised: 1-8-13

City of Galt
WWTP Facilities Master Plan

This chapter presents an evaluation of potential treatment train upgrade and expansion alternatives for the WWTP under the three design phases evaluated in this Facilities Master Plan. The major topics discussed in this chapter as follows:

- Current Permitted Design Condition Upgrade Alternatives
- Future Permitted Design Condition Expansion Alternatives
- Ultimate Buildout Design Condition Expansion Alternatives
- Approaches to Facilities Expansion
- Estimated Upgrade and Expansion Project Costs
- Comparative Evaluation of Alternatives

6.1 CURRENT PERMITTED DESIGN CONDITION UPGRADE ALTERNATIVES

This section presents an evaluation of potential improvements for ensuring reliable compliance with the City's effluent limitations under the Current Permitted Design Condition. The following specific topics are addressed:

- Project Components Common to All Alternatives
- Oxidation Ditch Upgrade Alternatives
- Secondary Clarifier Capacity Needs
- Tertiary Filtration and UV Disinfection Facilities Requirements
- Summary of Potential Upgrade Alternatives

6.1.1 Project Components Common to All Alternatives

This section describes common improvements that are necessary to ensure reliable compliance with the City's effluent limitations under the Current Permitted Design Condition, regardless of which upgrade alternative is selected.

6.1.1.1 Secondary Clarifier Retrofits

Regardless of which secondary process upgrade alternative is selected, the existing secondary clarifiers should be retrofitted to increase hydraulic and solids capacities by providing multiple draw off locations and to replace worn equipment at or near the end of its useful life. Table 6-1 lists the recommended clarifier retrofits that should be provided under the Current Permitted Design Condition upgrade project.

Table 6-1. Proposed Retrofits for Existing Secondary Clarifiers

Retrofit	Benefit
Replace existing sludge scrapers with suction ducts	· Provide multiple draw off points · Increase sludge removal rates
Install density current baffles	· Reduce secondary effluent TSS concentration and turbidity · Reduce likelihood of solids carryover during high flows or poor settleability periods
Replace existing energy dissipating inlets and flocculation wells	· Reduce inlet velocities to improve settling performance at high flows and increase hydraulic capacity
Replace existing rake arms, inlet columns, bridges and mechanism drives	· Replace equipment at or near the end of its useful life · Accommodate new spiral blades or suction ducts (rake arms)

6.1.1.2 Headworks and Mixed Liquor Splitter Box Modification

The existing Headworks Splitter Box, which divides influent evenly among the oxidation ditches, and Mixed Liquor Splitter Box, which divides oxidation ditch effluent evenly among the clarifiers, were designed to split flow to four treatment units. Two of the four outlets in the splitter boxes are currently isolated by knockout walls and are fitted with pipe stubs. As a third (and ultimately a fourth) treatment unit is constructed, knockout walls will be removed, new weir gates will be installed, and the outlet pipes will be extended.

6.1.1.3 Solids Pumping Modification

With each new clarifier installed, the RAS and WAS pumping system will also need to be expanded. The existing Secondary Sludge Pump Station was designed to accommodate the RAS/WAS pumping for up to four secondary clarifiers. However, due to the arrangement of the pump station, pumping systems for the first and second clarifiers are likely to remain independent from the pump systems for the third and fourth clarifiers.

With the installation of each new clarifier (up to four total), one dedicated RAS and WAS pump will be installed. In addition, a second RAS pump should also be provided as a standby unit upon installation of the next (third) clarifier. However, because the standby pumps can be used to serve two clarifiers, only one additional RAS pump will be needed upon installation of a fourth clarifier. (A redundant WAS pump is not needed. The remaining operational WAS pumps can be used to compensate for any one offline WAS pump.) Also note that the existing scum pump station can be utilized for up to four clarifiers, and as such will not require expansion at the Current Permitted Design Condition.

6.1.1.4 Solids Handling Improvements

As discussed in Chapter 5 (Biosolids Handling Alternatives), the addition of supplemental mechanical dewatering is the lowest-cost approach for addressing the City's solids handling capacity needs at the Current Permitted Design Condition. Therefore, the unit costs presented in Table 5-10 have been incorporated into the costs of the alternatives under the Current Permitted Design presented herein. The amount of supplemental dewatering capacity required is dependent on the treatment alternative selected.

The following improvements to the sludge lagoons are also assumed to be included under the Current Permitted Design upgrade project:

- Installation of larger and more robust surface aerators to provide better mixing and homogenization of biosolids for more consistent dewatering performance.
- Construction of an additional vehicle access ramp to the existing lagoon berm to improve lagoon access.
- Construction of a pedestrian access stairway to the top of the lagoon berm.

6.1.1.5 SCADA Improvements

The SCADA system at the WWTP needs to be expanded to accommodate the following additional needs:

- Migrate the MSC SCADA system to the WWTP
- Provide SCADA monitoring and flow-pacing control capabilities for recent and ongoing Secondary Sludge Pump Station improvements
- Add a local off switch to the UV chemical wash station blower

6.1.1.6 Flow Equalization

The City is currently using the 5.8 million gallon Auxiliary Basin to store supernatant return flows so they can be returned to the WWTP during off-peak load hours. To provide even more control of loads entering the WWTP, the Filtrate Pump Station conveyance piping (which currently conveys filtrate from the Deskin Drying Beds to the headworks) will be modified to allow for diversion of the filtrate flows to the Auxiliary Basin for temporary storage and return during off-peak hours. (Note that the filtrate pump station can also be used to convey pond water back to the WWTP and is used to convey drainage water from select plant areas. If the filtrate pump station is being used heavily for one of these purposes, flows would likely be directly discharged to the headworks).

The volume of solids processing return flows generated in a given day is much lower than the total capacity of the Auxiliary Basin. Therefore, the basin could also be used for equalization (EQ) of influent flows to reduce the pressure on the primary and secondary treatment processes that are designed for peak flow conditions (e.g., the secondary clarifiers, and if installed, primary treatment).

To provide the flexibility to divert flows above an amount established by the WWTP staff, the existing sluice gate on the pipe leading to the Auxiliary Basin would need to be fitted with an automatic actuator. In addition, the larger Auxiliary Basin return pump would be fitted with a variable frequency drive (VFD) so that flows could be returned back to the treatment plant at a variable rate. Finally, SCADA system would be configured so the City staff could set the desired peak flow through the WWTP given the conditions at hand. During later design phases, the City will need to better define the level of SCADA automation that will be provided for both diverting flows and returning them back to the WWTP.

At the Current Permitted Design Condition it is recommended that the City not rely on an equalization process on a day-to-day basis for controlling influent flow rates through the treatment plant the following reasons:

- Daily diversions to the Auxiliary Basin and return of flows to the WWTP would create a significant amount of operational complexity that is not desired at this time.
- The City has the ability to divert undisinfected secondary effluent to the irrigation storage reservoir and subsequently to the land application area to offset the operational redundancy needs of the recently constructed tertiary filtration and UV disinfection facilities. Moreover, the tertiary processes have the capacity to handle the anticipated Current Design Condition PHWWFs with all units in service. Therefore, influent equalization does not provide much added reliability benefits for the tertiary facilities. Additional detail regarding secondary effluent diversions is provided in the Pond Use Analysis TM (Appendix F).

Therefore, it is recommended that the Auxiliary Basin only be used to equalize influent flows under relatively rare conditions. Specifically, it is envisioned the equalization basins would be used if the secondary effluent is exhibiting poor settling properties and/or if a primary or secondary treatment unit is offline for maintenance. During these conditions, flows through the WWTP would be limited to a set amount based on what can be handled by the secondary facilities that are online. Excess flows would be directed to the Auxiliary Basin and held there until such at time that all facilities are back online (and/or solids settleability has improved).

An analysis of what peak flows could be conveyed through the WWTP over a 30 day period with no return flow to the treatment process (including the solids handling flows) is provided in the Pond Use Analysis TM (Appendix F). A comparison of these flows to the design flows under the three design conditions evaluated in this Facilities Plan is presented in Table 6-2. An evaluation of how limiting the flow through the plant to the values presented in Table 6-2 impacts the sizing of downstream facilities is provided later in this Chapter.

Table 6-2. Comparison of Influent Peak Flows to Flows with Raw Wastewater Equalization

Design Condition	Winter Flow Condition		Summer Flow Conditions	
	Influent PHWWF	WWTP Peak Flow with EQ	Influent PHDWF	WWTP Peak Flow with EQ
Current Permitted Design Condition	9.0	5.9	5.0	3.8
Future Permitted Design Condition	13.5	8.8	7.5	5.8
Ultimate Buildout Design Condition	18.0	12.1	10.0	7.9

As additional winter season data is collected (and City staff becomes more comfortable using the equalization facilities), the use of the equalization facilities on a more regular basis to further reduced plant capital or operational costs may become a desired practice. Therefore, consideration of more frequent equalization should be included as part of the pre-design efforts under the Future Permitted Design Project and Ultimate Buildout Design projects.

6.1.1.7 Required Building Improvements

The existing Control Building will need to be remodeled to accommodate a larger and refurbished laboratory and to better serve the needs of existing and future City staff stationed at the WWTP. The existing Chemical/Maintenance Building should be also modified to improve vehicle access and to better serve the WWTP's maintenance needs. In addition to the improvements to the existing buildings listed above, a covered storage area should be constructed to protect the portable generators, pumps and other mobile equipment that are typically stationed at the WWTP.

6.1.1.8 Arsenic Treatment at Potable Water System Groundwater Wellheads

As discussed in the Arsenic Management Plan TM (Appendix C), the City will need to construct facilities to store and decant backwash water from the City's arsenic wellhead treatment systems and to dewater the settled solids so they can be hauled offsite for disposal. For purposes of the WWTP facility planning efforts, it is assumed that backwash dewatering equipment will be installed at the Carillion and Industrial Park groundwater treatment plants. The costs presented in the Technical Memorandum *Arsenic Treatment Residuals Disposal Assessment* (ECO:LOGIC, 2008) are incorporated into this chapter.

6.1.2 Oxidation Ditch Upgrade Alternatives

This section presents the results of the secondary process modeling analysis[1] that was used to evaluate potential alternatives for providing reliable nitrification and denitrification at the Current Permitted Design Condition. The following alternatives were considered:

- Alternative A: Expand Oxidation Ditches to Provide Primary/Secondary Treatment Needs
- Alternative B: Construct a Separate Anoxic Basin for Denitrification
- Alternative C: Provide Primary Treatment to Reduce Loads to Oxidation Ditches
- Alternative D: Provide Alternative Solids Treatment to Reduce Demands on Oxidation Ditches
- Alternative E: Expand Clarifiers to Allow Mixed Liquor Concentrations in the Oxidation Ditches to be Increased

A more detailed discussion of secondary process modeling assumptions and results is presented in the WWTP Secondary Process Modeling TM (Appendix B).

6.1.2.1 Secondary Process Modeling Assumptions

The major oxidation ditch performance assumptions that were used for secondary process modeling evaluation of the five oxidation ditch alternatives (in addition to the three design loading conditions discussed in Chapter 2 (Basis of Design) are provided in Table 6-3. The modeling analysis was also used to evaluate whether the reliability/redundancy criteria can be satisfied for each alternative as follows:

- Either satisfy the effluent BOD, ammonia, and nitrate criteria at the annual average design conditions (and average temperatures) with one unit out of service, or
- The treatment units must be fitted (at a higher cost) with equipment that can be removed and/or maintained while the treatment unit remains in service.

Finally, some landfills will accept solids that do not meet Class B standards. Therefore, if an oxidation ditch is temporarily offline, the City could operate the remaining units with an MCRT that is lower than what is needed to meet Class B standards. Any non-compliance solids could then be kept separated from the solids that are to be land applied, and hauled to a landfill for disposal. Because the City's existing facilities would allow for such flexibility, the reliability/redundancy also considers the following two different potential scenarios:

1. The MLSS is maintained at a level that ensures a minimum 25 day MCRT is achieved, thereby meeting the Class B biosolids treatment requirements[2].
2. The MLSS is maintained at a level that ensures a minimum 15 day MCRT is achieved.

[1] The EnviroSim BioWIN® secondary process modeling program was used to develop the models.

[2] Note that under Alternative D the City would be relying on a separate process for solids treatment so an MCRT of 25 days is not needed to meet Class B standards and Scenario 1 does not apply.

Table 6-3. Major Oxidation Ditch Performance Criteria Assumptions for Secondary Process Modeling

Parameter	Value	Basis
Effluent Ammonia (as N)	Average < 0.5 mg/L Maximum < 1 mg/L	Performance standards are established at levels less than the effluent criteria to minimize the potential for effluent violations under variable loading and temperature conditions.
Effluent Nitrate (as N)	Average < 8 mg/L Maximum < 10 mg/L	
Effluent BOD	Average < 3 mg/L Maximum < 5 mg/L	
Return Activated Sludge (RAS) Flow	Paced at 75 percent of influent flow minus WAS Flow	Typical value
WAS Flow	Assume a constant flow rate. Values are determined for each alternative, as needed, to maintain the desired minimum MCRT. Maximum day loading condition WAS flow rates are set equal to rates used under the Maximum month loading condition.	
Minimum Mixed Liquor Suspended Solids (MLSS) Concentration	2,000 mg/L	Assumed minimum MLSS to provide overall aeration system efficiency.
Minimum Mean Cell Residence Time (MCRT)	25 days	Minimum for all alternatives that rely on the oxidation ditches to produce Class B biosolids.
	15 days	Minimum if solids treatment is being provided in a separate process.
Aeration System Configuration	Two Surface Aerators per Reactor	For comparison purposes, the aeration system design is assumed to be consistent with the configuration and performance of existing equipment. However, in future design phases, specific equipment will be evaluated to determine the most efficient approach to meeting the system aeration needs.
Standard Aeration Efficiency	3.5 lb O_2/HP-hr	
α_F	0.5	Factors typical for surface aerator systems range from 0.5 to 1.0.

6.1.2.2 Alternative A – Additional Oxidation Ditch Capacity Only

Alternative A entails construction of a third oxidation ditch to increase the overall secondary treatment capacity. A summary of the process modeling results for this alternative are shown in Table 6-4.

Table 6-4. Alternative A (Oxidation Ditch Only) Secondary Process Modeling Results Current Permitted Design Condition

Design Condition[(a)]	# of Ditches	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations			Total HP Required per Reactor[(a)]
		MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	
Total Capacity							
Average	3	2,597	25	0.31	2.5	1.0	90
Maximum Month		3,425	25	0.39	8.2	1.1	100
Maximum Day		—	—	0.44	3.3	1.2	110
Firm Capacity							
Annual Average	2	4,082	25	0.51	0.0	1.7	105
		3,410	21	0.45	0.0	1.5	110

(a) The two existing surface aerators can provide a total of 120 HP per reactor.

As shown in Table 6-4, reliable treatment should be achieved with three oxidation ditches if the MLSS concentrations range between 2,600 mg/L during the average loading conditions up to approximately 3,400 mg/L during maximum month loading conditions.

The results in Table 6-4 also show that the horsepower (HP) requirements for the aeration system for each reactor would be less than what can currently be provided. However, it is assumed that new diffuser-based aeration equipment (and related monitoring and control facilities) will be installed to ensure simultaneous nitrification/denitrification can be achieved. (Note that in later design phases it may be determined that the existing equipment can be retrofitted to provide the level of control needed, thereby reducing project costs.)

The results in Table 6-4 demonstrate that the City should also be able to provide reliable treatment at the annual average conditions if one of the reactors if offline for maintenance. However, to maintain a 25-day MCRT, the mixed liquor would need to increase to approximately 4,100 mg/L. As discussed in more detail later in this chapter, this higher MLSS concentration may be acceptable if the SVI is low enough and there is adequate clarifier capacity. However, if settling became an issue, the City could lower the MLSS and simply haul the dewatered biosolids that do not meet Class B standards to a landfill for disposal.

6.1.2.3 Alternative B – Separate Anoxic Basin

Alternative B entails construction of a separate anoxic basin as an alternative means of facilitating denitrification. Therefore, instead of retrofitting the ditches with the control facilities needed to accommodate simultaneous nitrification/denitrification, a separate basin would be constructed to provide the anoxic reactor volume needed to achieve reliable treatment. One common anoxic basin, equal to approximately 33 percent of the total reactor volume (or 1 million gallons), would be located upstream of both the existing oxidation ditches. In addition, pumping facilities would be installed in the oxidation ditches to return mixed liquor from the ditches to the anoxic basin to achieve denitrification. A summary of the process modeling results for this alternative are shown in Table 6-5.

Table 6-5. Alternative B (Separate Anoxic Basin) Secondary Process Modeling Results Current Permitted Design Condition

Design Condition[(a)]	# of Ditches	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations			Total HP Required per Reactor[(a)]
		MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	
Total Capacity							
Average	2	3,097	25	0.31	4.1	1.1	120
Maximum Month		3,994	25	0.39	6.4	1.2	140
Maximum Day		—	—	0.41	8.2	1.4	160
Firm Capacity							
Average	1	6,207	25	0.51	0.1	1.7	210
		3,989	15	0.54	1.2	1.4	240

[(a)] The two existing surface aerators can provide a total of 120 HP per reactor.

As shown in Table 6-5, reliable treatment should be achieved if a separate 1 million gallon anoxic basin is constructed, and the MLSS concentrations ranges from 3,100 under the average loading conditions up to 4,000 mg/L under maximum month conditions. This mixed liquor concentration is near the upper end of what can be readily settled in conventional secondary clarifiers. An evaluation of clarifier requirements at these MLSS levels is provided later in this chapter.

Although the aeration system would not need to be automatically controlled to provide reliable simultaneous nitrification/denitrification, some automation would be desirable to provide increased process efficiency. In addition, as shown in Table 6-5 the existing system's aeration capacity would not be adequate to meet treatment needs, so new surface aerators would need to be installed.

The results in Table 6-5 also demonstrate that, if a reactor were offline for service, the City would need to maintain a MLSS concentration of 6,200 mg/L to provide reliable treatment at the average conditions and still maintain a 25-day MCRT. The settleability of effluent with a MLSS concentration this high would only be possible if the SVI was low, and there were an adequate number of clarifiers. However, the City could lower the MLSS to the design level of 4,000 mg/L and haul the dewatered biosolids that do not meet Class B standards to a landfill for disposal.

Under this alternative, the aeration equipment in each reactor would consist of two surface aerators. Each aeration unit would be capable of providing 100 percent of the aeration needs so that treatment will not be compromised if one aerator is offline. If an entire reactor were offline, the two aeration units in the remaining online reactor would also be capable of meeting the entire systems aeration needs as shown in Table 6-5.

6.1.2.4 Alternative C – Primary Microscreening

Alternative C entails the addition of primary microscreening to the existing treatment train to reduce the organic loading to the oxidation ditches, thereby offsetting the capacity needed in the oxidation ditches and reducing the aeration requirements. In addition to the microscreens, Alternative C would entail replacement of aeration facilities with a diffused air system that has the controls needed to ensure reliable simultaneous nitrification/denitrification.

Microscreening is a relatively new process, which relies on a wire cloth with a mesh size from 0.1 to 1.0 mm to remove solids from the raw wastewater. Microscreening, which has been used primary in Europe, provides an equivalent level of constituent removal as conventional primary clarification process. A comparison of the major operational considerations associated with both microscreening and conventional primary treatment is provided in Table 6-6. As shown, microscreens provide a number of added benefits in terms of operational flexibility and require less supporting infrastructure.

Estimated percent removals of key constituents for the secondary process modeling effort for two different potential sizes of microscreens were provided by a primary microscreening equipment manufacturer and are shown in Table 6-7. Given the uncertainties with removals that would be achieved at the Galt WWTP, the estimated lower removals associated with the larger screen size were used for the modeling analysis.

Finally, primary microscreens are sized based on anticipated peak flows, and it is envisioned that three units – each having the capacity to treat up to 3.0 mgd – will be installed. As discussed previously, the City will be able to equalize flows so that flow through the treatment plant can be maintained at less than 6.0 mgd during periods when a unit is out of service. Therefore, it will not be necessary to provide a redundant microscreening unit.

Table 6-6. Comparison of Major Operational Requirements for Microscreening and Conventional Primary Clarification

Parameter	Primary Microscreening	Primary Clarification
Headlosses	2-3 feet	2-3 feet
Solids Treatment Needs	Screenings are washed and compacted as they are separated from the liquid stream. The resulting product has a 20 to 50 percent solid content that can be hauled offsite to a landfill.	The resulting product has a solids content of 2 to 6 percent. A primary solids treatment facility (likely anaerobic digesters) would need to be constructed at the WWTP site.
Anaerobic Co-Digestion of Primary and Secondary Solids[(a)]	It would be possible to use the higher solids content product in an anaerobic co-digestion process. However, specialized equipment – similar to the equipment used to feed manure and food wastes to anaerobic digesters - would be required.	Co-digestion of solids from primary and secondary clarification processes is a well-established (and often preferred) solids handling strategy.
Solids Dewatering	Annual average secondary solids production rates are reduced, resulting in less required dewatering equipment.	Any reduction in secondary solids would be offset by the need to dewater treated primary solids.
Solids Disposal	The City would immediately be incurring costs for hauling and disposal of primary solids. However, the existing land disposal site could be used for secondary solids disposal for a significantly longer period of time (likely offsetting the near-term hauling costs).	Land disposal capacity needs would be generally unchanged, and the City would need to consider additional disposal options before the Future Permitted Design Condition (i.e., ADWF = 4.5) is reached.
Energy Production	As noted above, it would be possible to use the primary solids in more conventional anaerobic reactors – thereby resulting in the production of biogas, which could be used for energy generation. However, materials with higher percent solids content offer even more opportunities for energy production – particularly as newer technologies like pyrolysis and gasification become more widely available.	Without dewatering, energy production opportunities would be limited to those intended for liquid wastes – such as anaerobic digestion. Because of the high liquid content, heat-based energy generation options (i.e., pyrolysis or gasification) would also require a dewatering process.

[(a)] Anaerobic treatment is more energy efficient than aerobic treatment and it provides an opportunity for biogas generation (and eventual use). However, secondary solids can only be reliably treated in an anaerobic process if they are being co-digested with primary solids.

Table 6-7. Primary Microscreening Constituent Removal

Parameter	Units	Estimated Percent Removal via Microscreening	
		Mesh Size = 300 µm	Mesh Size = 160 µm
Organics			
BOD-total	mg/L	29%	44%
BOD-soluble	mg/L	5.2%	12%
COD-total	mg/L	30%	50%
COD-soluble	mg/L	0%	22%
Solids			
TSS	mg/L	52%	62%
VSS	mg/L	48%	59%
Inert Suspended Solids	mg/L	81%	81%
Nitrogen			
Ammonia as N	mg/L	5.1%	5.1%
Nitrate as N	mg/L	0%	0%
TKN-total	mg/L	3.9%	3.9%
TKN-soluble	mg/L	0%	0%
Phosphorus			
Phosphorus-total	mg/L	11%	22%
Phosphorus-soluble	mg/L	0%	0%

A summary of the process modeling results for this alternative is shown in Table 6-8. As shown, the City should be able to provide reliable treatment with two oxidation ditches if microscreening is installed. In addition, because of the reduced loads, the required MCRT of 25 days could be maintained with MLSS concentrations that range between 2,000 under the average design loading conditions up to 2,400 mg/L. Moreover, as further detailed in the WWTP Secondary Process Modeling TM (Appendix B), Alternative C would have approximately 50% less volume of secondary solids produced annually.

Alternative C also has the lowest aeration demands of all the alternatives. However, the reduced demand is not proportional to the reduction in influent loads because the efficiencies realized from the higher MLSS concentrations in some of the other alternatives would not occur under Alternative C.

The results in Table 6-8 also demonstrate that, if a reactor were offline for service, the City would need to maintain a MLSS concentration of 3,800 mg/L to provide reliable treatment at the average conditions and still maintain a 25-day MCRT. Similar to Alternative A, this higher MLSS concentration may be acceptable if the SVI is low enough and there is adequate clarifier capacity. However, if settling became an issue, the City could lower the MLSS and simply haul the dewatered biosolids that do not meet Class B standards to a landfill for disposal.

Table 6-8. Alternative C (Primary Microscreening) Secondary Process Modeling Results Current Permitted Design Condition

Design Condition[(a)]	# of Ditches	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations			Total HP Required per Reactor[(a)]
		MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	
Total Capacity							
Average	2	2,016	28	0.33	3.8	1.0	120
Maximum Month		2,372	25	0.53	4.6	1.2	130
Maximum Day		—	—	0.47	6.8	1.3	150
Firm Capacity							
Average	1	3,810	25	0.51	0.0	1.8	205
		2,417	14	0.47	0.1	1.5	220

[(a)] The two existing surface aerators can provide a total of 120 HP per reactor.

Moreover, if a reactor was offline for maintenance, the amount of air needed in the remaining reactor exceeds what would be provided at the maximum day load conditions with both reactors online. Therefore, the aeration system design must take this additional aeration requirement into account.

Regardless of what approach to redundancy is selected, at least one reactor would need to be retrofitted with the new equipment while in service, as it is not likely that one basin could provide reliable treatment with the current equipment installed. For purposes of this analysis, it has been assumed that new aeration diffusers would be installed on rails to allow for access and maintenance while the basin is in service. Additional evaluation of the options for meeting the redundancy requirements should be considered in design if Alternative C is selected as the preferred approach.

Finally, as noted in Table 6-6, primary microscreening will require approximately 2 to 3 feet of additional head through the WWTP between the headworks and the oxidation ditches. A review of the hydraulic grade line through this portion of the WWTP reveals that hydraulic improvements at the headworks are needed to provide this additional head. Specifically, the walls in bar screen and grit influent area need to be raised approximately 1.5 ft. In addition, the bar screen itself will need to be mounted 1.5 ft higher, which will require installation of a new bar screen. (Note that bar screen replacement is discussed later in this chapter as a maintenance improvement that does not necessarily need to occur when the plant is upgraded for the Current Permitted Design Condition. However, if Alternative C were selected as the preferred option, bar screen replacement would move forward as a near-term project.) The Live Oak Pump Station will also experience the additional head as a higher discharge pressure. It is assumed that the resulting change in pumping rates can be accommodated or mitigated.

6.1.2.5 Alternative D – Separate Biosolids Treatment

Alternative D entails construction of a separate facility for aerobic solids digestion, which would allow the City to reduce the MCRT in the oxidation ditches to a minimum of 15 days. Similar to Alternatives A and C, Alternative D would also entail replacement of aeration facilities and installing controls in the oxidation ditches to ensure reliable simultaneous nitrification/denitrification. A summary of the process modeling results for this alternative are shown in Table 6-9.

Table 6-9. Alternative D (Separate Biosolids Treatment) Secondary Process Modeling Results Current Permitted Design Condition

Design Condition[(a)]	# of Ditches	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations			Total HP Required per Reactor[(a)]
		MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	
Total Capacity							
Average	2	2,533	15	0.37	2.1	1.1	125
Maximum Month		3,243	15	0.58	3.7	1.3	140
Maximum Day		—	—	0.51	7.2	1.4	170
Firm Capacity							
Average	1	5,067	15	0.58	0.0	1.7	220

(a) The two existing surface aerators can provide a total of 120 HP per reactor.

As shown in Table 6-9, the City should be able to provide reliable treatment with two oxidation ditches with a minimum MCRT of 15 days. In this case, MLSS concentrations would range from 2,500 under the average loading conditions up to 3,200 mg/L under maximum month conditions.

However, the oxidation ditch aeration requirements are the greatest under this alternative at the Current Permitted Design Condition for the following two reasons:

- Reduced efficiency because of the lower MLSS concentrations.
- The lower MCRT reduces the robustness of the system, and short-term diurnal peaks require additional aeration to ensure effluent concentrations are satisfied.

The results in Table 6-9 also demonstrate that, if a reactor were offline for service, the City would need to maintain a MLSS concentration of 5,000 mg/L to meet a 15-day MCRT. It is unlikely that the effluent would be readily settleable at this high of a MLSS concentration – even with three clarifiers online. Therefore, for purposes of this analysis, it has been assumed that all new aeration diffuser equipment would be installed on rails to allow for access and maintenance while the basin is in service.

The aerobic digesters were sized to ensure reliable solids stabilization during the maximum month loading conditions. Volume requirements for aerobic digestion under the Current Permitted Design Condition are based on the following parameters:

- Total and volatile suspended solids content of the WAS was provided by the secondary process model;
- With an aeration demand of approximately 1.0 lb oxygen per pound of VSS per day, a minimum VSS destruction of 38% will be possible;
- Minimum SRT of 60 days;
- Estimated digested sludge solids concentration of 2%; and
- Estimated unusable portion of digester(s) of 10%.

Assuming two digesters will be constructed under the Current Permitted Design Condition, each digester will be approximately 1.2 MG. Assuming a typical of depth of 20-25 feet, circular digesters would have a diameter of 90-100 feet.

6.1.2.6 Alternative E – Additional Clarifier Capacity

Alternative E would involve replacing the aeration and related monitoring and control facilities in the existing oxidation ditches with the facilities needed to provide reliable simultaneous nitrification/denitrification. However, in lieu of increasing the reactor volume to meet the MCRT requirements, the MLSS concentrations would be allowed to increase to the level needed to provide the required MCRT. Thus, this alternative relies primarily on increased clarifier capacity. A summary of the process modeling results for this alternative are shown in Table 6-10.

Table 6-10. Alternative E (Clarifier Capacity) Secondary Process Modeling Results Current Permitted Design Condition

Design Condition[(a)]	# of Ditches	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations			Total HP Required per Reactor[(a)]
		MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	
Total Capacity							
Annual Average	2	4,082	25	0.510	0.0	1.7	120
Maximum Month		5,183	25	0.550	0.0	1.5	140
Maximum Day		—	—	0.380	6.2	1.4	160
Firm Capacity							
Annual Average	1	8,071	25	0.500	0.0	2.1	215
		5,205	15	0.520	0.0	1.9	225

(a) The two existing surface aerators can provide a total of 120 HP per reactor.

As shown in Table 6-10, the City should be able to provide reliable treatment with two oxidation ditches, with an MCRT at 25 days, if the MLSS is increased to 4,100 under the average design loading conditions and up to 5,200 mg/L under maximum month design conditions. This would be at the upper end of what MLSS levels can be maintained in oxidation ditch reactors without impacting system efficiency. An evaluation of how these high MLSS levels could affect clarifier performance is provided in the next section of this chapter.

The results in Table 6-10 also demonstrate that, if a reactor were offline for service, the City would need to maintain a MLSS concentration of 8,100 mg/L to provide reliable treatment at the average conditions and still maintain a 25-day MCRT, which would not be acceptable for reliable settling, even if the SVI was low. Therefore, the only option would be to lower the MLSS and haul the dewatered biosolids that do not meet Class B standards to a landfill for disposal.

Similar to Alternative C, the newly installed aeration system should ensure the oxygen demands can be met if a basin be offline for service. Nevertheless, regardless of what approach to redundancy is selected, at least one reactor would need to be retrofitted with the new equipment while in service, as it is not likely that one basin could provide reliable treatment with the current equipment installed. For purposes of this analysis, it has been assumed that all new aeration diffuser equipment would be installed on rails to allow for access and maintenance while the basin is in service. Additional evaluation of the options for meeting the redundancy requirements should be considered in later design phases if Alternative E is selected as the preferred approach.

6.1.3 Secondary Clarifier Capacity Needs

This section presents a discussion of the secondary clarifier capacity needs for each of the five secondary process alternatives listed above. The following topics are addressed:

- Design Criteria
- State Point Analysis Results

6.1.3.1 Design Criteria

As discussed above in this chapter, the existing clarifiers will be retrofitted with equipment that provides multiple draw off points for the settled sludge. Therefore, a spreadsheet state point analysis tool, which provides a graphical means of predicting and evaluating performance of clarifiers with multi-point draw offs, was used to evaluate the clarification capacity needed under the five alternatives discussed in the previous section.

The criteria used for the state point analysis are presented in Table 6-11. As shown, one factor considered in the clarifier state point analysis is the fact that the City's Auxiliary Basin system could be used for EQ of influent flows to reduce the pressure on the secondary clarifiers – particularly if effluent is exhibiting poor settling (i.e., high SVI conditions). Therefore, Table 6-11 provides the minimum required treatment capacity with equalization. The state point analysis was used to determine how many clarifiers are required to provide at least this minimum amount of treatment.

Table 6-11. Secondary Clarifier State Point Analysis Criteria Current Permitted Design Condition

Parameter		Design Criterion
Diameter		80 ft
RAS Flow		75% of Influent Flow up to 1.94 mgd[a] per Clarifier
Maximum SVI[b]	All Clarifiers in Service[c]	250
	One Clarifier Out of Service[d]	200
Design Influent Flow Peak[e]	Winter Months[c]	9.0 mgd
	Summer Months[d]	5.0 mgd
Minimum Required Treatment with EQ[f]	Winter Months[c]	5.9 mgd
	Summer Months[d]	3.8 mgd
Design MLSS	Alternative A	3,400
	Alternative B	4,000
	Alternative C	2,400
	Alternative D	3,200
	Alternative E	5,200

(a) Best Efficiency Point (BEP) capacity of recently installed submersible RAS pumps.
(b) Bulking and poor settling characteristics can occur as a result from both high MCRTs and low DO levels in the oxidation ditches, both of which could occur under the alternatives being considered. Therefore, the historically high SVI values observed at the WWTP could be experienced in the future and should be accounted for in the clarifier analysis.
(c) All clarifiers would be in service during the winter month flow conditions.
(d) Assume a clarifier may be out of service for up to 30 days during the low-flow summer months.
(e) Design PHWWF and PHDWF at the Current Permitted Design Condition.
(f) The evaluation used to determine the sustained peak WWTP design flows with EQ facilities online are discussed in the Pond Use Analysis TM (Appendix F).

6.1.3.2 State Point Analysis Results

A comparison of the state point analysis results for each alternative given the design conditions discussed above is presented in Table 6-12. As shown, the equalization facilities may be needed under all the alternatives, except Alternative C, if the solids are exhibiting poor settling properties during the winter months.

Table 6-12. Secondary Clarifier State Point Analysis of Alternatives Current Permitted Design Condition

# of Clarifiers	Discharge Season	Maximum Flow that Meets State Point Analysis mgd	Equalization Potentially Needed? (Maximum Flow < Design Influent Flow)	Design Flow Rate	Overflow Rate, gal/ft^2/day	Solids Loading Rate, lb/ft^2/day
Alternative A (Oxidation Ditch Only): Design MLSS = 3,400						
3	Winter	7.9	Yes: 7.9 < 9.0	7.9	524	26
2	Summer	6.8	No: 6.8 > 5.0	5.0	497	25
Alternative B (Separate Anoxic Basin): Design MLSS = 4,000						
4	Winter	7.8	Yes: 7.8 < 9.0	7.8	388	26
3	Summer	7.7	No: 7.7 > 5.0	5.0	497	22
Alternative C (Primary Microscreening): Design MLSS = 2,400						
2	Winter	9.1	No: 9.1 > 9.0	9.0	895	26
1	Summer	5.6	No: 5.6 > 5.0	5.0	995	28
Alternative D (Separate Biosolids Treatment): Design MLSS = 3,200						
3	Winter	8.7	Yes: 8.7 < 9.0	8.7	577	26
2	Summer	7.5	No: 7.5 > 5.0	5.0	497	24
Alternative E (Clarifier Capacity Only): Design MLSS = 5,200						
8	Winter	6.1	Yes: 6.1 < 9.0	6.1	149	13
7	Summer	9.2	No: 6.7 > 5.0	5.0	142	12

Based on the information presented in Table 6-12, the viability of Alternative B is uncertain. Specifically, the analysis indicates that a total of four clarifiers would be needed to treat a flow greater than 5.9 mgd, which is the minimum sustained peak determined to be achievable during the winter month determined through the EQ analysis presented in the Pond Use Analysis TM (Appendix F). Therefore, Alternative B relies too much on clarifier capacity to achieve treatment performance standards, and a more efficient approach to treatment would be Alternative A. Consequently, Alternative B has been removed from further consideration.

Similar to Alternative B, Alternative E requires an excessive number of clarifiers to treat a constant flow greater than 6.0 mgd during the winter months and the viability of meeting this design standard is uncertain. In addition, the reliance on eight clarifiers to provide reliable treatment at the Current Permitted Design Condition is not reasonable. Therefore, Alternative E has been removed from further consideration.

6.1.4 Tertiary Filtration and UV Disinfection Facilities Requirements

The design capacity of the tertiary filtration and UV disinfection facilities is based on anticipated peak flows through the WWTP, and the existing tertiary filtration and UV disinfection facilities were designed to handle the peak flow at the Current Permitted Design Condition when all units are in service. Moreover, the Filter Influent Pump Station is designed to passively overflow to the Effluent Storage Reservoir if the treatment capacity is exceeded. Therefore, if a filtration or disinfection unit is out of service, flows in excess of the remaining capacity could be diverted.

(Note that the City is allowed to apply disinfected secondary effluent to the irrigation areas.) An evaluation of the diversion volumes that would accumulate in the Effluent Storage Reservoir under an assumed "worst-case" condition where a unit were out of service for an extended period of time is presented in the Pond Use Analysis TM (Appendix F). Based on this analysis, it is concluded that the existing filtration and UV disinfection facilities are adequately sized for the Current Permitted Design Condition and additional facilities are not necessary.

6.1.5 Summary of Potential Upgrade Alternatives

Table 6-13 provides a summary of the number of major process units under existing conditions and those required under each of the three viable alternatives at the Current Permitted Design Condition. Schematic diagrams for each alternative are provided in Figures 6-1, 6-2 and 6-3.

Table 6-13. Treatment Process Unit Installed Quantities for Existing Conditions and Viable Alternatives at the Current Permitted Design Condition

Process/Equipment	Existing Conditions	Alternative A: Oxidation Ditch Only	Alternative C: Primary Microscreening	Alternative D: Separate Biosolids Treatment
Microscreens	—	—	3	—
Vortex Grit Chambers	1	1	—	1
Oxidation Ditches	2	3	2	2
Secondary Clarifiers	2	3	2	3
Filter Feed Pumps	3	3	3	3
Tertiary Filters	3	3	3	3
UV Channels	3	3	3	3
Aerobic Digesters	—	—	—	2
Sludge Lagoons	2	2	2	2
Screw Presses[(a)]	—	2	1	2

[(a)] From Chapter 5 (Biosolids Handling Alternatives).

6.2 FUTURE PERMITTED DESIGN CONDITION (4.5 MGD ADWF) EXPANSION ALTERNATIVES

This section describes the alternatives for expanding the WWTP to address capacity needs at the Future Permitted Design Condition. The following specific topics are addressed:

- Project Components Common to All Alternatives
- Oxidation Ditch Expansion Alternatives
- Secondary Clarifier Capacity Needs

- Tertiary Filtration and UV Disinfection Facilities Requirements
- Summary of Potential Expansion Alternatives

6.2.1 Project Components Common to All Alternatives

This section describes common improvements that are necessary to ensure reliable compliance with the City's effluent limitations under the Future Permitted Design Condition, regardless of which expansion alternative is selected.

6.2.1.1 Effluent Flow Metering Improvements

The existing Effluent Flow Metering facility should be replaced with a new facility located near the Effluent Diversion Structure. The new facility should include a magnetic flow metering device sized to minimize head losses under peak flow conditions. The meter should be installed in a vault with either sufficient gravity drainage for incidental inflow during wet weather, or with a drainage pump automatically activated by water level inside the vault.

Following completion of the new metering facility, the existing Outfall Junction Structure and Effluent Flow Metering facilities near the Storage Reservoir dam should be demolished, and the existing 36-inch final effluent pipeline should be extended to discharge at an appropriate location within the existing Skunk Creek stream bed. This extension will replace the existing 18-inch outfall pipe between the Outfall Junction Structure and Skunk Creek.

6.2.1.2 Hydraulic Improvements

As discussed in Chapter 4 (Capacity Analysis of Existing Wastewater Treatment Facilities), the following hydraulic improvements will need to be made when the WWTP is expanded to the Future Permitted Design Condition[3]:

- Replace the above grade piping between Grit Removal Facility and Headworks Splitter Box with 30-inch diameter pipe.
- Replace the buried piping between the RAS pipe junction and Headworks Splitter Box with a 42-inch diameter pipe.
- Install a new 30-inch diameter overflow pipe between an existing manhole located along the 36-inch overflow line (which runs between the Filter Feed Pump Station and the Effluent Diversion Structure) and Pond 4.

6.2.1.3 Electrical Service and Switchgear

The existing electrical service and switchgear will need to be upgraded as the electrical loads at the WWTP increase. Although a detailed evaluation of the current and anticipated loads has not been developed, it is estimated that the service will need to be upgraded when the City expands the WWTP to meet the Future Permitted Design Condition and the improvements will serve the

[3] Note that 36" pipes will need to be installed between the headworks splitter box and any future oxidation ditches. (Existing piping is 30" diameter.)

City through the Ultimate Buildout Design Condition. The improvements are anticipated to include a new utility (i.e., Sacramento Municipal Utility District (SMUD)) transformer, new switchgear, and re-feeding from the new switchgear to the existing switchgear.

6.2.1.4 Standby Generation

The existing generator was installed as part of the 2010 tertiary improvements, and combines with the plant's original generator to provide up to 900 kW of available backup power. It is anticipated that additional standby generation will be required at the Future Permitted Design Condition, and the expansion project will include installing two additional 600 kW generators identical to the unit installed in 2010. (Note that the existing main switchgear installed in 2010 was designed to accept these new units.) It should be noted, however, that the detailed design of the improvements required under Current Permitted Design Condition could demonstrate the need for additional back-up generation capacity as part of the near-term (i.e., current capacity upgrade) project.

6.2.1.5 Headworks and Mixed Liquor Splitter Box Improvements

As discussed in the Current Permitted Design Condition evaluation, the existing Headworks and Mixed Liquor Splitter Box can accommodate up to four downstream treatment units. If more than four treatment units are required (applicable only for the Headworks Splitter Box under Alternative A), it is anticipated that a new splitter box will be constructed with room for a total of six flow splits. (Note that this structure would accommodate expansion beyond the planning horizon covered by this Facilities Master Plan.)

6.2.1.6 Solids Pumping Improvements

As discussed in the Current Permitted Design Condition evaluation, the existing Secondary Sludge Pump Station can accommodate the RAS/WAS pumping needs for up to four secondary clarifiers. Two RAS pumps and one WAS pump will be installed when the third clarifier is added, and a third RAS pump and second WAS pump will be installed when the fourth clarifier is added. Also note that the existing scum pump station can be utilized for up to four clarifiers, and as such will not require expansion at the Future Permitted Design Condition.

6.2.1.7 Biosolids Handling and Disposal

It is assumed that the supplemental mechanical dewatering system will be further expanded under the Future Permitted Design Condition to meet the additional solids handling needs. However, the amount of supplemental dewatering capacity required is dependent on the treatment alternative selected. In addition, one redundant solids dewatering unit would be provided under each alternative.

As discussed in Chapter 5 (Biosolids Handling Alternatives) the existing land disposal facilities are not adequately sized to handle the solids generated at the Future Permitted Design Condition. For purposes of the analysis presented in this chapter, it is assumed the City will incur added operation costs to haul biosolids to a landfill for disposal.

Nevertheless, it is recommended that the City reevaluate the available solids handling options to consider the following factors:

- Actual solids generation rates given operating data collected after the new facilities are online,
- Newly available (or improved) technologies for solids treatment and/or dewatering,
- Opportunities for onsite energy generation using the wastewater solids, and
- Condition and operational constraints of continued operation of the drying beds.

6.2.2 Oxidation Ditch Expansion Alternatives

This section presents the results of the secondary process modeling analysis used to evaluate the reactor operating conditions at the Future Permitted Design Condition. The three remaining secondary process alternatives[4] and a new hybrid alternative were evaluated, as follows:

- Alternative A: Expand Oxidation Ditches to Provide Primary/Secondary Treatment Needs
- Alternative C: Provide Primary Treatment to Reduce Loads to Oxidation Ditches
- Alternative D: Provide Alternative Solids Treatment to Reduce Demands on Oxidation Ditches
- Alternative F: Provide Primary Treatment to Reduce Loads to Oxidation Ditches and Provide an Alternative Solids Treatment to Reduce Demands on Oxidation Ditches

6.2.2.1 Alternative A – Additional Oxidation Ditch Capacity Only

Under Alternative A, each oxidation ditch will handle the flow and loads associated with approximately 1.0 mgd of ADWF into the plant. However, under the Future Permitted Design Condition, the ADWF into the plant will be 4.5 mgd. Therefore, in lieu of constructing a smaller reactor to accommodate the added loads associated with the 0.5 mgd of ADWF, five reactors (each capable of treating flows associated with approximately 1.0 mgd of ADWF into the plant) will be provided. This configuration will also best position the City for expansion to handle the Ultimate Buildout Design Condition, when the ADWF is expected to be 6.0 mgd.

Because there will be a different ratio of influent loads to reactor volume at the Future Permitted Design Condition under Alternative A, a secondary process model was developed to define the anticipated operating conditions. A summary of the process modeling results for this alternative are shown in Table 6-14.

[4] Alternative B, construct a separate anoxic basin, and Alternative E, expansion of the clarifiers to provide improved settling of higher mixed liquor concentrations (both in lieu of expanding the Oxidation Ditches), have been eliminated from consideration.

Table 6-14. Alternative A (Oxidation Ditches Only) Secondary Process Modeling Results Future Permitted Design Condition

Design Condition[(a)]	# of Ditches	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations			Total HP Required per Reactor[(a)]
		MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	
Total Capacity							
Annual Average	5	2,327	25	0.27	6.2	1.0	80
Maximum Month		3,454	25	0.44	4.2	1.1	95
Maximum Day		—	—	0.43	5.7	1.2	100
Firm Capacity							
Annual Average	4	2,908	25	0.27	4.9	1.0	100

(a) The two existing surface aerators can provide a total of 120 HP per reactor.

As shown in Table 6-14, reliable treatment should be achieved with five oxidation ditches if the MLSS concentrations range between 2,300 mg/L during the average loading conditions up to approximately 3,500 mg/L during maximum month loading conditions. In addition, the horsepower (HP) requirements for the aeration system for each reactor would be slightly less than what is needed at the Current Permitted Design Condition (and less than what is currently provided).

The results in Table 6-14 demonstrate that the City should also be able to provide reliable treatment at the annual average conditions if one of the reactors if offline for maintenance.

6.2.2.2 Alternative C – Primary Microscreening

One new oxidation ditch reactor will be added to handle the loads associated with the increase in ADWF of 1.5 mgd at the Future Permitted Design Condition. Therefore, the ratio of influent loads to reactor volume will be the same at both the Current and Future Permitted Design Conditions for Alternative C and the modeling analysis presented in Table 6-8 for the Current Permitted Design Conditions are also applicable to the Future Permitted Design Condition.

It should be noted, however, the microscreening units will provide more capacity than what is required at the Future Permitted Design Condition, which could result in improved performance. Specifically, each microscreening unit is capable of handling a flow up to 3 mgd, and the peak flow at the Future Permitted Design Condition is anticipated to be 13.5 mgd. Therefore, a total of five microscreening units will be provided resulting in a total capacity of 15 mgd[5]. However, the difference in treatment performance associated with excess microscreening capacity cannot reasonably be defined without pilot testing. Therefore, any added treatment benefit that would occur has not been accounted for in this analysis.

[5] If necessary, the EQ facilities will be used during periods when a unit is out of service and flows greater than 12 mgd are anticipated.

In addition, as noted previously, there may be opportunities for onsite solids treatment and/or energy generation with the microscreening alternative. Therefore, if the City elects to pursue microscreening as a treatment option, it is recommended that a review of the solids handling options be considered when planning for the Future Permitted Design Condition expansion. Nevertheless, for purposes of this Facilities Plan, it has been assumed that the City would continue the practice of hauling the microscreening solids to a landfill for disposal.

Meeting the treatment standards with a unit out of service would be more readily achievable under this alternative at the Future Permitted Design Condition because the City would lose only a third of the total system capacity with a reactor down instead of half of the capacity. Therefore, the City should be able to take a reactor offline for maintenance of aeration and mixing equipment, and a specialized installation allowing for access to the reactor's equipment with the reactor online is not necessary. Additional evaluation of the aeration capacity needed to ensure the redundancy requirements can be met will need to be developed in later design phases if Alternative C is selected as the preferred approach.

6.2.2.3 Alternative D – Separate Biosolids Treatment

Expansion of Alternative D for the Future Permitted Design Condition would involve constructing a third oxidation ditch and a third 1.2 MG solids digester. Similar to Alternative C, secondary reactor performance of Alternative D at the Future Permitted Design Condition is described by the parameters shown in Table 6-9 for the Current Permitted Design Condition. Moreover, the City should be able to take a reactor offline for maintenance of aeration and mixing equipment, and a specialized installation allowing for access to the reactor's equipment with the reactor online is not necessary.

6.2.2.4 Alternative F – Microscreening/Solids Treatment Hybrid

Alternative F combines the treatment technologies featured in Alternatives C and D as the WWTP is expanded to meet the Future Permitted Design Condition and beyond. Instead of constructing a third oxidation ditch at the Future Permitted Design Condition, Alternative F would either supplement primary microscreening installed for Alternative C at the Current Permitted Design Condition with aerobic digestion, or vice-versa. The combination of reduced loading to the secondary process and shifting solids stabilization from the oxidation ditches to aerobic digesters would result in sufficiently low MLSS concentrations and MCRTs in the oxidation ditches to continue to provide reliable nitrification and denitrification without additional treatment volume.

Under Alternative F, each oxidation ditch will ultimately need to be able handle the flow and loads associated with approximately 2.0 mgd of ADWF into the plant. However, under the Future Permitted Design Condition, the ADWF into the plant will be 4.5 mgd. Therefore, in lieu of constructing a third reactor to accommodate the added loads associated with the 0.5 mgd of ADWF, the aeration capacity in the two reactors would be increased so that each would be capable of treating flows associated with approximately 2.25 mgd of ADWF. This configuration will allow the City to postpone construction of a third reactor until the Ultimate Buildout Design Condition expansion project, when the ADWF is expected to be 6.0 mgd.

A summary of the process modeling results for this alternative are shown in Table 6-15.

Table 6-15. Alternative F (C-D Hybrid) Secondary Process Modeling Results Future Permitted Design Condition

Design Condition[a]	# of Ditches	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations			Total HP Required per Reactor[a]
		MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	
Total Capacity							
Annual Average	2	1,992	16	0.36	5.8	1.1	185
Maximum Month		2,421	15	0.66	6.0	1.3	200
Maximum Day		—	—	0.60	8.1	1.5	240
Firm Capacity							
Annual Average	1	3,784	15	0.40	3.5	1.4	400

[a] The two existing surface aerators can provide a total of 120 HP per reactor.

As shown in Table 6-15, the City should be able to provide reliable treatment at the Future Permitted Design Condition with two oxidation ditches if primary microscreening is provided and the minimum MCRT is 15 days. In this case, the MLSS concentrations would range from 2,000 under the average loading conditions up to 2,400 mg/L under maximum month conditions. However, the aeration requirements per reactor are higher for this alternative than for either Alternative C or D under the Current Permitted Design Condition. (Alternative C requires 150 HP per reactor and Alternative D requires 180 HP). Therefore, additional aeration capacity will need to be installed if the City were to implement Alternative F at the Future Permitted Design Condition. This would be achieved by increasing the blower capacity. (However, the installation of additional diffusers into the reactors is assumed to not be required.)

The results in Table 6-15 also demonstrate that, if a reactor were offline for service, the City would need to maintain a MLSS concentration of 3,800 mg/L to provide reliable treatment at the average conditions and still maintain a 15-day MCRT. At a peak dry weather flow of 7.5 mgd the City should be able to achieve reliable settling with three clarifiers online if the SVI is less than 200, if the SVI is higher some equalization would be needed. In addition, the added HP required per reactor could be provided if a diffused air system that has a common blower facility were installed in place of the surface aerators. Therefore, it may be possible to operate with only one basin online. Nevertheless, the aeration and mixing equipment installed at the Current Permitted Design Condition under both Alternative C and D would be on rails to allow for access and maintenance while the basin is in service.

As further detailed in the WWTP Secondary Process Modeling TM (Appendix B), the peak month solids generation under Alternative F is approximately 4,000 pounds per day, or more than 50 percent less than the solids that would be generated if microscreening was not also included (i.e., solids generated under Alternative D at the Future Permitted Design Condition). Each 1.2 MG digester installed under Alternative D would be able to handle approximately 2,750 pounds of solids per day. Therefore, if the City were to implement Alternative F by adding microscreening to Alternative D, the 1.2 MG digesters would exceed what is needed for the Future Permitted Design Condition.

Finally, it should be noted that if the City were to implement Alternative F, one option would be to construct an anaerobic solids treatment process to process both the primary screenings and the secondary solids. (Anaerobic treatment is not a viable option for treatment of secondary solids only.) Although not detailed in this Facilities Master Plan, this approach could result in long-term savings due to reduced hauling costs. In addition, the City could consider an onsite energy generation project using biogas from the anaerobic digestion process, thus further reducing overall WWTP operating costs.

6.2.3 Secondary Clarifier Capacity Needs

This section presents a discussion of the secondary clarifier capacity needs for each of the four secondary process alternatives listed above. The following topics are addressed:

- Design Criteria
- State Point Analysis Results

6.2.3.1 Design Criteria

A spreadsheet state point analysis tool was used to evaluate the clarification capacity needed under the four alternatives discussed in the previous section. Similar to the clarifier analysis completed for Current Permitted Design Condition, the analysis of the clarifier needs at the Future Permitted Design Condition accounts for the ability to use EQ facilities if the secondary effluent is exhibiting poor settling (i.e., high SVI conditions). The criteria used for the state point analysis, including the anticipated sustained flow under EQ basin operations conditions, are presented in Table 6-16.

Table 6-16. Secondary Clarifier State Point Analysis Criteria Future Permitted Design Condition

Parameter		Design Criterion
Diameter		80 ft
RAS Flow		75% of Influent Flow up to 1.94 mgd[a] per Clarifier
Maximum SVI[b]	All Clarifiers in Service[c]	250
	One Clarifier Out of Service[d]	200
Design Influent Flow Peak[e]	Winter Months	13.5 mgd
	Summer Months	7.5 mgd
Minimum Required Treatment with EQ[f]	Winter Months	8.8 mgd
	Summer Months	5.8 mgd
Design MLSS	Alternative A	3,500
	Alternative C	2,400
	Alternative D	3,200
	Alternative F	2,400

(a) Best Efficiency Point (BEP) capacity of recently installed submersible RAS pumps.
(b) Bulking and poor settling characteristics can occur as a result from both high MCRTs and low DO levels in the oxidation ditches, both of which could occur under the alternatives being considered. Therefore, the historically high SVI values observed at the WWTP could be experienced in the future and should be accounted for in the clarifier analysis.
(c) All clarifiers would be in service during the winter month flow conditions.
(d) Assume a clarifier may be out of service for up to 30 days during the low-flow summer months.
(e) Design PHWWF and PHDWF at the Future Permitted Design Condition.
(f) The evaluation used to determine the sustained peak WWTP design flows with EQ facilities online are discussed in the Pond Use Analysis TM (Appendix F).

6.2.3.2 State Point Analysis Results

A comparison of the state point analysis results for each alternative given the design conditions discussed above is presented in Table 6-17. As shown, the EQ facilities may be needed under Alternatives A and D if the solids are exhibiting poor settling properties during the winter months.

Table 6-17. Secondary Clarifier State Point Analysis of Alternatives Future Permitted Design Condition

# of Clarifiers	Discharge Season	Maximum Flow that Meets State Point Analysis mgd	Equalization Potentially Needed? (Maximum Flow < Design Influent Flow)	Design Flow Rate	Overflow Rate, gal/ft^2/day	Solids Loading Rate, lb/ft^2/day
Alternative A (Oxidation Ditch Only): Design MLSS = 3,500						
4	Winter	9.9	Yes: 9.9 < 13.5	9.9	492	26
3	Summer	9.6	No: 9.6 > 7.5	7.5	497	26
Alternative C (Primary Microscreening): Design MLSS = 2,400						
3	Winter	13.6	No: 13.6 > 13.5	13.5	902	26
2	Summer	11.2	No: 11.2 > 7.5	7.5	746	23
Alternative D (Separate Biosolids Treatment): Design MLSS = 3,200						
4	Winter	11.6	Yes: 11.6 < 13.5	11.6	577	26
3	Summer	11.2	No: 11.2 > 7.5	7.5	497	24
Alternative F (C-D Hybrid): Design MLSS = 2,400						
3	Winter	13.6	No: 13.6 > 13.5	13.5	902	26
2	Summer	11.2	No: 11.2 > 7.5	7.5	746	23

6.2.4 Tertiary Filtration and UV Disinfection Facilities Requirements

As discussed above and in the Pond Use Analysis TM (Appendix F), the tertiary facilities (including pumping, filtration and disinfection) do not require a redundant treatment unit since undisinfected secondary flows can be discharged to the existing storage facilities for land disposal. Similarly, the tertiary facilities could also be sized to handle a peak flow slightly lower than the PHWWF amount. Therefore, at the Future Permitted Design Condition, one additional unit for each of the tertiary processes (filter feed pumps, tertiary filters and UV disinfection channels) should be installed to increase the number of units at each process to four (providing a peak capacity of 12 mgd).

6.2.5 Summary of Potential Expansion Alternatives

Table 6-18 provides a summary of the number of major process units required under each of the four viable alternatives at the Current Permitted Design Condition. Schematic diagrams for each alternative are provided in Figures 6-4 through 6-7.

Table 6-18. Treatment Process Unit Installed Quantities for Viable Alternatives at the Future Permitted Design Condition

Process/Equipment	Alternative A: Oxidation Ditch Only	Alternative C: Primary Microscreening	Alternative D: Separate Biosolids Treatment	Alternative F: C-D Hybrid
Microscreens	—	5	—	5
Vortex Grit Chambers	2	—	2	—
Oxidation Ditches	5	3	3	2
Secondary Clarifiers	4	3	4	3
Filter Feed Pumps	4	4	4	4
Tertiary Filters	4	4	4	4
UV Channels	4	4	4	4
Aerobic Digesters	—	—	3	2
Sludge Lagoons	2	2	2	2
Screw Presses[(a)]	5	3	5	3

[(a)] From Chapter 5 (Biosolids Handling Alternatives). If rotary fan presses are provided instead of screw presses, the installed quantity of fan presses will likely be less than those shown for screw presses.

6.3 ULTIMATE BUILDOUT DESIGN CONDITION (6.0 MGD ADWF) EXPANSION ALTERNATIVES

This section describes the alternatives for expanding the WWTP to address capacity needs at the Future Permitted Design Condition. The following specific topics are addressed:

- Project Components Common to All Alternatives
- Oxidation Ditch Expansion Alternatives
- Secondary Clarifier Capacity Needs
- Tertiary Filtration and UV Disinfection Facilities Requirements
- Summary of Potential Expansion Alternatives

6.3.1 Project Components Common to All Alternatives

This section describes common improvements that are necessary under the Ultimate Buildout Design Condition, regardless of which expansion alternative is selected.

6.3.1.1 Headworks and Mixed Liquor Splitter Box Improvements

As discussed previously, the existing Headworks and Mixed Liquor Splitter Box will be modified to accommodate up to four downstream treatment units. If more than four treatment units are required (applicable only to the Mixed Liquor Splitter Box under Alternative A), it is anticipated that a new splitter box will be constructed with room for a total of six flow splits.

6.3.1.2 Solids Pumping Improvements

As discussed in the Current Permitted Design Condition evaluation, the existing Secondary Sludge Pump Station can accommodate the RAS/WAS pumping needs for up to four secondary clarifiers. Two RAS and WAS pumps will be installed when the third clarifier is added, and a third RAS pump and second WAS pump will be installed when the fourth clarifier is added. If more than four clarifiers are needed for a given alternative (applicable to Alternatives A and D), a dedicated RAS/WAS pumping system will be installed in a small structure adjacent to the additional clarifier(s).

6.3.1.3 Biosolids Handling and Disposal

As discussed previously, it is assumed that the supplemental mechanical dewatering system will be further expanded to meet the City's solids handling needs, including one redundant solids dewatering unit. In addition, it is assumed the City will incur added operation costs to haul biosolids to a landfill for disposal that cannot be applied to the City's irrigated properties due to capacity limitations. Nevertheless, because of changing regulations and the likelihood that current cutting-edge solids handling technologies will be become viable options, it is recommended that the City reevaluate the available solids handling options prior to implementation of these expansion projects.

6.3.2 Oxidation Ditch Expansion Alternatives

This section presents the results of the secondary process modeling analysis used to evaluate the reactor operating conditions at the Ultimate Buildout Design Condition. The four remaining secondary process alternatives[6] were evaluated, as follows:

- Alternative A: Expand Oxidation Ditches to Provide Primary/Secondary Treatment Needs
- Alternative C: Provide Primary Treatment to Reduce Loads to Oxidation Ditches
- Alternative D: Provide Alternative Solids Treatment to Reduce Demands on Oxidation Ditches
- Alternative F: Provide Primary Treatment to Reduce Loads to Oxidation Ditches and Provide an Alternative Solids Treatment to Reduce Demands on Oxidation Ditches

6.3.2.1 Alternative A – Additional Oxidation Ditch Capacity Only

Under Alternative A, a sixth oxidation ditch would be constructed to provide the capacity needed to handle the flow and loads associated with the Ultimate Buildout Design Condition (i.e., 6.0 mgd ADWF). The ratio of influent loads to reactor volume at the Ultimate Buildout Design Condition under Alternative A is the same as what would occur at the Current Permitted Design Condition. Therefore, the modeling results presented in Table 6-4 for the Alternative A under the

[6] Alternative B, construct a separate anoxic basin, and Alternative E, expansion of the clarifiers to provide improved settling of higher mixed liquor concentrations (both in lieu of expanding the Oxidation Ditches), have been eliminated from consideration.

Current Permitted Design Conditions are also applicable to the Ultimate Buildout Design Condition.

Similar to the previously discussed design conditions, the City should also be able to provide reliable treatment at the annual average conditions if one of the reactors if offline for maintenance.

6.3.2.2 Alternative C – Primary Microscreening

Under Alternative C a fourth oxidation ditch reactor will be added to handle the loads associated with Ultimate Buildout Design Condition, and the ratio of influent loads to reactor volume will be the same at both the Current and Future Permitted Design Conditions. In addition, one microscreening unit (for a total of six units) would be added to provide a total treatment capacity of 18 mgd[7]. Therefore, the Alternative C modeling results presented in Table 6-8 for the Current Permitted Design Conditions are also applicable to the Ultimate Buildout Design Condition.

In addition, as noted previously, a review of the primary solids handling options should occur when planning for the Ultimate Buildout Design Condition expansion. Nevertheless, for purposes of this Facilities Plan, it has been assumed that the City would continue the practice of hauling the microscreening solids to a landfill for disposal.

Finally, meeting the treatment standards with a unit out of service should be readily achievable (because the City would lose only a fourth of the total system capacity with a reactor down). Therefore, the City should be able to take a reactor offline for maintenance of aeration and mixing equipment, and a specialized installation allowing for access to the reactor's equipment with the reactor online is not necessary.

6.3.2.3 Alternative D – Separate Biosolids Treatment

Expansion of Alternative D for the Ultimate Buildout Design Condition would involve constructing a fourth oxidation ditch and 1.2 MG aerobic solids digester. Similar to Alternatives A and C, secondary reactor performance of Alternative D at the Future Permitted Design Condition is described by the performance at the Current Permitted Design Condition (Table 6-9). Moreover, for purposes of this Facilities Planning effort, it is assumed that specialized aeration system installation allowing for access to the reactor's equipment with the reactor online is not necessary.

6.3.2.4 Alternative F – Microscreening/Solids Treatment Hybrid

For Alternative F a third oxidation ditch would be added to handle the loads associated with the Ultimate Buildout Design Condition. In addition, a total of six primary microscreening units would be added to provide a total treatment capacity of 18 mgd[8].

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[7] If necessary, the equalization facilities will be used during periods when a unit is out of service and flows greater than 15 mgd are anticipated.

[8] If necessary, the equalization facilities will be used during periods when a unit is out of service and flows greater than 15 mgd are anticipated.

As noted previously, there will be a different ratio of influent loads to reactor volume at the Future Permitted Design Condition and the Ultimate Buildout Design Condition under Alternative F. Therefore, a secondary process model was developed to define the anticipated operating conditions at the Ultimate Buildout Design Condition. A summary of the process modeling results for this alternative are shown in Table 6-19.

Table 6-19. Alternative F (C-D Hybrid) Secondary Process Modeling Results Ultimate Buildout Design Condition

Design Condition[a]	# of Ditches	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations			Total HP Required per Reactor[a]
		MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	
Total Capacity							
Annual Average	3	2,000	19	0.37	4.1	1.1	160
Maximum Month		2,496	17	0.70	8.2	1.2	185
Maximum Day		—	—	0.60	4.9	1.4	205
Firm Capacity							
Annual Average	2	2,537	15	0.42	3.4	1.2	245

[a] The two existing surface aerators can provide a total of 120 HP per reactor.

As shown in Table 6-19, to provide reliable treatment at the Ultimate Buildout Design Condition with three oxidation ditches, the MLSS would need to be increased to 2,000 under the average loading conditions and to 2,500 mg/L under maximum month conditions. This would result in a higher MCRT than the minimum 15-day requirement.

It should also be noted that the aeration requirements per reactor are lower than what is needed at the Future Permitted Design Condition (240 HP). However, the *total* additional HP required (135 HP, or 615 HP minus 480 HP) is less than the aeration needs for one basin. Therefore, assuming a common blower system is installed, some of the aeration capacity provided at the Future Permitted Design Condition would be distributed into the third reactor that is constructed under the Ultimate Buildout Design Condition project.

The results in Table 6-19 also demonstrate that, if a reactor were offline for service, the City could provide reliable treatment with a MLSS concentration of 2,500 mg/L and a 15-day MCRT. Moreover, if a reactor was offline for maintenance, the diffused air system should be capable of providing the needed additional air to the remaining online reactor. Therefore, the City should be able to take a reactor offline for maintenance of aeration and mixing equipment, and a specialized installation allowing for access to the reactor's equipment with the reactor online is not necessary.

As discussed previously for Alternative F, each of the 1.2 MG digester installed under Alternative F at the Future Permitted Design Condition would be able to handle approximately 2,750 pounds of solids per day (or 5,500 pounds per day total). Moreover, as further detailed in the WWTP Secondary Process Modeling TM (Appendix B), the peak month solids generation under Alternative F at the Ultimate Buildout Design Condition is approximately 5,600 pounds per day. Nevertheless, given the uncertainties with the solids production rates with primary microscreening, it has been assumed for purposes of this Facilities Plan that a third, 1.2 MG digester would be constructed under Alternative F to handle the loads associated with the Ultimate Buildout Design Condition.

Finally, as noted previously, Alternative F offers an opportunity for the City to implement an anaerobic treatment process (and biogas recovery/energy generation system). Should Alternative F be identified as a preferred option, this approach to solids treatment/handling should be further considered.

6.3.3 Secondary Clarifier Capacity Needs

This section presents a discussion of the secondary clarifier capacity needs for each of the four secondary process alternatives listed above. The following topics are addressed:

- Design Criteria
- State Point Analysis Results

6.3.3.1 Design Criteria

A spreadsheet state point analysis tool was used to evaluate the clarification capacity needed under the four alternatives discussed in the previous section. Similar to the previous design conditions discussed, the analysis of the clarifier needs at the Ultimate Buildout Design Condition accounts for the ability to use equalization facilities if the secondary effluent is exhibiting poor settling (i.e., high SVI conditions). The criteria used for the state point analysis, including the anticipated sustained flow under equalization basin operations conditions, are presented in Table 6-20.

Table 6-20. Secondary Clarifier State Point Analysis Criteria (Ultimate Buildout Design Condition)

Parameter		Design Criterion
Diameter		80 ft
RAS Flow		75% of Influent Flow up to 1.94 mgd[a] per Clarifier
Maximum SVI[b]	All Clarifiers in Service[c]	250
	One Clarifier Out of Service[d]	200
Design Influent Flow Peak[e]	Winter Months	18 mgd
	Summer Months	10 mgd
Minimum Required Treatment with EQ[f]	Winter Months	12.1 mgd
	Summer Months	7.9 mgd
Design MLSS	Alternative A	3,400
	Alternative C	2,400
	Alternative D	3,200
	Alternative F	2,200

(a) Best Efficiency Point (BEP) capacity of recently installed submersible RAS pumps.
(b) Bulking and poor settling characteristics can occur as a result from both high MCRTs and low DO levels in the oxidation ditches, both of which could occur under the alternatives being considered. Therefore, the historically high SVI values observed at the WWTP could be experienced in the future and should be accounted for in the clarifier analysis.
(c) All clarifiers would be in service during the winter month flow conditions.
(d) Assume a clarifier may be out of service for up to 30 days during the low-flow summer months.
(e) Design PHWWF and PHDWF at the Future Permitted Design Condition.
(f) The evaluation of sustained peak WWTP design flows with EQ facilities online is presented in the Pond Use Analysis TM (Appendix F).

6.3.3.2 State Point Analysis Results

A comparison of the state point analysis results for each alternative given the design conditions discussed above is presented in Table 6-21. As shown, the equalization facilities may be needed under all the alternatives if the solids are exhibiting poor settling properties during the winter months.

Table 6-21. Secondary Clarifier State Point Analysis of Alternatives

# of Clarifiers	Discharge Season	Maximum Flow that Meets State Point Analysis mgd	Equalization Needed? (Maximum Flow < Design Influent Flow)	Design Flow Rate	Overflow Rate, gal/ft^2/day	Solids Loading Rate, lb/ft^2/day
Alternative A (Oxidation Ditch Only): Design MLSS = 3,400						
5	Winter	13.2	Yes: 13.2 < 18.0	13.2	525	26
4	Summer	13.5	No: 13.5 > 10.0	10.0	497	25
Alternative C (Primary Microscreening): Design MLSS = 2,400						
3	Winter	13.6	Yes: 13.6 < 18.0	13.6	902	26
2	Summer	11.2	No: 11.2 > 10.0	10.0	995	28
Alternative D (Separate Biosolids Treatment): Design MLSS = 3,200						
5	Winter	14.5	Yes: 14.5 < 18.0	14.5	577	26
4	Summer	14.7	No: 14.7 > 10.0	10.0	497	24
Alternative F (C-D Hybrid): Design MLSS = 2,200						
3	Winter	15.3	Yes: 15.3 < 18.0	15.3	1,015	26
2	Summer	12.5	No: 12.5 > 10.0	10.0	995	25

6.3.4 Tertiary Filtration and UV Disinfection Facilities Requirements

As discussed above and in the Pond Use Analysis TM (Appendix F), the tertiary facilities (including pumping, filtration and disinfection) do not require a redundant treatment unit since undisinfected secondary flows can be discharged to the existing storage facilities for land disposal. Similarly, the tertiary facilities could also be sized to handle a peak flow slightly lower than the PHWWF amount. Therefore, at the Future Permitted Design Condition, one additional unit for each of the tertiary processes (filter feed pumps, tertiary filters and UV disinfection channels) should be installed to increase the number of units at each process to five (providing a peak capacity of 15 mgd).

6.3.5 Summary of Potential Expansion Alternatives

Table 6-22 provides a summary of the number of major process units required under each of the four viable alternatives at the Ultimate Buildout Design Condition. Schematic diagrams for each alternative are provided in Figures 6-8 through 6-11.

Table 6-22. Treatment Process Unit Installed Quantities for Viable Alternatives at the Ultimate Buildout Design Condition

Process/Equipment	Alternative A: Oxidation Ditch Only	Alternative C: Primary Microscreening	Alternative D: Separate Biosolids Treatment	Alternative F: C-D Hybrid
Microscreens	—	6	—	6
Vortex Grit Chambers	2	—	2	—
Oxidation Ditches	6	4	4	3
Secondary Clarifiers	5	3	5	3
Filter Feed Pumps	5	5	5	5
Tertiary Filters	5	5	5	5
UV Channels	5	5	5	5
Aerobic Digesters	—	—	4	3
Sludge Lagoons	2	2	2	2
Screw Presses[(a)]	6	3	6	3

[(a)] From Chapter 5 (Biosolids Handling Alternatives). If rotary fan presses are provided instead of screw presses, the installed quantity of fan presses will likely be less than those shown for screw presses.

6.4 APPROACHES TO FACILITIES EXPANSION

The following six alternatives have been evaluated for meeting the City's long-term secondary process treatment needs:

- Alternative A: Expand Oxidation Ditches to Provide Primary/Secondary Treatment Needs
- Alternative B: Construct a Separate Anoxic Basin for Denitrification
- Alternative C: Provide Primary Treatment to Reduce Loads to Oxidation Ditches
- Alternative D: Provide Alternative Solids Treatment to Reduce Demands on Oxidation Ditches
- Alternative E: Expand Clarifiers to Allow Mixed Liquor Concentrations in the Oxidation Ditches to be Increased
- Alternative F: Provide Primary Treatment to Reduce Loads to Oxidation Ditches and Provide an Alternative Solids Treatment to Reduce Demands on Oxidation Ditches

Of these, Alternative B and E have been eliminated because they would require MLSS concentrations in the 4,000 to 5,000 mg/L range under the Current Permitted Design Condition, thus resulting in poor settling and too much emphasis on the need for clarification capacity. Therefore, cost estimates for these two alternatives have not been developed.

Although each of these alternatives is presented separately in the discussion above, the City could elect to move from one alternative to another under one of the future expansion projects. For example, the City could elect to construct the facilities needed to implement Alternative A at the Current Permitted Design Condition, and then move to implement either Alternative C or Alternative D at the Future Permitted Design Condition by adding either primary microscreening or aerobic digestion facilities, respectively (these approaches are denoted as Alternative C2 and Alternative D2, respectively, in Figure 6-12 and the cost tables presented later in this section).

On the other hand, once the City has committed to a given alternative at the Future Permitted Design Condition, moving toward a different alternative at the Ultimate Buildout Design Condition may not be the most economical approach. For example, if the City implements Alternative D at the Future Permitted Design Condition, a total of four clarifiers and a new vortex grit chamber will need to be constructed. However, if the City wanted to implement Alternative F at the Ultimate Buildout Design Condition, only three clarifiers would be needed and the vortex grit chambers would be removed. Therefore, this approach would result in the construction of facilities at the Future Permitted Design Condition that would not ultimately be necessary.

Figure 6-12 provides a flow chart that demonstrates the various approaches to expansion that would provide the highest use of constructed facilities. The cost estimates discussed in the next sections of this chapter consider the range of expansion options reflected in this figure.

6.5 ESTIMATED UPGRADE AND EXPANSION PROJECT COSTS

This section presents a comparison of the capital and net present value (NPV) costs for each of the treatment alternatives described above. The following major topics are addressed:

- Capital Costs
- NPV Costs
- Major Maintenance Project Costs
- Operations and Electrical Building Expansion

6.5.1 Capital Costs

Planning-level cost estimates for the various treatment alternatives at the three design conditions are presented in Tables 6-23, 6-24 and 6-25. The following specific factors were used in developing these costs:

- **Component Costs:** Cost estimates of improvements recommended at this level of planning are necessarily preliminary in nature. In most cases, pre-design and detailed design efforts will be necessary to refine and confirm preliminary cost estimates. To the extent practicable, schedules of values from recently completed WWTP improvement projects (including the 2010 Galt improvement project) were used to estimate the component costs (adjusted according to units of capacity and a “power law” factor). Other cost sources were used where this was approach not possible, particularly for those components associated with modifications to existing facilities. All costs were adjusted to current costs.

Table 6-23. Estimated Capital Costs of Treatment Train Alternatives, million $
Current Permitted Design Condition (3.0 MGD)

Project Component[a][b]	Alternative A Oxidation Ditch Only	Alternative C Primary Micro-screening	Alternative D Separate Solids Treatment
Preliminary/Primary Treatment			
Headworks Retrofit (Raise Walls)	-	0.19	-
Headworks (Replace Bar Screens)	-	0.17	-
Headworks (Flow Equalization)	0.15	0.15	0.15
Primary Treatment (Microscreens)	-	1.49	-
Oxidation Ditches			
Modify Headworks Splitter Box	0.03	-	-
New Oxidation Ditch	4.27	-	-
Retrofit Existing Ditches	2.06	2.42	2.42
Secondary Clarifiers			
Existing Clarifier Retrofit	1.30	1.30	1.30
Construct a Third Clarifier	2.18	-	2.18
New RAS/WAS Pumps	0.35	-	0.35
Modify Mixed Liquor Splitter Box	0.03	-	0.03
Solids Processing			
Aerobic Solids Digestion	-	-	4.08
Solids Dewatering (Screw Presses)	1.27	1.00	1.27
Sludge Lagoon Improvements	0.32	0.32	0.32
Common Ancillary Improvements			
Building Improvements	0.49	0.49	0.49
Arsenic Treatment at Wellheads	0.56	0.56	0.56
SCADA Improvements	0.08	0.08	0.08
SUBTOTALS	13.09	8.16	13.23
Estimating and Construction Contingencies[c]	3.93	2.45	3.97
Total Construction Bid Cost (Nov. 2012)	17.02	10.61	17.20
Adjustment for Inflation to Construction Midpoint[d]	1.02	0.64	1.03
Total Construction Bid Cost at Midpoint	**18.04**	**11.24**	**18.23**
Project Allowances[e]	6.04	3.77	6.11
Total Project Cost	**24.09**	**15.01**	**24.34**

[a] Estimated in current (November 2012) dollars unless otherwise indicated.

[a] Includes distributed costs estimated as a percentage of the component cost according to the following allowances: mobilization and demobilization (3%); demolition, bypassing, and shutdowns (1%); site work, including general earthwork and paving (5%); site piping for process flows and utilities (10%); and electrical and instrumentation (15%).

[a] Based on an estimating and bidding contingency allowance of 25% and a construction contingency of 5%.

[a] Adjusted to mid-point of construction at 3% per year. Anticipated date of construction contract award is December 2013, and a 24-month construction period is anticipated.

[a] Total Allowances are 33.5%, which include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction management, including O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%).

Table 6-24. Estimated Capital Costs of Treatment Train Alternatives, million $
Future Permitted Design Condition (4.5 MGD)

Project Component[(a)(b)]	Alternative A Oxidation Ditch Only	Alternative C Primary Micro-screening	Alternative C2 Add Micro-screening to Alt. A	Alternative D Separate Solids Treatment	Alternative D2 Add Solids Treatment to Alt. A	Alternative F Add Microscreening to Alt. D
Preliminary/Primary Treatment						
Headworks Retrofit (Raise Walls)	-	-	0.19	-	-	0.19
Headworks (Replace Bar Screens)	-	-	0.17	-	-	0.17
Primary Treatment (Microscreens)	-	0.92	2.40	-	-	2.40
New Vortex Grit Chamber	0.53	-	-	0.53	0.53	-
Oxidation Ditches						
Modify Headworks Splitter Box	-	0.03	-	0.03	-	-
New Headworks Splitter Box	0.54	-	-	-	-	-
New Oxidation Ditch(es)/Blowers	8.54	4.46	-	4.46	-	0.50
Secondary Clarifiers						
Modify Existing Mixed Liquor Splitter Box	0.03	0.03	-	0.03	-	-
New Mixed Liquor Splitter Box	-	-	-	-	-	-
New Secondary Clarifier(s)	2.18	2.18	-	2.18	-	-
Secondary Sludge Pumping	0.24	0.35	-	0.24	-	-
Solids Processing						
Aerobic Solids Digestion	-	-	-	2.04	6.12	-
Solids Dewatering (Screw Presses)	1.43	0.98	0.49	1.43	1.43	0.49
Tertiary Improvements						
Filter Feed Pump Station	0.05	0.05	0.05	0.05	0.05	0.05
Tertiary Filters	0.64	0.64	0.64	0.64	0.64	0.64
UV Disinfection	0.80	0.80	0.80	0.80	0.80	0.80
Common Ancillary Improvements						
Relocate Effluent Flow Meter	0.04	0.04	0.04	0.04	0.04	0.04
Misc. Hydraulic Improvements	0.09	0.09	0.09	0.09	0.09	0.09
Electrical Service and Switchgear	0.63	0.63	0.63	0.63	0.63	0.63
Backup Generator	0.39	0.39	0.39	0.39	0.39	0.39
SUBTOTALS	16.13	11.58	5.88	13.59	10.72	6.38
Estimating and Construction Contingencies[(c)]	4.84	3.47	1.76	4.08	3.22	1.91
Total Construction Bid Cost (Nov. 2012)	20.97	15.06	7.65	17.66	13.94	8.29
Adjustment for Inflation to Construction Midpoint[(d)]	4.82	3.46	1.76	4.06	3.21	1.91
Total Construction Bid Cost at Midpoint	**25.79**	**18.52**	**9.40**	**21.72**	**17.15**	**10.20**
Project Allowances[(e)]	8.64	6.20	3.15	7.28	5.74	3.42
Total Project Cost	**34.43**	**24.72**	**12.55**	**29.00**	**22.89**	**13.61**

(a) Estimated in current (November 2012) dollars unless otherwise indicated.
(b) Includes distributed costs estimated as a percentage of the component cost according to the following allowances: mobilization and demobilization (3%); demolition, bypassing, and shutdowns (1%); site work, including general earthwork and paving (5%); site piping for process flows and utilities (10%); and electrical and instrumentation (15%).
(c) Based on an estimating and bidding contingency allowance of 25% and a construction contingency of 5%.
(d) Adjusted to mid-point of construction at 3% per year. Anticipated date of construction contract award is December 2018, and a 24-month construction period is anticipated.
(e) Total Allowances are 33.5%, which include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction management, including O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%).

Table 6-25. Estimated Capital Costs of Treatment Train Alternatives, million $
Ultimate Buildout Design Condition (6.0 MGD)

Project Component[a][b]	Alternative A Oxidation Ditch Only	Alternative C Primary Micro-screening	Alternative D Separate Solids Treatment	Alternative F Add Microscreening to Alt. D	Alternative F2 Add Digestion to Alt. C2	Alternative F3 Add Microscreening to Alt. D2
Preliminary/Primary Treatment						
Headworks Retrofit (Raise Walls)	-	-	-	-	-	0.19
Headworks (Replace Bar Screens)	-	-	-	-	-	0.35
Primary Treatment (Microscreens)	-	0.46	-	0.46	0.46	2.86
Oxidation Ditches						
Modify Headworks Splitter Box	0.03	0.03	0.03	0.03	-	-
Oxidation Ditch(es)	4.27	4.46	4.46	4.46	-	-
Secondary Clarifiers						
New Mixed Liquor Splitter Box	0.33	-	0.33	-	-	-
Modify Mixed Liquor Splitter Box	-	-	0.33	-	-	-
Secondary Clarifier(s)	2.18	-	2.18	-	-	-
Secondary Sludge Pumping	0.54	-	0.54	-	-	-
Aerobic Solids Digestion						
Aerobic Solids Digestion	-	-	2.04	2.04	6.12	-
Solids Dewatering (Screw Presses)	0.52	-	0.52	-	-	-
Tertiary Improvements						
Filter Feed Pump Station	0.05	0.05	0.05	0.05	0.05	0.05
Tertiary Filters	0.64	0.64	0.64	0.64	0.64	0.64
UV Disinfection	0.80	0.80	0.80	0.80	0.80	0.80
SUBTOTALS	9.35	6.44	11.59	8.48	8.07	4.89
Estimating and Construction Contingencies[c]	2.81	1.93	3.48	2.54	2.42	1.47
Total Construction Bid Cost (Nov. 2012)	12.16	8.37	15.06	11.02	10.49	6.35
Adjustment for Inflation to Construction Midpoint[d]	9.12	6.28	11.30	8.27	7.87	4.77
Total Construction Bid Cost at Midpoint	**21.28**	**14.65**	**26.36**	**19.29**	**18.36**	**11.12**
Project Allowances[e]	7.13	4.91	8.83	6.46	6.15	3.73
Total Project Cost	**28.41**	**19.56**	**35.19**	**25.76**	**24.51**	**14.84**

[a] Estimated in current (November 2012) dollars unless otherwise indicated.
[b] Includes distributed costs estimated as a percentage of the component cost according to the following allowances: mobilization and demobilization (3%); demolition, bypassing, and shutdowns (1%); site work, including general earthwork and paving (5%); site piping for process flows and utilities (10%); and electrical and instrumentation (15%).
[c] Based on an estimating and bidding contingency allowance of 25% and a construction contingency of 5%.
[d] Adjusted to mid-point of construction at 3% per year. Anticipated date of construction contract award is December 2030, and a 24-month construction period is anticipated.
[e] Total Allowances are 33.5%, which include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction management, including O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%)

- **Distributed Costs:** These include costs not included in the component cost estimates at the planning level. The costs were determined by applying multipliers to the total of the component costs. Distributed cost allowances were as follows: mobilization and demobilization (3%); demolition, bypassing, and shutdowns (1%); sitework, including general earthwork and paving (5%); site piping for process flows and utilities (10%); and electrical and instrumentation (15%).
- **Estimating Contingency Allowance:** This allowance was applied to the sum of the component and distributed costs to account for possible costs that may not have been identified in the conceptual description of a given alternative. For purposes of this memorandum, an allowance of 30 percent was used as estimating contingency, as is typically done for planning level cost estimating. This amount also includes a 5 percent construction contingency. The sum of the component and distributed costs and the estimating contingency allowance yield the estimated construction bid cost.
- **Escalation to Construction Midpoint**: An escalation rate of 3 percent per year is used to inflate the construction costs to the midpoint of construction. The anticipated dates of construction contract awards for the Current Permitted Design Condition, Future Permitted Design Condition and Buildout Design Condition improvements are: December, 2013; December, 2018; and December, 2030, respectively. Construction periods of 24 months are anticipated for all improvement projects.
- **Project Allowances:** Additional project allowances were applied to sum of the estimated construction bid cost and the adjustment for inflation to construction midpoint to account for the engineering, administration, legal, environmental and other costs that will be incurred in addition to the bid construction price for the project various. Allowances include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction management, including O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%).

Each project for which a cost estimate is presented in the above tables is assumed to be constructed under a single contract awarded through a competitive bidding process. These planning-level estimates of project costs require refinement as more information becomes available during the preliminary and final design phases. Further, this estimate of construction costs has been made without detailed engineering data and developed using known costs for similar facilities, scale-up and scale-down factors, cost curves and generic cost guides. It is normally expected that an estimate of this type would be accurate within a range of plus 30 percent to minus 15 percent. This means that bids can be expected to fall within a range of 30 percent over the estimate to 15 percent under the estimates presented (AACE, 2005).

6.5.2 NPV Costs

NPV costs, which account for both capital and operation and maintenance (O&M) components, have also been prepared. The estimated NPV costs for incremental expansions to meet the three design conditions, as well as the combined life cycle costs at the end of the two expansion phases, are presented in Table 6-26.

Table 6-26. Estimated NPV for Treatment Alternatives

Project Component[(a),(b)]	Alternative A Oxidation Ditch Only	Alternative C Primary Micro-screening	Alternative C2 Add Micro-screening to Alt. A	Alternative D Separate Solids Treatment	Alternative D2 Add Solids Treatment to Alt. A	Alternative F Add Microscreening to Alt. D	Alternative F2 Add Digestion to Alt. C2	Alternative F3 Add Microscreening to Alt. D2
Current Permitted Capacity (3.0 MGD)								
Total Project Cost	24.09	15.01	-	24.34	-	-	-	-
PW Total Project Cost	21.85	13.61	-	22.07	-	-	-	-
PW O&M	6.74	6.66	-	6.94	-	-	-	-
NPV	**28.59**	**20.27**	**-**	**29.01**	**-**	**-**	**-**	**-**
Future Permitted Capacity (4.5 MGD), Halfway to General Plan Buildout								
Total Project Cost	34.43	24.72	12.55	29.00	22.89	13.61	-	-
PW Total Project Cost	23.30	16.73	8.50	19.63	15.49	9.21	-	-
PW O&M	16.46	15.46	15.46	16.52	16.52	15.06	-	-
NPV	39.76	32.19	23.96	36.15	32.01	24.27	-	-
Total Project Cost Through Future Permitted Capacity (4.5 MGD), Halfway to General Plan Buildout								
Total Project Cost	58.51	39.73	36.64	53.34	46.98	37.95	-	-
PW Total Project Cost	45.15	30.34	30.34	41.70	37.34	31.29	-	-
PW O&M	23.21	22.12	22.21	23.46	23.26	22.00	-	-
NPV	**68.35**	**52.46**	**52.55**	**65.16**	**60.60**	**53.28**	**-**	**-**
Ultimate Capacity (6.0 MGD), General Plan Buildout								
Total Project Cost	28.41	19.56	19.56	35.19	35.19	25.76	24.51	14.84
PW Total Project Cost	10.71	7.37	7.37	13.26	13.26	9.71	9.24	5.59
PW O&M	26.48	23.89	23.89	26.94	26.94	23.92	23.92	23.92
NPV	37.19	31.26	31.26	40.21	40.21	33.63	33.16	29.52
Total Project Cost Through Ultimate Capacity (6.0 MGD), General Plan Buildout								
Total Project Cost	86.92	59.29	56.20	88.53	82.17	63.71	61.15	61.82
PW Total Project Cost	55.85	37.71	37.71	54.96	50.60	40.99	39.58	42.93
PW O&M	49.69	46.00	46.09	50.40	50.21	45.92	46.13	47.18
NPV	**105.54**	**83.72**	**83.80**	**105.37**	**100.81**	**86.91**	**85.71**	**90.12**

[(a)] Estimated in current (November 2012) dollars unless otherwise indicated.

Reasonably accurate determinations of O&M costs for each alternative are critical to determining the present worth of the alternatives. The following factors were used in developing these O&M cost estimates:

- **Labor Costs:** Annual O&M labor hour requirements for most unit processes, as well as general WWTP functions and administrative activities, were estimated using published curves and adjustment factors (Culp, 1979; USEPA, 1973) taking into account the WWTP's size, level of automation, equipment conditions, and so on. Labor hour requirements for solids handling requirements were based on the assumptions presented in Chapter 5 (Biosolids Handling Alternatives). The assumed average hourly labor rate was $40, including benefits.
- **Materials Costs:** Material costs were estimated for certain unit processes based on recommendations by manufacturers (e.g., UV disinfection equipment), historical WWTP experience (biosolids dewatering bed sand media) and engineer's estimates/allowances (e.g., tertiary filter equipment).
- **Power Costs:** Power costs were determined according to the estimated power demands of duty equipment, estimated annual operating hours, and a composite power cost of $0.095 per kilowatt hour (kWh) reflecting the WWTP's existing mix of power from the SMUD and the solar facility located at the WWTP.
- **Chemical Costs:** Chemical costs were determined according to estimated doses for polymers and control chemicals (e.g., sodium hypochlorite). Unit processes requiring steady or intermittent chemical dosing include oxidation ditches (for filamentous bacteria and/or foam control), clarification and filtration (for enhanced coagulation and flocculation), and biosolids dewatering (for flocculation).
- **Parts Replacement & Repairs Costs:** An additional allowance for annual repair and replacement of materials was included at a rate of 0.5 percent of a given alternative's estimated construction cost.
- **Solids Hauling and Landfill Disposal Costs:** Costs are assumed to vary depending on what type of solid is being hauled, as summarized in Table 6-27.

Table 6-27. Solids Hauling and Landfill Disposal Cost Assumptions

Parameter	Primary Microscreening Solids	Secondary Solids Exceeding Available Land Disposal Capacity	Arsenic Sludge
Solids Concentration	35%	25%	20%
Destination	Forward Landfill (Manteca)	Kiefer Landfill (Sacramento)	Hay Road Landfill (Vacaville)
Roundtrip to Destination	70 miles	50 miles	[a]
Average Travel Speed	35 mph		[a]
Hauling Costs	$95/hr		[a]
Landfill Tipping Fee	$40/ton	$48/ton	[a]

[a] Disposal costs are based on information presented in the *Arsenic Treatment Residuals Disposal Assessment* Technical Memorandum (ECO:LOGIC, 2008), which does not indicate travel distance, speed, hauling cost or tipping fee.

Finally, the NPV calculations reflected in Table 6-26 were carried out using the following governing parameters:

- Planning periods (duration, from initial construction, to construction of next phase):
  - Current Permitted Design Condition improvements = 6 years
  - Future Permitted Design Condition improvements = 12 years
  - Buildout Design Condition improvements = 20 years
- Discount rate = 5% annually
- Inflation rate = 3% annually

6.5.3 Major Maintenance Project Costs

This section presents a discussion of improvements intended to address various deficiencies previously described in Chapter 3 (Existing Facilities) and Chapter 4 (Existing Wastewater Treatment Plant Conditions Assessment). Some of these projects do not, necessarily, need to be completed at this time. However, they will likely be necessary before the City reaches the Future Permitted Design Condition. Because completion of these projects will not be critical for ensuring reliable treatment, it is envisioned that these improvements will be made over a 10-year period as City funds become available. However, the City may elect to move one or more of the more critical items listed below into the one of the major WWTP upgrade/expansion projects discussed should funds be available.

An overview of the individual major maintenance projects is provided below followed by the total estimated capital costs for completing these maintenance projects.

6.5.3.1 Overview of Improvements

6.5.3.1.1 Headworks and Screening Improvements

The existing mechanical climber screen at the Headworks and auger are nearing the end of their useful life. When this occurs, the existing equipment will need to be replaced with a new mechanical climber screen and washer-compactor. It is recommended that a ½- to ¾-inch screen be installed.

In addition, to improve the overall reliability of the headworks process, a backup mechanical climber screen should be installed in the existing spare channel when the upgrades are made. The manual isolation gates upstream and downstream of the screen channels currently in use should also be replaced with automatic gates. Additional monitoring and control capabilities should also be added for the new climber screens and gates.

Finally, the following minor improvements to the headworks control system are needed:

- Replace and relocate panels to grade
- Provide level-based rake controls
- Provide auger failure alarm
- Add SCADA monitoring and control capabilities for existing auger and rake

6.5.3.1.2 Vortex Grit Removal Improvements

The damaged and missing liner inside the vortex grit chamber should be replaced with a new liner to protect the concrete from damage from abrasive tank contents. Additionally, the scour piping inside the grit chamber should be modified to reduce compaction and improve re-suspension of settled grit material prior to grit classification. Finally, two easily accessible cleanouts should be installed on the grit chamber drain line to further assist in the removal of grit material that settles in the grit chamber.

6.5.3.1.3 Filter Feed Pump Station Improvements

A permanent submersible pump and appurtenant equipment should be installed for automatic drainage of the existing bypass valve vault at the Filter Feed and Utility Water Pump Station. The pump should be controlled by level measured inside the vault. Discharge from the pump should be directed to the adjacent pump station wet well.

The high pressure utility water (W3) pumps should be relocated to the UV effluent channel to provide the WWTP's utility water system with disinfected tertiary effluent. No additional pumps will be necessary. As an alternative project, the City could install strainers on the existing W3 pump discharge piping to prevent solids in the secondary effluent from entering the WWTP's utility water system; both projects are estimated to have similar costs.

6.5.3.1.4 Tertiary Filter Protection

Screens should be installed in the influent chambers of the existing tertiary filters to aid in the removal of nuisance solids that pass through the preliminary and secondary treatment systems prior to becoming lodged in the filter backwash troughs. The screens should be constructed of stainless steel, and should be easily removed by operators for inspection and cleaning.

6.5.3.1.5 UV Disinfection Improvements

An enclosure around the UV structure would improve worker comfort and help reduce wear associated with inclement weather, dust and debris.

6.5.3.1.6 Sludge Lagoon Liner Replacement

The existing Hypalon liners were installed in the early 1990's (southern lagoon) and 2000's (northern lagoon), and may be reaching the end of their useful lives. As discussed in the Existing Wastewater Treatment Plant Conditions Assessment TM (Appendix E), seams in the liners are coming apart in several areas and have been patched twice in the past. To determine if and when repair or replacement of the liners is warranted, the City should conduct a thorough inspection of the liners prior to the issuance of contract documents for the Current Permitted Design Condition upgrade project.

6.5.3.1.7 Biosolids Polymer Facility

To prevent accidents from occasional polymer spills in the Biosolids Polymer Facility, a spill containment unit should be provided capable of capturing contents of one or more polymer totes or drums.

To assist City staff in maneuvering polymer totes, a portable gantry crane or similar device should be provided. To prevent damage to the asphalt near the entry to the Biosolids Polymer Facility, particularly from crane and pallet jack wheels on hot summer days, a portion of the asphalt surface near the entry should be removed and replaced with a concrete landing.

6.5.3.1.8 Irrigation Pump Station Control Panels

The control panels on both irrigation pump stations are nearing the end of their useful life and should be replaced.

6.5.3.2 Estimated Major Maintenance Capital Costs

Planning-level capital cost estimates for the major maintenance improvements listed above are provided in Table 6-28. As noted previously in this chapter, the microscreening project will need to include replacement of the bar screens due to hydraulic constraints in the headworks facility. Therefore, Table 6-28 presents a capital cost estimate for the maintenance improvements needed under alternatives with and alternatives without microscreening.

The major assumptions used to develop the capital cost estimates for the various alternatives discussed previously were used in developing the costs presented in Table 6-28 for the maintenance-related improvements, with the following exceptions:

- **Distributed Costs:** Distributed cost allowances were as follows: mobilization and demobilization (3%); demolition, bypassing, and shutdowns (1%); and electrical and instrumentation (10%).
- **Escalation to Construction Midpoint**: An anticipated date of construction is assumed to be July 2017 for all projects. This is the expected average for all the projects. Some will likely be completed up to three year prior to this date, and others up to three years beyond this date.

6.5.4 Operations and Electrical Building Expansion

The existing Operations and Electrical Building should be remodeled and expanded to accommodate additional City water system operations staff. If implemented, this project will likely be completed concurrently with the Current Permitted Design Condition upgrade project. However, the costs associated with the Operations and Electrical Building expansion will be borne through the City's water department funds. Therefore, the cost for expansion of the Operations and Electrical Building is presented separately from the remainder of the project costs.

Table 6-28. Estimated Capital Cost of Major Maintenance Improvements

Project Component[a,b]	Alternatives without Microscreening	Alternatives with Microscreening
Headworks (Replace Bar Screen)	143,000	—
Headworks (Redundant Bar Screens)	343,000	343,000
Headworks (Automatic Gates on Bar Screen Channels)	209,000	209,000
Headworks (Control System Improvements)	51,000	51,000
Vortex Grit Chamber Improvements	14,000	—
Filter Feed Pump Station Improvements	60,000	60,000
Mesh Screen to Protect Filter Units	8,000	8,000
UV Disinfection System Enclosure	181,000	181,000
Sludge Lagoon Liner Replacement	58,000	58,000
Polymer Facility Improvements	10,000	10,000
Irrigation Pump Station Control Panels	66,000	66,000
SUBTOTALS	1,143,000	986,000
Estimating and Construction Contingencies[c]	343,000	296,000
Total Construction Bid Cost (Nov. 2012)	1,486,000	1,282,000
Adjustment for Inflation to Construction Midpoint[d]	208,000	179,000
Total Construction Bid Cost at Midpoint	1,694,000	1,461,000
Project Allowances[e]	567,000	489,000
Total Project Cost	2,261,000	1,950,000

(a) Estimated in current (November 2012) dollars unless otherwise indicated.
(b) Includes distributed costs estimated as a percentage of the component cost according to the following allowances: mobilization and demobilization (3%); demolition, bypassing, and shutdowns (1%); and electrical and instrumentation (10%).
(c) Based on an estimating and bidding contingency allowance of 25% and a construction contingency of 5%.
(d) Adjusted to mid-point of construction at 3% per year. Anticipated year of construction mid-point is July 2017.
(e) Total Allowances are 33.5%, which include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction management, including O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%).

Planning-level cost estimates for expanding the Operations and Electrical Building are provided in Table 6-29. These costs also include providing a water division SCADA computer and software at the Operations and Electrical building. The major assumptions used to develop the capital cost estimates for the various alternatives discussed previously were used in developing the costs for the maintenance-related improvements, with the following exceptions:

- **Distributed Costs:** Distributed cost allowances were as follows: mobilization and demobilization (3%); sitework (5%); and electrical and instrumentation (10%).
- **Escalation to Construction Midpoint**: Although it is anticipated that this project will be completed concurrently with the Current Permitted Design Condition upgrade project, the inflation adjustment conservatively assumes date of mid-point construction date of July 2017.

Table 6-29. Estimated Capital Cost for Operations and Electrical Building Expansion

Project Component[a,b]	Estimated Cost
Building Expansion	532,000
Transfer Water System SCADA	27,000
SUBTOTALS	559,000
Estimating and Construction Contingencies[c]	168,000
Total Construction Bid Cost (Nov. 2012)	727,000
Adjustment for Inflation to Construction Midpoint[d]	102,000
Total Construction Bid Cost at Midpoint	829,000
Project Allowances[e]	278,000
Total Project Cost	1,107,000

(a) Estimated in current (November 2012) dollars unless otherwise indicated.
(b) Includes distributed costs estimated as a percentage of the component cost according to the following allowances: mobilization and demobilization (3%); site work, including general earthwork and paving (5%); and electrical and instrumentation (10%).
(c) Based on an estimating and bidding contingency allowance of 25% and a construction contingency of 5%.
(d) Adjusted to mid-point of construction at 3% per year. Anticipated year of construction mid-point is July 2017.
(e) Total Allowances are 33.5%, which include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction management, including O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%).

6.6 COMPARATIVE EVALUATION OF ALTERNATIVES

This section presents a comparison of alternatives based on the performance of alternatives related to factors that are important to the City. This includes cost, operation, maintenance and other factors that have a bearing on the operation and administration of the WWTP. The following specific topics are addressed:

- Ranking Criteria Overview
- Rating and Comparison of Alternatives

6.6.1 Ranking Criteria Overview

The criteria considered in the evaluation of the alternatives are discussed below. These evaluation criteria have been developed to reflect the specific needs of the City. Because not all of the criteria have equal importance, weighting factors have been assigned. The most important criteria receive the maximum weight while relatively unimportant criteria receive a low weight. The weighting scale is from 15, for the very important criteria, to 1, for those with a low importance.

6.6.1.1 Life Cycle Net Present Value

The life cycle costs presented herein include both the phased construction costs and the present worth for the annual O&M costs associated with the alternative. Cost is a very important factor and is assigned a weight of 15 for the rating of the alternatives.

6.6.1.2 Operability

Generally, low-technology, less-complex alternatives are favored. Although it is understood that O&M costs are reflected in the NPV of the alternatives, it is typically difficult to increase staffing levels. As a result, new or expanded unit processes that require staff attention are likely to divert staff from other WWTP functions. A high score will be given to alternatives that require minimal daily attention. Alternatives that are complex with equipment that requires daily operator attention will receive a lower score.

Operability is important from a perspective of day-to-day operations and has a close link to long term performance of the system. This criterion is assigned a weight of 6 for rating of alternatives.

6.6.1.3 Ability to Meet Current and Future Permits (Effectiveness)

Effectiveness is a measure of how well an alternative will comply with discharge requirements, both current and future. All of the alternatives that are evaluated must meet the WDRs issued by the Regional Board. There will be some differences in the overall effectiveness of alternatives, including a certain amount of risk that an alternative will or will not achieve the stated requirements. Thus, proven technologies will receive the highest rating while newer technologies that will require pilot testing will receive a lower rating.

Because only those alternatives that will meet the stated requirements are being evaluated, the relative importance of this criterion is somewhat less than the maximum. This criterion is assigned a weight of 5 for rating of alternatives.

6.6.1.4 Reliability

The reliability factor considers the susceptibility of an alternative to process or mechanical failures as well as the consequence of a major system failure. Relatively simple systems with the ability to operate over a wide range of conditions without disruption of performance will receive a high score, while systems that are required to operate within narrow tolerances, are highly mechanical or are susceptible to disruption will receive a lower score.

While an important criterion, all alternatives being considered will be proven technology or tested prior to implementation. Also, some of the same factors that influence reliability are reflected in the operability criterion. For these reasons, this criterion is assigned a weight of 5.

6.6.1.5 Flexibility

The flexibility criterion relates to how well an alternative can adapt to possible future constraints or changes. These would include WWTP expansions to address additional (unforeseen) capacity needs, new regulatory requirements and/or new technologies. Systems that are modular and could be expanded would rank higher. In addition, alternatives that would have the capacity to meet (or be readily adaptable to meet) more stringent requirements should receive a higher score. Finally, alternatives that provide a viable and adaptable first phase that is compatible with one or more the options would receive a high ranking.

Treatment requirements and technologies for treatment and energy generation from wastewater are changing rapidly. Alternatives that give the City the most flexibility to adjust to these industry changes will provide the highest long-term benefit. Therefore, this criterion is fairly very important and should receive a weight of 7 for ranking the alternatives.

6.6.1.6 Investment Life

Investment life is a measure of how long the structures and equipment associated with a treatment alternative are anticipated to last. Concrete basins are likely to remain useful for many years, and are typically capable of being retrofitted with new equipment as the need arises in the future. Specialized mechanical equipment, on the other hand, is more likely to require repair or replacement before reaching the end of its useful life. Alternatives with a greater emphasis on concrete basins with mechanical equipment/technologies with longer service histories should receive a higher score, while alternatives with more specialized mechanical equipment with shorter service histories should receive lower scores.

This criterion is not very significant given the alternatives under consideration and should receive a lower weight. This criterion is assigned a weight of 2 for rating of alternatives.

6.6.1.7 Energy Efficiency

The energy efficiency criterion is a measure of how efficiently an alternative utilizes energy while achieving the WWTP's treatment goals. Although it is understood that energy costs are reflected in the NPV of the alternatives, energy efficiency can impact public and institutional perception of a project and may influence the availability of certain energy efficiency incentives offered by utilities. As a result, alternatives with lower annual energy costs will receive a higher score, while systems with high energy consumption will receive lower scores.

The WWTP already generates a significant amount of solar electricity and enjoys a reduced composite (utility grid plus solar) cost of electricity. This criterion is assigned a weight of 2.

6.6.1.8 Risk of Negative Secondary Impacts

Any change to the WWTP's treatment train will result in certain secondary (or "ripple effect") changes to other processes, the impact of which may be positive, negative or neutral. Alternatives relying on new unit processes with the potential to adversely impact the operation of existing processes will receive lower scores, while alternatives featuring process changes likely to result in positive or neutral impacts to existing operations will receive higher scores.

All alternatives being considered will be proven technology or tested prior to implementation, providing the City with further opportunities to assess the risks of negative secondary impacts. Therefore, this criterion is assigned a weight of 3.

6.6.2 Rating and Comparison of Alternatives

A rating of the alternatives was developed for each project phase. For each criteria listed above, the alternatives were rated 3 for "most favorable," 2 for "less favorable," and 1 for "unfavorable."

6.6.2.1 Current Permitted Design Condition

The results of the rating evaluation for the Current Design Conditions are presented in Table 6-30.

As indicated, although not the lowest cost option, Alternative A received the highest rating. The high rating assigned to Alternative A was a result of the following factors:

- With respect to O&M considerations, Alternative A does not introduce a new treatment process and should not result in significantly increased workload. (Alternative C ranked the lowest because it introduces a new process that requires regular maintenance and hauling of solids.)
- Alterative A ranked the highest with respect to reliability because it simply expands on the existing, robust system. Alternative C introduces a new, relatively untested technology in the industry and ranks the lowest. Alternative D introduces aerobic treatment of biosolids, which is a well-tested technology but new to the City.
- Alternative A also ranks highest with respect to flexibility because it provides an additional oxidation ditch and clarifier, reducing the risk of treatment plant upsets during important maintenance activities by allowing a unit to be taken out of service. In addition, implementation of Alternative A at the Current Permitted Design Condition does not preclude implementation of Alternatives C, D, or F at the Future Permitted Design Condition.
- Finally, Alternative A ranks the highest with respect to risk of negative secondary impacts. Alternative C requires solids to be disposed of in a landfill, which may not be a viable option long-term. Alternative D, may not be the best long-term solids handling option due to energy use and potential future greenhouse gas regulations.

Therefore, Alternative A is the recommended option at the Current Permitted Design Condition.

Table 6-30. Rating of Alternatives at the Current Permitted Design Condition

Category	Weighting Factor	Alternative A Oxidation Ditch Only		Alternative C Primary Microscreening		Alternative D Separate Biosolids Treatment	
		Rating	Weighted Rating	Rating	Weighted Rating	Rating	Weighted Rating
Life Cycle Costs	15	3	45	4	60	2	30
Capital	—	1	—	2	—	1	—
O&M	—	2	—	2	—	1	—
O&M Considerations	6	9	54	4	24	5	30
Different Skills	—	3	—	2	—	1	—
Workload	—	3	—	1	—	2	—
Spare Parts	—	3	—	1	—	2	—
Ability to Meet Current and Future Permits	5	2	10	1	5	3	15
Reliability	5	3	15	1	5	2	10
Flexibility	7	3	21	2	14	1	7
Investment Life	2	3	6	2	4	3	6
Energy Efficiency	2	2	4	3	6	1	2
Risk of Negative Secondary Impacts	3	3	9	1	3	2	6
Total Weighted Rating	—	—	**164**	—	**121**	—	**106**

6.6.2.2 Future Permitted Design Condition

The results of the rating evaluation for the Future Design Conditions are presented in Table 6-31. As indicated, Alternatives C/C2 received the highest rating. (If Alternative A is implemented for the Current Permitted Design Condition, Alternative C2 would be implemented at the Future Permitted Design Condition.) In addition to being the lowest cost option, Alternative C received favorable ratings for the following factors:

- Alternative C provides the highest level of flexibility in that the City could implement either Alternative C or Alternative F at the next expansion.
- Alternative C also ranked highest with respect to energy efficiency. Microscreens use very little energy to remove solids, reducing energy demands that would otherwise occur in downstream processes. In addition, microscreening offers an opportunity for future energy generation using gasification, pyrolysis or anaerobic digestion.

Alternative C does *not* rank very high for reliability and ability to meet future effluent limitations. This is primarily due to the fact that it is newer technology that has not been tested on plants like the Galt WWTP. However, it is expected that more wastewater treatment facilities that rely on microscreening will be constructed over the next few years, and data from these facilities (as well as pilot testing at the Galt WWTP) could increase the overall ranking of Alternative C.

However, microscreening is not demonstrated to be a viable option for the Galt WWTP, Alternative D/D2 would be recommended. (Note that Alternative F would not be an efficient approach to treatment if the City implements Alternative A at the Current Permitted Design Condition. This is because only two oxidation ditches would be needed, and the City will have already constructed a third ditch at the Current Permitted Design Condition.)

Therefore, assuming future installations of microscreens demonstrate this as a viable approach for Galt, Alternative C is the recommended option at the Future Permitted Design Condition.

Table 6-31. Rating of Alternatives at the Future Permitted Design Condition

		Alternative A		Alternative C/C2		Alternative D/D2		Alternative F	
		Oxidation Ditch Only		Primary Microscreening		Separate Biosolids Treatment		Hybrid of Alternative C and D	
	Weighting Factor	Rating	Weighted Rating	Rating	Weighted Rating	Rating	Weighted Rating	Rating	Weighted Rating
Life Cycle Costs	15	2	30	5	75	3	45	6	90
Capital	—	1	—	3	—	2	—	3	—
O&M	—	1	—	2	—	1	—	3	—
O&M Considerations	6	8	48	5	30	6	36	3	18
Different Skills	—	3	—	2	—	2	—	1	—
Workload	—	3	—	2	—	2	—	1	—
Spare Parts	—	2	—	1	—	2	—	1	—
Ability to Meet Current and Future Permits	5	2	10	1	5	3	15	3	15
Reliability	5	3	15	1	5	2	10	1	5
Flexibility	7	1	7	3	21	2	14	1	7
Investment Life	2	3	6	2	4	3	6	2	4
Energy Efficiency	2	2	4	3	6	1	2	2	4
Risk of Negative Secondary Impacts	3	3	9	2	6	3	9	1	3
Total Weighted Rating	—	—	**129**	—	**152**	—	**137**	—	**146**

6.6.2.3 Ultimate Buildout Design Condition

The results of the rating evaluation for the Ultimate Buildout Conditions are presented in Table 6-32. As indicated, Alternatives C/C2 received the highest rating for the reasons listed above. However, Alternative F also received a similarly high rating and should be considered at the Ultimate Buildout Condition for the following reasons:

- Alterative F ranks the highest with respect to ability to meet future discharge regulations. This is because this alternative provides a separate biosolids treatment process. If regulations on biosolids treatment requirement for land application become more stringent, the City will require an additional treatment process beyond what can be provided in the oxidation ditches. In addition, long-term disposal of primary solids in a landfill could also be restricted and/or become cost prohibitive. Therefore, a separate biosolids treatment process may be the best approach for handling the primary solids (in addition to the secondary solids).
- Alternative F provides an opportunity for solids digestion in an anaerobic process. This would allow the City to generate biogas that could be used to run a generator at the WWTP site.

However, other options for energy generation using the primary solids may be preferred by the time the City is ready to construct the facilities to satisfy the Ultimate Buildout Design Condition.

Therefore, pending future regulations of solids disposal and the development of energy generation options for primary solids, either Alternative C or Alternative F is the recommended option at the Ultimate Buildout Design Condition.

Table 6-32. Rating of Alternatives at the Ultimate Buildout Design Condition

		Alternative A		Alternative C/C2		Alternative D/D2		Alternative F/F2/F3	
		Oxidation Ditch Only		Primary Microscreening		Separate Biosolids Treatment		Hybrid of Alternative C and D	
	Weighting Factor	Rating	Weighted Rating	Rating	Weighted Rating	Rating	Weighted Rating	Rating	Weighted Rating
Life Cycle Costs	15	3	45	5	75	2	30	5	75
Capital	—	1	—	3	—	1	—	3	—
O&M	—	2	—	2	—	1	—	2	—
O&M Considerations	6	8	48	5	30	6	36	3	18
Different Skills	—	3	—	2	—	2	—	1	—
Workload	—	3	—	2	—	2	—	1	—
Spare Parts	—	2	—	1	—	2	—	1	—
Ability to Meet Current and Future Permits	5	2	10	1	5	3	15	3	15
Reliability	5	3	15	1	5	2	10	1	5
Flexibility	7	1	7	3	21	2	14	3	21
Investment Life	2	3	6	2	4	3	6	2	4
Energy Efficiency	2	2	4	2	4	1	2	3	6
Risk of Negative Secondary Impacts	3	3	9	2	6	2	6	1	3
Total Weighted Rating	—	—	**144**	—	**150**	—	**119**	—	**147**

6.7 REFERENCES

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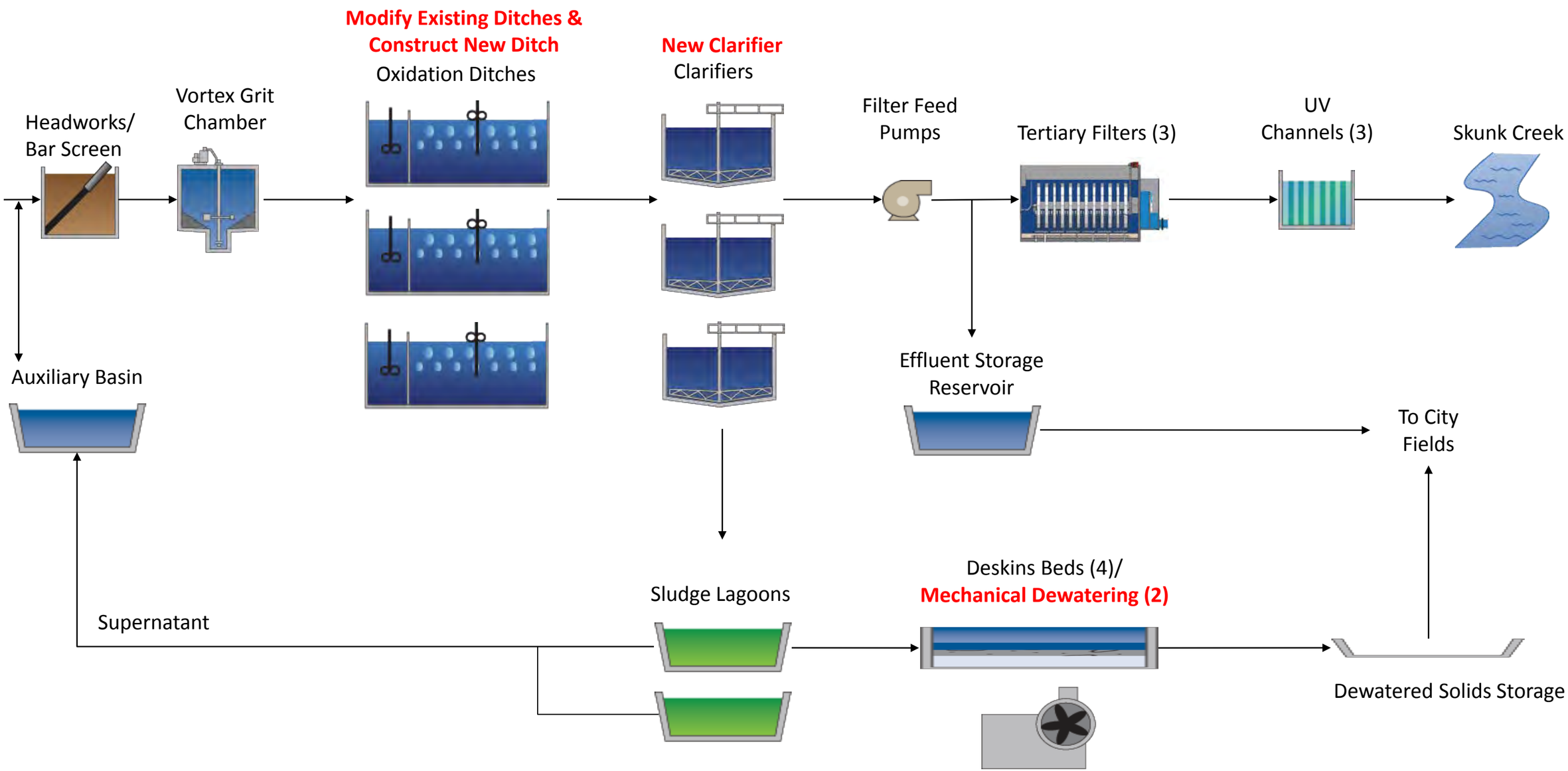

FIGURE 6-1
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE A SCHEMATIC
CURRENT PERMITTED DESIGN CONDITION
(3.0 MGD ADWF)

WEST YOST ASSOCIATES
Consulting Engineers

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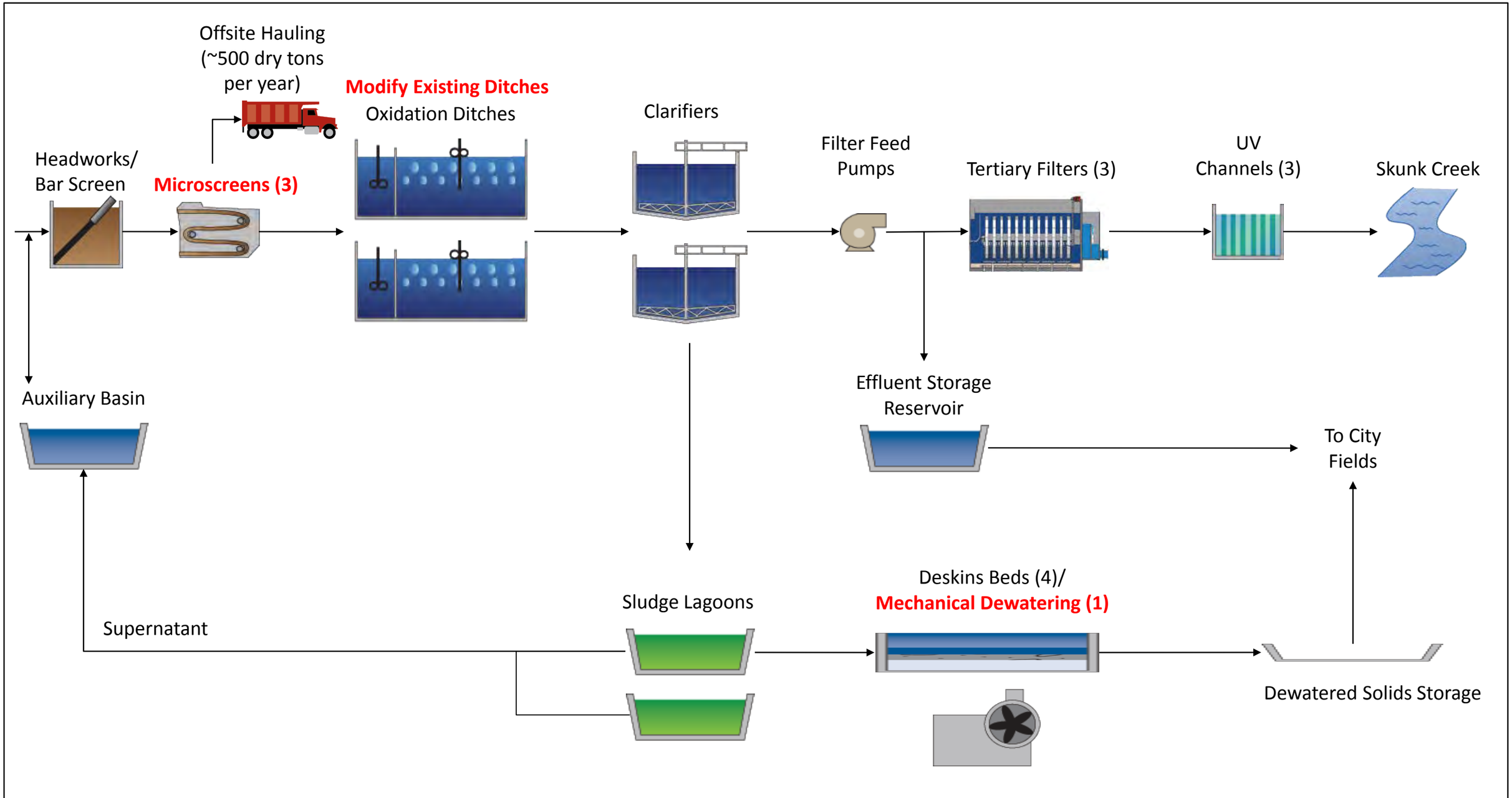

FIGURE 6-2
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE C SCHEMATIC
CURRENT PERMITTED DESIGN CONDITION
(3.0 MGD ADWF)

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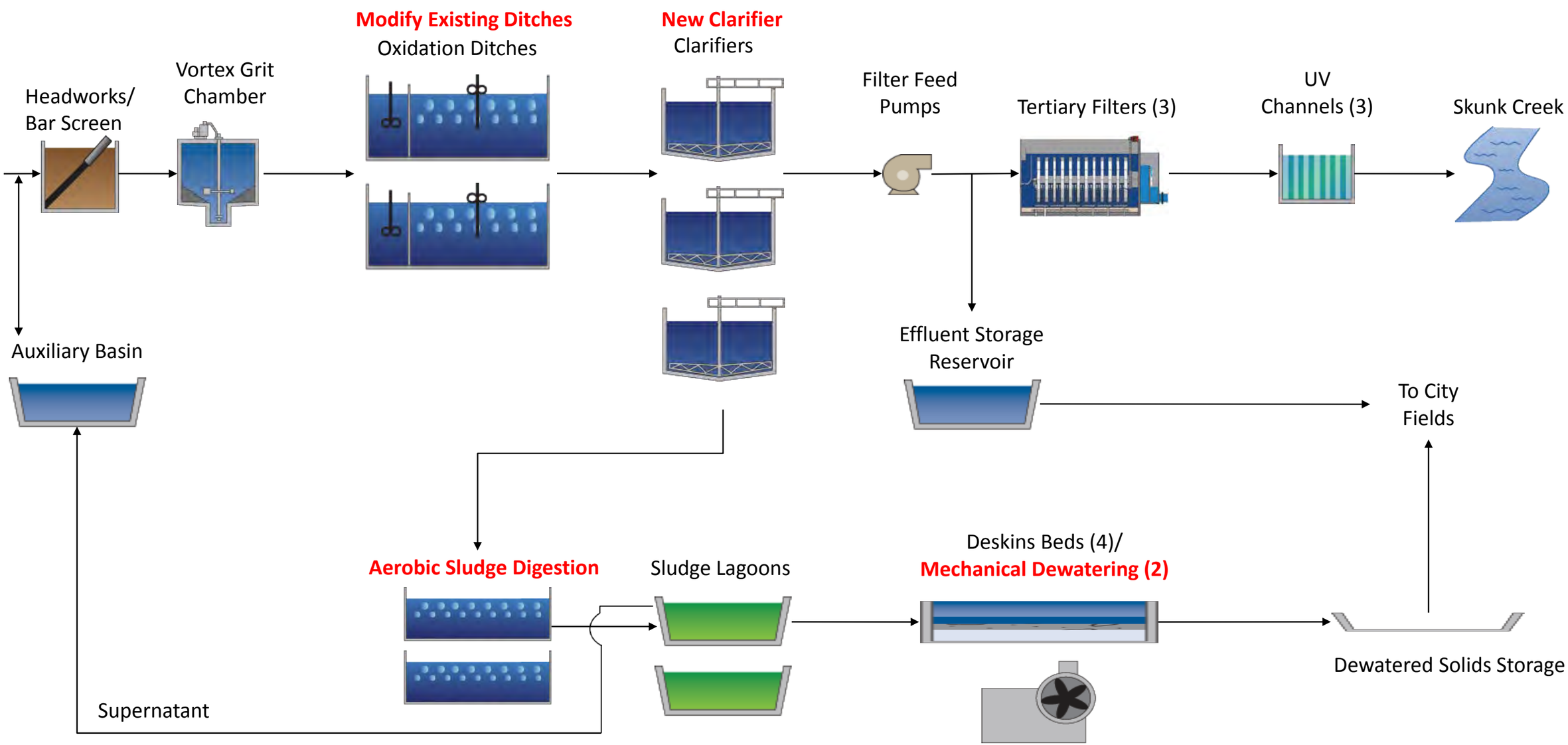

FIGURE 6-3
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE D SCHEMATIC
CURRENT PERMITTED DESIGN CONDITION
(3.0 MGD ADWF)

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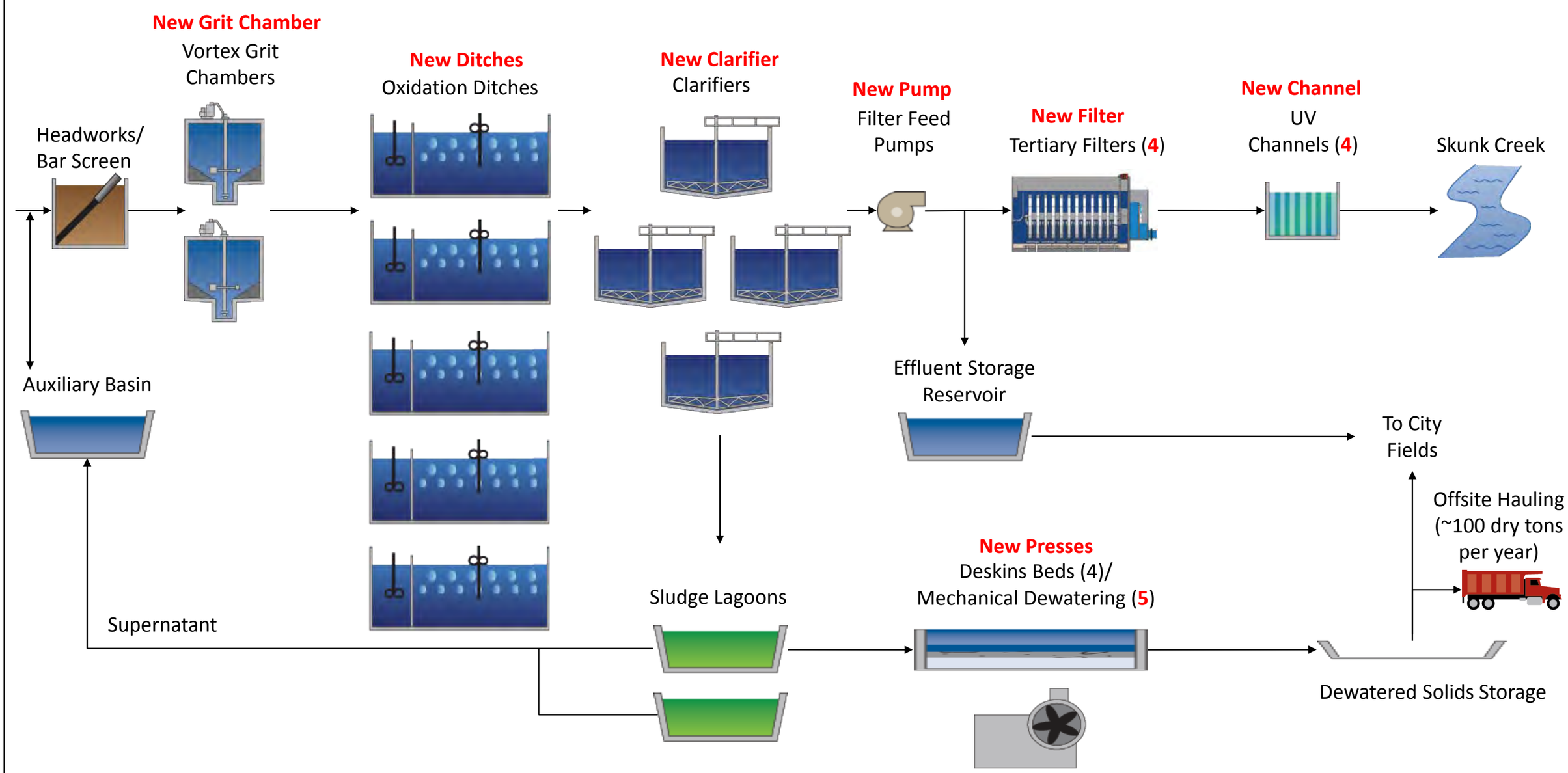

FIGURE 6-4
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE A SCHEMATIC
FUTURE PERMITTED DESIGN CONDITION
(4.5 MGD ADWF)

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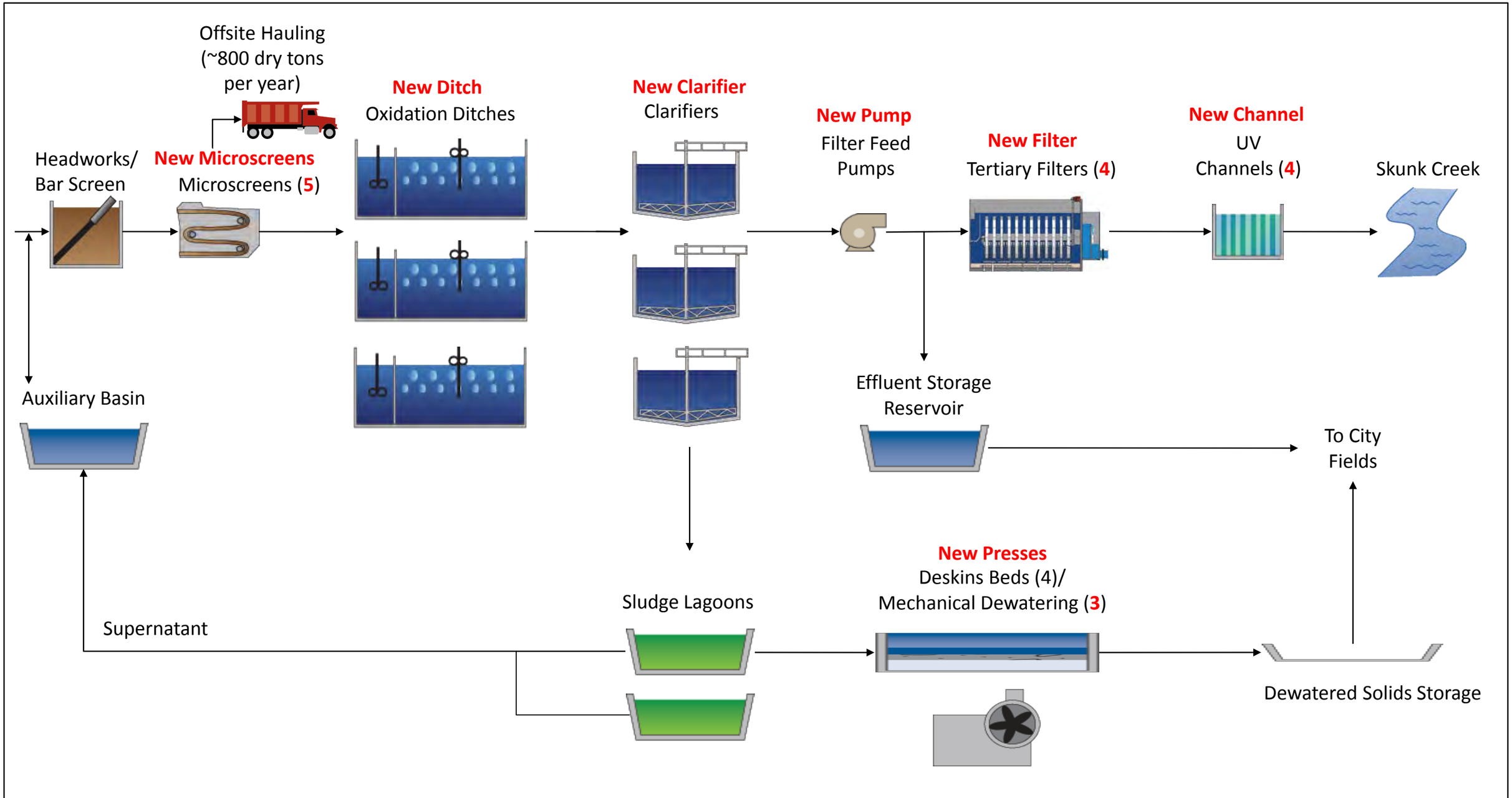

FIGURE 6-5
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE C SCHEMATIC
FUTURE PERMITTED DESIGN CONDITION
(4.5 MGD ADWF)

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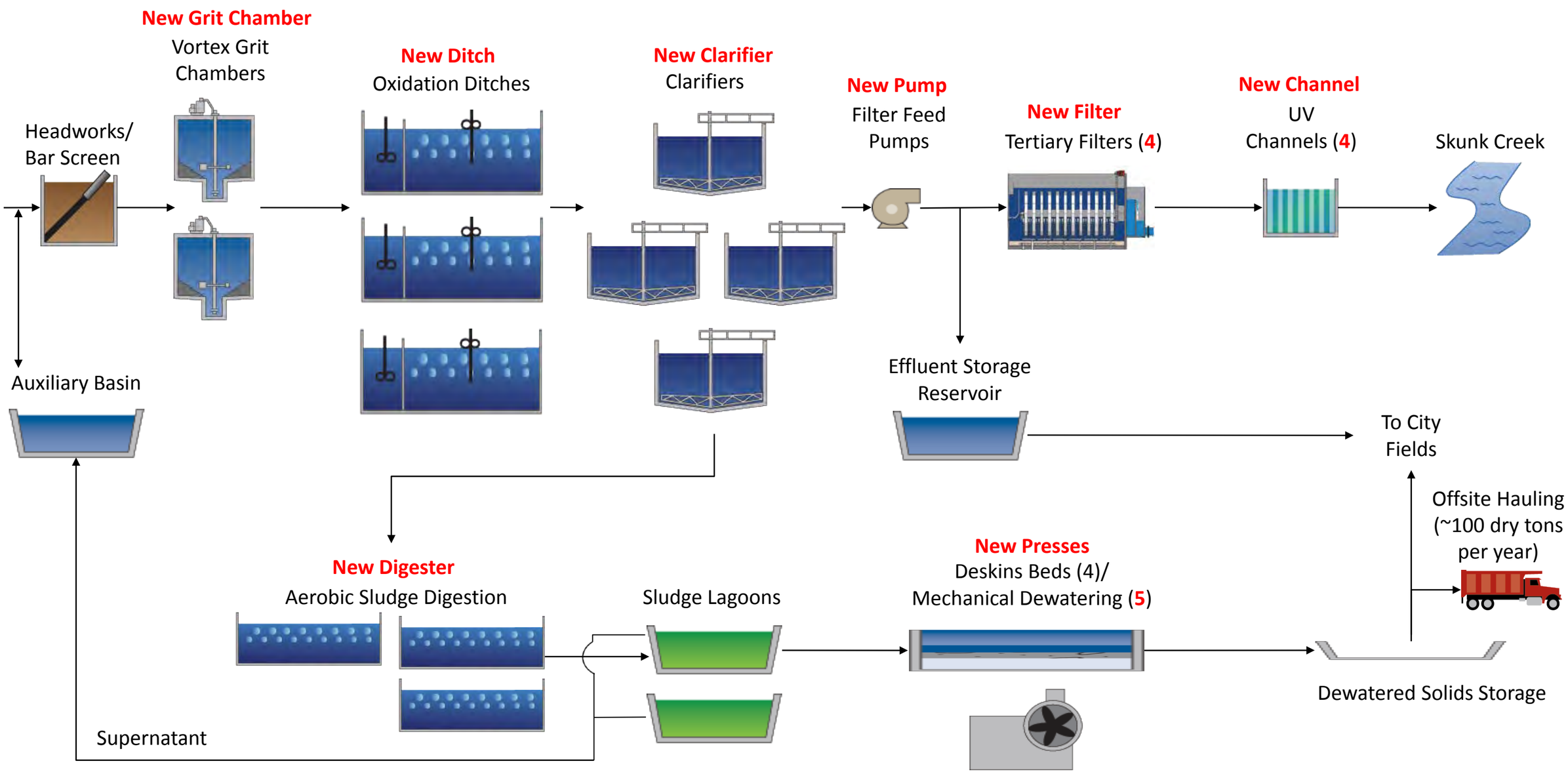

FIGURE 6-6
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE D SCHEMATIC
FUTURE PERMITTED DESIGN CONDITION
(4.5 MGD ADWF)

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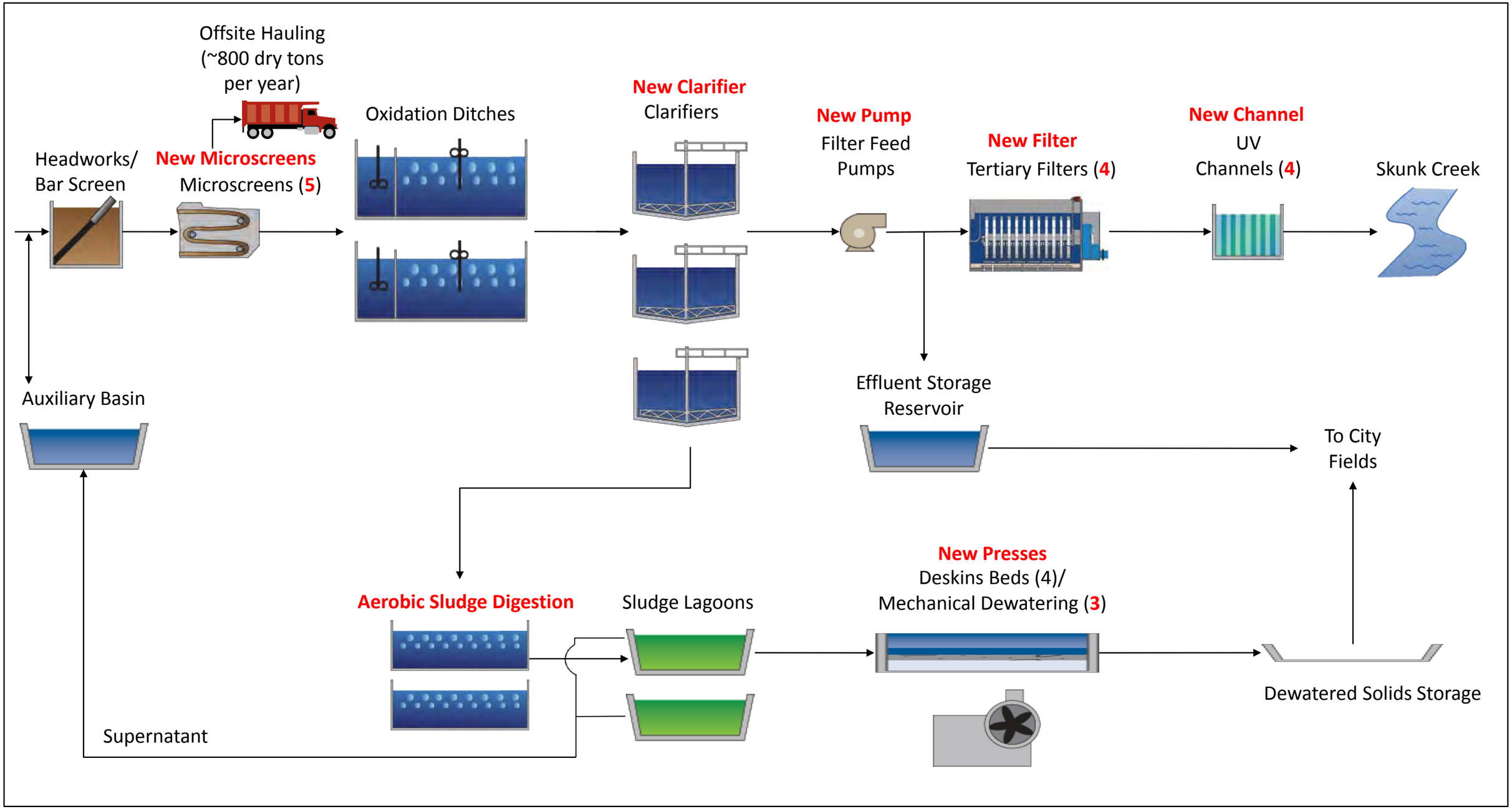

FIGURE 6-7
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE F (HYBRID C+D) SCHEMATIC
FUTURE PERMITTED DESIGN CONDITION
(4.5 MGD ADWF)

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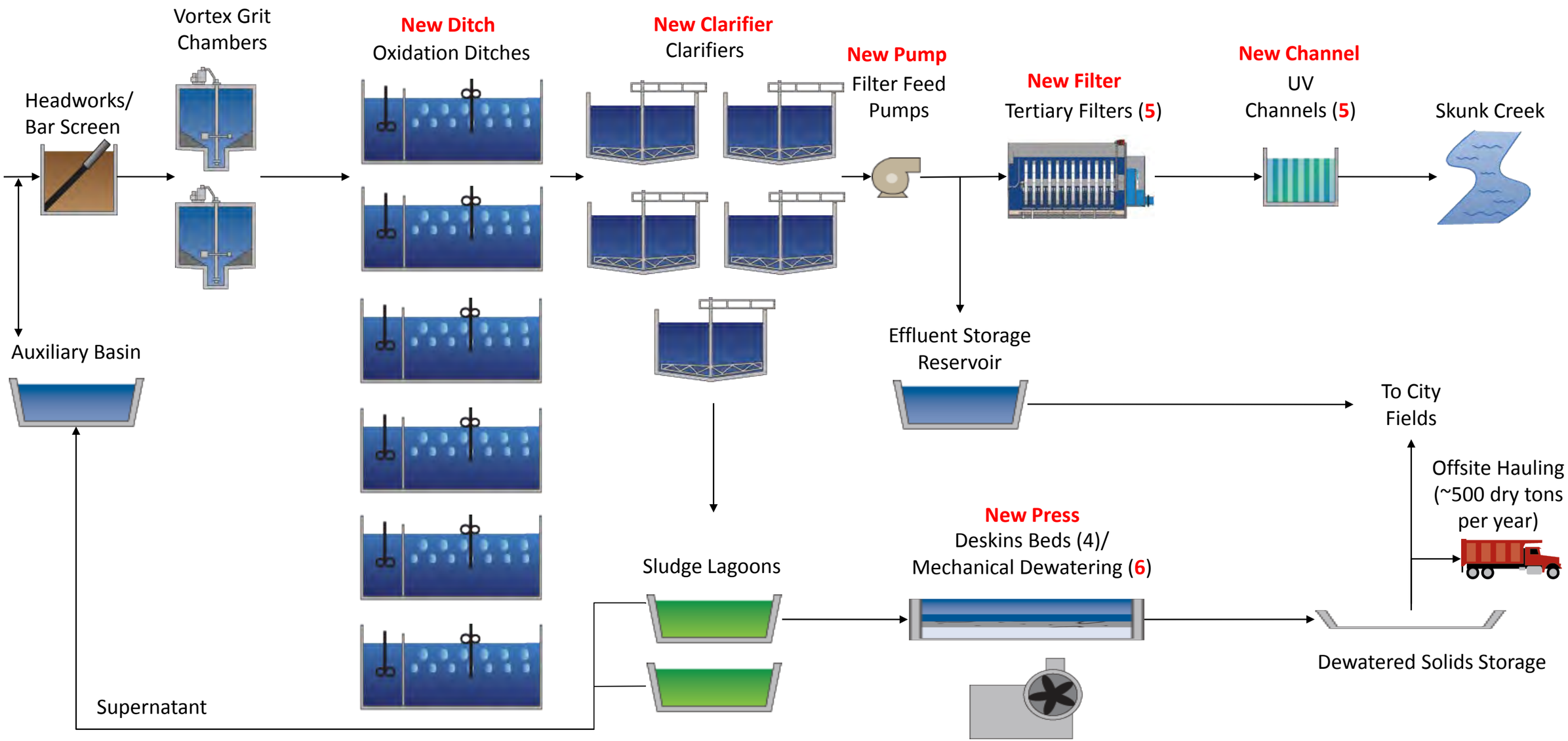

FIGURE 6-8
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE A SCHEMATIC
ULTIMATE BUILD-OUT DESIGN CONDITION
(6.0 MGD ADWF)

WEST YOST ASSOCIATES
Consulting Engineers

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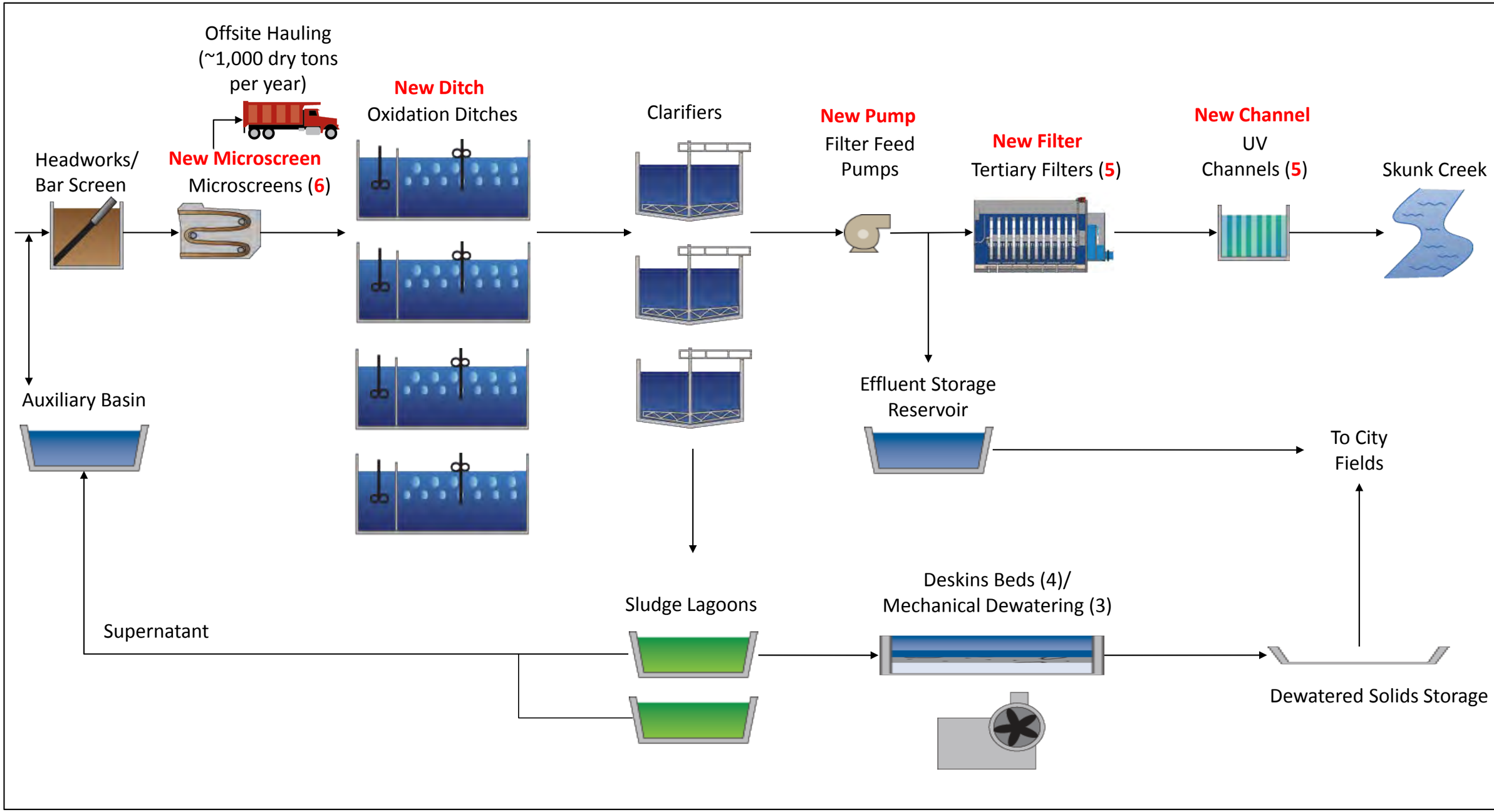

FIGURE 6-9
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE C SCHEMATIC
ULTIMATE BUILD-OUT DESIGN CONDITION
(6.0 MGD ADWF)

WEST YOST ASSOCIATES
Consulting Engineers

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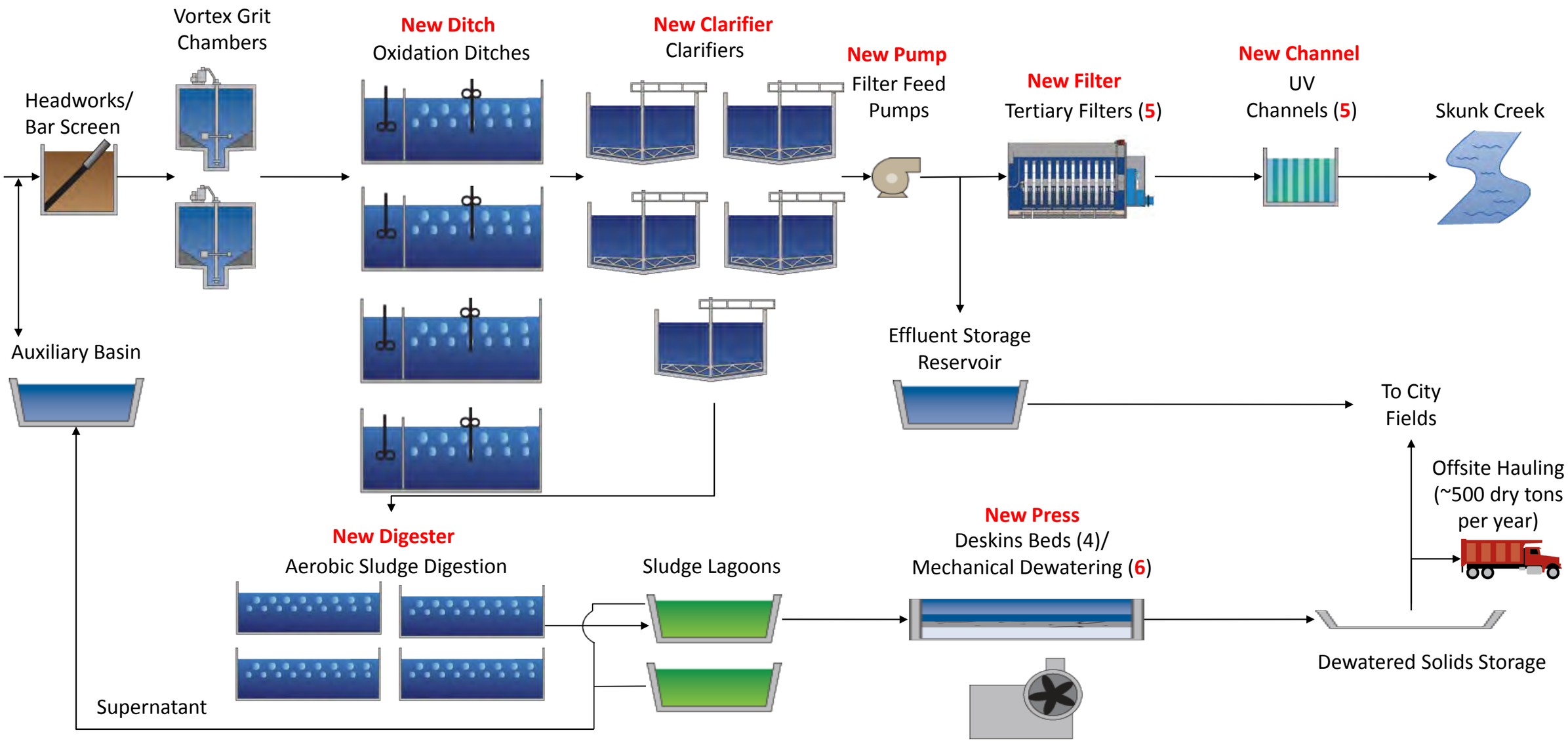

FIGURE 6-10
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE D SCHEMATIC
ULTIMATE BUILD-OUT DESIGN CONDITION
(6.0 MGD ADWF)

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Consulting Engineers

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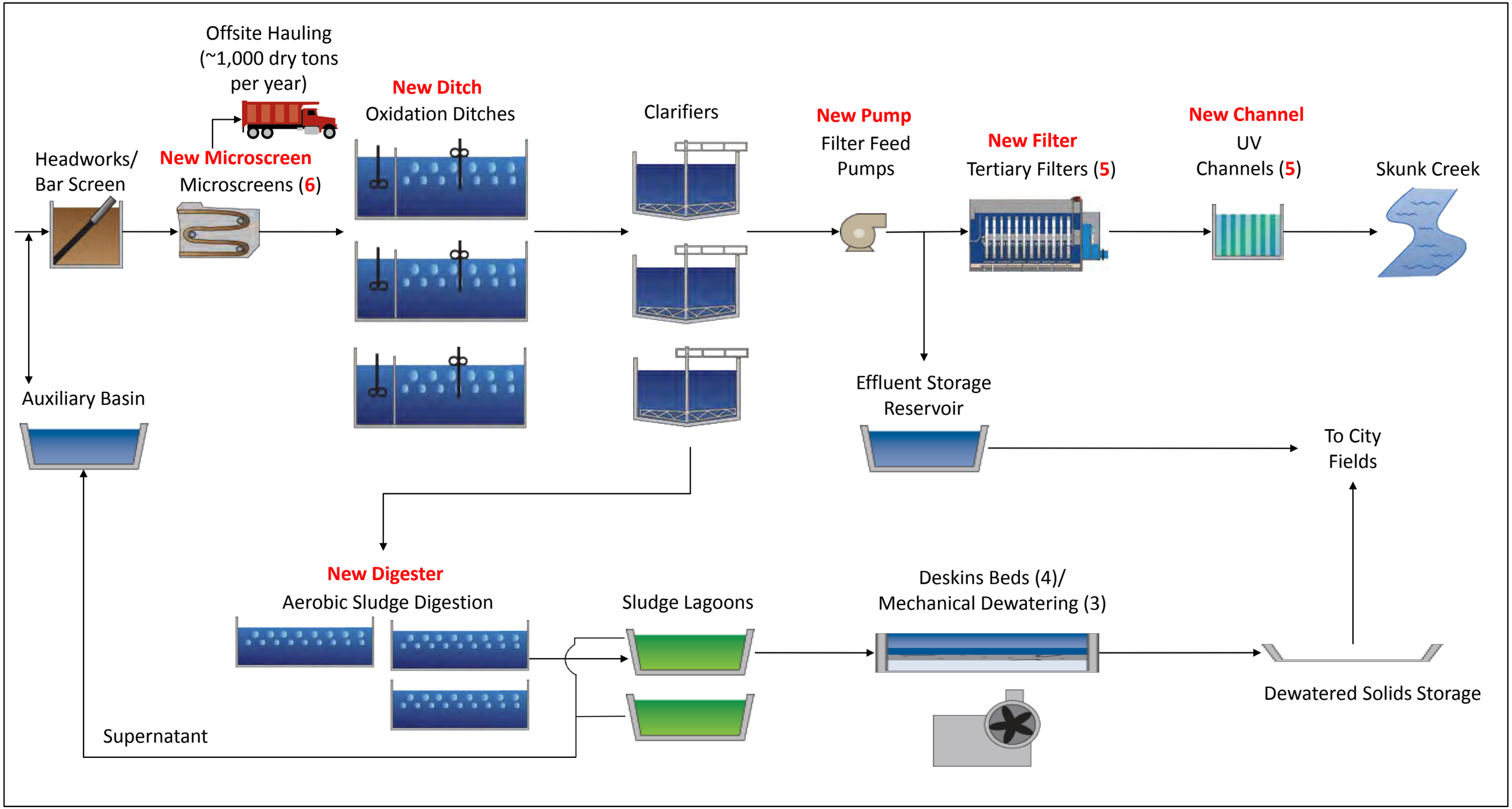

FIGURE 6-11
City of Galt
WWTP Facilities Master Plan
ALTERNATIVE F (HYBRID C+D) SCHEMATIC
ULTIMATE BUILD-OUT DESIGN CONDITION
(6.0 MGD ADWF)

WEST YOST ASSOCIATES
Consulting Engineers

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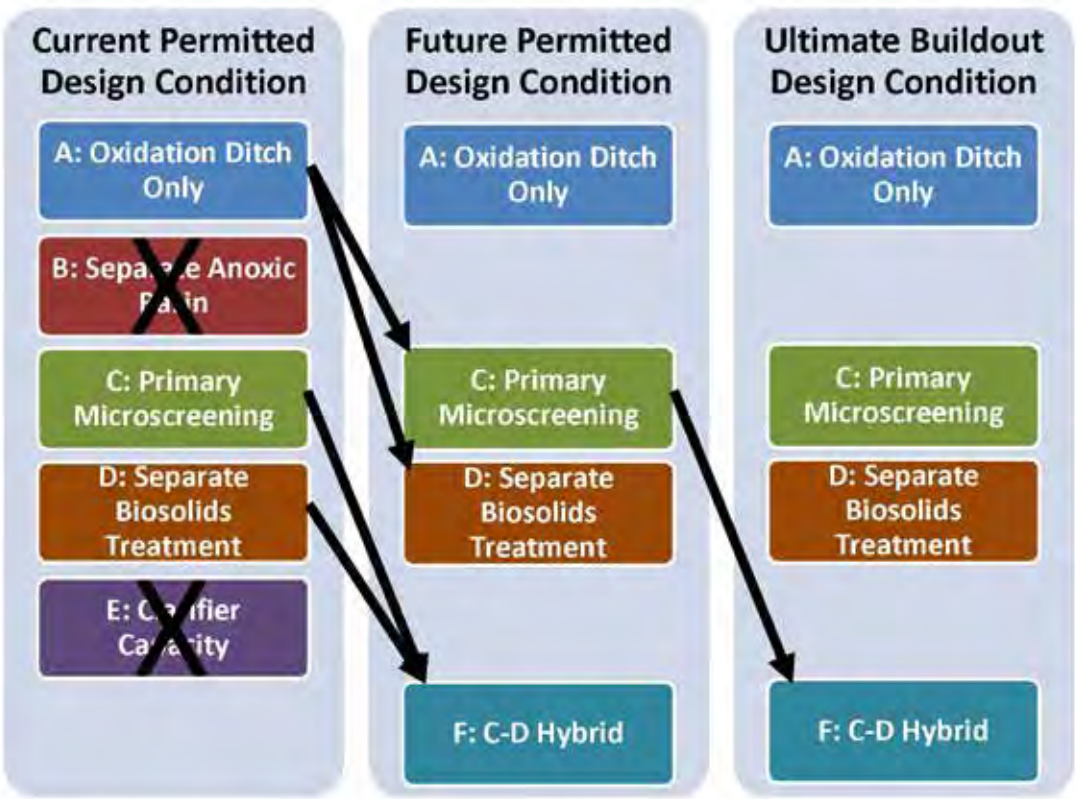


Figure 6-12. Flow Chart of Potential Expansion Options

CHAPTER 7
Recommended Facilities Implementation Plan

This chapter presents an implementation plan for the recommendations in this Facilities Master Plan. The following topics are addressed:

- Implementation Strategy
- Site Layout
- Detailed Implementation Plan and Schedule

7.1 IMPLEMENTATION STRATEGY

An adaptive management approach to facility expansion is recommended for each of the three design phases discussed in this Facilities Master Plan (i.e. Current Permitted Design Condition, Future Permitted Design Condition, and Ultimate Buildout Design Condition). At each phase, the City will need to make certain planning decisions to determine the next steps. This implementation strategy is described for each design phase in the sections below and is shown graphically in Figure7-1.

7.1.1 Immediate Improvements Strategy: Current Permitted Design Condition

It is recommended that the City implement Alternative A for the Current Permitted Design Condition upgrade project. As described in detail in Chapter 6, Alternative A involves constructing a third oxidation ditch/clarifier treatment train at the WWTP (and modifying the existing train s) to ensure the WWTP can both meet the nitrate and ammonia limits and treat the City's biosolids to Class B standards at the Current Permitted Design Condition. In addition, the Immediate Improvements Phase includes the following major facility improvements:

- Modify the headworks to allow for equalization of influent and solids treatment return flows
- Construct supplemental mechanical dewatering facilities
- Provide sludge lagoon improvements to improve mixing and access
- Remodel Control Building for improved laboratory and staff utilization
- Construct covered equipment storage facilities
- Provide maintenance building improvements to improve vehicle access
- Migrate Municipal Service Center SCADA system to WWTP and add various monitoring and control capabilities to existing equipment
- Construct backwash solids capture systems at two City supply wells to ensure compliance with the WWTP arsenic requirements
- Conduct a Water Effects Ratio Study and request recalculation of the effluent limit for copper.

The estimated City costs for the Immediate Improvements Phase project are summarized in Table 7-1.

Table 7-1. Estimated Costs for Immediate Improvements[a], million dollars Current Permitted Design Condition (3.0 mgd)

Cost Component	Cost
Estimate of Probable Construction Bid (December 2013 Dollars)	18.0
Project Allowances[b]	6.0
Water Effects Ratio Study (allowance)	0.1
Total Project Cost	**24.1**
Annual O&M Cost (2014 – 2019)	1.23
Net Present Value (2012 Dollars)	**28.6**

(a) From Tables 6-29 and 6-32. Estimated costs represent planning-level estimates using available information and the assumptions stated throughout the Facilities Master Plan. An inflation allowance has been included, which may be higher or lower than actual inflation and/or the effects of other market forces.

(b) Total Allowances are 33.5%, which include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction period services including construction management, O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%).

A number of other improvements to address existing deficiencies (broadly termed "Major Maintenance" projects in Chapter 6) may also be added to the project constructed at Current Permitted Design Condition, pending further City consideration of 1) the relative priority of the improvements and 2) the potential availability of funds given the actual bid price for the project. If not added to the Current Permitted Design upgrade project, these improvements (at a total estimated cost of $2.0 to $2.3 million) are expected to be completed as smaller projects that are implemented over time between the completion of the Current Permitted Design Condition upgrade project and the Future Permitted Design Condition expansion project.

Finally, if the City determines that relocating water division staff to the WWTP is preferred, the Immediate Improvements Phase project may also include expansion of the Operations and Electrical (O&E) Building to accommodate these staff.

7.1.2 Phase 2 Strategy: Future Permitted Design Condition

For the Future Permitted Design Condition expansion project, it is recommended the City implement Alternative C, which involves adding primary microscreening to the WWTP treatment train as a means of increasing the overall system capacity and offsetting the need to construct additional oxidation ditch/clarifier treatment trains. Because Phase 2 is not expected to be necessary for several years, it is expected that a number microscreening projects will have been installed by this time so that the City can better evaluate whether this technology will be appropriate for the WWTP.

The long-term viability of the microscreening approach (Alternative C) is also highly dependent on the City's ability to dispose of captured microscreening solids at a landfill site at a reasonable cost. For purposes of this Facilities Master Plan, the cost for hauling and disposal of the microscreening solids has been assumed to increase proportionately to inflation, but no more. However, changes in regulatory requirements could result in dramatic

increases in disposal costs, or add treatment requirements for the screened solids. The cost of such treatment would be over and above what is provided for in this Facilities Master Plan.

Therefore, as a first step in implementing the Future Permitted Design Condition expansion project, the City should complete a primary microscreening technology review to verify that Alternative C is the preferred approach. Topics that would need to be considered include:

- Have impacts of primary microscreening on downstream plant operations been adequately evaluated, especially at plants with similar oxidation ditch operations?
- Are there new or pending restrictions on disposal of un-treated microscreening solids at landfills?
- Do long-term operating and solids disposal costs make microscreening more expensive than initially anticipated and/or cost prohibitive?
- Are there more cost effective solids treatment and disposal options that should be considered?

If this analysis demonstrates that microscreening is not a reliable option for the Galt WWTP under the Future Permitted Design Condition expansion project, Alternative D should be implemented. As described in Chapter 6, Alternative D involves constructing a new secondary biosolids treatment process at the WWTP to increase the overall system capacity and offset the need to construct an additional oxidation ditch/clarifier secondary treatment train.

The estimated City costs at Phase 2 for Alternatives C and D are summarized in Table 7-2.

Table 7-2. Estimated Costs for the Phase 2 Expansion Project[(a)], million dollar Future Permitted Design Condition (4.5 mgd)

Cost Component	Immediate Improvements >>	Cost	
		Alternative A	
	Phase 2 >>	Alternative C2	Alternative D2
Estimate of Probable Construction Bid (2018 Dollars, based on 3% annual inflation)		9.4	17.1
Project Allowances[(b)]		3.2	5.7
Total Project Cost		**12.6**	**22.8**
Annual O&M Cost (2020 – 2031)		1.67	1.78
Net Present Value (2012 Dollars)		**24.0**	**31.9**
Total Net Present Value for Immediate Improvements and Phase 2		**52.6**	**60.5**

(a) From Tables 6-30 and 6-32. Estimated costs represent planning-level estimates using available information and the assumptions stated throughout the Facilities Master Plan. An inflation allowance has been included, which may be higher or lower than actual inflation and/or the effects of other market forces.

(b) Total Allowances are 33.5%, which include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction management, including O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%).

7.1.3 Phase 3 Strategy: Ultimate Buildout Design Condition

As noted above, the long-term viability of the microscreening approach (Alternative C) is highly dependent on the City's ability to dispose of un-treated microscreening solids at a nearby landfill site at a reasonable cost. Given the uncertainties regarding this assumption over the 20 to 30 year planning horizon of this Facilities Master Plan, it will likely be necessary to re-examine the long-term viability of microscreening solids disposal as the City prepares to implement Phase 3. As part of this effort, the City should also determine if treatment and land application of secondary solids is also a viable long-term approach given the limited available biosolids disposal capacity of the existing land application area. Therefore, as the ADWF nears 4.5 mgd (i.e. the capacity installed at the Future Permitted Design Condition), it is recommended that the City complete a Solids Treatment and Disposal Alternatives Study to answer, at a minimum, the following questions:

- Are there new or pending Class B biosolids regulations that could preclude or impact onsite land-application?
- Do long-term operating and solids disposal costs make onsite land disposal of secondary solids more expensive than initially anticipated and/or cost prohibitive?
- Are there new or pending restrictions on disposal of secondary solids and/or un-treated microscreening solids at landfills?
- Are the costs of hauling and/or landfill disposal primary or secondary solids increasing or likely to increase?
- Do long-term operating and solids disposal costs make microscreening more expensive than initially anticipated and/or cost prohibitive?
- What technologies are available for co-treatment of primary microscreening solids and secondary solids and what are their costs?
- Are there any offsite biomass or biogas energy generating facilities that would accept primary or secondary solids as an alternative to landfill disposal?
- Are there new/better technologies available for energy generation from primary and secondary solids (and possibly other materials) that can decrease WWTP energy-related operating costs and/or provide revenue?
- Are there options for expanding the land application area?

Assuming that Alternative C is implemented for the Future Permitted Design Condition, the Solids Treatment and Disposal Alternatives Study would also help the City determine which of the following two treatment train approaches should be implemented at the Ultimate Buildout Design Condition expansion project:

- Continue to expand the WWTP primary, secondary, and tertiary facilities in kind (i.e. Alternative C); or
- Construct a separate biosolids treatment facility to increase overall system capacity and thereby avoiding the need to expand the secondary facilities (i.e. Alternative F).

Specifically, if the Solids Treatment and Disposal Alternatives Study demonstrates that there is a viable, long term disposal option for both secondary solids treated in the oxidation ditches *and* un-treated microscreening solids, then the City should implement Alternative C – otherwise Alternative F will likely be the preferred approach.

On the other hand, if Alternative D is implemented for the Future Permitted Design Condition, it is expected that Alternative F would be implemented at the Ultimate Buildout Design Condition – regardless of the outcome of the Solids Treatment and Disposal Alternatives Study. This recommendation is based on the following assumptions:

- Adding microscreening at this stage will be less expensive than expanding the secondary treatment and solids handling processes as demonstrated in this Facilities Master Plan, and
- When City is ready to implement the Future Permitted Design Condition expansion project, primary microscreening (or potentially some other innovative primary treatment technology) will have been demonstrated as a viable technology.

To verify these assumptions, the City would need to re-evaluate the microscreening technology at an early planning stage for Phase 3 in addition to completing the Solids Treatment and Disposal Alternatives Study described above.

Finally, the Solids Treatment and Disposal Alternatives Study may demonstrate that an alternative strategy is needed for disposal of microscreening solids under Alternative F – regardless of whether Alternative C or D is implemented at the Future Permitted Design Condition. Technologies for co-treating primary solids with secondary solids (as well as other wastes) while generating energy are expanding rapidly, and it is possible that the recommended Solids Treatment and Disposal Alternatives Study will demonstrate that anaerobic co-treatment of primary and secondary solids is a preferred strategy due to the following benefits:

- Co-treatment provides a reliable approach for meeting EPA biosolids regulations for land application of both primary and secondary solids (eliminates landfill disposal)
- Co-treatment will allow for secondary solids to be handled -anaerobically, reducing WWTP energy costs
- Energy could potentially be cost-effectively generated onsite from the anaerobic treatment biogas as compared to the cost of purchasing power.

However, because the City would have already committed to an aerobic secondary solids treatment system if Alternative D is implemented at the Future Permitted Design Condition, the savings associated with constructing a primary and secondary solids co-treatment system at Phase 3 of would not be realized as it would be if the City moved from Alternative C to Alternative F at Phase 3.

The estimated City costs at Phase 3 for implementing either Alternative C or F – as related to whether Alternative C or D is implemented at Phase 2 - are summarized in Table 7-3.Also shown is the total estimated net present value cost for all three phases combined.

Table 7-3. Estimated Costs for the Phase 3 Expansion Project, million dollar Ultimate Buildout Design Condition (6.0 mgd)

Cost Component		Cost(b)		
	Immediate Improvements >>	Alternative A		
	Phase 2 >>	Alternative C2		Alternative D2
	Phase 3 >>	Alternative C2	Alternative F2	Alternative F3
Estimate of Probable Construction Bid (2030 Dollars, based on 3% annual inflation)		14.6	18.4	11.1
Project Allowances[(b)]		4.9	6.2	3.7
Total Project Cost		**19.5**	**24.6**	**14.8**
Annual O&M Cost (20 years, 2032 – 2051)		2.09	2.10	2.10
Net Present Value		**31.2**	**33.2**	**29.5**
Total Net Present Value for All Phases		**83.8**	**85.8**	**90.0**

(a) From Tables 6-31 and 6-32. Estimated costs represent planning-level estimates using available information and the assumptions stated throughout the Facilities Master Plan. An inflation allowance has been included, which may be higher or lower than actual inflation and/or the effects of other market forces.

(b) Total Allowances are 33.5%, which include: City construction materials and services (0.5%); City design and construction period costs (0.5%); predesign (2%); design services (15%); construction management, including O&M manuals and record drawings (15%); and other City materials and services, including environmental (0.5%).

7.2 RECOMMENDED SITE LAYOUT

Expansion of the central area of the WWTP (i.e., the area containing major structures and process equipment) is recommended to be limited to the presently vacant areas within the existing central area and portions of the agricultural fields south of the existing central area.

Figure 7-2 depicts the recommended facilities layout if the City determined that Alternative C (Microscreening) was the preferred approach at both Phases 2 and 3.

Figure 7-3 depicts the recommended layout of facilities if the City determined that determined that Alternative C (Microscreening) was the preferred approach at Phase 2 and Alternative F (Microscreening/Solids Treatment Hybrid) was the preferred approach at Phase 3. Note that if the City were to implement Alternative D at Phase 2 and Alternative F at Phase 3, the ultimate site layout shown in Figure 7-3 would be the same. However, the City would construct two of the aerobic digesters at Phase 2, and the third digester and the entire microscreening facility at Phase 3.

7.3 IMPLEMENTATION PLAN

A preliminary schedule for the implementation of the three WWTP phases discussed in this Facilities Master Plan and for the Arsenic Compliance Plan discussed in Appendix C is presented in Figure 7-4. Additional discussion of the proposed schedule for each phase is provided below.

7.3.1 Immediate Improvements Plan: Current Permitted Design Condition

The primary driver for the timeline of the Current Permitted Design Condition (Immediate Improvements) is the September 1, 2015 compliance date for meeting effluent ammonia and nitrate/nitrite requirements in the WDRs. However, even if the City immediately initiates design of the Immediate Improvements Phase upon completion of this Facilities Master Planning effort, the Immediate Improvements will not likely be fully completed by September 2015. Nevertheless, with careful staging of the construction project, the oxidation ditch expansion and improvement projects necessary to meet the ammonia and nitrate/nitrite limitations should be completed by the compliance date. Moreover, it should be noted that the City could get mandatory minimum penalties relief from ammonia and nitrate/nitrate violations for up to a 90-day start-up period under the protection of a Facilities Startup Plan. Therefore, there is some flexibility to allow the start-up of the oxidation ditches to extend beyond the September 1, 2015 deadline.

7.3.2 Arsenic Compliance Plan

The schedule shown for the Arsenic Compliance Plan is consistent with the recommendations outlined in Appendix C. As shown, the schedule provides the City with the flexibility to determine the best long-term strategy for handling the arsenic treatment backwash solids.

7.3.3 Phase 2 Plan: Future Permitted Design Condition

The schedule presented in Figure 7-4 for the Future Permitted Design Condition is based on an annual wastewater flow increase of 3.4 percent per year after 2014 (as presented in Figure 2-1 in Chapter 2, Basis of Design). In accordance with the City's WDRs, when the flow projection shows that capacity of any part of the facilities may be exceeded in four years, the City must have a plan in place for the next flow increase. Assuming a 3.4 percent growth rate, the City will need to have selected a preferred project for the Future Permitted Design Capacity expansion when flows reach approximately 2.6 mgd. (New, large developments within Galt's wastewater service area occurring prior to the 2.6 mgd mark may also trigger the initiation of facilities planning). Therefore, it is recommended that the facilities planning efforts begin when flows are approximately 85 percent of 3.0 MGD.

As shown in Figure 7-4, if growth rates are as predicted, the Phase 2 microscreening technology and reliability review will need to begin in mid to late-2015, or around the time the Current Permitted Design Condition project is being completed. This schedule provides an additional 3 years for the microscreening technology to become established in the U.S. The list below includes topics that should be addressed in the microscreening technology review, as well as suggested conditions that would indicate the viability of microscreening for Galt:

- Number and service durations of primary microscreening installations in the U.S. and California, particularly at WWTPs utilizing an extended aeration treatment process.
  - Three to five WWTPs in U.S., preferably in California, with microscreening operating for two or more years.
  - At least one WWTP in U.S. with microscreening installed upstream of extended aeration secondary processes.

- General performance and owner satisfaction surveys of microscreening installations in the U.S.
  - Reliable BOD and TSS removal near manufacturer's stated ranges.
  - Generally not operator or maintenance intensive.
- Operation and maintenance issues.
  - Typical issues limited to periodic inspection and preventive maintenance.
- Impact of microscreening on secondary solids settleability, thickening and/or dewaterability.
  - Predictable changes in settleability and dewaterability.
  - Reasonable requirements for coagulant dosing in thickening and/or dewatering facilities.
  - No secondary process upsets attributable to microscreening.
- Microscreening solids disposal.
  - Reliable solids cake concentration and structure.
  - Dewatered solids generally accepted by landfills in California.
- Availability of pilot units.
  - Pilot units from multiple manufacturers available within a 12-month window.
- Comparisons between pilot and full scale performance.
  - Full scale installation performance generally equal to or better than pilot unit.
- Manufacturer assistance available to owners before and after warranty period.
  - Available for startup training and post-warranty period technical assistance.

If the technology and reliability review reveals that microscreening is *not* a viable alternative, the City should move forward with the implementation of Alternative D. If the review indicates that microscreening is viable, the City will move forward with implementing Alternative C. In this case, pilot studies of a microscreening unit from one or more manufacturer(s) will need to be included as part of the predesign of the Phase 2 improvements.

7.3.4 Phase 3 Plan: Ultimate Buildout Design Condition

The schedule presented in Figures 7-4 for the Ultimate Buildout Design Condition is also based on an annual wastewater flow increase of 3.4 percent per year after 2014 (as presented in Figure 2-1 in Chapter 2, Basis of Design). To ensure a plan is in place for the Phase 3 capacity increase, the City will need to have selected a preferred project for Future Permitted Design Capacity expansion project when flow reach approximately 3.9 mgd. (Again, a large development within Galt's wastewater service area occurring prior to the 3.9 mgd mark may also trigger the initiation of facilities planning).

The microscreening technology and reliably review discussed above will inform the recommended solids handling studies discussed below. Therefore, the microscreening technology and reliably review is shown as the first step in the Phase 3 planning process. (If in the next 15 years microscreening becomes a well-established technology, the level of effort for the microscreening technology and reliably review can likely be reduced.)

As discussed previously in this Chapter, technologies for treatment of solids and/or using solids to generate energy are evolving rapidly. In addition, the City's land application area disposal capacity is limited and new regulations could impact the City's ability to dispose of solids onsite (and potentially at landfills). Therefore, if not triggered earlier by new regulatory constraints, the City should complete a Solids Treatment and Disposal Alternatives Study in preparation for implementing Phase 3, Ultimate Buildout Design Phase expansion project. Specifically, the City should review current regulations for solids disposal and identify whether new technologies for using solids to generate energy (e.g., pyrolysis and gasification) are a viable option for the WWTP long-term.

Depending on the number and complexity of solids handling options being considered, the Solids Treatment and Disposal Alternatives Study could take up to one year to complete. Therefore, it is recommended that the facilities planning efforts for Phase 3 begin when flows are approximately 80 to 85 percent of 4.5 mgd, or approximately 3.7 to 3.8 mgd. The list below includes topics that should be addressed as part of this review

- Review anticipated changes to biosolids landfill disposal and land application regulations.
  - Identify if changes to regulations could require additional stabilization, pathogen reduction or vector attraction reduction prior to land application of Class B biosolids.
  - Determine if restrictions on landfill disposal are likely to result in significantly higher disposal costs.
- Review the capacity for the oxidation ditches to meet more stringent regulations (if applicable).
- Identify other potential solids treatment technologies that can reliably meet more stringent requirements (if applicable).
- Identify available technologies for processing primary solids for energy production (e.g., pyrolysis and gasification processes).
  - Number and service durations of primary solids processing systems in the U.S. and California, particularly at WWTPs utilizing a primary microscreening treatment process.
  - Thickening and dewatering requirements for primary solids feed stock.
  - Estimated capital and O&M costs.
  - Energy production potential.
  - Availability of solids processing (e.g., pyrolysis and gasification) pilot units.

- Review anaerobic digestion technology for co-processing of primary and secondary solids:
  - Thickening and dewatering requirements for primary and secondary solids feed stocks.
  - Estimated capital and O&M costs.
  - Energy production potential.
- Review of the capacity of the existing land application area to accept biosolids.
  - Consider both the secondary Class B solids as well as any primary solids that may be treated onsite for land application disposal.
- Identify a preferred long-term solids disposal plan
  - Reevaluate potential new biosolids land application areas (as appropriate)
  - Compare continued land application to other potentially viable disposal options.
- Develop a recommended Solids Treatment and Disposal Plan

If through the above efforts it is determined that modifications to the solids treatment and disposal system are needed, then the City should implement Alternative F. However, if the review of regulations reveals that continued landfill disposal is the preferred long-term strategy, then Alternative C is the likely best approach for the Phase 3, Ultimate Buildout Design Condition project.

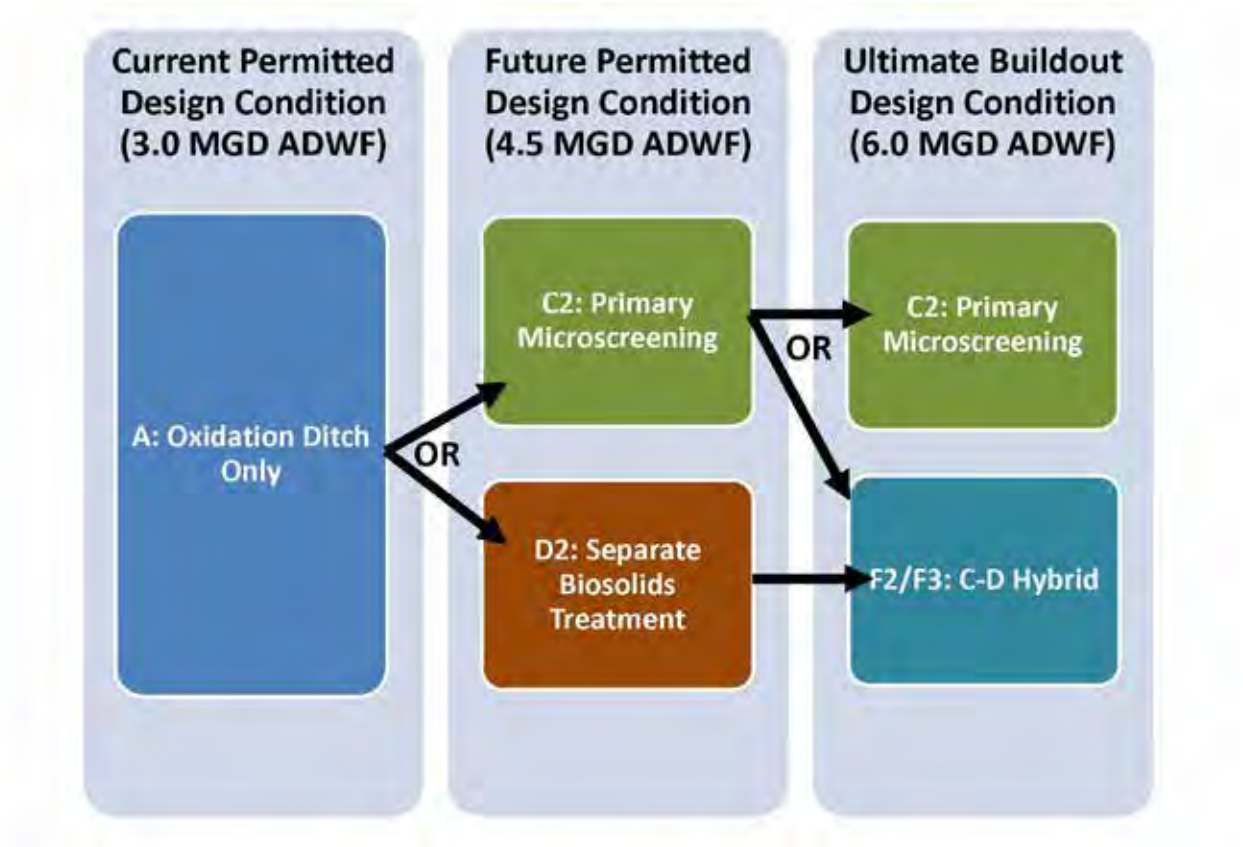
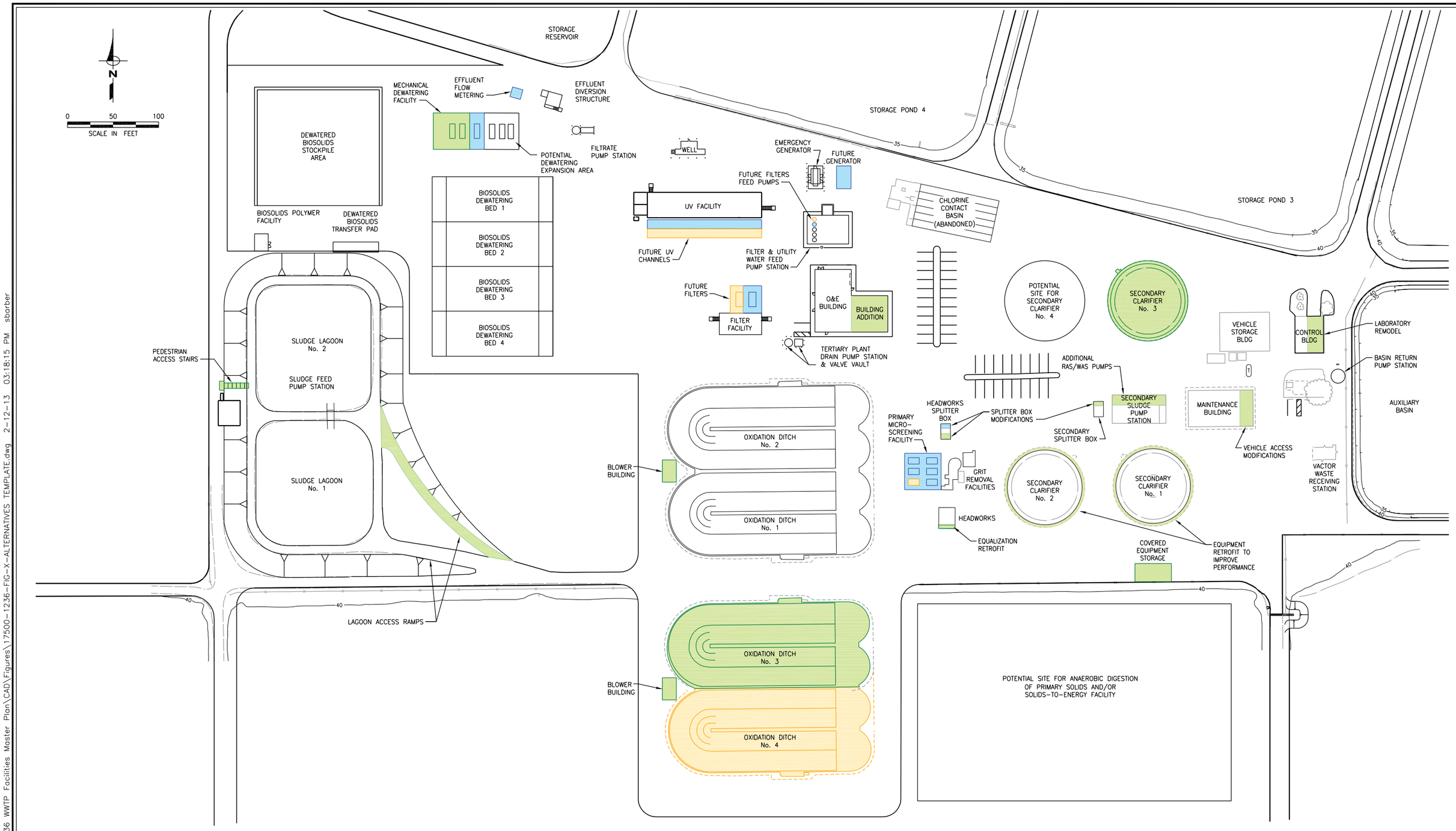


Figure 7-1. Flow Chart of the Recommended Facilities Master Plan Implementation Strategy



LEGEND

- FACILITIES INSTALLED FOR CURRENT PERMITTED DESIGN CONDITION
- FACILITIES INSTALLED FOR FUTURE PERMITTED DESIGN CONDITION
- FACILITIES INSTALLED FOR ULTIMATE BUILD-OUT DESIGN CONDITION

FIGURE 7-2

City of Galt
WWTP Facilities Master Plan
SITE LAYOUT FOR ALTERNATIVE C: MICROSCREENING

WEST YOST ASSOCIATES
Consulting Engineers

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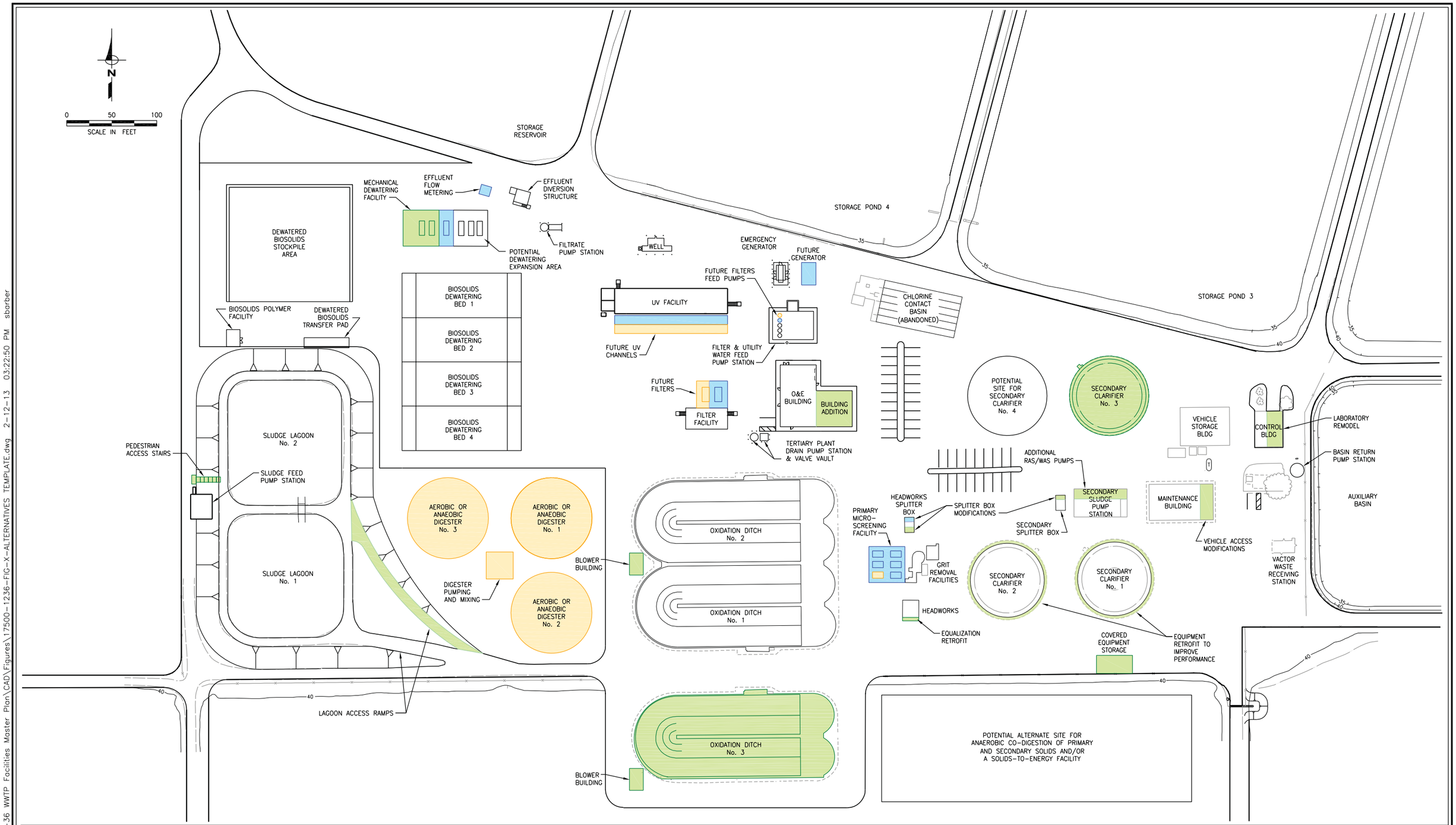

LEGEND

- FACILITIES INSTALLED FOR CURRENT PERMITTED DESIGN CONDITION
- FACILITIES INSTALLED FOR FUTURE PERMITTED DESIGN CONDITION
- FACILITIES INSTALLED FOR ULTIMATE BUILD-OUT DESIGN CONDITION

FIGURE 7-3

City of Galt

WWTP Facilities Master Plan

SITE LAYOUT FOR ALTERNATIVE F

(MICROSCREENING/SOLIDS TREATMENT HYBRID)

WEST YOST ASSOCIATES
Consulting Engineers

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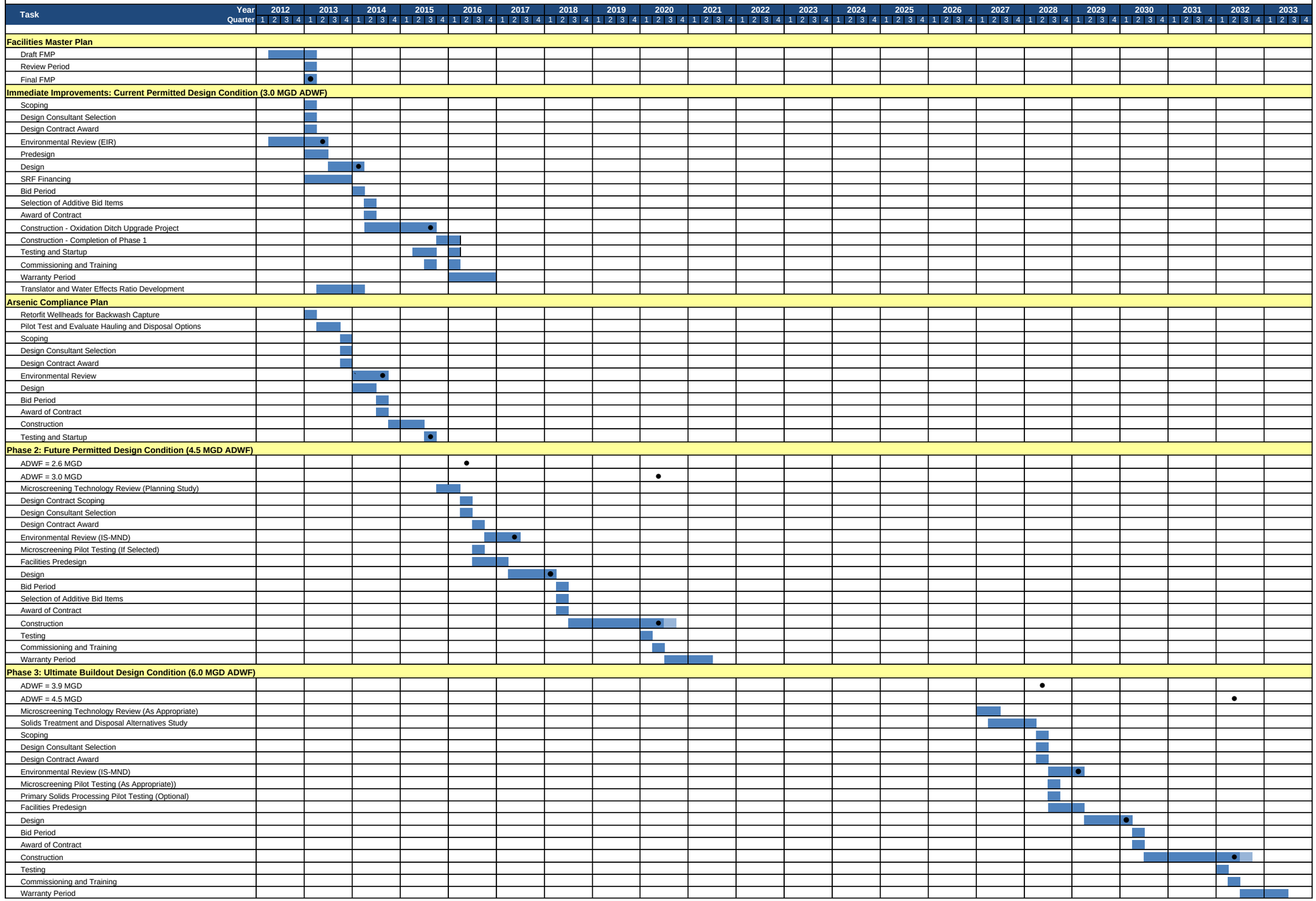
Figure 7-4 - Preliminary Schedule for Upgrade and Expansion Projects



APPENDIX A

Water Quality Sampling Plan

Sampling Plan for Secondary Process Modeling: City of Galt WWTP

Prepared for

City of Galt

April 2012

WEST YOST ASSOCIATES
Consulting Engineers

175-00-12-36

Table of Contents

1.0 Project Schedule 1

2.0 Pre-Sampling Preparation 1
- 2.1 Laboratory Involvement 1
- 2.2 Sample Bottle Acquisition and Labeling 2
- 2.3 Equipment Preparation 2

3.0 Two-Week Daily Sampling 3
- 3.1 Sample Locations 3
- 3.2 Timing of Samples and Readings 4
- 3.3 Analytical Requirements 4
- 3.4 Data Collection Log Sheet 6

4.0 48-Hour Diurnal Sampling 6
- 4.1 Timing of Samples 6
- 4.2 Analytical Requirements 6
- 4.3 Data Collection Log Sheet 7

5.0 WWTP Recycle Streams Sampling 7
- 5.1 Deskin Bed Filtrate 8
- 5.2 Sludge Lagoon Decant 9

List of Tables

Table 1. Two-Week Program Onsite Measurements 4

Table 2. Two-Week Program Sampling Locations and Analyses 5

Table 3. Sample Container Volumes and Analyses for Two-Week Sample Collection 5

Table 4. Diurnal Sampling Locations and Analyses 7

Table 5. Sample Container Volumes and Analyses for Diurnal Sample Collection 7

Table 6. Sample Container Volumes and Analyses for Recycle Stream Sample Collection 8

List of Figures

Figure 1. Oxidation Ditch Monitoring Locations 9

The purpose of this document is to present a plan for wastewater sample collection and analysis in support of modeling of the secondary treatment processes at the City of Galt (City) Wastewater Treatment Plant (WWTP). The major aspects of this sampling plan include the following:

- Project Schedule
- Pre-Sampling Preparation
- Two-Week Daily Sampling
- 48-Hour Diurnal Sampling
- WWTP Recycle Streams Sampling

WWTP operational changes in preparation for the sampling effort were discussed in a separate memorandum titled Sampling Program Prerequisite Steps, dated March 2, 2012. It should be noted, however, that the proposed sampling period begins later than described in that memorandum.

1.0 PROJECT SCHEDULE

The proposed sampling effort has two major components. The first is a 14-day sampling effort consisting of daily grab and composite samples (see Two-Week Sampling Program below). The second is a 48-hour diurnal sampling effort with samples collected every two hours that will occur over the last two days of the 14-day sampling period.

The proposed schedule for the two-week program is for sampling to begin on Sunday, April 22 and continuing through Saturday, May 5. The proposed schedule for the 48-hour diurnal sampling effort is for sampling to begin late on Thursday, May 3 and continuing through Saturday, May 5. The logic of the latter schedule is to capture both weekday and weekend diurnal variation and will coincide with the last three days of the two-week program.

2.0 PRE-SAMPLING PREPARATION

The major preparatory steps prior to actual sample collection include:

- Laboratory Involvement
- Sample Bottle Acquisition and Labeling
- Equipment Preparation

2.1 Laboratory Involvement

The expectation of this sampling plan is that the City will utilize the services of Caltest Analytical Laboratory (Caltest) in Napa, CA. Caltest is a high-precision laboratory known for their low-level analytical techniques and overall reliability.

It is recommended that Caltest provide all sample pick-ups and container drop-offs. On weekdays, they will provide sample container drop-off and sample pick-up at no charge. There is a fee for weekend drop-off and pick-up. The delivered sample containers will have the required sample preservative in them (where applicable).

Samples to be analyzed for soluble BOD, COD, TKN, and phosphorus must be filtered by Caltest immediately upon arrival at the lab. (The instructions to the laboratory need to specify that Caltest perform this filtering.) Because filtered samples are not preserved in the field, and because certain analyses (e.g., BOD) otherwise have very short hold times it is essential to schedule pick-up of samples on a daily basis.

Oxidation ditch samples to be analyzed for ammonia, nitrate and nitrite, and influent samples to be analyzed for dissolved arsenic require field filtration by WWTP staff. Caltest will provide disposable filtration units for this field filtering.

It is recommended that for the first three days of the two-week sampling program, the analysis of certain oxidation ditch samples (specified below) are to be expedited on a three-day turnaround time. (The expedited turn-around will increase the cost for those samples.) Advanced notice of expedited turnaround times should be provided to the lab to ensure readiness. The results obtained will then be used to make in-program adjustments, if appropriate, for the remainder of the oxidation ditch samples.

2.2 Sample Bottle Acquisition and Labeling

The analytical laboratory will provide the sample bottles, including those containing sample preservative, where applicable. City staff will need to label the bottles appropriately. Each sample bottle label must include the following information:

- Location of collected sample
- Date and time of collected sample
- Sequential number of collected sample (1 through 14 for daily samples; 1 through 24 for diurnal samples)
- Initials of persons collecting the sample
- Analyses to be performed
- Presence of sample preservative (where applicable)

The date and time should be written on the sample container when the sample is collected. The other information may be written on the container in advance of sample collection.

2.3 Equipment Preparation

The equipment needed for the sampling includes the following:

- Composite samplers (two – the existing influent sampler plus one for secondary effluent that may later be used for side stream sampling). It is recommended that the sampler tubing be replaced with new tubing prior to initiation of the sampling program.
- Portable continuous dissolved oxygen (DO) meters (two portable meters in addition to the existing permanently installed oxidation ditch DO sensors) with logging devices that provide for at least 48 hours of continuous data recording.

- Computer or other device needed to download data from the portable DO meter logging devices.
- Portable temperature/pH meter(s).
- Refrigeration facilities and/or ice chests w/ice.

The portable DO meters need to be set up to collect DO readings in either of the two oxidation ditches, as described in the next section (see Sample Locations). All meters must be adequately calibrated in advance of collection of field readings.

3.0 TWO-WEEK DAILY SAMPLING

The two-week daily sampling effort will involve a combination of onsite measurement (flow, pH and DO), composite samples (influent and secondary effluent), and grab samples (oxidation ditch zones and WAS). The major issues related to the two-week sampling effort include:

- Sample Locations
- Timing of Samples and Readings
- Analytical Requirements
- Data Collection Log Sheet

3.1 Sample Locations

The onsite measurements, grab samples, and composite samples will be collected at the following sampling locations:

- Influent – Composite samples and onsite measurements are to be collected at the existing composite sampler location. Flow-weighting of composite samples will utilize the existing influent flow meter.
- Secondary Effluent – Composite samples and onsite measurements are to be collected at the filter feed pump station wet well or other suitable location. If possible, flow-weighting of composite samples should utilize the existing UV system flow meter. If flow weighting is not possible, a diurnal flow curve should be programmed into the composite sampler. If the sampler lacks that capability, time-weighted composites will be necessary.
- Oxidation Ditches – Grab samples and onsite measurements are to be collected at the locations indicated in Tables 1 and 2 and as shown in Figure 1. Grab samples and readings will only be collected from one of the two oxidation ditches, as both ditches operate in parallel and are expected to have equivalent performance. As noted previously, the City should request that the analytical results for the first three days of monitoring be provided within a three-day turnaround time. The data collected during the first three days will then be used to determine if the oxidation ditch sampling locations should be modified to better capture the data needed to define the oxidation ditch performance. If a change to the monitoring locations is proposed, a revised figure showing the new monitoring locations will be provided to the City.

- RAS/WAS – Grab samples will be collected as the usual sampling location for this stream. RAS flow monitoring will occur using the existing meter and estimated WAS flow rates will be recorded daily.

All wastewater samples and field measurements should be taken from below the flow surface (to avoid floating materials), and away from the edge of the given flow channel.

3.2 Timing of Samples and Readings

Grab samples from the oxidation ditch should be timed to coincide with the peak diurnal flow/loading condition of the day. The influent and secondary effluent onsite readings and the RAS/WAS samples could be collected shortly before or after collection of the oxidation ditch grab samples. Note that the influent pH and temperature should also be collected when flow is being actively pumped to the WWTP from the Live Oak pump station. DO measurements must be collected continuously.

Each composite sample will be collected over a 24-hour period, with a morning to morning schedule being adequate. All composite samples should be collected as flow-weighted composites, if possible. The total volume collected by the composite sampler should exceed the minimum volume requirements for laboratory analysis (see Table 3 below).

3.3 Analytical Requirements

Onsite measurements to be collected are summarized in Table 1. The laboratory analyses for the two-week sampling effort are summarized in Table 2. The sample bottle sizes, analyses to be performed filtration method, and need for sample preservation at each sampling location are shown in Table 3.

All samples must be maintained at 4°C at all times, either with a refrigerator or using ice chests. Hold times are as low as 48 hours from the time of sample collection for some of the analyses, and filtration will also be necessary on a very short time for certain analyses; therefore, prompt delivery to the laboratory is critical for this sampling program.

All composite samples will need to be decanted off from the composite container into the appropriate containers indicated in Table 3. With regard to the influent composite sample, it is critically important that the sample collected in the composite bottle be adequately mixed prior to pouring off into the laboratory sample container.

Table 1. Two-Week Program Onsite Measurements

Parameter	Influent	Secondary Effluent	Oxidation Ditch Locations 1, 4, 6	RAS	WAS
Flow	X	X		X	X
Dissolved oxygen			X		
pH	X	X			
Temperature	X	X			

Table 2. Two-Week Program Sampling Locations and Analyses

Parameter	Influent	Secondary Effluent	Oxidation Ditches		RAS/WAS
			Location 2 only	Locations 2–5	
Nitrogen					
Ammonia (as N)	X	X		X	
Nitrate (as N)	X	X		X	
Nitrite (as N)	X	X		X	
TKN-total	X		X		
TKN-soluble	X	X			
Organics					
BOD-total	X				
BOD-soluble	X	X			
COD-total	X	X	X		
COD-soluble	X	X			
Phosphorous					
Phosphorous-total	X				
Phosphorous-soluble	X				
Solids/Alkalinity					
TSS	X	X	X		X
VSS	X		X		X
Alkalinity	X	X			

Table 3. Sample Container Volumes and Analyses for Two-Week Sample Collection

Analyses	Sampling Location	Volume/Container	Filtration	Preservative
Ammonia; nitrate; nitrite; TKN-total; COD-total; phosphorous-total	Influent	1 liter PE	none	H_2SO_4
TKN-soluble; BOD-total; BOD-soluble; COD-soluble; phosphorous-soluble; TSS; VSS; alkalinity[(a)]	Influent	2 liter PE	Caltest (soluble parameters only)	none
Ammonia, nitrate; nitrite; COD-total	Secondary effluent	1 liter PE	none	H_2SO_4
TKN-soluble; BOD-soluble; COD-soluble; TSS; alkalinity[(a)]	Secondary effluent	2 liter PE	Caltest (soluble parameters only)	none
Ammonia; nitrate; nitrite	Ox-ditch zones 2–5	500 mL PE	Field filtered by WWTP Staff	H_2SO_4
TKN-total; COD-total; TSS; VSS	Ox-ditch zone 2	500 mL PE	none	none
TSS; VSS	RAS/WAS	500 mL PE	none	none

PE = polyethylene
[(a)] Filtration for analysis of soluble parameters to be performed by Caltest (must be requested).

3.4 Data Collection Log Sheet

A data collection log sheet will need to be produced by the City for the study. The data log sheet will need to provide for recording the following information:

- Influent and effluent pH and temperature readings and timing of measurements
- Timing and type of oxidation ditch grab sample collection at all locations
- Timing of RAS/WAS grab sample collection
- WAS flow rate
- Timing of composite sample collection (start and finish)
- Timing of download of DO meters

4.0 48-HOUR DIURNAL SAMPLING

The diurnal sampling will consist of the collection of grab samples at the same sampling locations identified above for the two-week program. The major issues related to diurnal sampling include:

- Timing of Samples
- Analytical Requirements
- Data Collection Log Sheet

4.1 Timing of Samples

As indicated above, the diurnal sampling event is proposed for May 3–5 (5:00 p.m. Thursday to 3:00 p.m. Saturday). The event will consist of samples being collected every two hours for a 48-hour period. Because the sample collection is on two-hour intervals, the last sample of the 48-hour cycle would occur approximately 44 hours after the first sample.

The sequence for collection of diurnal samples for each two hour interval should be as follows:

- Influent
- Oxidation ditch
- Secondary effluent

Influent samples will need to be collected when there is flow (i.e., Live Oak is running).

4.2 Analytical Requirements

The analyses to be performed at the various sampling locations are indicated in Table 4 below. The sample bottle sizes, analyses to be performed filtration method, and need for sample preservation at each sampling location are shown in Table 5.

All samples must be maintained at 4°C, either with a refrigerator or using ice chests. Hold times are as low as 48 hours from the time of sample collection for some of the analyses, and filtration will also be necessary on a very short time for certain analyses; therefore, prompt delivery to the laboratory is critical for this sampling program.

Table 4. Diurnal Sampling Locations and Analyses

Parameter	Influent	Secondary Effluent	Oxidation Ditch Zones 2–5
Ammonia (as N)	X	X	X
Nitrate (as N)		X	X
Nitrite (as N)		X	X
TKN-total	X	X	
COD-total	X	X	

Table 5. Sample Container Volumes and Analyses for Diurnal Sample Collection

Analyses	Sampling Location	Volume/Container	Filtration	Preservative
Ammonia; TKN-total; COD-total	Influent	1 liter PE	none	H_2SO_4
Ammonia, nitrate; nitrite; TKN-total, COD-total	Secondary effluent	1 liter PE	none	H_2SO_4
Ammonia; nitrate; nitrite	Ox-ditch zones 2–5	500 mL PE	Field filtered by WWTP Staff	H_2SO_4
PE = polyethylene				

4.3 Data Collection Log Sheet

A data collection log sheet will need to be produced by the City for the diurnal sampling period. The data log sheet will need to provide for recording the timing and type of sample collection at all locations.

5.0 WWTP RECYCLE STREAMS SAMPLING

There are two major recycle streams that affect the loadings into the WWTP, and therefore, should be monitored for certain key constituents. These two streams include:

- Deskin Bed Filtrate
- Sludge Lagoon Decant

5.1 Deskin Bed Filtrate

It is understood that the Deskin bed filtrate recycle stream will be off-line during the sampling activities described above. Either before or after the two-week daily and 48-hour diurnal sampling programs, it is recommended that the Deskin bed filtrate recycle be analyzed for the following constituents:

- Alkalinity
- Ammonia
- Arsenic-total
- Arsenic-dissolved
- BOD-total
- COD-total
- COD-soluble
- Nitrate/nitrite
- pH (field measured)
- TKN-total
- TSS
- VSS

A total of 10 samples are recommended, to be analyzed for the aforementioned constituents, with the exception of total and dissolved arsenic, for which a total of four samples plus one filter blank (nine tests in total) will be adequate. The exact timing and sampling methodology is dependent on the filtrate pump station data that has yet to be obtained and analyzed. Samples should be collected from the filtrate pump station wet well or other suitable location at times when the Deskin beds are in operation. The sample bottle sizes, analyses to be performed filtration method, and need for sample preservation at each sampling location are shown in Table 6.

Table 6. Sample Container Volumes and Analyses for Recycle Stream Sample Collection

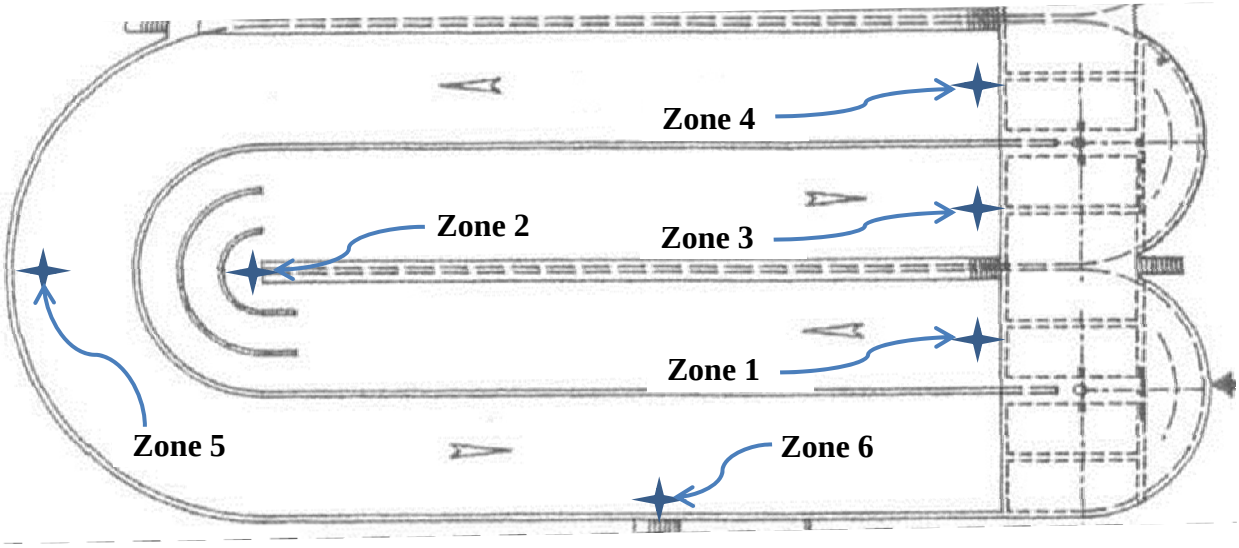
Analyses	Volume/Container	Filtration	Preservative
Ammonia; nitrate/nitrite; TKN-total; COD-total	1 liter PE	none	H_2SO_4
BOD-total; COD-soluble; TSS; VSS; alkalinity	2 liter PE	Caltest (soluble parameters only)	none
Arsenic-total	500 mL PE	none	H_2NO_3
Arsenic-dissolved	500 mL PE	Field filtered by WWTP Staff	none

5.2 Sludge Lagoon Decant

The sludge lagoon decant flows can be sampled during any of the above monitoring activities. The constituents of interest to be monitored in this recycle stream are the same as those for the Deskin bed filtrate recycle stream; thus the sample specifications in Table 5 apply to the sludge lagoon decant recycle stream as well.

Daily grab samples are considered adequate, as there is not expected to be any significant diurnal variation in the samples. As with the Deskin bed filtrate recycle stream samples, a total of 10 daily samples are recommended, to be analyzed for the aforementioned constituents, with the exception of total and dissolved arsenic, for which a total of four samples will be adequate.

Figure 1. Oxidation Ditch Monitoring Locations



APPENDIX B

WWTP Secondary Process Modeling

WEST YOST ASSOCIATES
Consulting Engineers

TECHNICAL MEMORANDUM NO. 7

DATE: January 2, 2012 Project No.: 175-00-12-36.005

TO: Mark Clarkson, City of Galt Public Works Department

FROM: Charles Hardy, R.C.E. #71015

REVIEWED BY: Kathryn Gies, R.C.E. #65022

SUBJECT: WWTP Secondary Process Modeling

INTRODUCTION

This technical memorandum (TM) presents an overview of West Yost Associates' (West Yost) secondary process computer modeling analyses completed for the City of Galt's (City's) Wastewater Treatment Plant (WWTP) Facilities Master Plan (Facilities Plan). BioWin software, which is commonly used for modeling wastewater treatment processes, was chosen as the preferred platform for the WWTP model. The modeling analyses have been used to determine the size and configuration of secondary treatment facilities needed to provide nitrogen removal and reliably meet ammonia and nitrate effluent limits in the City's Waste Discharges Requirements (WDRs) Order No. R5-2010-0099.

The modeling of the WWTP secondary process to identify recommended improvements included the following tasks: collecting influent and process data of the existing WWTP, using the data to develop a calibrated secondary process model for the WWTP, and then using the calibrated model to evaluate potential alternative facility improvements under near-term and long-term conditions. Accordingly, this TM presents the following topics:

- Special Sampling Program
- Secondary Process Model Development
- Alternatives Evaluation Design Criteria
- Secondary Process Alternatives Evaluation
- Expansion Alternatives Evaluation

SPECIAL SAMPLING PROGRAM

A special sampling program was performed at the WWTP during late April and early May 2012 to fill data gaps and identify influent and process trends for key constituents to be used in the modeling of the secondary treatment process. This section provides a discussion of the sampling program implementation and a summary of results from the daily and diurnal sampling.

DRAFT

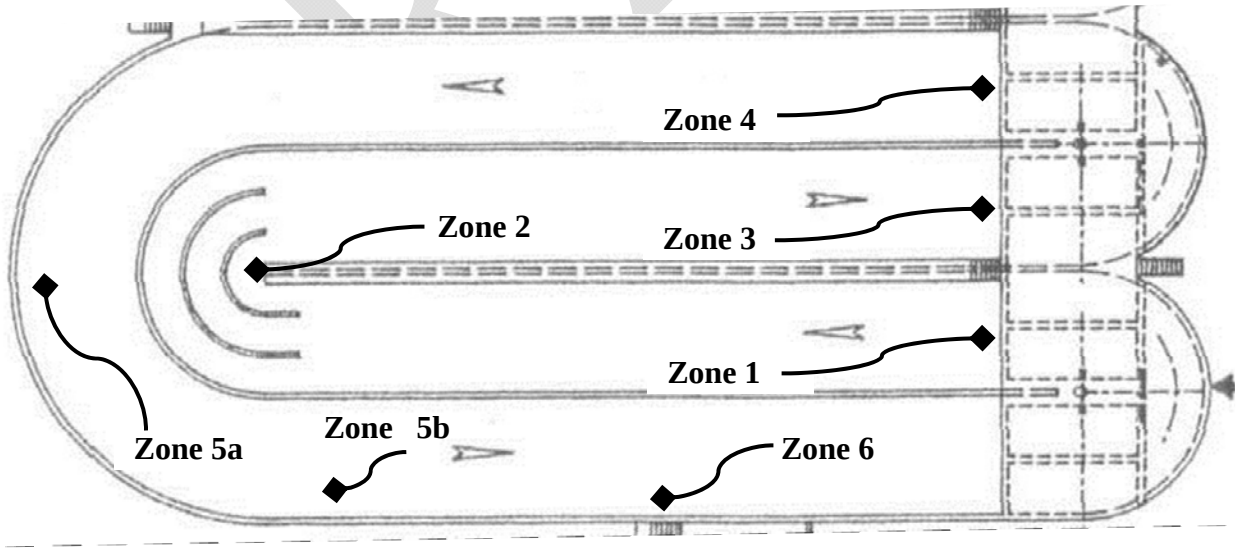
Sampling Program Implementation

West Yost prepared the April 2012 *Sampling Plan for Secondary Process Modeling: City of Galt WWTP* (Sampling Plan), which details sampling and analysis procedures for the City staff to follow in collecting and analyzing wastewater samples in support of the secondary process modeling. The Sampling Plan specified a Sampling Program with two major components: a 14-day sampling effort of daily grab and composite samples and a 48-hour diurnal sampling effort of grab samples collected every two hours over the last two days of the 14-day sampling period. Sampling and measurements were planned for only one of the two oxidation ditches since both ditches operate in parallel and are expected to have equivalent performance.

From April 23 through May 6, 2012, City staff implemented the Sampling Program, with the diurnal samples collected every two hours from 4:00 PM on May 3 to 2:00 PM on May 6. Sampling of the recycle streams (dewatering beds filtrate and sludge lagoon supernatant) was performed May 15 through 17, 2012, following the main sampling, per the Sampling Plan.

Specific sampling zones had been defined in the Sampling Plan to evaluate changes in water quality through the oxidation ditches and support calibration of the secondary process model, and the entire Sampling Program was performed according to the Sampling Plan except for changes related to dissolved oxygen (DO) measurements. Specifically, the locations for DO measurements were shifted over the sampling period from those specified in the Sampling Plan, based on results from the first few days of sampling and in an effort to better capture the oxidation ditch zone in which the DO levels appeared to be quickly dropping. As a result, Zone 5b was added during the Sampling Program between Zone 5 (now called Zone 5a) and Zone 6, as indicated in Figure 1.

Figure 1. Oxidation Ditch Monitoring Zone Sampling Locations



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The locations and dates of the DO measurements are as follows:

- Zones 1, 4, 6 (per Sampling Plan): April 23 through April 26 (3:29 PM)
- Zones 4, 5a, 6: April 26 (3:30 PM) through May 2 (10:29 AM)
- Zones 4, 5b, 6: May 2 (10:30 AM) through May 6

After moving the portable DO probe from Zone 1 to Zone 5a and then Zone 5b and not finding the location of the apparent drop in DO, it was suspected that the permanent Zone 6 DO probe was producing inaccurate DO measurements (*i.e.*, DO was not as low as indicated) and likely required recalibration. After completing the Sampling Program, a portable probe was placed in Zone 6 with the permanent probe. Comparable DO measurements were taken for both of these probes in Zone 6 from midnight on May 15 through the afternoon of May 20, with a break during the morning of May 17 when the City staff had the permanent probe recalibrated. DO measurements were significantly lower for the permanent probe prior to the recalibration and nearly identical following the recalibration, as shown in the figure in Attachment A, indicating that the permanent probe was indeed reporting inaccurately low DO levels during the Sampling Program. The Zone 6 DO results were thus not relied on during the subsequent modeling efforts.

Influent Characterization

A summary of the secondary process influent daily and diurnal sampling data is provided below.

Influent Daily Data Summary

The secondary process influent flows consist of both the wastewater entering the WWTP via the City's collection system and the WWTP's two solids processing recycled stream (Deskin drying bed filtrate and sludge lagoon supernatant). Results of the daily sampling of the influent and recycle streams – including average, maximum, and minimum concentrations; standard deviations; and number of samples for each parameter – are summarized in Table 1 (influent), Table 2 (filtrate), and Table 3 (supernatant).

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Table 1. Summary of Influent Results from Daily Sampling (April 23-May 6, 2012)

Parameter	Units	Values			Standard Deviation	Number of Samples
		Average	Maximum	Minimum		
Onsite Measurements						
Flow[a]	mgd	2.02	2.15	1.91	0.08	14
pH	s.u.	8.0	8.3	7.6	0.2	14
Temperature	°C	22.3	22.7	21.8	0.3	14
Organics						
BOD-total[b]	mg/L	206	262	161	23	13
BOD-soluble[c]	mg/L	69	84	52	8	13
COD-total	mg/L	498	720	345	80	14
COD-soluble	mg/L	142	191	102	23	14
Solids/Alkalinity						
TSS[d]	mg/L	210	294	188	28	13
VSS[e]	mg/L	183	246	160	23	13
ISS[f]	mg/L	26	48	16	9	12
Alkalinity (as $CaCO_3$)	mg/L	239	250	225	6	14
Nitrogen						
Ammonia (as N)	mg/L	25	28	24	1.1	14
Nitrate (as N)	mg/L	0.010	0.010	0.010	0.000	14
Nitrite (as N)	mg/L	0.006	0.012	0.002	0.003	14
TKN-total	mg/L	36	38	35	1.3	14
TKN-soluble[g]	mg/L	27	29	25	1.1	13
Phosphorus						
Phosphorous-total	mg/L	4.5	6.1	3.4	0.74	14
Phosphorous-soluble	mg/L	2.7	3.1	2.4	0.19	14

(a) Flow data were recorded at five-minute intervals, but values presented here are for daily averages of that data.
(b) Uncharacteristically low BOD-total result of 60 mg/L for the 5/3/12 sample was removed from the data set.
(c) Uncharacteristically low BOD-soluble result of non-detect (ND) with a detection limit of 5 mg/L for the 5/3/12 sample was removed from the data set,
(d) Uncharacteristically low TSS result of 172 mg/L for the 4/30/12 sample was removed from the data set. The reported value is more consistent with VSS results (corresponding 4/30/12 VSS result is 170 mg/L).
(e) Uncharacteristically low VSS result of 60 mg/L for the 4/23/12 sample was removed from the data set.
(f) ISS data have been calculated as the difference between paired TSS and VSS data.
(g) TKN-soluble analysis was inadvertently not requested for the first day of sampling (4/23/12).

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Table 2. Summary of Filtrate Results from Daily Sampling (May 15-17, 2012)

Parameter	Units	Values			Standard Deviation	Number of Samples
		Average	Maximum	Minimum		
Organics						
BOD-total	mg/L	125	258	7	89	9
BOD-soluble[a]	mg/L	92				
COD-total	mg/L	230	414	13	151	9
COD-soluble	mg/L	170	263	24	91	9
Solids/Alkalinity						
TSS	mg/L	53	176	1	54	9
VSS	mg/L	36	98	1	31	9
ISS[b]	mg/L	21	78	0	27	7
Alkalinity (as $CaCO_3$)	mg/L	449	857	38	320	9
Nitrogen						
Ammonia (as N)	mg/L	74	160	3	56	9
Nitrate (as N)	mg/L	63	110	27	32	9
Nitrite (as N)	mg/L	2.9	5.2	0.4	1.3	9
TKN-total	mg/L	79	160	3	59	9
TKN-soluble[a]	mg/L	58				
Arsenic						
Arsenic-total	mg/L	47	66	18	17	9
Arsenic-soluble	mg/L	41	55	18	14	8[c]

(a) TKN-soluble and BOD-soluble levels were not measured directly. Average levels for these have been estimated based on the average TKN-total and BOD-total levels, respectively, multiplied by the ratio of average COD-soluble and COD-total levels.

(b) ISS data have been calculated as the difference between paired TSS and VSS data. The first two sets of data (5/15/12 10:10 AM and 11:10 AM had VSS values slightly greater than TSS values, so ISS values have not been calculated with those data.)

(c) Results for arsenic-soluble in the filtrate were not reported by the analytical laboratory for the 5/15/12 11:10 AM sample.

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Table 3. Summary of Supernatant Results from Daily Sampling (May 15-17, 2012)

Parameter	Units	Values			Standard Deviation	Number of Samples
		Average	Maximum	Minimum		
Organics						
BOD-total	mg/L	314	388	214	52	9
BOD-soluble[a]	mg/L	277				
COD-total	mg/L	467	524	375	48	9
COD-soluble	mg/L	413	550	282	83	9
Solids/Alkalinity						
TSS	mg/L	80	95	66	8	9
VSS	mg/L	51	64	36	9	9
ISS[b]	mg/L	30	44	16	8	9
Alkalinity (as $CaCO_3$)	mg/L	798	843	749	39	9
Nitrogen						
Ammonia (as N)	mg/L	147	160	130	12	9
Nitrate (as N)	mg/L	0.01	0.01	0.01	0.00	9
Nitrite (as N)	mg/L	0.002	0.002	0.002	0.000	9
TKN-total	mg/L	156	170	140	11	9
TKN-soluble[a]	mg/L	137				
Arsenic						
Arsenic-total	mg/L	26	29	23	2	9
Arsenic-soluble	mg/L	22	24	21	1	9

(a) TKN-soluble and BOD-soluble levels were not measured directly. Average levels for these have been estimated based on the average TKN-total and BOD-total levels, respectively, multiplied by the ratio of average COD-soluble and COD-total levels.
(b) ISS data have been calculated as the difference between paired TSS and VSS data.

Influent Diurnal Data Summary

The diurnal variability in the WWTP influent flow was also characterized for key constituents. The results of each two-hour influent sample are presented graphically in Attachment B for all of the analyzed influent constituents – flow (Figure B-1) ammonia and TKN (Figure B-2), and COD (Figures B-3).

WWTP Process Characterization

A summary of the secondary process daily and diurnal sampling data is provided below.

Process Daily Data Summary

The secondary process monitoring consisted of collecting samples for key parameters at the following locations: oxidation ditch zones 2, 3, 4, and 5; the secondary effluent, and RAS flows. Results of the daily sampling at these locations – including average, maximum, and minimum concentrations; standard deviations; and number of samples for each parameter – are summarized in Table 4.

In addition to the daily data collected during the special monitoring program presented in Table 4, City staff also provided West Yost with the following requested daily process data for all of 2011 (The full data is for June 17, 2010 through March 2, 2012.):

- Influent Temperature
- Oxidation ditch Mixed Liquor Suspended Solids (MLSS), Mixed Liquor Volatile Suspended Solids (MLVSS), and ammonia
- Secondary clarifier effluent TSS
- Return Activate Sludge (RAS) flow, TSS, and VSS
- Waste Activate Sludge (WAS) flow, TSS, and VSS

These data were summarized and used, as needed, in the model development and calibration as discussed in other sections of this TM.

Process Diurnal Data Summary

The diurnal variability in the WWTP process data was also characterized for key constituents. The results of each two-hour influent sample are presented graphically in Attachment B for all of the analyzed process constituents – DO (Figure B-4), ammonia (Figure B-5), nitrate (Figure B-6), nitrite (Figure B-7), TKN (Figure B-8), and COD (Figure B-9).

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Table 4. Summary of Process Results from Daily Sampling (April 23-May 6, 2012)

Parameter	Units	Values			Standard Deviation	Number of Samples
		Average	Maximum	Minimum		
Secondary Effluent						
Ammonia (as N)	mg/L as N	0.32	0.60	0.11	0.14	14
TKN-soluble	mg/L	0.09	0.56	0.00	0.14	14
Nitrate (as N)	mg/L as N	19.4	21.0	17.0	1.1	14
Nitrite (as N)	mg/L	0.06	0.16	0.03	0.03	14
TSS	mg/L	1.9	4.0	1.0	0.9	14
VSS	mg/L	1.9	4.0	1.0	1.0	12
S-BOD	mg/L	5.1	6.0	5.0	0.3	14
T-COD	mg/L	15.9	25.0	13.0	4.0	14
S-COD	mg/L	15.9	22.0	13.0	3.6	14
pH	s.u.	7.3	7.7	7.0	0.2	12
Temperature	°C	21.8	22.5	21.2	0.5	12
Oxidation Ditch Zone 2						
TSS	mg/L	2,860	3,280	1,450	465	12
VSS	mg/L	2,308	2,660	1,150	384	12
Ammonia (as N)	mg/L as N	0.24	0.57	0.08	0.14	11
TKN-total	mg/L	187	220	150	21	12
Nitrate (as N)	mg/L as N	18.2	21.0	15.0	1.7	11
Nitrite (as N)	mg/L	0.32	0.55	0.17	0.10	11
T-COD	mg/L	3,398	5,520	2,750	717	12
Oxidation Ditch Zone 3						
Ammonia (as N)	mg/L as N	0.15	0.32	0.04	0.08	11
Nitrate (as N)	mg/L as N	18.7	21.0	16.0	1.6	11
Nitrite (as N)	mg/L	0.32	0.62	0.16	0.12	11
Oxidation Ditch Zone 4						
Ammonia (as N)	mg/L as N	0.09	0.11	0.04	0.02	11
Nitrate (as N)	mg/L as N	18.5	21.0	16.0	1.6	11
Nitrite (as N)	mg/L	0.29	0.55	0.15	0.12	11
Oxidation Ditch Zone 5						
Ammonia (as N)	mg/L as N	0.08	0.10	0.04	0.02	11
Nitrate (as N)	mg/L as N	18.7	21.0	16.0	1.5	11
Nitrite (as N)	mg/L	0.26	0.42	0.18	0.08	11
RAS						
TSS	mg/L	11,797	14,000	7,380	2,140	13
VSS	mg/L	9,492	11,100	6,790	1,425	13

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BioWin Wastewater Fractions

The characterization data was used to develop influent water quality fraction for the BioWin model. Default values in BioWin can be used for many of these parameters, but several parameters are site specific and should be estimated from actual process data. The process specific parameters used for the BioWin analysis are summarized in Table 5. For nine additional influent wastewater fractions (not shown in Table 5) – all of which are related to masses of different types of organisms commonly found in wastewater treatment facilities relative to total COD mass – process-specific information was not collected, Therefore, the default values ($1.00*10^{-4}$ g/g) have been used.

Table 5. Summary of Wastewater Fractions (g/g) for Influent and Recycle Streams

WW Fraction	Description (Ratios)	Default Values[a],[b]		Selected Values[b]			Basis[c]
		Raw	Settled	Influent	Filtrate	Sludge Lagoon	
F_{bs}	Readily biodegradable COD, including acetate, to total COD	0.160	0.270	0.172	0.338	0.352	Data
F_{ac}	Volatile acid (acetate) to readily biodegradable COD	0.150	0.150	0.150	0.150	0.150	Default
F_{xsp}	Particulate (non-colloidal) COD to slowly biodegradable COD	0.750	0.500	0.840	0.350	0.065	Data
F_{us}	Unbiodegradable, soluble COD to total COD	0.050	0.080	0.029	0.062	0.031	Data
F_{up}	Unbiodegradable, particulate COD to total COD	0.130	0.080	0.270	0.081	0.080	Data
F_{na}	Ammonia to TKN	0.660	0.750	0.694	0.943	0.943	Data
F_{nox}	Particulate organic nitrogen to biodegradable organic nitrogen	0.500	0.250	0.500	0.250	0.250	Default
F_{nus}	Unbiodegradable, soluble TKN to total TKN	0.020	0.020	0.020	0.020	0.020	Default
F_{upN}	Nitrogen to unbiodegradable particulate COD	0.035	0.035	0.035	0.035	0.035	Default
F_{po4}	Phosphate to total phosphorus	0.500	0.750	0.500	0.750	0.750	Default
F_{upP}	Phosphorus to unbiodegradable particulate COD	0.011	0.011	0.011	0.011	0.011	Default

(a) Default values are included in the BioWin model and Influent Specifier (IS) tools for both raw and settled wastewater. Default values for raw wastewater have been selected for the influent, and default values for settled wastewater have been selected for the filtrate and supernatant (sprntnt.), when no better information is available (*i.e.*, main parameter in fraction was not analyzed for).

(b) All values are in units of mass of parameter in the numerator per mass of parameter in the denominator (generally, grams of COD).

(c) Some of the fractions are internally estimated in the IS tool based on values input by the user. F_{xsp}, and F_{up} can be modified in the IS tool by the user until appropriate influent levels of typical parameters (e.g., TSS, BOD, COD) are estimated.

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SECONDARY PROCESS MODEL DEVELOPMENT

This section provides a discussion of the development and calibration process for steady-state and dynamic secondary process models of the existing WWTP.

Steady State Model

A steady-state model of one train of the WWTP secondary process was developed. The unit process design parameters (*e.g.*, size, settling rates) were inserted into the model based on the WWTP design plans, with the oxidation ditch modeled as eleven, equally sized units – similar to the Sampling Program zones – two with aeration and nine without.

In addition to the wastewater fractions discussed above, the BioWin model requires input of some wastewater influent water quality parameters. The steady-state influent parameters assumed for the calibration of the model were based on the average concentration measured during the Sampling Program (see Table 1), and are presented in Table 6. Note that calcium and magnesium are two required BioWin parameters not measured as part of the Sampling Program. Given that no better information was available, influent loadings were assumed to be equivalent to typical domestic wastewater concentrations (80 and 15 mg/L, respectively).

Table 6. BioWin Model Steady-State WWTP Influent Concentrations (Current Conditions)

Parameter	Units	Input Value
Flow	mgd	1.01
COD-total	mg/L	498
TKN-total	mg/L	36
Phosphorus-total	mg/L	4.5
Nitrate (as N)	Mg/L	0
pH	—	7.3
Alkalinity (as $CaCO_3$)	mg/L	239
ISS	mg/L	26
Calcium	mg/L	80
Magnesium	mg/L	15
Dissolved Oxygen	mg/L	0.0

The main model parameters adjusted to calibrate the steady-state model were the WAS rate and the horsepower (HP) provided by each aeration unit. First, the WAS rate was adjusted to achieve the observed MLSS level in the oxidation ditches. Second, the oxygen input parameters were adjusted. During the sampling period, the City was operating both of the oxidation ditch aeration units on the "high" setting (*i.e.*, rated at 60 HP). According to the equipment manufacturer, the standard oxygen transfer rate (SOTR) for the equipment is 3.5 pounds of oxygen per HP-hour. However, the SOTR will vary depending on the submergence level of the aeration equipment. In addition, wear and tear on the aeration equipment could affect performance. To simplify the

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analysis, the SOTR was held constant at 3.5 pounds of oxygen per HP-hour and the HP provided was adjusted until the model predicted the observed performance. The resulting adjusted input value was determined to be 58 HP.

The calibrated model predicts effluent ammonia and nitrate levels very close to the observed level, as indicated in Table 7.

Table 7. Comparison of Observed Values and Predicted Secondary Process Model Results under Existing Conditions

Source	Influent Conditions			Oxidation Ditch Conditions[a]	Effluent Concentrations[b]		
	Flow Condition	Flow mgd	Temp. °C	MLSS mg/L	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L
Observed[b]	Current	2.0	22.3	2,997	0.32	19.36	< 5.1[c]
Model				2,861	0.19	19.46	1.08

(a) Observed oxidation ditch MLSS and MCRT are average values from the City's records for June 17, 2010-March 2, 2012, with MCRT outliers of less than 10 days or above 100 days removed to give an average representative of normal operating conditions.
(b) Observed effluent concentrations are the average secondary effluent values from the April 23-May 6, 2012 Sampling Program.
(c) Total BOD was not measured as part of the Sampling Program, but soluble BOD was measured, so the average value for soluble BOD is presented. Also, only 1 of the 14 results (6 mg/L) was reported above the method detection limit (MDL) of 5 mg/L, and the average is estimated based on all other (non-detect) results set equal to the MDL.

Dynamic Model

Once the steady-state model was determined to be adequately predicting the observed average performance of the WWTP, diurnal influent data collected during the Sampling Program was added to the calibrated steady-state model to complete a dynamic evaluation. The purpose of this evaluation was to verify that, given the model configuration and influent wastewater characteristics described, the model accurately predicts performance as observed by the diurnal data.

Measured diurnal influent parameters have been described above and are presented graphically in Attachment B. In addition, the following influent parameters are assumed to be equal to their steady-state values (in mg/L) (presented in Table 6) under all diurnal conditions:

- Nitrate
- Alkalinity
- pH
- Calcium
- Magnesium
- DO

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The base model was run under dynamic conditions to evaluate whether the model predicted operating conditions (DO) and effluent conditions (COD, ammonia, nitrate, nitrite, TKN) similar to those observed for the diurnal component of the Sampling Program. The results of this effort are provided in Attachment C. As shown, the dynamic model adequately predicted DO levels in the oxidation ditch and adequately predicted varying effluent levels for ammonia, nitrate, and nitrite. For COD, the dynamic model results in a relatively constant effluent COD, similar to the average of the observed COD effluent levels but not capturing the observed diurnal variation in effluent COD. In addition, the dynamic model appears to significantly overpredict effluent TKN, although the reported effluent TKN values are atypically low, similar to the average value (0.09 mg/L) observed for soluble TKN from the daily sampling of secondary effluent.

Overall, the model results agree well enough with the observed conditions for there to be confidence in using the model configuration and wastewater characteristics data for modeling of alternatives.

ALTERNATIVES EVALUATION DESIGN CRITERIA

This section provides a discussion of the criteria used in the modeling evaluation of the various WWTP secondary process alternatives. Process modeling of the WWTP alternatives requires defining the model inputs and desired outputs. The following specific topics are addressed below:

- Effluent Water Quality Requirements Operational Parameters
- Design Load Conditions
- Design Concentrations

Effluent Water Quality Requirements and Operational Parameters

Regulatory effluent limitations are the primary drivers for the design of improvements to the WWTP and its subsequent operation. The WDRs, adopted by the Central Valley Regional Water Quality Control Board (Regional Board) on September 23, 2010, establish final effluent limitations for a variety of constituents, including the nitrogen species “ammonia nitrogen” and “nitrate plus nitrite”; the effluent limitations for nitrogen compounds are the primary focus of the current planning and design efforts for the WWTP.

Setting target effluent levels below the effluent limitations is a prudent measure to allow for the expected variation of influent flow and load conditions described above, which can impact effluent conditions. Target effluent levels for nitrogen species have thus been established to ensure that the regulatory requirements can be reliably met under a variety of influent flow and load conditions, in particular expected worst-case conditions. Since both average monthly and maximum daily effluent limitations are specified in the WDRs, target effluent levels have been established for both average monthly and maximum daily conditions. The effluent limitations relevant to the current analysis, as well as the target effluent levels, are presented in Table 8.

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Table 8. Effluent Limitations from Waste Discharge Requirements (WDRs) Order No. R5-2010-0099 and Target Effluent Levels for Nitrogen Compounds

Constituent	Units	Effluent Limitation			Target Effluent Level	
		Average Monthly	Average Weekly	Maximum Daily	Average Monthly	Maximum Daily
Ammonia Nitrogen, Total (as N)	mg/L	1.7	—	3.3	< 0.5	< 1
Nitrate plus Nitrite, Total (as N)	mg/L	10	—	—	< 8	< 10
Biochemical Oxygen Demand (5-day @ 20°C)	mg/L	10	15	20	< 3	< 5

(a) Mass effluent limitations (in lb/day) are based on a design average dry weather flow of 4.5 million gallons per day. Target mass effluent levels have also been estimated based on this flow rate.
(b) The WDRs inadvertently list the units for the arsenic effluent limitation as "mg/L", but the correct units are "µg/L".
(c) Effluent limitations for pH are for a range (instantaneous minimum of 6.5 and instantaneous maximum of 8.2).

The desired WWTP operational parameters are summarized in Table 9Table .

Table 9. Desired Secondary Process Operational Parameters

Existing WWTP Parameter	Value	Basis
Return Activated Sludge (RAS) Flow	Paced at 75 percent of influent flow minus WAS flow	Average calculated value, January 2011 to March 2012
Waste Activated Sludge (WAS) Flow (per treatment train)	Varies	Assume a constant flow rate. Flow value is set as needed to maintain desired MLSS concentrations
MLSS Concentration	3,000 mg/L 3,500 mg/L	Assumed operating range for maximum MLSS conditions
(Aerated) Mean Cell Residence Time (MCRT)	25 days	Set to meet Class B pathogen reduction requirements, allowing land application of biosolids

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Design Load Conditions

The following discussion presents the influent steady-state design loads for the parameters measured during the Sampling Program and other input parameters required by the BioWin model. Also presented are the diurnal loads for the parameters required by the BioWin model.

The influent load design values were developed based on the data collected under the Sampling Program and the projected load information provided in Chapter 2 (Basis of Design) of the Facilities Master Plan. Moreover, design loads are presented for each of the three design phases being considered in the Facilities Master Plan (Current Permitted Design Condition, Future Permitted Design Condition, and Ultimate Buildout Design Condition).

Steady-State Influent Loads

Steady-state conditions influent design loads have been estimated as follows:

- For parameters measured or estimated as part of the Sampling Program (nitrate, COD, TSS, VSS, ISS, and total phosphorus), average loadings were directly determined from the average observed concentrations (presented in Table 1) Table 1.and the design condition ADWF (*i.e.*, 3.0 mgd, 4.5 mgd and 6.0 mgd). For maximum month and maximum day conditions, the peaking factors of 1.3 and 1.6, respectively, were applied to the calculated average loads.
- Alkalinity, pH, calcium and magnesium are not likely to vary proportionally with the other flow and load parameters and were thus assumed to be the same for all load conditions. (Average calcium and magnesium concentrations were assumed to be equivalent to typical domestic wastewater concentrations (80 and 15 mg/L, respectively)).
- Influent DO was also not measured but has been assumed to be negligible (*i.e.*, 0 ppd), as is typical for domestic wastewater.

The resulting influent loadings are summarized for each of the flow conditions of interest – Current Permitted Design Condition (3.0 mgd), Future Permitted Design Condition (4.5 mgd), and Ultimate Buildout Design Condition (6.0 mgd) – in Table 10, Table 11, and Table 12, respectively.

DRAFT

Table 10. Influent Load Conditions for Secondary Process Design
Current Permitted Design Condition (3.0 mgd ADWF)

Parameter	Units	Load Condition		
		ADL[a]	MML[b]	MDL[b]
Organics				
BOD-total	ppd	5,100	6,700	8,200
BOD-soluble	ppd	1,700	2,200	2,700
COD-total	ppd	12,500	16,200	20,000
COD-soluble	ppd	3,500	4,600	5,700
Solids/Alkalinity				
TSS	ppd	5,200	6,800	8,400
VSS	ppd	4,600	6,000	7,300
ISS	ppd	700	800	1,000
Alkalinity (as CaCO3)	ppd	6,000		
pH	s.u.	7.3		
Nitrogen				
Ammonia as N	ppd	600	800	1,000
Nitrate as N	ppd	0.0	0.0	0.0
TKN-total	ppd	900	1,200	1,500
TKN-soluble	ppd	700	900	1,100
Phosphorus				
Phosphorus-total	ppd	100	100	200
Phosphorus-soluble	ppd	100	100	100
Other Required Parameters				
Calcium	ppd	2,000		
Magnesium	ppd	400		
Dissolved Oxygen	ppd	0.0		

[a] ADL values have been estimated based on average influent data from the Sampling Program (Table 1) and an ADWF of 3.0 mgd.
[b] MML and MDL values are established based on ADL values multiplied by the appropriate peaking factor (1.3 for MML and 1.6 for MDL).

DRAFT

Table 11. Influent Load Conditions for Secondary Process Design
Future Permitted Design Condition (4.5 mgd ADWF)

Parameter	Units	Load Condition		
		ADL[a]	MML[b]	MDL[b]
Organics				
BOD-total	ppd	7,700	10,000	12,300
BOD-soluble	ppd	2,600	3,300	4,100
COD-total	ppd	18,700	24,300	29,900
COD-soluble	ppd	5,300	6,900	8,500
Solids/Alkalinity				
TSS	ppd	7,900	10,200	12,600
VSS	ppd	6,900	8,900	11,000
ISS	ppd	1,000	1,300	1,600
Alkalinity (as CaCO3)	ppd	9,000		
pH	s.u.	7.3		
Nitrogen				
Ammonia as N	ppd	900	1,200	1,500
Nitrate as N	ppd	0.0	0.0	0.0
TKN-total	ppd	1,400	1,800	2,200
TKN-soluble	ppd	1,000	1,300	1,600
Phosphorus				
Phosphorus-total	ppd	200	200	300
Phosphorus-soluble	ppd	100	100	200
Other Required Parameters				
Calcium	ppd	3,000		
Magnesium	ppd	600		
Dissolved Oxygen	ppd	0.0		

(a) ADL values have been estimated based on Sampling Program data for the influent.
(b) MML and MDL values are established based on ADL values multiplied by the appropriate peaking factor (1.3 for MML and 1.6 for MDL).

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Table 12. Influent Load Conditions for Secondary Process Design
Ultimate Buildout Design Condition (6.0 mgd ADWF)

Parameter	Units	Load Condition		
		ADL[a]	MML[b]	MDL[b]
Organics				
BOD-total	ppd	10,300	13,400	16,500
BOD-soluble	ppd	3,400	4,500	5,500
COD-total	ppd	24,900	32,400	39,900
COD-soluble	ppd	7,100	9,200	11,300
Solids/Alkalinity				
TSS	ppd	10,500	13,600	16,800
VSS	ppd	9,200	11,900	14,700
ISS	ppd	1,300	1,700	2,100
Alkalinity (as CaCO3)	ppd	12,000		
pH	s.u.	7.3		
Nitrogen				
Ammonia as N	ppd	1,300	1,600	2,000
Nitrate as N	ppd	0.0	0.0	0.0
TKN-total	ppd	1,800	2,400	2,900
TKN-soluble	ppd	1,300	1,700	2,100
Phosphorus				
Phosphorus-total	ppd	200	300	400
Phosphorus-soluble	ppd	100	200	200
Other Required Parameters				
Calcium	ppd	4,000		
Magnesium	ppd	800		
Dissolved Oxygen	ppd	0.0		

(a) ADL values have been estimated based on Sampling Program data for the influent.
(b) MML and MDL values are established based on ADL values multiplied by the appropriate peaking factor (1.3 for MML and 1.6 for MDL).

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Diurnal Influent Loads

The diurnal sampling data collected during the Sampling Program was used to define the design loads for the secondary process dynamic modeling. First, two-hour average load values were calculated from the two-hour interval flow and concentration data collected under the Sampling Program between May 3 at 4 PM and Saturday, May 5 at 4 PM. Second, the flow and load values were used to establish diurnal peaking factors for each two-hour period, as shown in Table 13. Finally, the peaking factors were applied to the steady-state load values (shown in Table Table 10, 11 and 12) to calculate the diurnal load values, as follows:

- Load factors for T-COD were used to calculate diurnal design loads for T-COD, TSS and VSS. (ISS was calculated as the difference in the two-hour average TSS and VSS load value.)
- Load factors for T-TKN were used to calculate diurnal design loads for T-TKN and T-phosphorus.
- Load factors for T-NH3 were used to calculate diurnal design loads for T-NH3.
- No diurnal variation was assumed for pH, calcium, magnesium, alkalinity, nitrate-N, and DO.

The resulting daily average, maximum month and maximum diurnal loads for the three design conditions are presented in Attachment D.

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Table 13. Diurnal Variation Flow and Load Peaking Factors

Part of Week	Time Interval, hr	Ratio of Hourly Average to Overall Average			
		Flow	T-COD	T-TKN	T-NH3
Weekdays	4:00 PM	1.00	1.19	0.90	0.86
	6:00 PM	1.28	1.32	1.25	1.19
	8:00 PM	1.00	1.09	0.92	0.89
	10:00 PM	1.17	1.20	1.11	0.96
	12:00 AM	0.65	0.74	0.62	0.58
	2:00 AM	0.42	0.35	0.38	0.36
	4:00 AM	0.49	0.34	0.40	0.42
	6:00 AM	0.92	0.42	0.68	0.76
	8:00 AM	1.28	0.95	1.79	1.90
	10:00 AM	1.17	1.28	1.79	1.87
	12:00 PM	1.08	1.11	1.08	1.16
	2:00 PM	1.07	1.06	0.91	0.88
Weekends	4:00 PM	1.09	0.96	0.86	0.81
	6:00 PM	1.14	1.06	1.02	1.02
	8:00 PM	1.07	1.16	1.01	0.95
	10:00 PM	1.16	1.22	1.13	1.12
	12:00 AM	0.81	0.80	0.75	0.69
	2:00 AM	0.58	0.62	0.52	0.54
	4:00 AM	0.47	0.31	0.34	0.37
	6:00 AM	0.61	0.27	0.40	0.47
	8:00 AM	1.21	0.59	1.11	1.30
	10:00 AM	1.59	2.71	2.60	2.67
	12:00 PM	1.45	1.96	1.41	1.29
	2:00 PM	1.28	1.28	1.01	0.90

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Design Concentrations

The BioWin software requires input of flow and water quality concentrations (rather than mass loadings). Therefore, this section presents the concentration information used for the modeling as follows:

- Influent steady-state design concentrations for all the parameters measured during the Sampling Program and other input parameters required by the BioWin model
- Solids return streams steady-state flow and concentration values required by the BioWin model
- Diurnal influent concentrations for the parameters required by the BioWin model.

Note that, although the flows increase, the concentration design values are expected to be the same for each of the three design phases being considered in the Facilities Master Plan (Current Permitted Design Condition, Future Permitted Design Condition, and Ultimate Buildout Design Condition).

Steady-State Influent Concentrations

Various combinations of flow, load, and temperature conditions have been analyzed through the modeling effort. The eleven combinations that were analyzed are summarized in Table 14. Influent concentrations that apply to each scenario described in Table 14 are shown in Table 15.

Table 14. Secondary Process Modeling Influent Flow and Load Combinations

Flow Condition[b]	Load Condition[a],[b]			
	Average Day Load (ADL)	Maximum Month Load (MML) [= 1.3 x ADL]	MML with Recycle Streams[c]	Maximum Day Load (MDL) = [1.6 x ADL]
Average Dry Weather Flow (ADWF)	21.6°C and 17.0°C	21.6°C and 17.0°C	17.0°C	
Average Day Maximum Month Flow (ADMMF) [= 1.3 x ADWF]		21.6°C and 17.0°C		21.6°C/14.5°C
Maximum Day Dry Weather Flow (MDDWF) [= 1.7 x ADWF]				21.6°C/14.5°C

(a) Values shown are temperature values used in the analyzed scenarios. Temperatures shown represent the following based on the City's June 2010 through March 2012 (weekly) influent temperature data: Average temperature = 21.6°C; Lowest monthly average temperature = 17.0°C; Lowest daily temperature = 14.5°C.
(b) Flow and load conditions are based on the information presented in West Yost's October 1, 2012 *City of Galt WWTP Flow and Load Projection* Technical Memorandum.
(c) The City can operate the biosolids dewatering facility and sludge lagoon such that recycle streams have minimal or no flow during MDL conditions, but recycle streams are likely to have significant flows during a month with MML, so the impact of the recycle streams under MML conditions needs to be evaluated.

DRAFT

Table 15. Secondary Process Modeling Raw Wastewater Influent Concentrations

Parameter	Units	Value by Scenario				
Flow Condition>>		ADWF		ADMMF		MDDWF
Load Condition>>		ADL	MML	MML	MDL	MDL
Organics						
BOD-total	mg/L	206	267	206	253	193
BOD-soluble	mg/L	69	89	69	84	65
COD-total	mg/L	498	648	498	613	469
COD-soluble	mg/L	142	184	142	174	133
Solids/Alkalinity						
TSS	mg/L	210	273	210	258	197
VSS	mg/L	183	238	183	226	172
ISS	mg/L	26	34	26	32	25
Alkalinity (as $CaCO_3$)[(a)]	mg/L	239				
pH[(a)]	—	7.3				
Nitrogen						
Ammonia as N	mg/L	25	33	25	31	24
Nitrate as N	mg/L	0.01	0.013	0.010	0.012	0.009
TKN-total	mg/L	36	47	36	45	34
TKN-soluble	mg/L	27	35	27	33	25
Phosphorus						
Phosphorus-total	mg/L	4.5	5.8	4.5	5.5	4.2
Phosphorus-soluble	mg/L	2.7	3.6	2.7	3.4	2.6
Other Required Parameters						
Calcium[(a)]	mg/L	80				
Magnesium[(a)]	mg/L	15				
Dissolved Oxygen[(a)]	mg/L	0.0				

[(a)] The following parameters were assumed to be the same across all scenarios: alkalinity, pH, calcium, magnesium, and dissolved oxygen. The values for alkalinity and pH are based on measured values from the Sampling Program. Values for calcium, magnesium, and dissolved oxygen are assumed based on typical domestic wastewater values.

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Steady-State Solids Return Flows and Concentrations

In addition to the influent flows, some of the secondary process modeling scenarios include the high strength (but low flow) return (recycle) flows from the solids handling processes (see Table 14Table). For purposes of the WWTP Facilities Master Plan, solids handling return flows were conservatively included in the steady-state modeling analysis presented herein, as follows:

- **Solids Return Concentrations:** If measured, the average observed concentrations from Table 2 and Table 3 were used. For parameters not directly measured, average influent concentrations were assumed to be representative of the solids return flows.
- **Sludge Lagoon Supernatant Flows:** Supernatant flow is estimated based on the annual average solids production rate and an assumed concentration of solids from 0.8 percent to 2.5 percent during the settling process. The resulting value of 0.0408 mgd was used for the Current Permitted Design Conditions condition and divided evenly per oxidation ditch modeled. For other design conditions, this flow was adjusted proportional to the influent ADWF increase above 3.0 mgd.
- **Dewatering System Filtrate Flows:** The volume and timing for filtrate return to the treatment process will vary depending on the selected solids processing alternative. For purposes of developing the preliminary steady-state modeling presented in the Facilities Master Plan, the peak flow was conservatively estimated as 0.374 mgd (which includes "clean water" used in the Deskin Beds). This value is based on one pump at a flow of 260 gallons per minute and represents a worst-case peak flow for the Current Permitted Design Condition (3.0 mgd), and was adjusted for the other design conditions proportional to the influent ADWF increase above 3.0 mgd.

A summary of the BioWin input parameters for the solids return flows are shown in Table 16.

The WWTP Facilities Master Plan addresses modifications to the solids handling processes that will affect the volume and strength of the return flows. In addition, improvements needed to allow the City to use the Auxiliary Storage Basin to equalize solids return flows, thus allowing for return of high strength loads during off-peak periods, are recommended in the Facilities Master Plan. Therefore, a more detailed dynamic modeling analysis will need to be performed during future design phases that accounts for the solids handling and equalization approach identified for the selected expansion alternative.

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Table 16. Secondary Process Modeling Recycle Stream Flows and Concentrations

Parameter	Unit	Filtrate Return	Supernatant
Flow[(a)]	mgd	0.374 / 0.561 / 0.748	0.0408 / 0.0612 / 0.0816
COD-total	mg/L	230	467
TKN-total	mg/L	79	156
Phosphorus – Total[(b)]	Mg/L	4.49	
Nitrate (as N)	mg/L	63	0.01
pH[(b)]	s.u.	7.3	
Alkalinity (as $CaCO_3$)	mg/L	449	798
ISS	mg/L	16	30
Calcium[(b)]	mg/L	80	
Magnesium[(b)]	mg/L	15	
Dissolved Oxygen[(b)]	mg/L	0.0	

(a) Values shown for Current Permitted Design Condition / Future Permitted Design Condition / Ultimate Buildout Design Condition
(b) Values are assumed to be equal to influent concentrations.

Diurnal Concentrations

Diurnal concentrations for the parameters required by BioWin were determined for following four conditions:

- ADWF combined with ADL (Table E-1)
- ADWF combined with MML (Table E-2)
- ADMMF combined with MML (Table E-3)
- ADMMF combined with MDL (Table E-4)
- MDDWF combined with MDLs (Table E-5)

The calculated diurnal concentration characteristics are provided in Attachment E. Finally, the following 5-day "worst-case" diurnal pattern was assumed for the modeling effort (temperature of 14.5°C):

- First two days at ADMMF combined with MML (Table E-2)
- The next two days at ADMMF combined with MDL (Table E-3)
- A final day at ADMMF combined with MML (Table E-2)

Other combinations of diurnal flow and load conditions may be considered at later design phases.

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SECONDARY PROCESS ALTERNATIVES EVALUATION

This section provides a discussion of alternatives for meeting the treatment needs at the Current Permitted Design Condition. Key topics presented include the following:

- Existing Facilities
- Upgrade Alternatives Evaluation
- Expansion Alternatives Evaluation

Existing Facilities

The calibrated steady-state model of the existing secondary process was used to evaluate treatment performance under the range of influent and solids return stream flow and loads detailed in Table 13. All of the scenarios were evaluated assuming the aeration system can be maximized at 60 HP. In addition, both target maximum MLSS values (3,000 mg/L and 3,500 mg/L, as described in Table 9) were considered by adjusting the WAS flow until the target MLSS values were achieved. (Target MCRT values were ignored for this exercise.) Predicted oxidation ditch conditions and effluent concentrations under each of the scenarios for these two MLSS values are presented, respectively, in Table 17 and Table 18.

Table 17. Secondary Process Model Results at Target MLSS Value of 3,000 mg/L

Influent Conditions					Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations		
Load Condition	Load Factor	Flow Condition	Flow mgd	Temp. °C	MLSS mg/L	Aerated MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L
ADL	1.0	ADWF	3.0	21.6	2,980	19	0.34	0.090	1.3
ADL	1.0	ADWF	3.0	17.0	2,970	19	0.42	9.4	1.2
MML	1.3 x ADL	ADWF	3.0	21.6	3,060	14	1.1	[a]	2.0
MML	1.3 x ADL	ADWF	3.0	17.0	3,050	14	1.3	[a]	1.8
MML	1.3 x ADL + Recycle Streams	ADWF	3.0	17.0	3,070	13	4.1	0.010	1.9
MML	1.3 x ADL	ADMMF	3.9	21.6	2,930	14	1.2	[a]	2.0
MML	1.3 x ADL	ADMMF	3.9	17.0	2,960	14	1.3	[a]	1.8
MML	1.3 x ADL	ADMMF	3.9	21.6	3,000	11	5.8	[a]	2.6
MDL	1.6 x ADL	ADMMF	3.9	14.5	3,034	12	7.1	[a]	2.1
MDL	1.6 x ADL	MDDWF	5.1	21.6	3,010	11	4.6	[a]	2.5
MDL	1.6 x ADL	MDDWF	5.1	14.5	3,000	11	7.2	[a]	2.2

[a] The BioWin model predicts complete denitrification under this scenario (*i.e.*, nitrate of 0 mg/L) due to the very low DO conditions that occur in portions of the oxidation ditch combined with the high recycle rates. In practice, it is expected that a high level of denitrification will occur, but that effluent nitrate levels will be greater than 0 mg/L.

Table 18. Secondary Process Model Results at Target MLSS Value of 3,500 mg/L

Influent Conditions					Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations		
Load Condition	Load Factor	Flow Condition	Flow mgd	Temp. °C	MLSS mg/L	Aerated MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L
ADL	1.0	ADWF	3.0	21.6	3,510	24	0.32	0.020	1.3
ADL	1.0	ADWF	3.0	17.0	3,490	23	0.40	7.5	1.2
MML	1.3 x ADL	ADWF	3.0	21.6	3,540	17	1.0	[a]	2.0
MML	1.3 x ADL	ADWF	3.0	17.0	3,510	17	1.2	[a]	1.8
MML	1.3 x ADL + Recycle Streams	ADWF	3.0	17.0	3,540	16	3.6	0.010	1.9
MML	1.3 x ADL	ADMMF	3.9	21.6	3,550	17	1.0	[a]	2.0
MML	1.3 x ADL	ADMMF	3.9	17.0	3,470	16	1.1	[a]	1.8
MML	1.3 x ADL	ADMMF	3.9	21.6	3,460	13	5.1	[a]	2.5
MDL	1.6 x ADL	ADMMF	3.9	14.5	3,510	13	7.2	[a]	2.3
MDL	1.6 x ADL	MDDWF	5.1	21.6	3,490	14	4.1	[a]	2.5
MDL	1.6 x ADL	MDDWF	5.1	14.5	3,490	13	5.9	[a]	2.3

[a] The BioWin model predicts complete denitrification under this scenario (*i.e.*, nitrate of 0 mg/L) due to the very low DO conditions that occur in portions of the oxidation ditch combined with the high recycle rates. In practice, it is expected that a high level of denitrification will occur, but that effluent nitrate levels will be greater than 0 mg/L.

The results in Table 17 and Table 18 indicate that the existing oxidation ditches have adequate capacity to nitrify and denitrify the City's wastewater at the ADL and MML conditions for the Current Permitted Design Condition – *if the target MCRT requirement did not need to be satisfied*. However, the effluent ammonia levels would likely be a concern at the MDL, cold temperature conditions. To address this concern, additional process modeling was performed for this condition using increased aerator capacity. This additional modeling, whose results are presented in Table 19, demonstrated that the ammonia limits could be satisfied under the worst-case MDL/ADMMF condition if the aeration capacity were increased by 25 percent (power supply rate increased from 60 to 75 HP).

Table 19. Secondary Process Model Results at Target MLSS Value of 3,000 mg/L with Aeration Power Increased to 75 Horsepower

Influent Conditions					Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations		
Load Condition	Load Factor	Flow Condition	Flow mgd	Temp. °C	MLSS mg/L	Aerated MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L
MDL	1.6 x ADL	ADMMF	3.9	14.5	3,016	12	1.0	0.57	1.4

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Nevertheless, even with additional aeration capacity, the limitations to the MLSS concentrations result in MCRT values much less than the recommended 25 to 30-day range needed to ensure Class B biosolids pathogen standards are satisfied. Therefore, the results presented in Table 17 and Table 18 demonstrate that, even with increased aeration capacity (and the ability to better control the aeration system), the existing oxidation ditches do not have adequate capacity to nitrify and denitrify the wastewater while still providing the MCRT needed for solids treatment.

Upgrade Alternatives Evaluation

The following alternatives for accomplishing the desired effluent levels have been considered in the development of the WWTP Facilities Master Plan:

- **Alternative A:** Construct a third oxidation ditch and modify oxidation ditches, as needed, to ensure reliable nitrification/denitrification
- **Alternative B:** Construct a separate anoxic basin (with mixed liquor return) for denitrification, effectively increasing the overall basin volume
- **Alternative C:** Reduce the organic loading to the oxidation ditches by providing primary treatment via microscreening and modify oxidation ditches, as needed, to ensure reliable nitrification/denitrification. (Influent parameters used for this alternative are presented in Attachment F.)
- **Alternative D:** Provide an alternative means of solids treatment so that the aerated MCRT can be reduced without impacting the City's ability to meet Class B biosolids requirements.
- **Alternative E:** Maintain two oxidation ditches, but improve the solids removal capacity of the clarifiers so that the MLSS levels in the oxidation ditches can be increased (while still maintaining at least a 25 day MCRT).

The models for all of the alternatives were adjusted to achieve the effluent water quality and operational requirements outlined in Tables 7 and 8, with the following exceptions:

- Alternatives B and E allows for a higher MLSS concentration than the recommended maximum of 3,500 mg/L.
- Alternative D allows for a minimum aerated MCRT of 15 days instead of the 25 days currently specified.

To simplify the modeling analysis of the alternatives, three design conditions identified in the existing facilities analysis as representing the range of operating conditions were used in evaluating potential alternatives. From Table 17 and Table 18, the following three scenarios were identified as representing the range of design conditions for the WWTP:

- **Annual Average:** ADWF and ADL conditions at the average monthly average temperature of 21.6°C, including recycle stream flows and loads.
- **Maximum Month:** ADWF and MML conditions at the minimum monthly average temperature of 17.0°C, including recycle stream flows and loads

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- **Maximum Day:** ADMMF and MDL conditions at a minimum historic temperature of 14.5°C

The Maximum Day design condition was also evaluated for each alternative under the range of potential operating conditions, as follows:

- **Maximum Day Option A**: The WAS flow rate is not adjusted from the maximum month condition, and MLSS is allowed to increase
- **Maximum Day Option B**: The WAS flow is adjusted to that MLSS is maintained at maximum month conditions (and MCRT drops accordingly).

Finally, the Annual Average design condition was evaluated for each alternative, assuming one oxidation ditch would be out of service, to represent the conditions during a maintenance period. For this analysis, a range of potential operating conditions were also considered (if appropriate), as follows:

- **Annual Average Maintenance Period Option A**: MLSS is allowed to increase, as needed, to meet MCRT requirements
- **Annual Average Maintenance Period Option B:** MLSS is maintained at maximum month conditions, and MCRT is allowed to drop accordingly.

Influent concentrations used in evaluating Alternatives A, B, D, E were discussed previously in this TM. For Alternative C (microscreening), the influent water quality (and wastewater fractions) would be different than the values used for the other analyses presented in this TM. A discussion of the influent water quality associated with Alternative C is presented in Attachment F. Wastewater fractions for the microscreened influent are assumed to be equal to the default values for settled wastewater shown in Table 5.

Results of the steady-state BioWin models developed for each alternative to evaluate the three design conditions (and the two optional bookends) listed above are shown in Table 20. Discussion and conclusions drawn from the alternatives evaluation modeling results is provided in Chapter 6 (WWTP Expansion Alternatives).

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Table 20. Results of Secondary Process Modeling of Alternatives for City of Galt Wastewater Treatment Plant

Design Conditions																	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations				Solids Production			
# of Ditches	Load Condition	Load Factor	COD Load, ppd	BOD Load, ppd	TSS Load, ppd	TKN Load, ppd	Flow Condition	Flow mgd	Reactor Detention Time, hours	WAS/ RAS Split, mgd	Return Streams	Temp. °C	alpha (f)	Total Air Per Basin, lb O_2	Air Zone 1a, hp	Air Zone 4a, hp	MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	TSS, mg/L	Total Solids (Per Train), ppd	Total Solids, ppd	Volatile Solids (Per Train) ppd	Volatile Solids ppd
Alternative A (Third Ditch)																										
3	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	36.0	0.019	Yes	21.6	0.5	298	45	40	2,597	25	0.310	2.5	1.0	1.5	1,306	3,918	1,030	3,090
3	MML	1.3 x ADL	16,200	6,700	6,800	1,200	ADWF	3.0	36.0	0.019	Yes	17.0	0.5	350	50	50	3,425	25	0.390	8.2	1.1	2.0	1,686	5,058	1,344	4,032
3	MDL	1.6 x ADL	20,000	8,200	8,400	1,500	ADMMF	3.9	27.7	0.019	No	14.5	0.5	385	55	55	4,283	26	0.440	3.3	1.2	2.6	2,017	6,051	1,613	4,839
3	MDL	1.6 x ADL	20,000	8,200	8,400	1,500	ADMMF	3.9	27.7	0.024	No	14.5	0.5	385	55	55	3,524	21	0.470	6.1	1.2	2.1	2,090	6,270	1,680	5,040
2	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	24.0	0.018	Yes	21.6	0.5	368	55	50	4,082	25	0.510	0.0	1.7	2.3	1,984	3,968	1,540	3,080
2	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	24.0	0.022	Yes	21.6	0.5	385	55	55	3,410	21	0.450	0.0	1.5	1.9	2,020	4,040	1,579	3,158
Alternative B (Separate Anoxic Basin)																										
2	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	24.0	0.024	Yes	21.6	0.5	420	60	60	3,097	25	0.310	4.1	1.1	1.8	2,009	4,018	1,548	3,096
2	MML	1.3 x ADL	16,200	6,700	6,800	1,200	ADWF	3.0	24.0	0.025	Yes	17.0	0.5	490	70	70	3,994	25	0.390	6.4	1.2	2.4	2,594	5,188	2,019	4,038
2	MDL	1.6 x ADL	20,000	8,200	8,400	1,500	ADMMF	3.9	18.5	0.025	No	14.5	0.5	560	80	80	5,043	26	0.410	8.2	1.4	3.0	3,127	6,254	2,445	4,890
2	MDL	1.6 x ADL	20,000	8,200	8,400	1,500	ADMMF	3.9	18.5	0.033	No	14.5	0.5	560	80	80	3,963	20	0.520	7.6	1.3	2.4	3,234	6,468	2,553	5,106
1	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	12.0	0.024	Yes	21.6	0.5	735	105	105	6,207	25	0.510	0.1	1.7	3.6	4,044	4,044	3,110	3,110
1	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	12.0	0.040	Yes	21.6	0.5	840	120	120	3,989	15	0.540	1.2	1.4	2.3	4,314	4,314	3,379	3,379
Alternative C (Microscreened Influent)																										
2	ADL	ADL	8,760	3,630	2,500	880	ADWF	3.0	24.0	0.016	Yes	21.6	0.5	420	60	60	2,016	28	0.330	3.8	1.0	1.2	872	1,744	735	1,470
2	MML	1.3 x ADL	11,400	4,700	3,300	1,100	ADWF	3.0	24.0	0.019	Yes	17.0	0.5	455	65	65	2,372	25	0.530	4.6	1.2	1.4	1,169	2,338	1,001	2,002
2	MDL	1.6 x ADL	14,000	5,800	4,000	1,400	ADMMF	3.9	18.5	0.019	No	14.5	0.5	525	75	75	2,908	26	0.510	3.7	1.3	1.8	1,374	2,748	1,196	2,392
2	MDL	1.6 x ADL	14,000	5,800	4,000	1,400	ADMMF	3.9	18.5	0.024	No	14.5	0.5	525	75	75	2,463	21	0.550	6.2	1.3	1.5	1,467	2,934	1,282	2,564
1	ADL	ADL	8,760	3,630	2,500	880	ADWF	3.0	12.0	0.018	Yes	21.6	0.5	718	105	100	3,810	25	0.510	0.0	1.8	2.2	1,858	1,858	1,520	1,520
1	ADL	ADL	8,760	3,630	2,500	880	ADWF	3.0	12.0	0.033	Yes	21.6	0.5	770	110	110	2,417	14	0.470	0.1	1.5	1.4	2,151	2,151	1,810	1,810
Alternative D (Separate Biosolids Facility)																										
2	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	24.0	0.031	Yes	21.6	0.5	438	65	60	2,533	15	0.370	2.1	1.1	1.5	2,108	4,216	1,680	3,360
2	MML	1.3 x ADL	16,200	6,700	6,800	1,200	ADWF	3.0	24.0	0.033	Yes	17.0	0.5	490	70	70	3,243	15	0.580	3.7	1.3	1.9	2,762	5,524	2,222	4,444
2	MDL	1.6 x ADL	20,000	8,200	8,400	1,500	ADMMF	3.9	18.5	0.033	No	14.5	0.5	595	85	85	4,039	15	0.510	7.2	1.4	2.4	3,297	6,594	2,670	5,340
2	MDL	1.6 x ADL	20,000	8,200	8,400	1,500	ADMMF	3.9	18.5	0.043	No	14.5	0.5	578	85	80	3,261	12	0.620	8.4	1.4	2.0	3,457	6,914	2,817	5,634
1	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	12.0	0.031	Yes	21.6	0.5	770	110	110	5,067	15	0.580	0.0	2.0	2.9	4,250	4,250	3,343	3,343
Alternative E (Existing with higher MLSS)																										
2	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	24.0	0.018	Yes	21.6	0.5	420	60	60	4,028	25	0.310	0.0	1.4	2.3	1,957	3,914	1,530	3,060
2	MML	1.3 x ADL	16,200	6,700	6,800	1,200	ADWF	3.0	24.0	0.019	Yes	17.0	0.5	490	70	70	5,183	25	0.430	0.1	1.4	3.0	2,555	5,110	2,019	4,038
2	MDL	1.6 x ADL	20,000	8,200	8,400	1,500	ADMMF	3.9	18.5	0.019	No	14.5	0.5	560	80	80	6,307	26	0.470	0.0	1.6	3.8	2,979	5,958	2,379	4,758
1	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	12.0	0.018	Yes	21.6	0.5	753	110	105	8,071	25	0.500	0.0	2.1	4.6	3,946	3,946	3,057	3,057
1	ADL	ADL	12,500	5,100	5,200	910	ADWF	3.0	12.0	0.030	Yes	21.6	0.5	788	115	110	5,205	15	0.520	0.0	1.9	3.0	4,226	4,226	3,322	3,322

Expansion Alternatives Evaluation

Additional expansion improvements have been identified in Chapter 6 (WWTP Expansion Alternatives) that would be needed for three of the viable alternatives (A, C, and D) under the Future Permitted Design Condition and Ultimate Buildout Design Condition. In addition, a fifth alternative (Alternative F) has been identified that combines the treatment technologies of Alternatives C and D as the WWTP is expanded.

The identified expansion alternatives have been modeled, as appropriate, to evaluate their feasibility. Some of the expansion alternatives would look similar on a secondary process "treatment train" basis to alternatives already modeled, so additional modeling analysis is not required. These alternatives include the following:

- Alternative C and D at the Future Permitted Design Condition because each is expected to have a 50 percent increase in oxidation ditch capacity (from 2 to 3 ditches) along with a 50 percent increase in flow (3.0 to 4.5 mgd).
- Alternatives A, C, and D at the Ultimate Buildout Design Condition because each is expected to have double the number of oxidation ditches (from 2 to 4) along with double the flow (3.0 to 6.0 mgd) as compared to the Current Permitted Design Condition.

The remaining expansion alternatives that require modeling include Alternatives A and F at the Future Permitted Design Condition and Alternative F at the Ultimate Buildout Design Condition. Results from models of these expansion alternatives, including models with one ditch out of service, are presented for the Permitted Design Condition and Ultimate Buildout Design Condition, respectively, in Table 21 and Table 22. Discussion and conclusions drawn from the alternatives evaluation modeling results is provided in Chapter 6 (WWTP Expansion Alternatives).

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Table 21. Results of Secondary Process Modeling of Expansion Alternatives at Future Permitted Design Condition for City of Galt Wastewater Treatment Plant

Design Conditions																	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations				Solids Production			
# of Ditches	Load Condition	Load Factor	COD Load, ppd	BOD Load, ppd	TSS Load, ppd	TKN Load, ppd	Flow Condition	Flow mgd	Reactor Detention Time, hours	WAS/ RAS Split, mgd	Return Streams	Temp. °C	alpha (f)	Total Air Per Basin, lb O_2	Air Zone 1a, hp	Air Zone 4a, hp	MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	TSS, mg/L	Total Solids (Per Train), ppd	Total Solids, ppd	Volatile Solids (Per Train) ppd	Volatile Solids ppd
Alternative A (Third Ditch)																										
5	ADL	ADL	18,700	7,700	7,900	1,400	ADWF	4.5	40.0	0.019	Yes	21.6	0.5	280	40	40	2,327	25	0.270	6.2	1.0	1.3	1,171	5,855	925	4,625
5	MML	1.3 x ADL	24,300	10,000	10,200	1,800	ADWF	4.5	40.0	0.019	Yes	17.0	0.5	333	50	45	3,454	25	0.440	4.2	1.1	2.0	1,688	8,440	1,342	6,710
5	MDL	1.6 x ADL	29,900	12,300	12,600	2,200	ADMMF	5.9	30.8	0.019	No	14.5	0.5	350	50	50	3,865	26	0.430	5.7	1.2	2.3	1,818	9,090	1,454	7,270
5	MDL	1.6 x ADL	29,900	12,300	12,600	2,200	ADMMF	5.9	30.8	0.021	No	14.5	0.5	350	50	50	3,553	24	0.440	6.8	1.2	2.1	1,845	9,225	1,479	7,395
4	ADL	ADL	18,700	7,700	7,900	1,400	ADWF	4.5	32.0	0.019	Yes	21.6	0.5	350	50	50	2,908	25	0.270	4.9	1.0	1.7	1,466	5,864	1,156	4,624
Alternative F (Microscreened Influent and Separate Biosolids Facility)																										
2	ADL	ADL	13,100	5,400	3,800	1,300	ADWF	4.5	16.0	0.028	Yes	21.6	0.5	648	95	90	1,992	16	0.360	5.8	1.1	1.1	1,509	3,018	1,287	2,574
2	MML	1.3 x ADL	17,100	7,100	4,900	1,700	ADWF	4.5	16.0	0.031	Yes	17.0	0.5	735	105	105	2,514	15	0.760	8.9	1.3	1.4	2,109	4,218	1,822	3,644
2	MDL	1.6 x ADL	21,000	8,700	6,000	2,100	ADMMF	5.9	12.3	0.031	No	14.5	0.5	840	120	120	3,075	16	0.590	7.8	1.5	1.9	2,369	4,738	2,079	4,158
2	MDL	1.6 x ADL	21,000	8,700	6,000	2,100	ADMMF	5.9	12.3	0.043	No	14.5	0.5	805	115	115	2,454	12	0.790	8.5	1.5	1.5	2,615	5,230	2,306	4,612
1	ADL	ADL	13,100	5,400	3,800	1,300	ADWF	4.5	8.0	0.030	Yes	21.6	0.5	1,400	200	200	3,784	15	0.400	3.5	1.4	2.2	3,083	3,083	2,633	2,633

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Table 22. Results of Secondary Process Modeling of Expansion Alternatives at Buildout Design Condition for City of Galt Wastewater Treatment Plant

Design Conditions																	Predicted Ox. Ditch Conditions		Predicted Effluent Concentrations				Solids Production			
# of Ditches	Load Condition	Load Factor	COD Load, ppd	BOD Load, ppd	TSS Load, ppd	TKN Load, ppd	Flow Condition	Flow mgd	Reactor Detention Time, hours	WAS/ RAS Split, mgd	Return Streams	Temp. °C	alpha (f)	Total Air Per Basin, lb O_2	Air Zone 1a, hp	Air Zone 4a, hp	MLSS mg/L	MCRT days	Ammonia mg/L as N	Nitrate mg/L as N	BOD mg/L	TSS, mg/L	Total Solids (Per Train), ppd	Total Solids, ppd	Volatile Solids (Per Train) ppd	Volatile Solids ppd
Alternative F (Microscreened Influent and Separate Biosolids Facility)																										
3	ADL	ADL	17,500	7,300	5,000	1,800	ADWF	6.0	18.0	0.024	Yes	21.6	0.5	560	80	80	2,000	19	0.370	4.1	1.1	1.2	1,287	3,861	1,092	3,276
3	MML	1.3 x ADL	22,800	9,400	6,500	2,300	ADWF	6.0	18.0	0.027	Yes	17.0	0.5	648	95	90	2,496	17	0.700	8.2	1.2	1.4	1,791	5,373	1,542	4,626
3	MDL	1.6 x ADL	28,000	11,600	8,000	2,800	ADMMF	7.8	13.8	0.027	No	14.5	0.5	718	105	100	3,052	19	0.600	4.9	1.4	1.8	2,010	6,030	1,760	5,280
3	MDL	1.6 x ADL	28,000	11,600	8,000	2,800	ADMMF	7.8	13.8	0.036	No	14.5	0.5	718	105	100	2,469	14	0.680	8.6	1.4	1.5	2,203	6,609	1,938	5,814
2	ADL	ADL	17,500	7,300	5,000	1,800	ADWF	6.0	12.0	0.030	Yes	21.6	0.5	858	125	120	2,537	15	0.420	3.4	1.2	1.5	2,061	4,122	1,758	3,516

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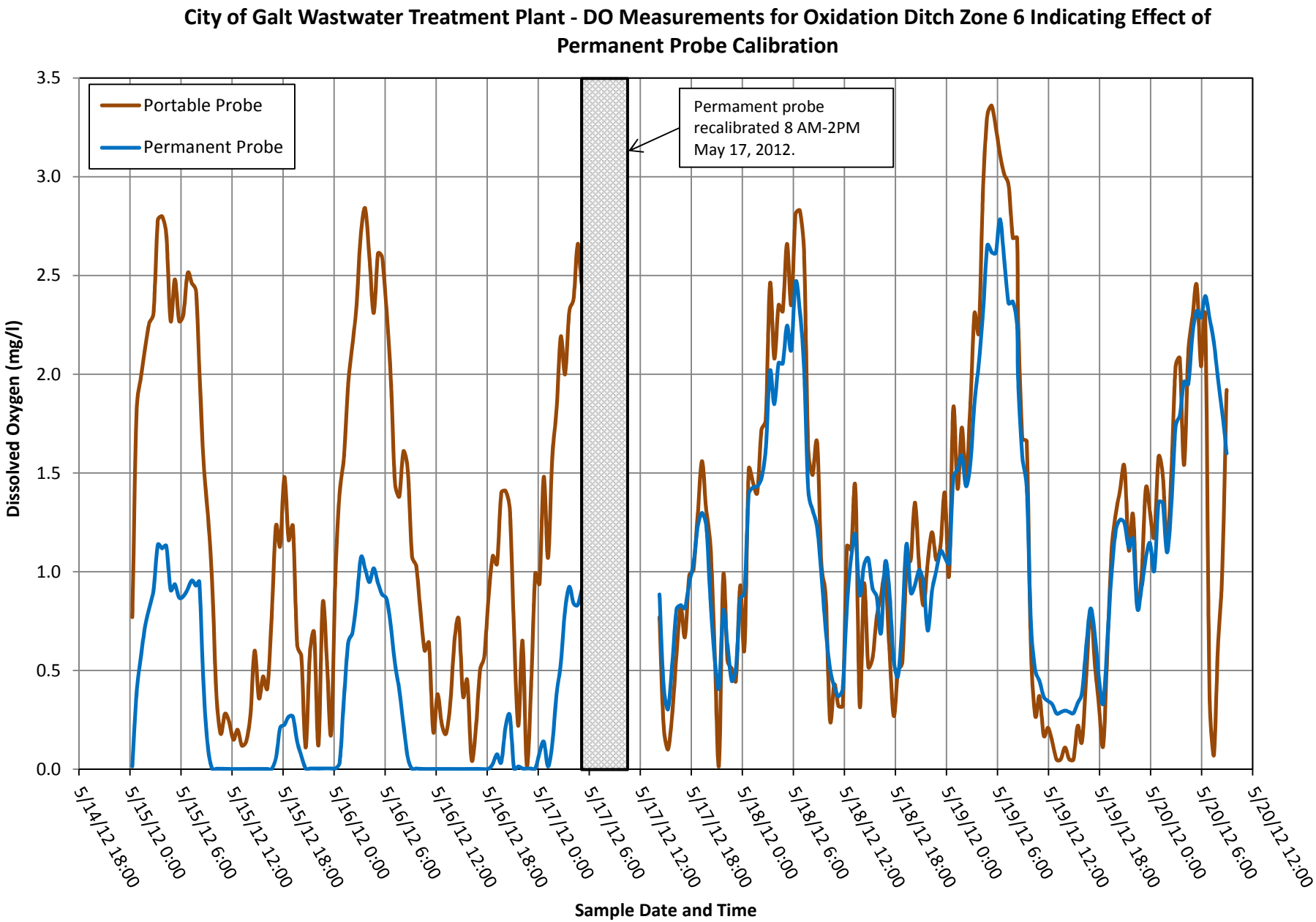
City of Galt Wastwater Treatment Plant - DO Measurements for Oxidation Ditch Zone 6 Indicating Effect of Permanent Probe Calibration
Portable Probe
Permanent Probe
Permament probe recalibrated 8 AM-2PM May 17, 2012.
Dissolved Oxygen (mg/l)
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
5/14/12 18:00
5/15/12 0:00
5/15/12 6:00
5/15/12 12:00
5/15/12 18:00
5/16/12 0:00
5/16/12 6:00
5/16/12 12:00
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5/19/12 0:00
5/19/12 6:00
5/19/12 12:00
5/19/12 18:00
5/20/12 0:00
5/20/12 6:00
5/20/12 12:00
Sample Date and Time

Figure B-1. Diurnal Influent Flow Per Oxidation Ditch
City of Galt WWTP

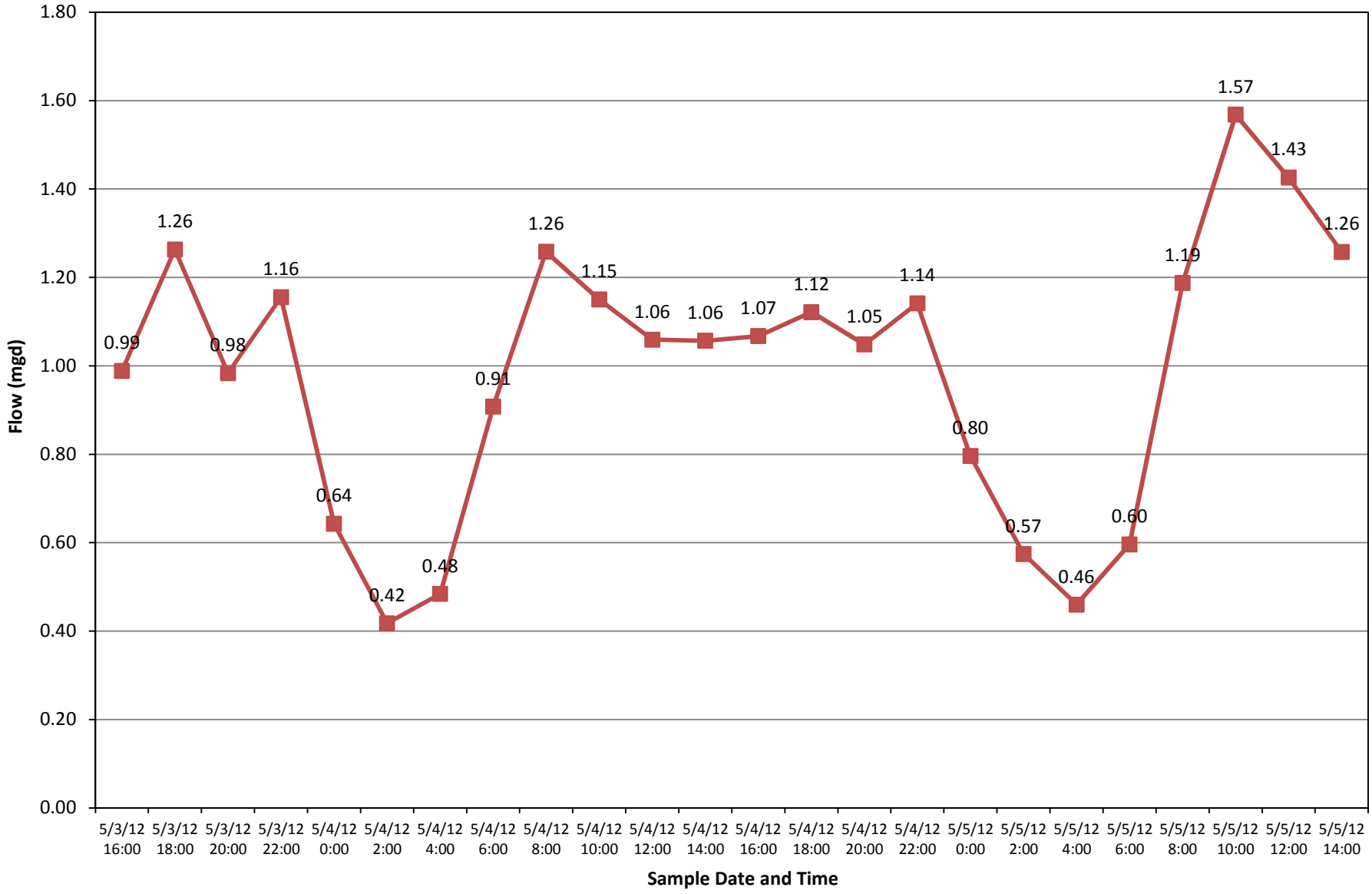


Figure B-2. Diurnal Influent TKN and Ammonia Levels
City of Galt WWTP

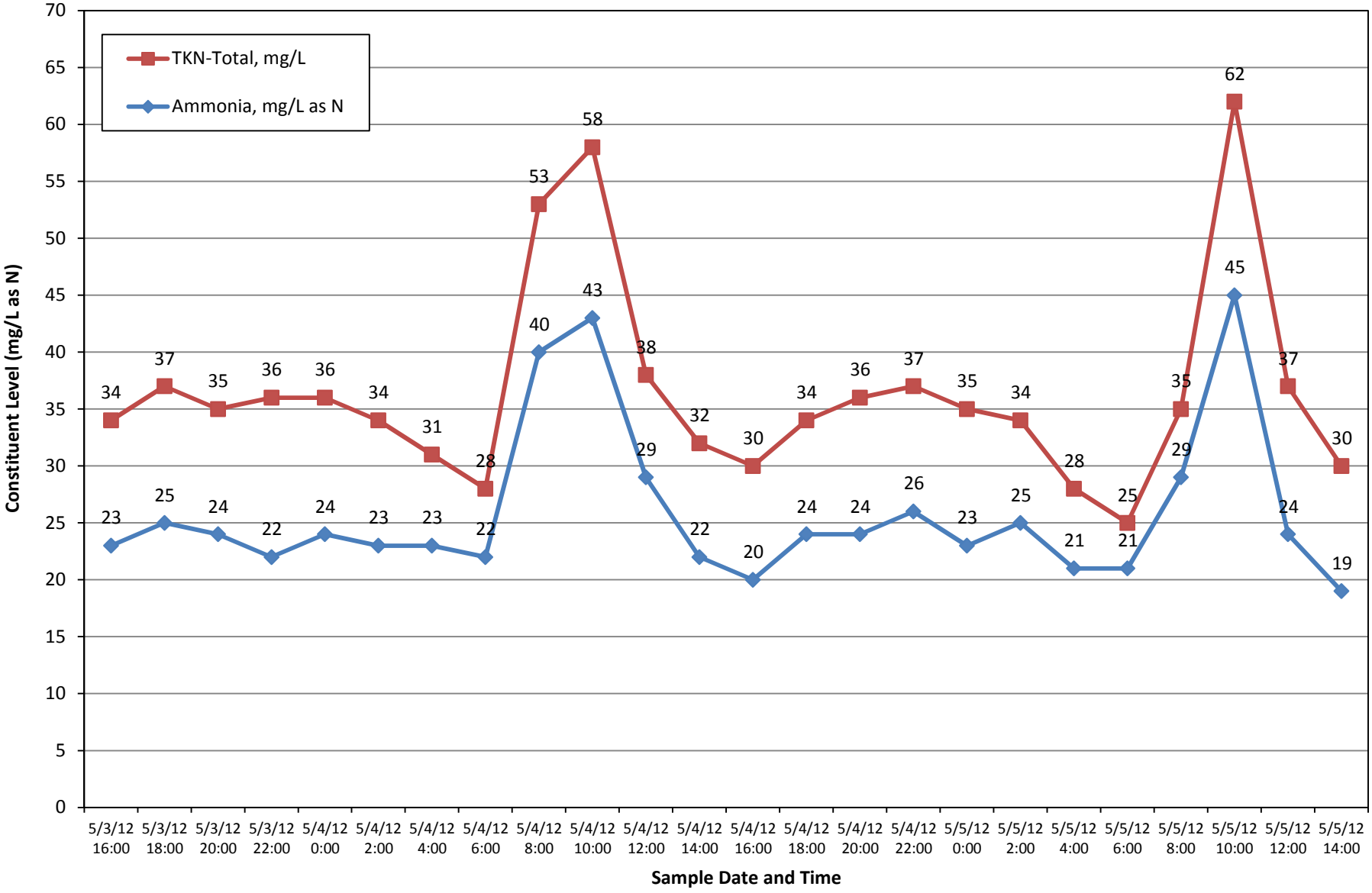


Figure B-3. Diurnal Influent COD Levels
City of Galt WWTP

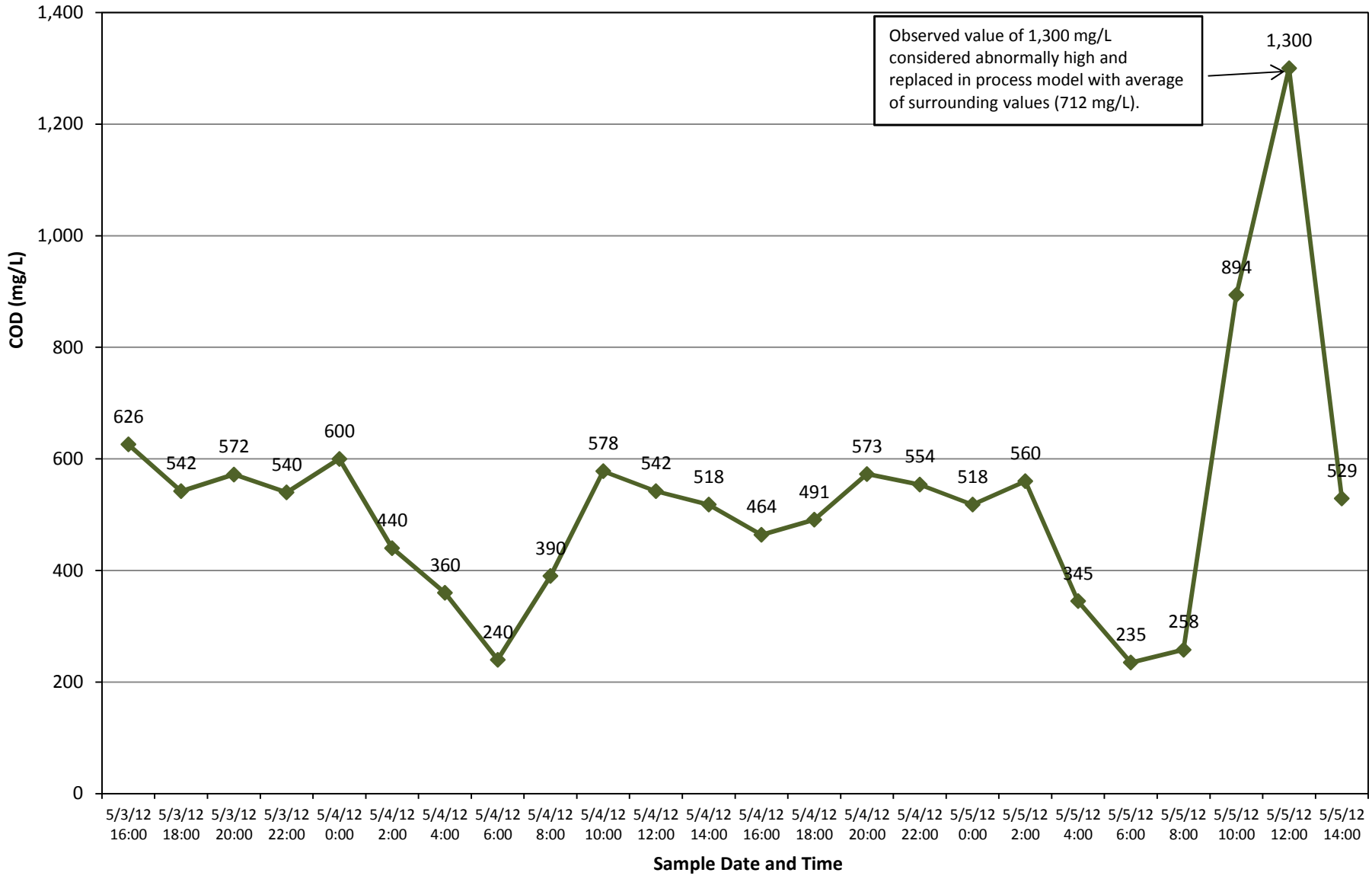


Figure B-4. Oxidation Ditch DO Measurments
City of Galt WWTP

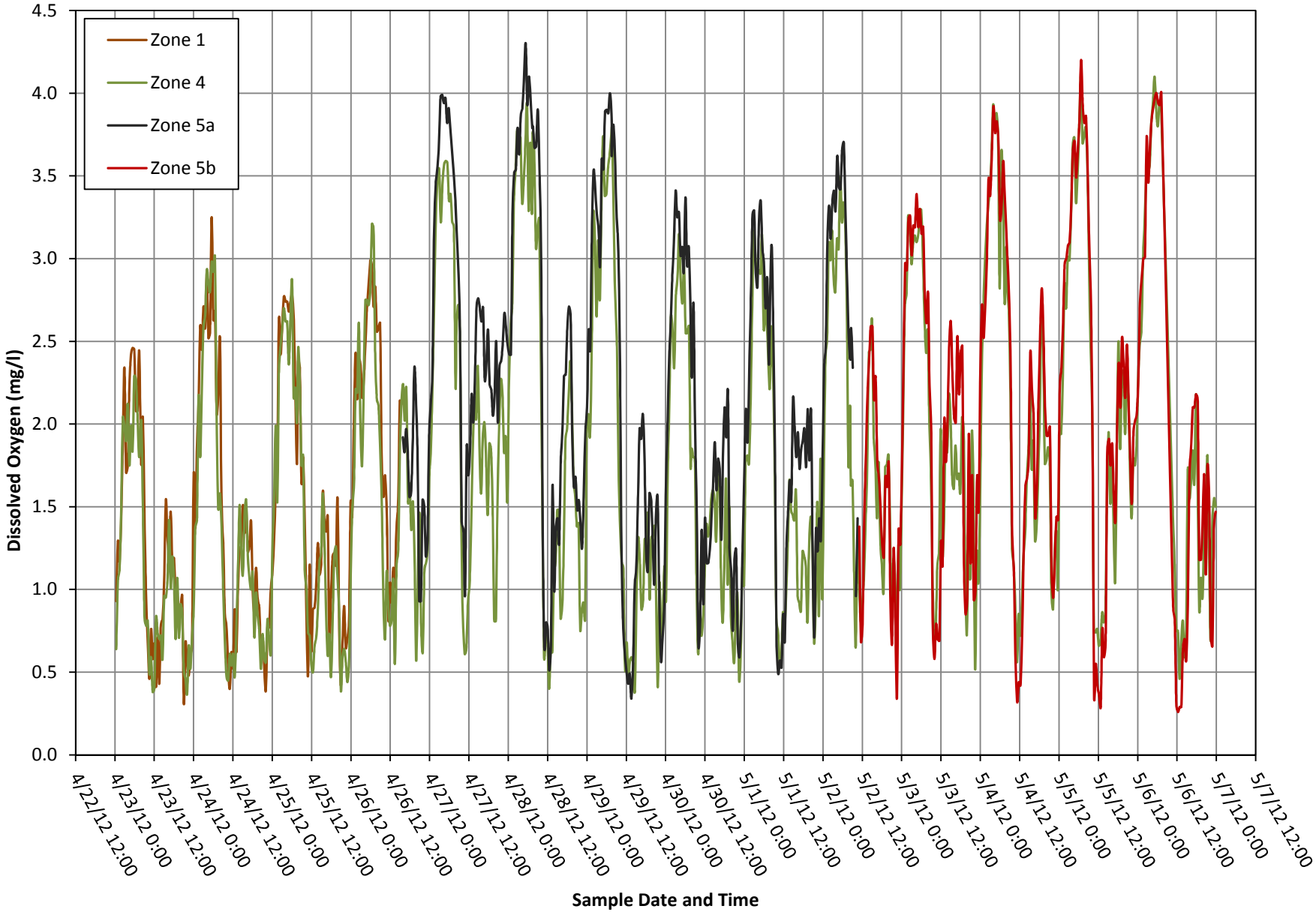


Figure B-5. Average Diurnal Ammonia Levels
City of Galt WWTP

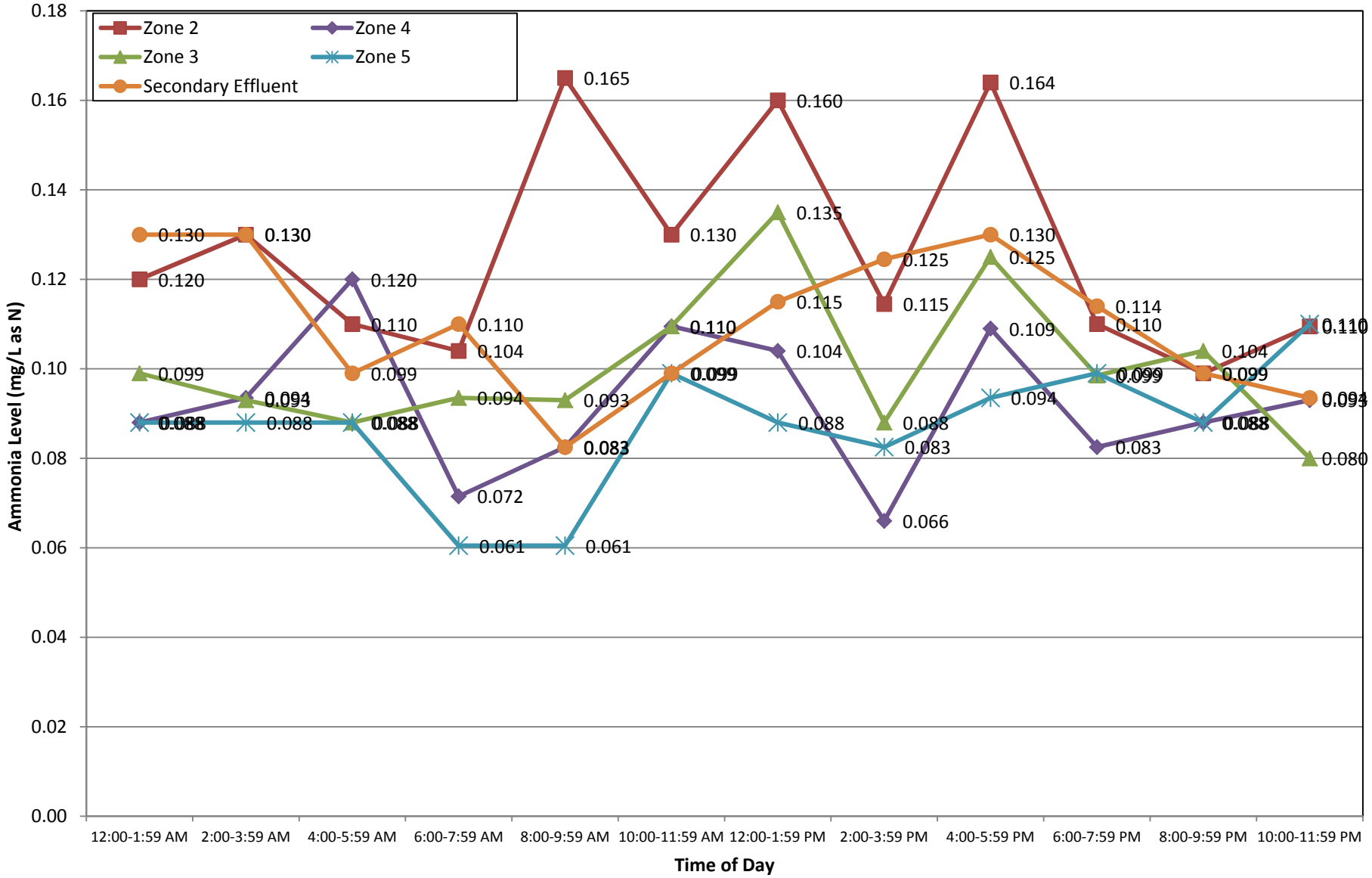


Figure B-6. Average Diurnal Nitrate Levels
City of Galt WWTP

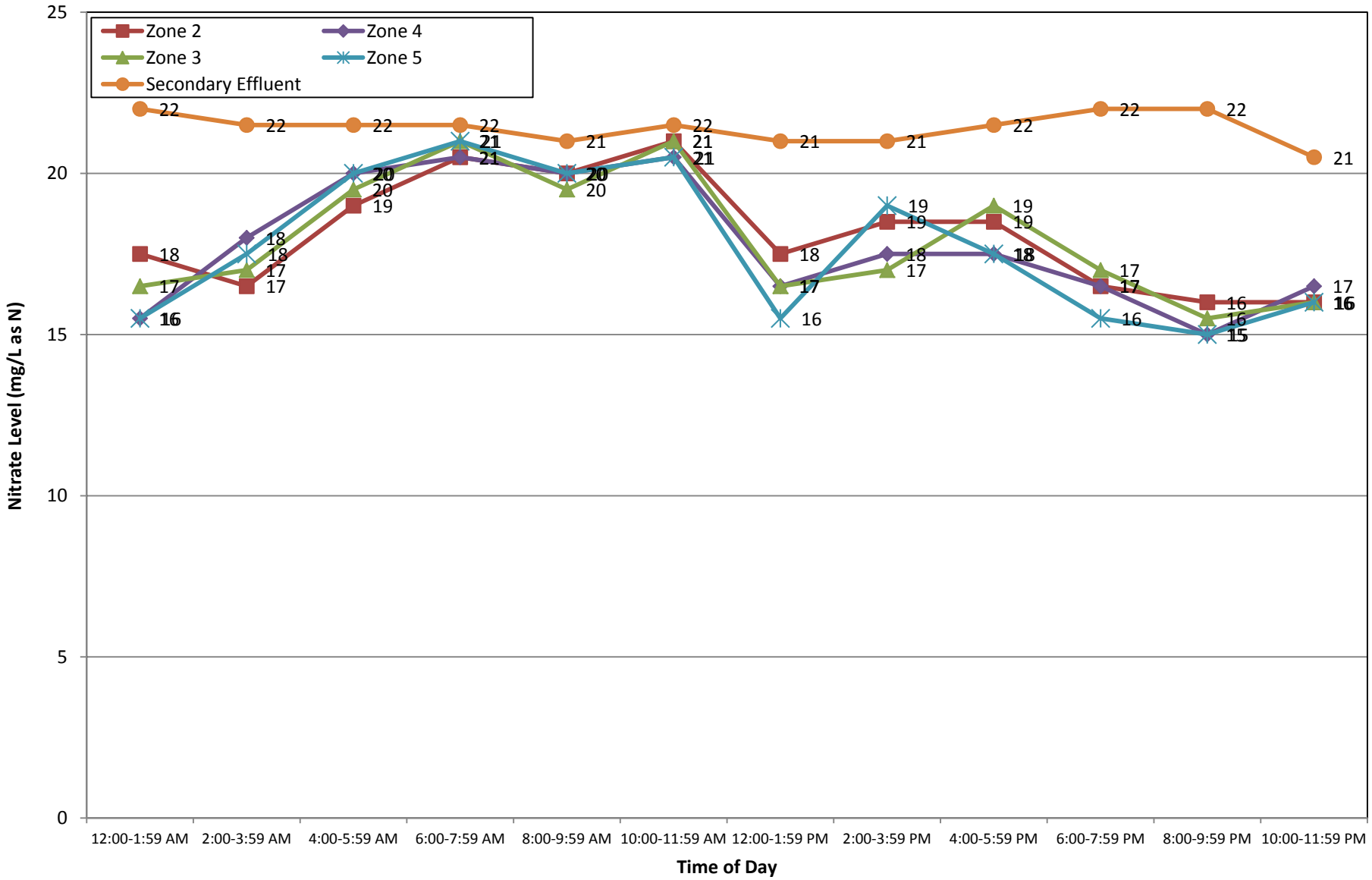


Figure B-7. Average Diurnal Nitrite Levels
City of Galt WWTP

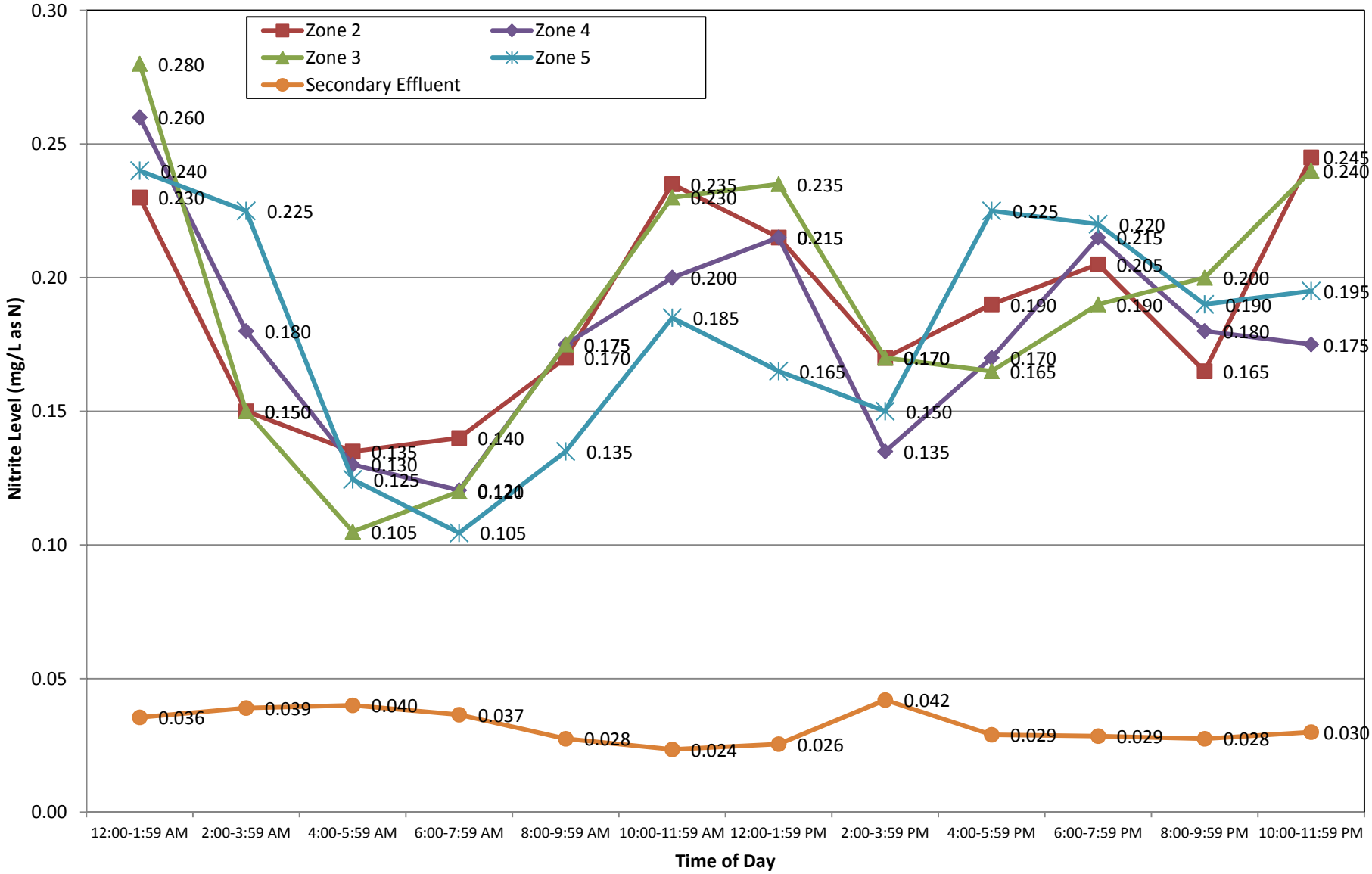


Figure B-8. Average Diurnal TKN Levels – Secondary Effluent
City of Galt WWTP

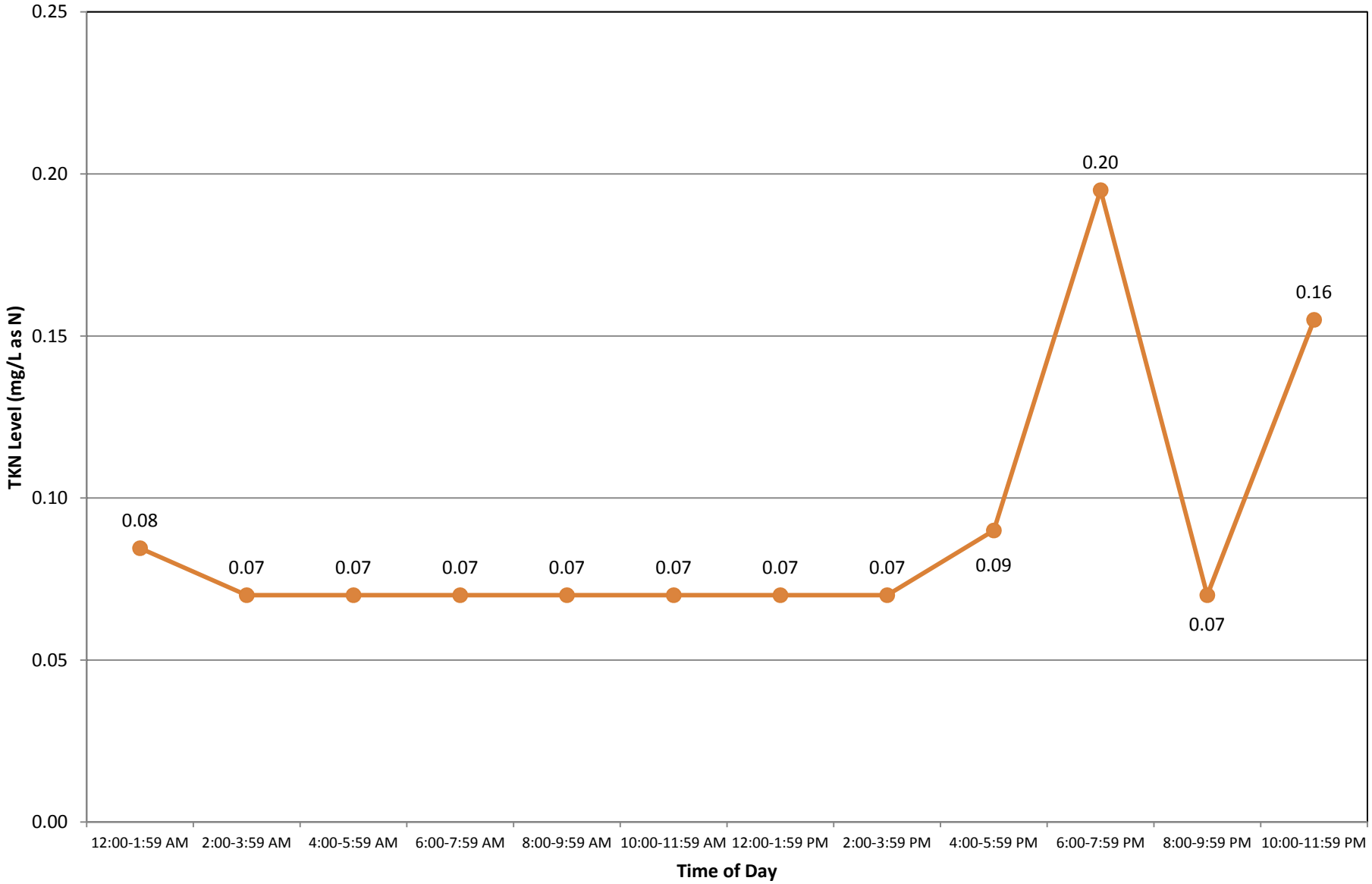
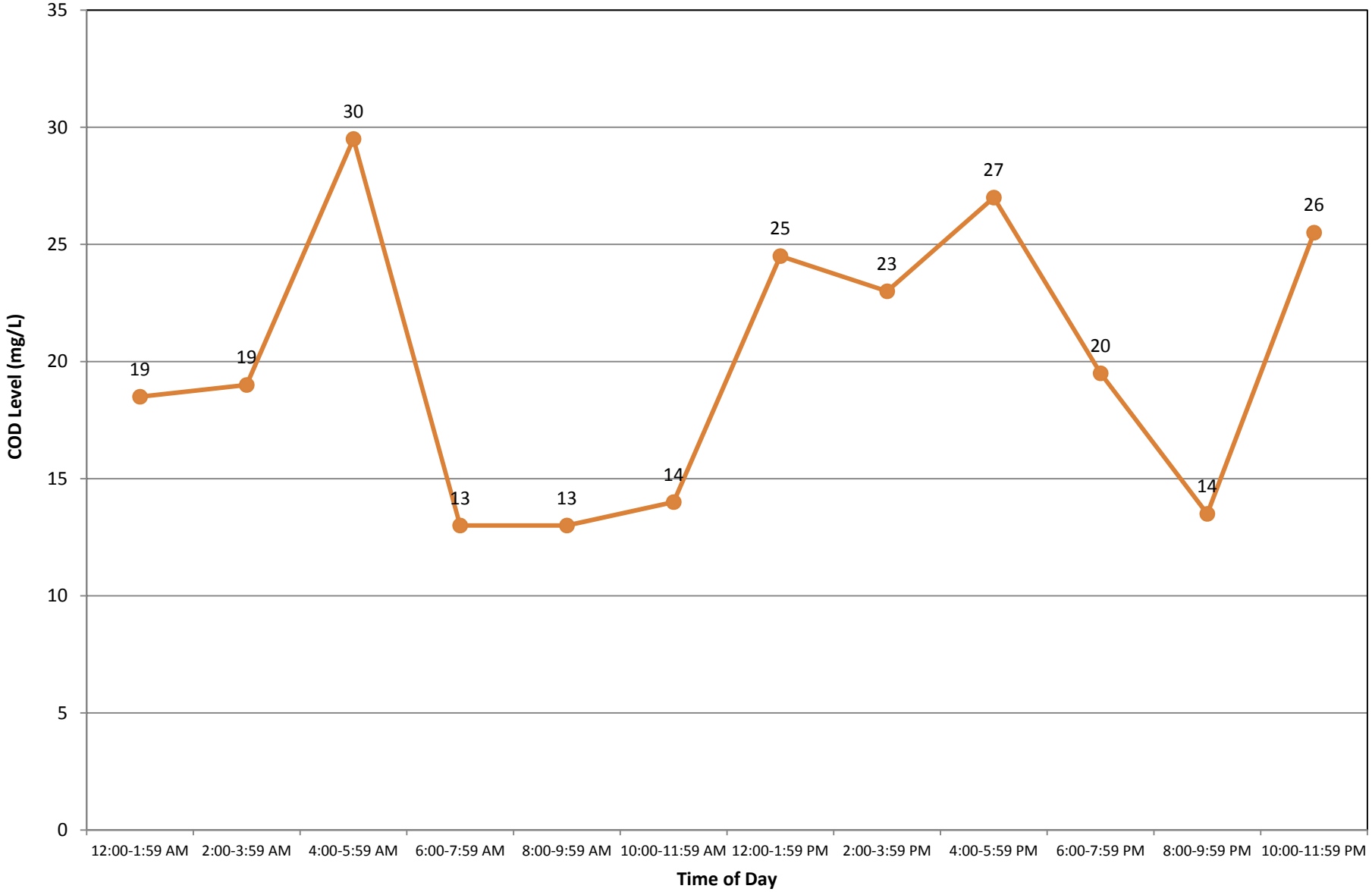


Figure B-9. Average Diurnal COD Levels – Secondary Effluent
City of Galt WWTP



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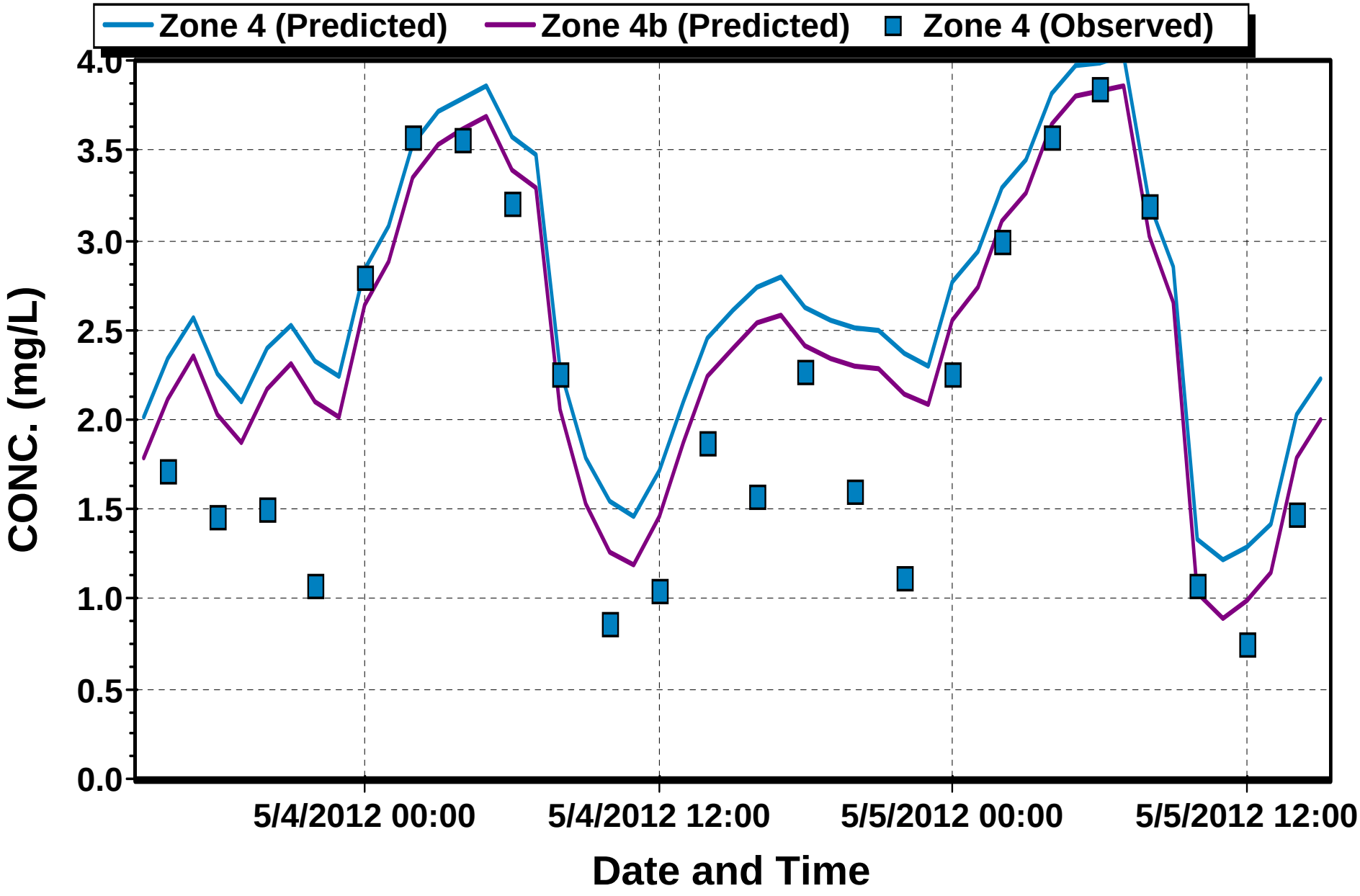
Galt WWTP Observed vs. Predicted Diurnal DO Levels - Zone 4



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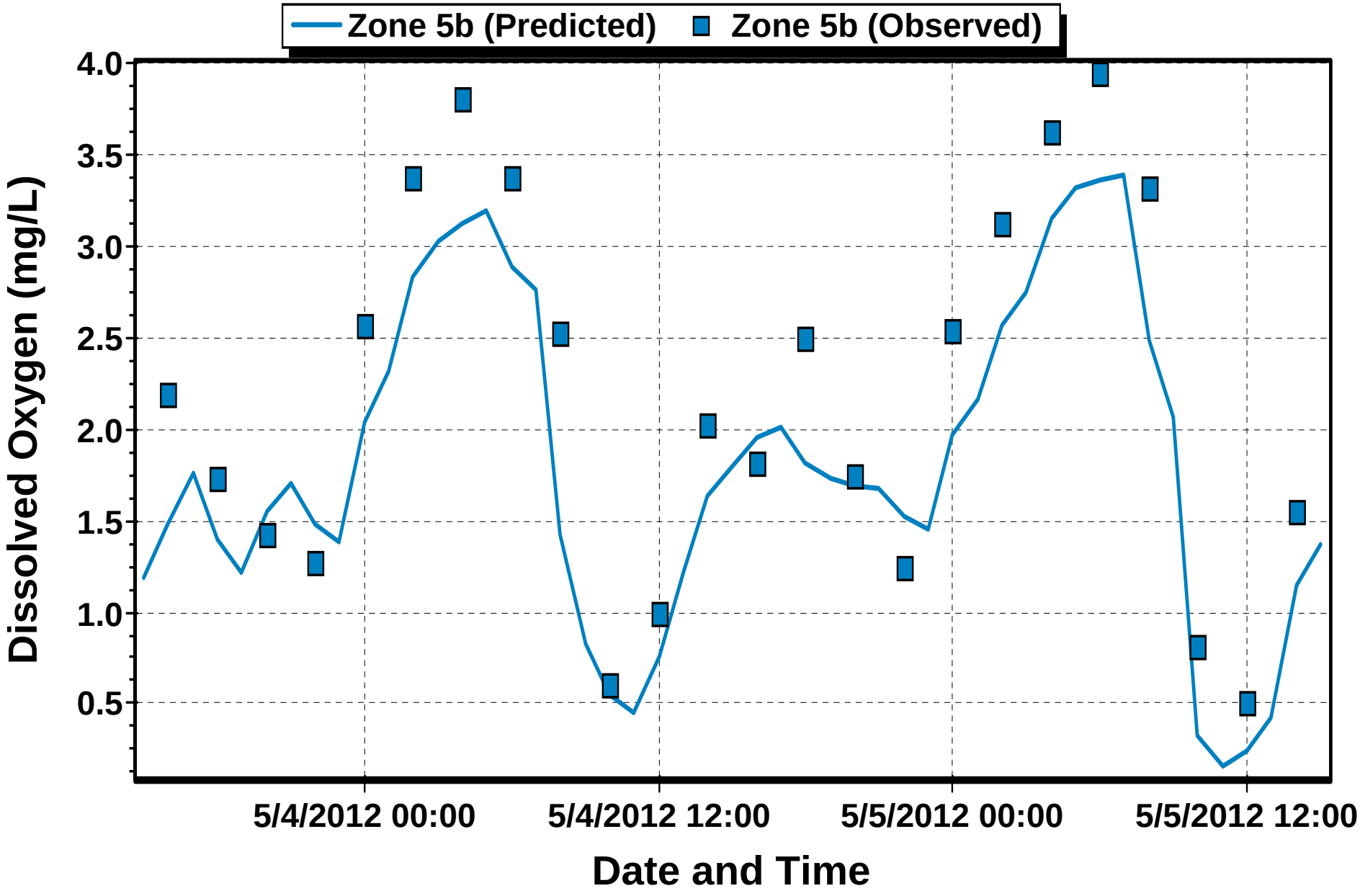
Galt WWTP Observed vs. Predicted Diurnal DO Levels - Zone 5b



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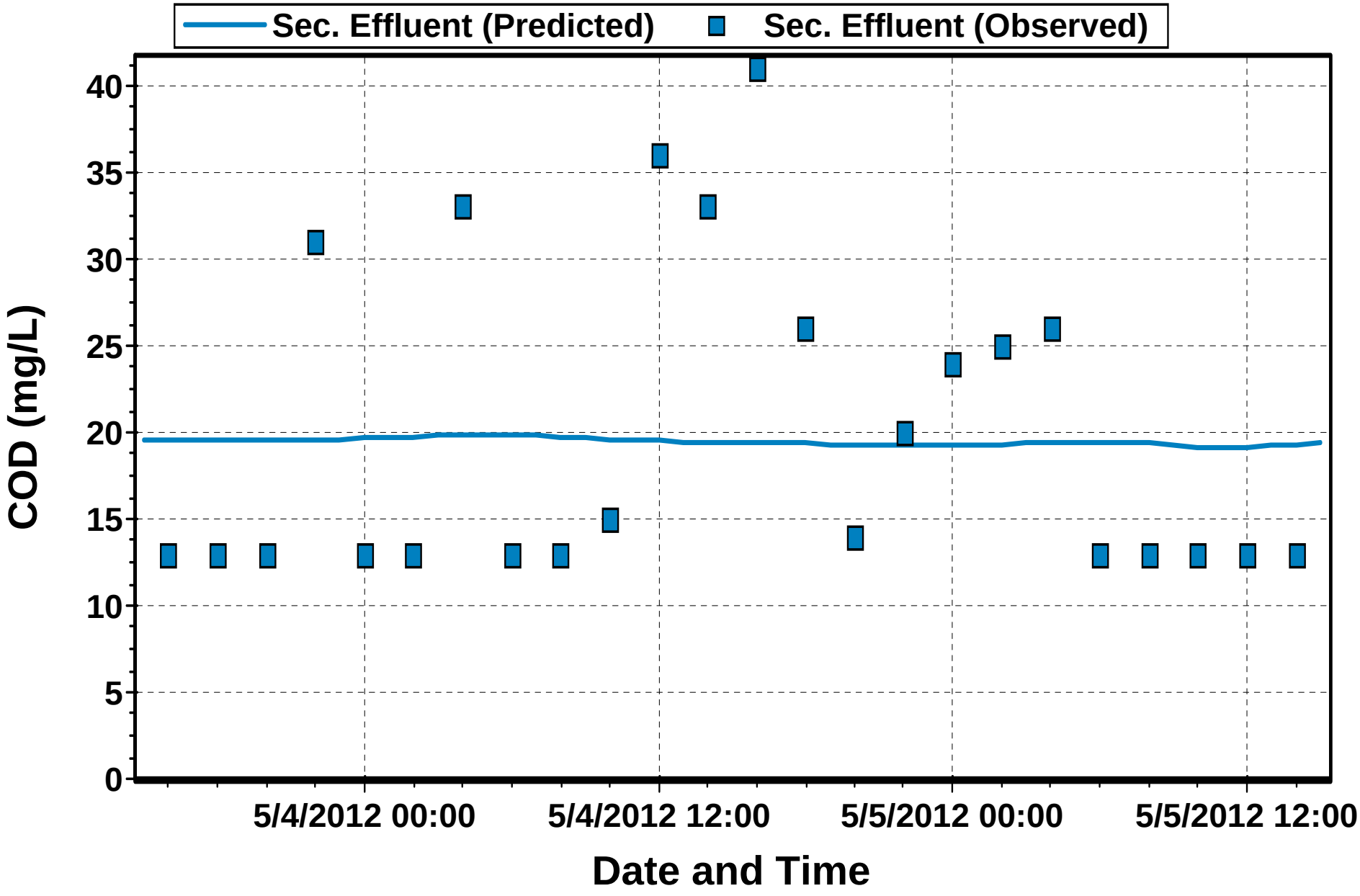
Galt WWTP Observed vs. Predicted COD Levels - Secondary Effluent



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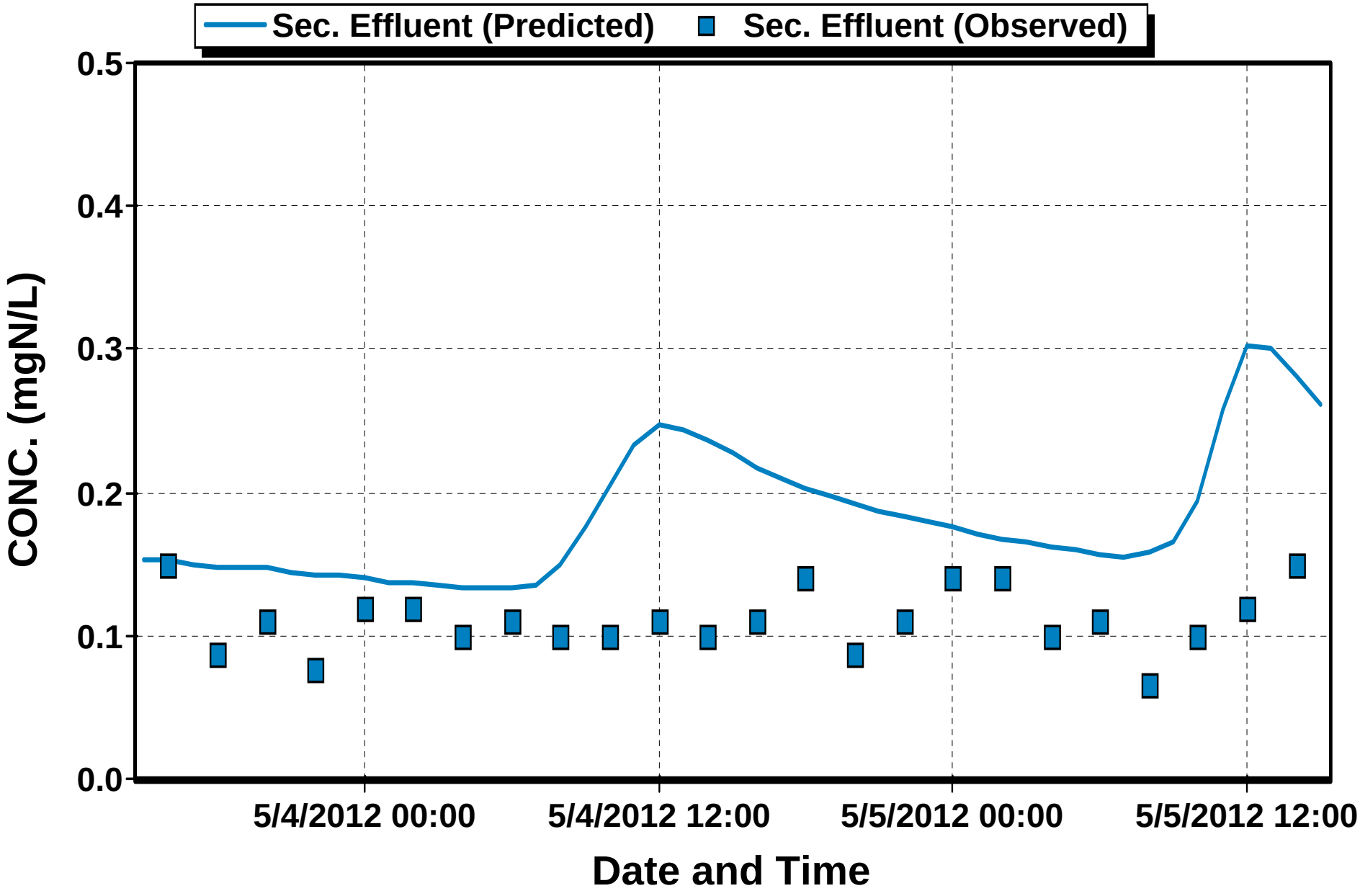
Observed vs. Predicted Diurnal Ammonia Levels - Secondary Effluent



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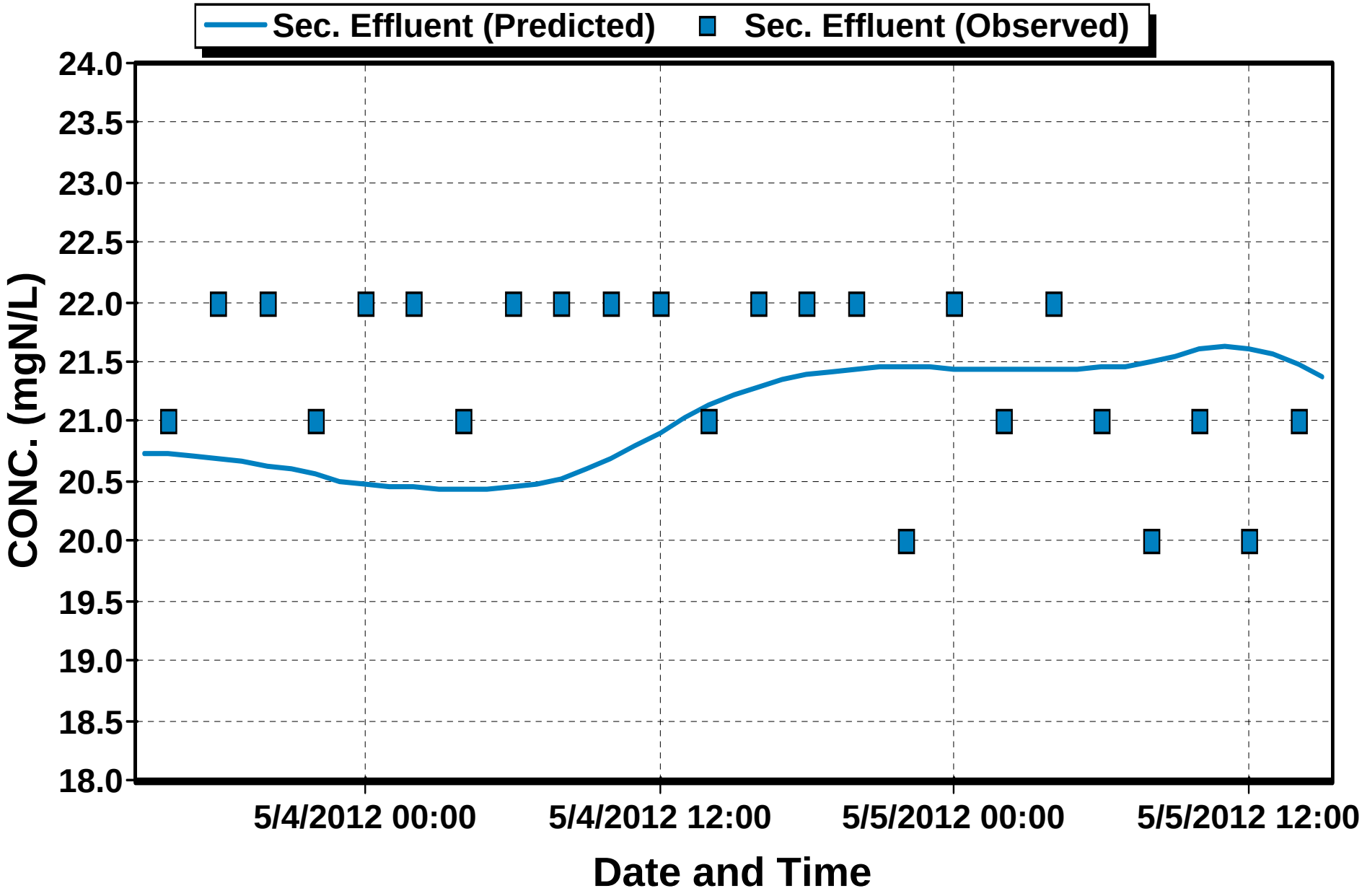
TP Observed vs. Predicted Diurnal Nitrate Levels - Secondary Effluent



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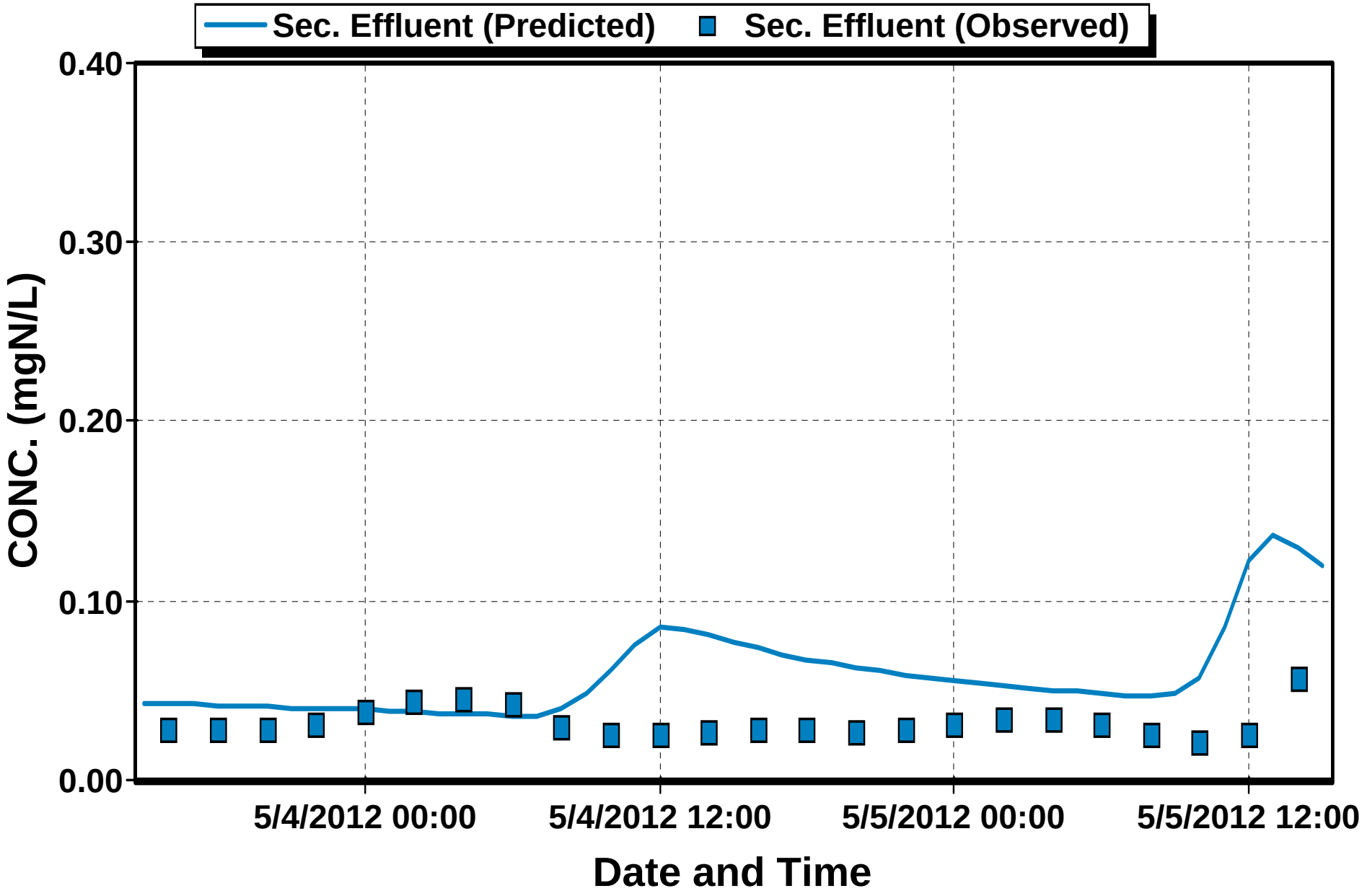
TP Observed vs. Predicted Diurnal Nitrite Levels - Secondary Effluent



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*N*TP Observed vs. Predicted Diurnal TKN Levels - Secondary Effluent

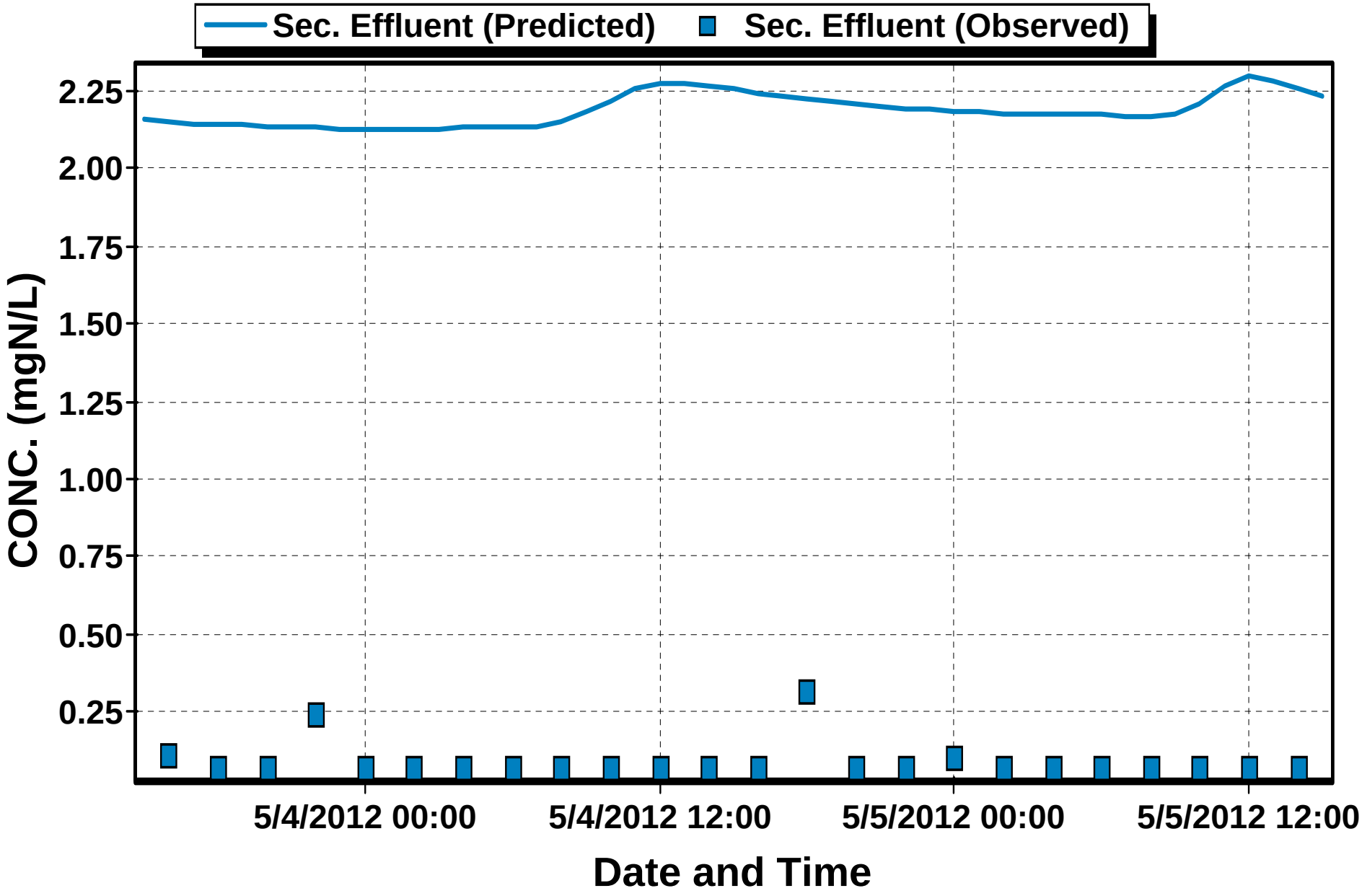

Table D-1. Two-Hour Diurnal Average Day Loads at Current Permitted Design Condition (3.0 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	14,900	800	500	100	0.0	7.3	6,000	6,200	5,500	700	2,000	400	0.0
6:00 PM	16,500	1,100	700	100	0.0	7.3	6,000	6,900	6,100	800	2,000	400	0.0
8:00 PM	13,600	800	500	100	0.0	7.3	6,000	5,600	5,000	600	2,000	400	0.0
10:00 PM	15,100	1,000	600	100	0.0	7.3	6,000	6,300	5,500	800	2,000	400	0.0
12:00 AM	9,300	600	400	100	0.0	7.3	6,000	3,900	3,400	500	2,000	400	0.0
2:00 AM	4,400	300	200	0	0.0	7.3	6,000	1,800	1,600	200	2,000	400	0.0
4:00 AM	4,200	400	300	0	0.0	7.3	6,000	1,700	1,500	200	2,000	400	0.0
6:00 AM	5,300	600	500	100	0.0	7.3	6,000	2,200	1,900	300	2,000	400	0.0
8:00 AM	11,800	1,600	1,100	200	0.0	7.3	6,000	4,900	4,400	500	2,000	400	0.0
10:00 AM	16,000	1,600	1,100	200	0.0	7.3	6,000	6,700	5,900	800	2,000	400	0.0
12:00 PM	13,900	1,000	700	100	0.0	7.3	6,000	5,800	5,100	700	2,000	400	0.0
2:00 PM	13,200	800	500	100	0.0	7.3	6,000	5,500	4,900	600	2,000	400	0.0
4:00 PM	12,000	800	500	100	0.0	7.3	6,000	5,000	4,400	600	2,000	400	0.0
6:00 PM	13,300	900	600	100	0.0	7.3	6,000	5,500	4,900	600	2,000	400	0.0
8:00 PM	14,500	900	600	100	0.0	7.3	6,000	6,000	5,300	700	2,000	400	0.0
10:00 PM	15,300	1,000	700	100	0.0	7.3	6,000	6,300	5,600	700	2,000	400	0.0
12:00 AM	10,000	700	400	100	0.0	7.3	6,000	4,100	3,700	400	2,000	400	0.0
2:00 AM	7,800	500	300	100	0.0	7.3	6,000	3,200	2,900	300	2,000	400	0.0
4:00 AM	3,800	300	200	0	0.0	7.3	6,000	1,600	1,400	200	2,000	400	0.0
6:00 AM	3,400	400	300	0	0.0	7.3	6,000	1,400	1,200	200	2,000	400	0.0
8:00 AM	7,400	1,000	800	100	0.0	7.3	6,000	3,100	2,700	400	2,000	400	0.0
10:00 AM	33,800	2,300	1,600	300	0.0	7.3	6,000	14,100	12,500	1,600	2,000	400	0.0
12:00 PM	24,500	1,300	800	100	0.0	7.3	6,000	10,200	9,000	1,200	2,000	400	0.0
2:00 PM	16,100	900	500	100	0.0	7.3	6,000	6,700	5,900	800	2,000	400	0.0
Average (Steady State Value)	12,500	900	600	100	0.0	7.3	6,000	5,200	4,600	700	2,000	400	0.0

Table D-2. Two-Hour Diurnal Max Month Loads at Current Permitted Design Condition (3.0 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	19,400	1,100	700	100	0.0	7.3	6,000	8,100	7,200	900	2,000	400	0.0
6:00 PM	21,400	1,500	1,000	100	0.0	7.3	6,000	9,000	7,900	1,100	2,000	400	0.0
8:00 PM	17,600	1,100	700	100	0.0	7.3	6,000	7,400	6,500	900	2,000	400	0.0
10:00 PM	19,500	1,300	800	100	0.0	7.3	6,000	8,200	7,200	1,000	2,000	400	0.0
12:00 AM	12,100	700	500	100	0.0	7.3	6,000	5,100	4,500	600	2,000	400	0.0
2:00 AM	5,700	500	300	0	0.0	7.3	6,000	2,400	2,100	300	2,000	400	0.0
4:00 AM	5,500	500	300	0	0.0	7.3	6,000	2,300	2,000	300	2,000	400	0.0
6:00 AM	6,800	800	600	100	0.0	7.3	6,000	2,900	2,500	400	2,000	400	0.0
8:00 AM	15,300	2,100	1,500	200	0.0	7.3	6,000	6,400	5,700	700	2,000	400	0.0
10:00 AM	20,800	2,100	1,500	200	0.0	7.3	6,000	8,700	7,700	1,000	2,000	400	0.0
12:00 PM	18,000	1,300	900	100	0.0	7.3	6,000	7,500	6,700	800	2,000	400	0.0
2:00 PM	17,100	1,100	700	100	0.0	7.3	6,000	7,200	6,300	900	2,000	400	0.0
4:00 PM	15,500	1,000	600	100	0.0	7.3	6,000	6,500	5,700	800	2,000	400	0.0
6:00 PM	17,200	1,200	800	100	0.0	7.3	6,000	7,200	6,400	800	2,000	400	0.0
8:00 PM	18,800	1,200	800	100	0.0	7.3	6,000	7,900	7,000	900	2,000	400	0.0
10:00 PM	19,800	1,400	900	100	0.0	7.3	6,000	8,300	7,300	1,000	2,000	400	0.0
12:00 AM	12,900	900	600	100	0.0	7.3	6,000	5,400	4,800	600	2,000	400	0.0
2:00 AM	10,100	600	400	100	0.0	7.3	6,000	4,200	3,700	500	2,000	400	0.0
4:00 AM	5,000	400	300	0	0.0	7.3	6,000	2,100	1,800	300	2,000	400	0.0
6:00 AM	4,400	500	400	0	0.0	7.3	6,000	1,800	1,600	200	2,000	400	0.0
8:00 AM	9,600	1,300	1,000	100	0.0	7.3	6,000	4,000	3,500	500	2,000	400	0.0
10:00 AM	43,900	3,100	2,100	300	0.0	7.3	6,000	18,400	16,200	2,200	2,000	400	0.0
12:00 PM	31,800	1,700	1,000	100	0.0	7.3	6,000	13,300	11,800	1,500	2,000	400	0.0
2:00 PM	20,800	1,200	700	100	0.0	7.3	6,000	8,700	7,700	1,000	2,000	400	0.0
Average (Steady State Value)	16,200	1,200	800	100	0.0	7.3	6,000	6,800	6,000	800	2,000	400	0.0

Table D-3. Two-Hour Diurnal Max Day Loads at Current Permitted Design Condition (3.0 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	23,900	1,400	900	200	0.0	7.3	6,000	10,000	8,700	1,300	2,000	400	0.0
6:00 PM	26,400	1,900	1,200	300	0.0	7.3	6,000	11,100	9,700	1,400	2,000	400	0.0
8:00 PM	21,700	1,400	900	200	0.0	7.3	6,000	9,100	7,900	1,200	2,000	400	0.0
10:00 PM	24,100	1,700	1,000	200	0.0	7.3	6,000	10,100	8,800	1,300	2,000	400	0.0
12:00 AM	14,900	900	600	100	0.0	7.3	6,000	6,300	5,400	900	2,000	400	0.0
2:00 AM	7,100	600	400	100	0.0	7.3	6,000	3,000	2,600	400	2,000	400	0.0
4:00 AM	6,700	600	400	100	0.0	7.3	6,000	2,800	2,500	300	2,000	400	0.0
6:00 AM	8,400	1,000	800	100	0.0	7.3	6,000	3,500	3,100	400	2,000	400	0.0
8:00 AM	18,900	2,700	1,900	400	0.0	7.3	6,000	8,000	6,900	1,100	2,000	400	0.0
10:00 AM	25,700	2,700	1,900	400	0.0	7.3	6,000	10,800	9,400	1,400	2,000	400	0.0
12:00 PM	22,200	1,600	1,200	200	0.0	7.3	6,000	9,300	8,100	1,200	2,000	400	0.0
2:00 PM	21,100	1,400	900	200	0.0	7.3	6,000	8,900	7,700	1,200	2,000	400	0.0
4:00 PM	19,100	1,300	800	200	0.0	7.3	6,000	8,000	7,000	1,000	2,000	400	0.0
6:00 PM	21,300	1,500	1,000	200	0.0	7.3	6,000	8,900	7,800	1,100	2,000	400	0.0
8:00 PM	23,200	1,500	1,000	200	0.0	7.3	6,000	9,700	8,500	1,200	2,000	400	0.0
10:00 PM	24,400	1,700	1,100	200	0.0	7.3	6,000	10,300	8,900	1,400	2,000	400	0.0
12:00 AM	15,900	1,100	700	100	0.0	7.3	6,000	6,700	5,800	900	2,000	400	0.0
2:00 AM	12,400	800	500	100	0.0	7.3	6,000	5,200	4,500	700	2,000	400	0.0
4:00 AM	6,100	500	400	100	0.0	7.3	6,000	2,600	2,200	400	2,000	400	0.0
6:00 AM	5,400	600	500	100	0.0	7.3	6,000	2,300	2,000	300	2,000	400	0.0
8:00 AM	11,800	1,700	1,300	200	0.0	7.3	6,000	5,000	4,300	700	2,000	400	0.0
10:00 AM	54,100	3,900	2,700	500	0.0	7.3	6,000	22,700	19,800	2,900	2,000	400	0.0
12:00 PM	39,200	2,100	1,300	300	0.0	7.3	6,000	16,500	14,300	2,200	2,000	400	0.0
2:00 PM	25,700	1,500	900	200	0.0	7.3	6,000	10,800	9,400	1,400	2,000	400	0.0
Average (Steady State Value)	20,000	1,500	1,000	200	0.0	7.3	6,000	8,400	7,300	1,000	2,000	400	0.0

Table D-4. Two-Hour Diurnal Average Day Loads at Future Permitted Design Condition (4.5 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	22,300	1,300	800	200	0.0	7.3	9,000	9,400	8,200	1,200	3,000	600	0.0
6:00 PM	24,700	1,800	1,100	300	0.0	7.3	9,000	10,400	9,100	1,300	3,000	600	0.0
8:00 PM	20,300	1,300	800	200	0.0	7.3	9,000	8,600	7,500	1,100	3,000	600	0.0
10:00 PM	22,500	1,600	900	200	0.0	7.3	9,000	9,500	8,300	1,200	3,000	600	0.0
12:00 AM	13,900	900	500	100	0.0	7.3	9,000	5,900	5,100	800	3,000	600	0.0
2:00 AM	6,600	500	300	100	0.0	7.3	9,000	2,800	2,400	400	3,000	600	0.0
4:00 AM	6,300	600	400	100	0.0	7.3	9,000	2,700	2,300	400	3,000	600	0.0
6:00 AM	7,900	1,000	700	100	0.0	7.3	9,000	3,300	2,900	400	3,000	600	0.0
8:00 AM	17,700	2,500	1,700	400	0.0	7.3	9,000	7,500	6,500	1,000	3,000	600	0.0
10:00 AM	24,000	2,500	1,700	400	0.0	7.3	9,000	10,100	8,900	1,200	3,000	600	0.0
12:00 PM	20,700	1,500	1,000	200	0.0	7.3	9,000	8,800	7,600	1,200	3,000	600	0.0
2:00 PM	19,800	1,300	800	200	0.0	7.3	9,000	8,400	7,300	1,100	3,000	600	0.0
4:00 PM	17,900	1,200	700	200	0.0	7.3	9,000	7,600	6,600	1,000	3,000	600	0.0
6:00 PM	19,900	1,400	900	200	0.0	7.3	9,000	8,400	7,300	1,100	3,000	600	0.0
8:00 PM	21,700	1,400	900	200	0.0	7.3	9,000	9,200	8,000	1,200	3,000	600	0.0
10:00 PM	22,800	1,600	1,000	200	0.0	7.3	9,000	9,600	8,400	1,200	3,000	600	0.0
12:00 AM	14,900	1,000	600	100	0.0	7.3	9,000	6,300	5,500	800	3,000	600	0.0
2:00 AM	11,600	700	500	100	0.0	7.3	9,000	4,900	4,300	600	3,000	600	0.0
4:00 AM	5,700	500	300	100	0.0	7.3	9,000	2,400	2,100	300	3,000	600	0.0
6:00 AM	5,100	600	400	100	0.0	7.3	9,000	2,100	1,900	200	3,000	600	0.0
8:00 AM	11,100	1,600	1,200	200	0.0	7.3	9,000	4,700	4,100	600	3,000	600	0.0
10:00 AM	50,600	3,600	2,400	500	0.0	7.3	9,000	21,400	18,700	2,700	3,000	600	0.0
12:00 PM	36,700	2,000	1,200	300	0.0	7.3	9,000	15,500	13,500	2,000	3,000	600	0.0
2:00 PM	24,000	1,400	800	200	0.0	7.3	9,000	10,100	8,900	1,200	3,000	600	0.0
Average (Steady State Value)	18,700	1,400	900	200	0.0	7.3	9,000	7,900	6,900	1,000	3,000	600	0.0

Table D-5. Two-Hour Diurnal Max Month Loads at Future Permitted Design Condition (4.5 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	29,000	1,600	1,000	200	0.0	7.3	9,000	12,200	10,600	1,600	3,000	600	0.0
6:00 PM	32,100	2,300	1,400	300	0.0	7.3	9,000	13,500	11,800	1,700	3,000	600	0.0
8:00 PM	26,400	1,700	1,100	200	0.0	7.3	9,000	11,100	9,700	1,400	3,000	600	0.0
10:00 PM	29,300	2,000	1,200	200	0.0	7.3	9,000	12,300	10,700	1,600	3,000	600	0.0
12:00 AM	18,100	1,100	700	100	0.0	7.3	9,000	7,600	6,600	1,000	3,000	600	0.0
2:00 AM	8,600	700	400	100	0.0	7.3	9,000	3,600	3,200	400	3,000	600	0.0
4:00 AM	8,200	700	500	100	0.0	7.3	9,000	3,400	3,000	400	3,000	600	0.0
6:00 AM	10,200	1,200	900	100	0.0	7.3	9,000	4,300	3,700	600	3,000	600	0.0
8:00 AM	23,000	3,200	2,300	400	0.0	7.3	9,000	9,700	8,400	1,300	3,000	600	0.0
10:00 AM	31,200	3,200	2,200	400	0.0	7.3	9,000	13,100	11,400	1,700	3,000	600	0.0
12:00 PM	26,900	1,900	1,400	200	0.0	7.3	9,000	11,300	9,900	1,400	3,000	600	0.0
2:00 PM	25,700	1,600	1,100	200	0.0	7.3	9,000	10,800	9,400	1,400	3,000	600	0.0
4:00 PM	23,200	1,500	1,000	200	0.0	7.3	9,000	9,800	8,500	1,300	3,000	600	0.0
6:00 PM	25,800	1,800	1,200	200	0.0	7.3	9,000	10,800	9,500	1,300	3,000	600	0.0
8:00 PM	28,200	1,800	1,100	200	0.0	7.3	9,000	11,800	10,300	1,500	3,000	600	0.0
10:00 PM	29,700	2,000	1,300	200	0.0	7.3	9,000	12,500	10,900	1,600	3,000	600	0.0
12:00 AM	19,400	1,300	800	100	0.0	7.3	9,000	8,100	7,100	1,000	3,000	600	0.0
2:00 AM	15,100	900	700	100	0.0	7.3	9,000	6,300	5,500	800	3,000	600	0.0
4:00 AM	7,400	600	400	100	0.0	7.3	9,000	3,100	2,700	400	3,000	600	0.0
6:00 AM	6,600	700	600	100	0.0	7.3	9,000	2,800	2,400	400	3,000	600	0.0
8:00 AM	14,400	2,000	1,600	200	0.0	7.3	9,000	6,000	5,300	700	3,000	600	0.0
10:00 AM	65,800	4,700	3,200	500	0.0	7.3	9,000	27,600	24,100	3,500	3,000	600	0.0
12:00 PM	47,600	2,500	1,600	300	0.0	7.3	9,000	20,000	17,400	2,600	3,000	600	0.0
2:00 PM	31,200	1,800	1,100	200	0.0	7.3	9,000	13,100	11,400	1,700	3,000	600	0.0
Average (Steady State Value)	24,300	1,800	1,200	200	0.0	7.3	9,000	10,200	8,900	1,300	3,000	600	0.0

Table D-6. Two-Hour Diurnal Max Day Loads at Future Permitted Design Condition (4.5 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	35,700	2,000	1,300	300	0.0	7.3	9,000	15,100	13,100	2,000	3,000	600	0.0
6:00 PM	39,500	2,800	1,800	400	0.0	7.3	9,000	16,700	14,500	2,200	3,000	600	0.0
8:00 PM	32,500	2,000	1,300	300	0.0	7.3	9,000	13,700	12,000	1,700	3,000	600	0.0
10:00 PM	36,000	2,500	1,400	300	0.0	7.3	9,000	15,200	13,200	2,000	3,000	600	0.0
12:00 AM	22,300	1,400	900	200	0.0	7.3	9,000	9,400	8,200	1,200	3,000	600	0.0
2:00 AM	10,600	800	500	100	0.0	7.3	9,000	4,500	3,900	600	3,000	600	0.0
4:00 AM	10,100	900	600	100	0.0	7.3	9,000	4,200	3,700	500	3,000	600	0.0
6:00 AM	12,600	1,500	1,100	200	0.0	7.3	9,000	5,300	4,600	700	3,000	600	0.0
8:00 AM	28,300	3,900	2,900	500	0.0	7.3	9,000	11,900	10,400	1,500	3,000	600	0.0
10:00 AM	38,400	3,900	2,800	500	0.0	7.3	9,000	16,200	14,100	2,100	3,000	600	0.0
12:00 PM	33,100	2,400	1,700	300	0.0	7.3	9,000	14,000	12,200	1,800	3,000	600	0.0
2:00 PM	31,600	2,000	1,300	300	0.0	7.3	9,000	13,300	11,600	1,700	3,000	600	0.0
4:00 PM	28,600	1,900	1,200	300	0.0	7.3	9,000	12,000	10,500	1,500	3,000	600	0.0
6:00 PM	31,800	2,200	1,500	300	0.0	7.3	9,000	13,400	11,700	1,700	3,000	600	0.0
8:00 PM	34,700	2,200	1,400	300	0.0	7.3	9,000	14,600	12,800	1,800	3,000	600	0.0
10:00 PM	36,500	2,500	1,700	300	0.0	7.3	9,000	15,400	13,400	2,000	3,000	600	0.0
12:00 AM	23,800	1,600	1,000	200	0.0	7.3	9,000	10,000	8,800	1,200	3,000	600	0.0
2:00 AM	18,600	1,200	800	200	0.0	7.3	9,000	7,800	6,800	1,000	3,000	600	0.0
4:00 AM	9,200	800	500	100	0.0	7.3	9,000	3,900	3,400	500	3,000	600	0.0
6:00 AM	8,100	900	700	100	0.0	7.3	9,000	3,400	3,000	400	3,000	600	0.0
8:00 AM	17,700	2,400	2,000	300	0.0	7.3	9,000	7,500	6,500	1,000	3,000	600	0.0
10:00 AM	80,900	5,700	4,000	800	0.0	7.3	9,000	34,100	29,800	4,300	3,000	600	0.0
12:00 PM	58,600	3,100	1,900	400	0.0	7.3	9,000	24,700	21,600	3,100	3,000	600	0.0
2:00 PM	38,400	2,200	1,400	300	0.0	7.3	9,000	16,200	14,100	2,100	3,000	600	0.0
Average (Steady State Value)	29,900	2,200	1,500	300	0.0	7.3	9,000	12,600	11,000	1,600	3,000	600	0.0

Table D-7. Two-Hour Diurnal Average Day Loads at Ultimate Buildout Design Condition (6.0 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	22,300	1,300	800	200	0.0	7.3	12,000	9,400	8,200	1,200	4,000	800	0.0
6:00 PM	24,700	1,800	1,100	300	0.0	7.3	12,000	10,400	9,100	1,300	4,000	800	0.0
8:00 PM	20,300	1,300	800	200	0.0	7.3	12,000	8,600	7,500	1,100	4,000	800	0.0
10:00 PM	22,500	1,600	900	200	0.0	7.3	12,000	9,500	8,300	1,200	4,000	800	0.0
12:00 AM	13,900	900	500	100	0.0	7.3	12,000	5,900	5,100	800	4,000	800	0.0
2:00 AM	6,600	500	300	100	0.0	7.3	12,000	2,800	2,400	400	4,000	800	0.0
4:00 AM	6,300	600	400	100	0.0	7.3	12,000	2,700	2,300	400	4,000	800	0.0
6:00 AM	7,900	1,000	700	100	0.0	7.3	12,000	3,300	2,900	400	4,000	800	0.0
8:00 AM	17,700	2,500	1,700	400	0.0	7.3	12,000	7,500	6,500	1,000	4,000	800	0.0
10:00 AM	24,000	2,500	1,700	400	0.0	7.3	12,000	10,100	8,900	1,200	4,000	800	0.0
12:00 PM	20,700	1,500	1,000	200	0.0	7.3	12,000	8,800	7,600	1,200	4,000	800	0.0
2:00 PM	19,800	1,300	800	200	0.0	7.3	12,000	8,400	7,300	1,100	4,000	800	0.0
4:00 PM	17,900	1,200	700	200	0.0	7.3	12,000	7,600	6,600	1,000	4,000	800	0.0
6:00 PM	19,900	1,400	900	200	0.0	7.3	12,000	8,400	7,300	1,100	4,000	800	0.0
8:00 PM	21,700	1,400	900	200	0.0	7.3	12,000	9,200	8,000	1,200	4,000	800	0.0
10:00 PM	22,800	1,600	1,000	200	0.0	7.3	12,000	9,600	8,400	1,200	4,000	800	0.0
12:00 AM	14,900	1,000	600	100	0.0	7.3	12,000	6,300	5,500	800	4,000	800	0.0
2:00 AM	11,600	700	500	100	0.0	7.3	12,000	4,900	4,300	600	4,000	800	0.0
4:00 AM	5,700	500	300	100	0.0	7.3	12,000	2,400	2,100	300	4,000	800	0.0
6:00 AM	5,100	600	400	100	0.0	7.3	12,000	2,100	1,900	200	4,000	800	0.0
8:00 AM	11,100	1,600	1,200	200	0.0	7.3	12,000	4,700	4,100	600	4,000	800	0.0
10:00 AM	50,600	3,600	2,400	500	0.0	7.3	12,000	21,400	18,700	2,700	4,000	800	0.0
12:00 PM	36,700	2,000	1,200	300	0.0	7.3	12,000	15,500	13,500	2,000	4,000	800	0.0
2:00 PM	24,000	1,400	800	200	0.0	7.3	12,000	10,100	8,900	1,200	4,000	800	0.0
Average (Steady State Value)	24,900	1,800	1,300	200	0.0	7.3	12,000	10,500	9,200	1,300	4,000	800	0.0

Table D-8. Two-Hour Diurnal Max Month Loads at Ultimate Buildout Design Condition (6.0 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	29,000	1,600	1,000	200	0.0	7.3	12,000	12,200	10,600	1,600	5,200	1,000	0.0
6:00 PM	32,100	2,300	1,400	300	0.0	7.3	12,000	13,500	11,800	1,700	5,200	1,000	0.0
8:00 PM	26,400	1,700	1,100	200	0.0	7.3	12,000	11,100	9,700	1,400	5,200	1,000	0.0
10:00 PM	29,300	2,000	1,200	200	0.0	7.3	12,000	12,300	10,700	1,600	5,200	1,000	0.0
12:00 AM	18,100	1,100	700	100	0.0	7.3	12,000	7,600	6,600	1,000	5,200	1,000	0.0
2:00 AM	8,600	700	400	100	0.0	7.3	12,000	3,600	3,200	400	5,200	1,000	0.0
4:00 AM	8,200	700	500	100	0.0	7.3	12,000	3,400	3,000	400	5,200	1,000	0.0
6:00 AM	10,200	1,200	900	100	0.0	7.3	12,000	4,300	3,700	600	5,200	1,000	0.0
8:00 AM	23,000	3,200	2,300	400	0.0	7.3	12,000	9,700	8,400	1,300	5,200	1,000	0.0
10:00 AM	31,200	3,200	2,200	400	0.0	7.3	12,000	13,100	11,400	1,700	5,200	1,000	0.0
12:00 PM	26,900	1,900	1,400	200	0.0	7.3	12,000	11,300	9,900	1,400	5,200	1,000	0.0
2:00 PM	25,700	1,600	1,100	200	0.0	7.3	12,000	10,800	9,400	1,400	5,200	1,000	0.0
4:00 PM	23,200	1,500	1,000	200	0.0	7.3	12,000	9,800	8,500	1,300	5,200	1,000	0.0
6:00 PM	25,800	1,800	1,200	200	0.0	7.3	12,000	10,800	9,500	1,300	5,200	1,000	0.0
8:00 PM	28,200	1,800	1,100	200	0.0	7.3	12,000	11,800	10,300	1,500	5,200	1,000	0.0
10:00 PM	29,700	2,000	1,300	200	0.0	7.3	12,000	12,500	10,900	1,600	5,200	1,000	0.0
12:00 AM	19,400	1,300	800	100	0.0	7.3	12,000	8,100	7,100	1,000	5,200	1,000	0.0
2:00 AM	15,100	900	700	100	0.0	7.3	12,000	6,300	5,500	800	5,200	1,000	0.0
4:00 AM	7,400	600	400	100	0.0	7.3	12,000	3,100	2,700	400	5,200	1,000	0.0
6:00 AM	6,600	700	600	100	0.0	7.3	12,000	2,800	2,400	400	5,200	1,000	0.0
8:00 AM	14,400	2,000	1,600	200	0.0	7.3	12,000	6,000	5,300	700	5,200	1,000	0.0
10:00 AM	65,800	4,700	3,200	500	0.0	7.3	12,000	27,600	24,100	3,500	5,200	1,000	0.0
12:00 PM	47,600	2,500	1,600	300	0.0	7.3	12,000	20,000	17,400	2,600	5,200	1,000	0.0
2:00 PM	31,200	1,800	1,100	200	0.0	7.3	12,000	13,100	11,400	1,700	5,200	1,000	0.0
Average (Steady State Value)	32,400	2,400	1,600	300	0.0	7.3	12,000	13,600	11,900	1,700	5,200	1,000	0.0

Table D-9. Two-Hour Diurnal Max Day Loads at Ultimate Buildout Design Condition (6.0 mgd ADWF)

Time hr	T-COD ppd	TKN-total ppd	NH4-N ppd	T-Phosphorus ppd	Nitrate ppd	pH s.u.	Alkalinity (as CaCO3) ppd	TSS ppd	VSS ppd	Inorganic SS ppd	Calcium ppd	Magnesium ppd	Dissolved Oxygen ppd
4:00 PM	35,700	2,000	1,300	300	0.0	7.3	12,000	15,100	13,100	2,000	6,400	1,200	0.0
6:00 PM	39,500	2,800	1,800	400	0.0	7.3	12,000	16,700	14,500	2,200	6,400	1,200	0.0
8:00 PM	32,500	2,000	1,300	300	0.0	7.3	12,000	13,700	12,000	1,700	6,400	1,200	0.0
10:00 PM	36,000	2,500	1,400	300	0.0	7.3	12,000	15,200	13,200	2,000	6,400	1,200	0.0
12:00 AM	22,300	1,400	900	200	0.0	7.3	12,000	9,400	8,200	1,200	6,400	1,200	0.0
2:00 AM	10,600	800	500	100	0.0	7.3	12,000	4,500	3,900	600	6,400	1,200	0.0
4:00 AM	10,100	900	600	100	0.0	7.3	12,000	4,200	3,700	500	6,400	1,200	0.0
6:00 AM	12,600	1,500	1,100	200	0.0	7.3	12,000	5,300	4,600	700	6,400	1,200	0.0
8:00 AM	28,300	3,900	2,900	500	0.0	7.3	12,000	11,900	10,400	1,500	6,400	1,200	0.0
10:00 AM	38,400	3,900	2,800	500	0.0	7.3	12,000	16,200	14,100	2,100	6,400	1,200	0.0
12:00 PM	33,100	2,400	1,700	300	0.0	7.3	12,000	14,000	12,200	1,800	6,400	1,200	0.0
2:00 PM	31,600	2,000	1,300	300	0.0	7.3	12,000	13,300	11,600	1,700	6,400	1,200	0.0
4:00 PM	28,600	1,900	1,200	300	0.0	7.3	12,000	12,000	10,500	1,500	6,400	1,200	0.0
6:00 PM	31,800	2,200	1,500	300	0.0	7.3	12,000	13,400	11,700	1,700	6,400	1,200	0.0
8:00 PM	34,700	2,200	1,400	300	0.0	7.3	12,000	14,600	12,800	1,800	6,400	1,200	0.0
10:00 PM	36,500	2,500	1,700	300	0.0	7.3	12,000	15,400	13,400	2,000	6,400	1,200	0.0
12:00 AM	23,800	1,600	1,000	200	0.0	7.3	12,000	10,000	8,800	1,200	6,400	1,200	0.0
2:00 AM	18,600	1,200	800	200	0.0	7.3	12,000	7,800	6,800	1,000	6,400	1,200	0.0
4:00 AM	9,200	800	500	100	0.0	7.3	12,000	3,900	3,400	500	6,400	1,200	0.0
6:00 AM	8,100	900	700	100	0.0	7.3	12,000	3,400	3,000	400	6,400	1,200	0.0
8:00 AM	17,700	2,400	2,000	300	0.0	7.3	12,000	7,500	6,500	1,000	6,400	1,200	0.0
10:00 AM	80,900	5,700	4,000	800	0.0	7.3	12,000	34,100	29,800	4,300	6,400	1,200	0.0
12:00 PM	58,600	3,100	1,900	400	0.0	7.3	12,000	24,700	21,600	3,100	6,400	1,200	0.0
2:00 PM	38,400	2,200	1,400	300	0.0	7.3	12,000	16,200	14,100	2,100	6,400	1,200	0.0
Average (Steady State Value)	39,900	2,900	2,000	400	0.0	7.3	12,000	16,800	14,700	2,100	6,400	1,200	0.0

Table E-1. Two-Hour Diurnal Concentrations for ADWF and ADL Conditions

Time hr	T-COD mg/L	TKN-total mg/L	NH4-N mg/L	T-Phosphorus mg/L	Nitrate mg/L as N	pH s.u.	Alkalinity mmol/L	TSS mg/L	VSS mg/L	Inorganic SS mg/L	Calcium mg/L	Magnesium mg/L	Dissolved Oxygen mg/L
4:00 PM	593	32	20	4.0	0.0	7.3	4.78	247	219	28	80	15	0.0
6:00 PM	514	34	22	3.1	0.0	7.3	4.78	215	190	25	80	15	0.0
8:00 PM	544	32	20	4.0	0.0	7.3	4.78	224	200	24	80	15	0.0
10:00 PM	514	34	20	3.4	0.0	7.3	4.78	215	187	27	80	15	0.0
12:00 AM	569	37	24	6.1	0.0	7.3	4.78	239	208	31	80	15	0.0
2:00 AM	414	28	19	0.0	0.0	7.3	4.78	170	151	19	80	15	0.0
4:00 AM	341	33	24	0.0	0.0	7.3	4.78	138	122	16	80	15	0.0
6:00 AM	230	26	22	4.3	0.0	7.3	4.78	95	82	13	80	15	0.0
8:00 AM	369	50	34	6.3	0.0	7.3	4.78	153	138	16	80	15	0.0
10:00 AM	547	55	38	6.8	0.0	7.3	4.78	229	202	27	80	15	0.0
12:00 PM	516	37	26	3.7	0.0	7.3	4.78	215	189	26	80	15	0.0
2:00 PM	491	30	19	3.7	0.0	7.3	4.78	205	182	22	80	15	0.0
4:00 PM	440	29	18	3.7	0.0	7.3	4.78	183	161	22	80	15	0.0
6:00 PM	466	32	21	3.5	0.0	7.3	4.78	193	172	21	80	15	0.0
8:00 PM	544	34	23	3.8	0.0	7.3	4.78	225	199	26	80	15	0.0
10:00 PM	527	34	24	3.4	0.0	7.3	4.78	217	193	24	80	15	0.0
12:00 AM	494	35	20	4.9	0.0	7.3	4.78	203	183	20	80	15	0.0
2:00 AM	534	34	21	6.8	0.0	7.3	4.78	219	199	21	80	15	0.0
4:00 AM	325	26	17	0.0	0.0	7.3	4.78	137	120	17	80	15	0.0
6:00 AM	224	26	20	0.0	0.0	7.3	4.78	92	79	13	80	15	0.0
8:00 AM	245	33	26	3.3	0.0	7.3	4.78	103	89	13	80	15	0.0
10:00 AM	848	58	40	7.5	0.0	7.3	4.78	354	314	40	80	15	0.0
12:00 PM	676	36	22	2.8	0.0	7.3	4.78	281	248	33	80	15	0.0
2:00 PM	503	28	16	3.1	0.0	7.3	4.78	210	185	25	80	15	0.0

Table E-2. Two-Hour Diurnal Concentrations for ADWF and MML Conditions

Time hr	T-COD mg/L	TKN-total mg/L	NH4-N mg/L	T-Phosphorus mg/L	Nitrate mg/L as N	pH s.u.	Alkalinity mmol/L	TSS mg/L	VSS mg/L	Inorganic SS mg/L	Calcium mg/L	Magnesium mg/L	Dissolved Oxygen mg/L
4:00 PM	772	44	28	4.0	0.0	7.3	4.78	322	286	36	80	15	0.0
6:00 PM	666	47	31	3.1	0.0	7.3	4.78	280	246	34	80	15	0.0
8:00 PM	704	44	28	4.0	0.0	7.3	4.78	296	260	36	80	15	0.0
10:00 PM	664	44	27	3.4	0.0	7.3	4.78	279	245	34	80	15	0.0
12:00 AM	740	43	31	6.1	0.0	7.3	4.78	312	275	37	80	15	0.0
2:00 AM	537	47	28	0.0	0.0	7.3	4.78	226	198	28	80	15	0.0
4:00 AM	447	41	24	0.0	0.0	7.3	4.78	187	163	24	80	15	0.0
6:00 AM	295	35	26	4.3	0.0	7.3	4.78	126	108	17	80	15	0.0
8:00 AM	478	66	47	6.3	0.0	7.3	4.78	200	178	22	80	15	0.0
10:00 AM	711	72	51	6.8	0.0	7.3	4.78	297	263	34	80	15	0.0
12:00 PM	668	48	33	3.7	0.0	7.3	4.78	278	249	30	80	15	0.0
2:00 PM	636	41	26	3.7	0.0	7.3	4.78	268	234	33	80	15	0.0
4:00 PM	568	37	22	3.7	0.0	7.3	4.78	238	209	29	80	15	0.0
6:00 PM	603	42	28	3.5	0.0	7.3	4.78	252	224	28	80	15	0.0
8:00 PM	705	45	30	3.8	0.0	7.3	4.78	296	263	34	80	15	0.0
10:00 PM	682	48	31	3.4	0.0	7.3	4.78	286	252	34	80	15	0.0
12:00 AM	637	44	30	4.9	0.0	7.3	4.78	267	237	30	80	15	0.0
2:00 AM	691	41	27	6.8	0.0	7.3	4.78	288	253	34	80	15	0.0
4:00 AM	428	34	26	0.0	0.0	7.3	4.78	180	154	26	80	15	0.0
6:00 AM	290	33	26	0.0	0.0	7.3	4.78	119	106	13	80	15	0.0
8:00 AM	318	43	33	3.3	0.0	7.3	4.78	132	116	17	80	15	0.0
10:00 AM	1101	78	53	7.5	0.0	7.3	4.78	462	406	55	80	15	0.0
12:00 PM	877	47	28	2.8	0.0	7.3	4.78	367	325	41	80	15	0.0
2:00 PM	650	38	22	3.1	0.0	7.3	4.78	272	241	31	80	15	0.0

Table E-3. Two-Hour Diurnal Concentrations for ADMMF and MML Conditions

Time hr	T-COD mg/L	TKN-total mg/L	NH4-N mg/L	T-Phosphorus mg/L	Nitrate mg/L as N	pH s.u.	Alkalinity mmol/L	TSS mg/L	VSS mg/L	Inorganic SS mg/L	Calcium mg/L	Magnesium mg/L	Dissolved Oxygen mg/L
4:00 PM	594	34	21	3.1	0.0	7.3	4.78	248	220	28	80	15	0.0
6:00 PM	513	36	24	2.4	0.0	7.3	4.78	216	189	26	80	15	0.0
8:00 PM	541	34	22	3.1	0.0	7.3	4.78	228	200	28	80	15	0.0
10:00 PM	511	34	21	2.6	0.0	7.3	4.78	215	189	26	80	15	0.0
12:00 AM	569	33	24	4.7	0.0	7.3	4.78	240	212	28	80	15	0.0
2:00 AM	413	36	22	0.0	0.0	7.3	4.78	174	152	22	80	15	0.0
4:00 AM	344	31	19	0.0	0.0	7.3	4.78	144	125	19	80	15	0.0
6:00 AM	227	27	20	3.3	0.0	7.3	4.78	97	83	13	80	15	0.0
8:00 AM	368	51	36	4.8	0.0	7.3	4.78	154	137	17	80	15	0.0
10:00 AM	547	55	39	5.3	0.0	7.3	4.78	229	203	26	80	15	0.0
12:00 PM	514	37	26	2.9	0.0	7.3	4.78	214	191	23	80	15	0.0
2:00 PM	490	31	20	2.9	0.0	7.3	4.78	206	180	26	80	15	0.0
4:00 PM	437	28	17	2.8	0.0	7.3	4.78	183	161	23	80	15	0.0
6:00 PM	464	32	22	2.7	0.0	7.3	4.78	194	173	22	80	15	0.0
8:00 PM	543	35	23	2.9	0.0	7.3	4.78	228	202	26	80	15	0.0
10:00 PM	525	37	24	2.7	0.0	7.3	4.78	220	194	27	80	15	0.0
12:00 AM	490	34	23	3.8	0.0	7.3	4.78	205	182	23	80	15	0.0
2:00 AM	532	32	21	5.3	0.0	7.3	4.78	221	195	26	80	15	0.0
4:00 AM	329	26	20	0.0	0.0	7.3	4.78	138	118	20	80	15	0.0
6:00 AM	223	25	20	0.0	0.0	7.3	4.78	91	81	10	80	15	0.0
8:00 AM	245	33	25	2.5	0.0	7.3	4.78	102	89	13	80	15	0.0
10:00 AM	847	60	41	5.8	0.0	7.3	4.78	355	313	42	80	15	0.0
12:00 PM	675	36	21	2.1	0.0	7.3	4.78	282	250	32	80	15	0.0
2:00 PM	500	29	17	2.4	0.0	7.3	4.78	209	185	24	80	15	0.0

Table E-4. Two-Hour Diurnal Concentrations for ADMMF and MDL Conditions

Time hr	T-COD mg/L	TKN-total mg/L	NH4-N mg/L	T-Phosphorus mg/L	Nitrate mg/L as N	pH s.u.	Alkalinity mmol/L	TSS mg/L	VSS mg/L	Inorganic SS mg/L	Calcium mg/L	Magnesium mg/L	Dissolved Oxygen mg/L
4:00 PM	731	43	28	6.1	0.0	7.3	4.78	306	266	40	80	15	0.0
6:00 PM	632	46	29	7.2	0.0	7.3	4.78	266	232	34	80	15	0.0
8:00 PM	667	43	28	6.2	0.0	7.3	4.78	280	243	37	80	15	0.0
10:00 PM	631	45	26	5.2	0.0	7.3	4.78	265	230	34	80	15	0.0
12:00 AM	701	42	28	4.7	0.0	7.3	4.78	297	254	42	80	15	0.0
2:00 AM	514	43	29	7.2	0.0	7.3	4.78	217	188	29	80	15	0.0
4:00 AM	419	38	25	6.3	0.0	7.3	4.78	175	156	19	80	15	0.0
6:00 AM	280	33	27	3.3	0.0	7.3	4.78	117	103	13	80	15	0.0
8:00 AM	455	65	46	9.6	0.0	7.3	4.78	192	166	26	80	15	0.0
10:00 AM	676	71	50	10.5	0.0	7.3	4.78	284	247	37	80	15	0.0
12:00 PM	634	46	34	5.7	0.0	7.3	4.78	266	231	34	80	15	0.0
2:00 PM	604	40	26	5.7	0.0	7.3	4.78	255	220	34	80	15	0.0
4:00 PM	539	37	23	5.6	0.0	7.3	4.78	226	197	28	80	15	0.0
6:00 PM	575	40	27	5.4	0.0	7.3	4.78	240	210	30	80	15	0.0
8:00 PM	670	43	29	5.8	0.0	7.3	4.78	280	245	35	80	15	0.0
10:00 PM	647	45	29	5.3	0.0	7.3	4.78	273	236	37	80	15	0.0
12:00 AM	604	42	27	3.8	0.0	7.3	4.78	255	220	34	80	15	0.0
2:00 AM	653	42	26	5.3	0.0	7.3	4.78	274	237	37	80	15	0.0
4:00 AM	401	33	26	6.6	0.0	7.3	4.78	171	145	26	80	15	0.0
6:00 AM	274	30	25	5.1	0.0	7.3	4.78	117	101	15	80	15	0.0
8:00 AM	301	43	33	5.1	0.0	7.3	4.78	127	110	18	80	15	0.0
10:00 AM	1044	75	52	9.6	0.0	7.3	4.78	438	382	56	80	15	0.0
12:00 PM	832	45	28	6.4	0.0	7.3	4.78	350	303	47	80	15	0.0
2:00 PM	618	36	22	4.8	0.0	7.3	4.78	260	226	34	80	15	0.0

Table E-5. Two-Hour Diurnal Concentrations for MDDWF and MDL Conditions

Time hr	T-COD mg/L	TKN-total mg/L	NH4-N mg/L	T-Phosphorus mg/L	Nitrate mg/L as N	pH s.u.	Alkalinity mmol/L	TSS mg/L	VSS mg/L	Inorganic SS mg/L	Calcium mg/L	Magnesium mg/L	Dissolved Oxygen mg/L
4:00 PM	559	33	21	4.7	0.0	7.3	4.78	234	204	30	80	15	0.0
6:00 PM	484	35	22	5.5	0.0	7.3	4.78	203	178	26	80	15	0.0
8:00 PM	510	33	21	4.7	0.0	7.3	4.78	214	186	28	80	15	0.0
10:00 PM	483	34	20	4.0	0.0	7.3	4.78	202	176	26	80	15	0.0
12:00 AM	536	32	22	3.6	0.0	7.3	4.78	227	194	32	80	15	0.0
2:00 AM	393	33	22	5.5	0.0	7.3	4.78	166	144	22	80	15	0.0
4:00 AM	320	29	19	4.8	0.0	7.3	4.78	134	119	14	80	15	0.0
6:00 AM	214	25	20	2.5	0.0	7.3	4.78	89	79	10	80	15	0.0
8:00 AM	348	50	35	7.4	0.0	7.3	4.78	147	127	20	80	15	0.0
10:00 AM	517	54	38	8.0	0.0	7.3	4.78	217	189	28	80	15	0.0
12:00 PM	485	35	26	4.4	0.0	7.3	4.78	203	177	26	80	15	0.0
2:00 PM	462	31	20	4.4	0.0	7.3	4.78	195	169	26	80	15	0.0
4:00 PM	412	28	17	4.3	0.0	7.3	4.78	172	151	22	80	15	0.0
6:00 PM	439	31	21	4.1	0.0	7.3	4.78	184	161	23	80	15	0.0
8:00 PM	512	33	22	4.4	0.0	7.3	4.78	214	188	26	80	15	0.0
10:00 PM	495	34	22	4.1	0.0	7.3	4.78	209	180	28	80	15	0.0
12:00 AM	462	32	20	2.9	0.0	7.3	4.78	195	169	26	80	15	0.0
2:00 AM	499	32	20	4.0	0.0	7.3	4.78	209	181	28	80	15	0.0
4:00 AM	307	25	20	5.0	0.0	7.3	4.78	131	111	20	80	15	0.0
6:00 AM	210	23	19	3.9	0.0	7.3	4.78	89	78	12	80	15	0.0
8:00 AM	230	33	25	3.9	0.0	7.3	4.78	97	84	14	80	15	0.0
10:00 AM	798	58	40	7.4	0.0	7.3	4.78	335	292	43	80	15	0.0
12:00 PM	636	34	21	4.9	0.0	7.3	4.78	268	232	36	80	15	0.0
2:00 PM	473	28	17	3.7	0.0	7.3	4.78	199	173	26	80	15	0.0

APPENDIX C

Arsenic Management Plan

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TECHNICAL MEMORANDUM NO. 9

DATE: December 21, 2012 Project No.: 175-00-12-36.006

TO: Mark Clarkson, City of Galt Public Works Department

FROM: Charles Hardy, R.C.E. #71015

REVIEWED BY: Jeff Pelz, R.C.E. #46088

SUBJECT: City of Galt WWTP Arsenic Management Plan

The purpose of this technical memorandum (TM) is to provide an action plan for reducing arsenic levels in the effluent of the City of Galt (City) Wastewater Treatment Plant (WWTP). Arsenic levels in the WWTP effluent consistently exceed the 10 µg/L water quality criteria applicable to the WWTP discharge. There are a variety of ways that arsenic levels can be reduced in the WWTP effluent, including source control upstream of the WWTP and enhanced removal at the WWTP. Potential management strategies for significantly reducing arsenic levels in the WWTP effluent below the permitted level by the September 1, 2015 compliance date are discussed. This TM specifically addresses the following topics:

- Background
- Evaluation of Arsenic Management Alternatives
- Recommended Arsenic Management Plan

BACKGROUND

This section provides background information relevant to management of arsenic in the WWTP influent and effluent, including the following specific topics:

- Regulatory requirements
- Arsenic levels in the WWTP influent and effluent
- Sources of arsenic in the WWTP influent
- Arsenic mass balance for existing conditions

Regulatory Requirements

Waste Discharge Requirements (WDRs), Order No R5-2010-0099 (NPDES No. CA0081434) – adopted by the Central Valley Regional Water Quality Control Board (Regional Board) on September 23, 2010 – permit discharge of treated effluent from the WWTP to both Skunk Creek and local groundwater. The WDRs include an average monthly effluent limitation (AMEL) and a groundwater limitation for arsenic of 10 micrograms per liter (µg/L).

In addition, the Regional Board adopted Time Schedule Order No. R5-2010-0100-001 (TSO) on December 15, 2010, implementing interim effluent limitations for the WWTP. The TSO includes a maximum daily effluent limitation (MDEL) for arsenic of 21 µg/L, effective until September 1, 2015. After this date, the City must fully comply with the WDRs' AMEL of 10 µg/L.

Arsenic Levels in the WWTP Influent and Effluent

The City has monitored the WWTP influent for arsenic on a weekly basis since June 2009 with (mostly) grab sampling. As part of the Monitoring and Reporting Program of the WDRs, the City also currently monitors the WWTP effluent for arsenic on a monthly basis with composite sampling. Prior to the adoption of the current WDRs (September 2010), the City was required to monitor monthly for arsenic in the WWTP effluent using grab samples. The historic influent and effluent arsenic levels are shown on Figure 1. A review of this data indicates the following:

- Influent and effluent arsenic levels vary considerably by season, with both influent and effluent levels higher in the dry season (May through October), most likely related to increased irrigation water use in this time period and thus increased arsenic mass loadings from water treatment systems, as discussed further below.
- In 2011, the City used the WWTP filtration facilities – installed in November 2010 – only when discharge to surface water was planned, so the summer 2011 effluent levels, which are relatively high, do not reflect the impacts of filtration during periods of higher influent arsenic levels. However, effluent levels for summer 2012, during which filtration was used, are similar to the summer 2011 levels, although influent levels are about 2-3 µg/L higher.
- Average effluent arsenic levels during the wet season (November-April) have decreased since the installation of WWTP filtration facilities
- Average effluent arsenic levels since November 2010 still trend above the 10 µg/L goal, particularly during the dry season and even with the use of filtration facilities.
- Average monthly removal rates during the wet season since November 2010 range from -3 to 43 percent and average 24 percent, based on filtered effluent data.
- Average monthly removal rates during the dry season since November 2010 range from 22 to 33 percent and average 28 percent, for unfiltered effluent data, and range from 27 to 38 percent and average 31 percent, for filtered effluent data.

Sources of Arsenic in the WWTP Influent

West Yost prepared a Pollution Prevention Plan (PPP) for the WWTP in April 2008 (West Yost, 2008), in which the predominant source of arsenic to the WWTP is identified as the City's drinking water supply. Monitoring data reviewed for the PPP indicated that, on average, 55 percent of the WWTP influent arsenic was in particulate form (the rest in dissolved form). Because the City was planning to install filtration at the WWTP, which was expected to enhance removal of particulate arsenic, the PPP includes a recommendation to review additional pollution prevention measures for arsenic only if arsenic effluent levels continue to be a compliance concern following installation of filtration at the WWTP. As discussed above, effluent arsenic levels do not appear to have been adequately improved even when the filtration facilities have been used.

The City's water treatment systems for arsenic removal – installed at several of the City's groundwater wells – can further increase arsenic levels in the WWTP influent, as discussed conceptually in the TM *Arsenic Treatment Residuals Disposal Assessment* prepared for the City in April 2008 (ECO:LOGIC, 2008). Potential arsenic levels in the WWTP influent following installation of the water treatment systems were estimated based on historic raw groundwater arsenic levels and expected performance of the water treatment systems.

The sources of arsenic need to be re-evaluated based on actual data collected since preparation of the 2008 reports and installation of the water treatment systems. Therefore, this section discusses more recent data collected for the City's Drinking Water Supply and Water Treatment Systems, which are sources of arsenic, in addition to briefly discussing recently collected data for the recycle streams at the WWTP, which return to the influent some of the arsenic removed during the treatment processes.

Drinking Water Supply and Water Treatment Systems

Arsenic is often naturally occurring in groundwater and is currently regulated by the United States Environmental Protection Agency (USEPA) drinking water standards at 10 µg/L to protect public health (USEPA, 2012). The City has had arsenic removal equipment installed and operating at three of the City's groundwater well sites since 2009 in an effort to meet the drinking water requirement.[1] In addition, several of the City's groundwater well sites have iron and manganese treatment systems that use different filtration media but can remove some arsenic as well. While these water treatment systems remove arsenic from the potable water via media filtration, they require backwashing of the media to ensure continued performance, and the City discharges the backwash with elevated arsenic levels to the sanitary sewer system, where it eventually makes it to the WWTP. Details of the water treatment systems relevant to the current evaluation are given in Table 1, based on visits to the well sites and information provided by the City staff. Recent (November 2010 through April 2011) raw water and treated water arsenic levels and treated water flows for the wells (in million gallons per day, mgd) are summarized by season in Table 2; summarizing the data by wet and dry seasons is helpful in analyzing potential seasonal impacts to the WWTP influent levels.

[1] The USEPA arsenic drinking water standard is also the governing factor (water quality criteria) for the value of the arsenic effluent limitation set for WWTP discharges in the WDRs.

Table 1. Details of Water Treatment Systems at City of Galt Groundwater Well Sites

Well Site	2011 Avg. Daily Production, mgd	Fe/Mn Treatment ?[a]	Arsenic Treatment ?	No. of Filtration Vessels	Approx. Dimensions of Backwash Storage Tank			Storage Tank Mixer ?
					Diameter, feet	Height, feet	Volume, gallons	
Carillion[b]	0.94	✓[c]	✓	2	26	16	63,000	✓
Fumasi	0.39	✓	✓	2	19	16	32,000	✓
Industrial Park	0.80	✓	✓	2	30	16	83,000	X
Creekside	0.00	✓	X	2				
Gateway	0.63	✓	X	2				
Golden Heights	0.01	✓	X	4				
Quail Hollow	0[d]	✓	X	3				
Monterey Bay	0.91	X	X					
River Oaks	0.97	X	X					

(a) Iron and manganese (Fe/Mn) treatment has historically been provided at most wells.
(b) The Carillion well site treats water from both an onsite well and an offsite well.
(c) A check ("✓") indicates the treatment is used at the well. An "X" indicates no such treatment at the well.
(d) The Quail Hollow well site is not in active use.

Table 2. Summary of Flows and Arsenic Levels at City of Galt Groundwater Well Sites

Well Site	Flow, mgd		Average Arsenic Levels, micrograms per liter, µg/L			
			Raw Water		Treated (Potable) Water	
	Nov.-Apr.[a]	May-Oct.[a]	Nov.-Apr.[a]	May-Oct.[a]	Nov.-Apr.[a]	May-Oct.[a]
Carillion	0.11	1.40	5.4	11.0	4.5	4.9
Fumasi	0.02	0.53	13.0	14.0	13.0[b]	4.5
Industrial Park	0.19	1.44	9.1	12.0	4.5	6.9
Creekside	—[c]	—	7.3[d]	7.0[e]	5.7[d]	5.9[e]
Gateway	0.65	0.64	8.8	8.7[f]	7.8	8.0[f]
Golden Heights	0.02	—	14.0	14.0[f]	10.0	10.0[f]
Quail Hollow	—	—	13.0[g]	—	—	—
Monterey Bay	1.02	1.13	10.0	9.9	10.0[h]	9.9[h]
River Oaks	0.73	1.29	12.5	13.0	12.5[h]	13.0[h]
Total	**2.75**	**6.43**				
Flow-Weighted Average					**9.6**	**7.3**

(a) "Nov.-Apr." data is for November 2010-April 2011 and "May-Oct." for May 2011-October 2011, unless otherwise noted.
(b) Average arsenic levels for Fumasi well's treated and raw water in Nov.-Apr. are estimated to be the same, based on only one raw water sample and three treated water samples, indicating little or no actual arsenic removal during this time period.
(c) Creekside and Quail Hollow well sites were reported as having no production during the time period.
(d) For November 2008-April 2009.
(e) For May 2009-October 2009.
(f) For May 2010-October 2010.
(g) For November 2009-April 2010.
(h) Water from Monterey Bay and River Oaks wells is not treated. Raw water arsenic levels are shown and used in the averages.

Recycle Streams at the WWTP

In addition to external sources of arsenic to the WWTP, the recycle streams within the WWTP can also affect effluent arsenic levels. While these recycle streams do not add arsenic mass loadings into the WWTP, they can influence the partitioning of arsenic between the liquid and solid streams of the WWTP, thus having competing influences on arsenic levels in the WWTP effluent and biosolids. The WWTP currently includes two recycle streams that can influence arsenic levels at the WWTP – supernatant from the sludge lagoons and filtrate from the biosolids dewatering system. Levels of arsenic in these recycle streams can be estimated based on recent sampling by City staff, and are summarized in Table 3, which also includes average flows for these recycle streams, based on recent flow data, and an average arsenic mass loading calculated from the average arsenic levels and flows.

Table 3. Summary of Arsenic Levels in Recycle Streams of City of Galt Wastewater Treatment Plant and Reclamation Facility

Recycle Stream	Arsenic Levels (May 15-17, 2012)			Average Flow, mgd	Average Arsenic Mass Load, pounds per day (ppd)
	Maximum, µg/L	Minimum, µg/L	Average, µg/L		
Sludge Lagoon Supernatant	29	23	26	0.018[(a)]	0.0039
Biosolids Dewatering Filtrate	66	18	47	0.030[(b)]	0.012

(a) Supernatant average flow estimated based on the average measured waste activated sludge flow and an assumed concentration of biosolids from 1 percent solids to 2.5 percent solids.
(b) Filtrate average flow estimated from City data for November 2011 through March 2012.

Arsenic Mass Balance for Existing Conditions

Mass balances of arsenic levels and mass loads from the City's water supply through to the WWTP effluent and biosolids have been developed for existing conditions, as shown schematically on Figures 2 and 3. Two mass balances have been developed, one for May 2011 through October 2011 (dry season) and one for November 2010 through April 2011 (wet season), to analyze arsenic cycling while taking into account the observed seasonal differences in WWTP influent arsenic levels.

These existing conditions mass balances have been developed based on available data, most of which has been summarized above. Many of the values used in the mass balances are then used as assumptions for the mass balances of the alternative arsenic management strategies discussed below. These major assumptions are detailed in Table 4. Also, as shown on the figures and discussed in Table 4, the main arsenic management procedure used under existing conditions is removal of arsenic in the WWTP processes, with "removal" being from the liquid waste stream and into the solids waste stream (*i.e.*, biosolids).

Table 4. Major Assumptions used to Develop Mass Balances for City of Galt Wastewater Treatment Plant and Reclamation Facility under Existing Conditions

Parameter	Nov.-Apr.	May-Oct.	Basis
Arsenic removal by water treatment systems (at wells with treatment)	Varies by well from 7-50% (concen-tration)[(a)]	Varies by well from 7-68% (concen-tration)	Nov. 2010 through Oct. 2011 data set, for each well and season, removal has been calculated from average arsenic levels for treated water relative to raw water.
Outdoor potable water usage	20%	66%	Water supply records compared to WWTP influent flow records, for Nov. 2010 through Oct. 2011.
Arsenic removal at WWTP	20% (concentration)		Average of the differences between paired data sets (*i.e.*, influent and effluent arsenic levels for the same day), for Nov. 2010 through Oct. 2011. Removal rates vary significantly by month and depend on whether filtration is being used at the WWTP; however, average removal rates for this time period are just over 20%. Based on the range of monthly removal rates for November-April, this assumption may over-predict wet season removal.
Ratio of arsenic returned to influent in sludge lagoon supernatant	10% (mass)	6% (mass)	Measured arsenic mass (for May 2012) in supernatant relative to estimated mass in waste activated sludge (WAS). (Arsenic mass in the WAS is less in the winter due to lower influent levels, resulting in a higher ratio).
Ratio of arsenic returned to influent in dewatering filtrate	32% (mass)	21% (mass)	Estimated arsenic mass load in the filtrate (See Table 3) relative to the arsenic mass load in the biosolids.
WWTP influent/ effluent flow	2.20 mgd		Average influent and effluent flow are assumed to be equal and consistent throughout the year.

[(a)] Percentages of arsenic removal/return are either based on relative concentrations or mass loads, as indicated.

EVALUATION OF ARSENIC MANAGEMENT ALTERNATIVES

This section provides an evaluation of three potential alternative scenarios for controlling arsenic levels in the WWTP effluent.

Summary of Alternatives

The alternatives developed initially are summarized as follows:

- Alternative 1: Collect backwash water from the wellhead treatment sites and discharge it directly to the sludge lagoons, where the backwash solids would be mixed with biosolids from the WWTP processes prior to land application. This alternative would likely entail use of the storage tanks currently at the well sites to store and settle solids in the backwash water, with only the “decant” water discharged to the sanitary sewer system and the settled solids collected and hauled to the WWTP.

- Alternative 2: Construct dewatering facilities at the wellhead treatment sites for the backwash water and dispose of the backwash solids offsite.
- Alternative 3: Prevent arsenic in the backwash water from re-solubilizing in the collection system, thus enhancing removal at the WWTP. Sequestering would likely be achieved by promoting non-alkaline (low pH) and/or high oxidation-reduction potential (redox) conditions (Masscheleyn, 1991).

Mass balances for all three of these alternatives were analyzed for both the wet and dry seasons. Based on the results of those analyses and following a discussion with City staff, the existing conditions scenario and the three alternatives were re-evaluated to consider the impacts of preferentially using a recently constructed Golden Heights well tapping a deeper aquifer with potentially lower arsenic levels. This well is assumed to have lower average arsenic levels[2], thereby reducing the need for arsenic treatment for the water supply as a whole and reducing backwash water flows. Schematics of the mass balances for the alternatives are shown on Figures 4-14.

The mass balances for the alternatives include a few assumptions additional to or different from those detailed in Table 4, as follows:

- For Alternatives 1 and 2 (both seasons), a WWTP removal of 1 percent is estimated, assuming most of the arsenic entering the WWTP is soluble. This assumption is based on the understanding that arsenic in the water supply, supernatant, and filtrate is mostly in a soluble form and not readily removed by the WWTP processes.
- For Alternative 3 during the dry season, a WWTP removal of 52 percent is estimated, assuming all of the arsenic from the backwash water is in particulate form. However, with increased use of the Golden Heights well, the WWTP removal is estimated to slightly decrease (to 47 percent) due to a lower level of particulate arsenic from the backwash water.
- For Alternative 3 during the wet season, WWTP removal is assumed to be the same as under existing conditions (20 percent).
- The Golden Heights well will have an untreated arsenic level of 5 µg/L, a capacity of 1,800 gpm, and can be used 22 hours per day (on average).

Analysis of Mass Balance Results for Existing Conditions and Alternatives

The potential impacts of the alternatives on both WWTP influent and effluent arsenic levels can be analyzed based on the mass balances, as discussed in this section. Results of the mass balances for the Existing Conditions and Alternatives are presented in Table 5.

[2] As shown in Table 2, the shallower wells at the Golden Heights site are reported with recent potable water arsenic levels of 10.0 µg/L; however, the additional, deeper well was placed into operation in late 2012, and City staff have indicated raw water from that well would have an expected arsenic level of 5 µg/L. The value of 5 µg/L has been used in the analysis of alternatives.

Table 5. Results of Arsenic Mass Balances for City of Galt Wastewater Treatment Plant and Reclamation Facility under Existing Conditions and Potential Alternative Arsenic Management Strategies

Scenario	Average Arsenic Levels, µg/L			
	WWTP Influent[a]		WWTP Effluent	
	Nov.-Apr.	May-Oct.	Nov.-Apr.	May-Oct.
Existing Conditions	11.2	18.4	9.0	14.7
Ex. Conditions with Preferential Use of Golden Heights Well	6.0	12.9	4.8	10.3
Alternative 1: Bypass of the WWTP by Backwash Water	10.0	10.7	9.9	10.6
Alternative 1 with Preferential Use of Golden Heights Well	[b]	8.1	[b]	8.0
Alternative 2: Dispose of Backwash Water Offsite	9.9	8.5	9.8	8.4
Alternative 2 with Preferential Use of Golden Heights Well	[b]	6.7	[b]	6.7
Alternative 3: Sequester Arsenic from the Backwash Water	11.2	19.8	8.9	10.6
Alternative 3 with Preferential Use of Golden Heights Well	[b]	13.7	[b]	8.0

[a] WWTP influent arsenic levels include contributions from the City's collection system and from the WWTP recycle streams.

[b] Alternatives that include preferential use of Golden Heights well were not fully assessed for the wet season (Nov.-Apr.) but would result in lower arsenic levels than has been determined for the dry season (May-Oct.) for the following reasons (1) lower backwash flows and resulting lower arsenic mass, and (2) more proportionate use of the low-arsenic Golden Heights well.

These results lead to the following conclusions relative to the arsenic effluent goal of 10 µg/L:

- Under the Existing Conditions, the goal can only be met (on average) during the wet season, when arsenic mass loadings from the backwash water are relatively low. (This conclusion agrees with the historic data, which also indicate significant variability in effluent data, such that existing conditions should not be expected to consistently result in meeting the goal, even during the wet season.)
- Under Alternative 1, average conditions may result in effluent levels meeting the goal; however, given uncertainties in the amount of arsenic that my enter the WWTP if the dewatered solids are disposed of in the sludge lagoons, this alternative should not be relied on for compliance without further study (and potentially pilot testing).
- Alternative 2 appears to result in compliance under average conditions. However, given uncertainties in the mass balance assumptions, this alternative alone (i.e. without preferential use of low arsenic wells) may not result in reliable compliance.
- Alternative 3 does not appear to effectively reduce effluent arsenic levels below the 10 µg/L goal – even with preferential use of the Golden Heights well.

- Maximizing use of the deep Golden Heights well is likely to be needed to significantly reduce arsenic levels in the WWTP influent and allow the City to meet the goal. In addition, changing the existing conditions only to preferentially use the Golden Heights well appears to meet the goal during the wet season. However, additional data is needed to confirm results for scenarios relying on the use of the Golden Heights well. The assumption of relatively low arsenic in the raw water for this well site needs to be verified.

Overall, further study – particularly of the new Golden Heights well and of the costs and other impacts of the alternatives – is needed to verify whether the proposed alternatives can indeed meet the City's 10 µg/L goal of effluent arsenic levels as indicated by this preliminary analysis.

Preliminary Evaluation of Alternatives

Based on the preceding analysis, any one of the three alternatives combined with preferential use of the Golden Heights well would appear to be viable options for the City to achieve year-round effluent levels of arsenic below 10 µg/L. However, many of the assumptions in the analysis, including the long-term arsenic level at the Golden Heights well and the partitioning of arsenic between dissolved and particulate phases in the collection system, need to be further verified before recommending a preferred alternative; a cost-benefit analysis should also be completed before such a recommendation can be adequately made.

RECOMMENDED ARSENIC MANAGEMENT PLAN

This section details the following:

- Monitoring Plan
- Evaluation of Monitoring Results
- Action Plan
- Schedule (for the arsenic management plan)

Monitoring Plan

The City has conducted three weeks of monitoring to collect the data needed to determine the influence of the force main downstream of the Live Oak Pump Station on dissolved arsenic levels entering the WWTP. In addition, the City has collected filtered effluent at the WWTP during the summer months of 2012 – providing valuable information regarding the removal potential during periods of peak well usage.

As indicated in Table 6, which details the sampling and analytical procedures, the City collected a total of 18 samples, including duplicate quality assurance/quality control (QA/QC) samples.

Table 6. Three-Week Arsenic Monitoring Plan for City of Galt Wastewater Treatment Plant and Reclamation Facility Arsenic Management Plan

Sampling Locations	Sampling Frequency	Analyses	Sample Volume/ Container	Filtration	Preservative	Maximum Holding Time	Analytical Method
Live Oak Pump Station WWTP Influent WWTP Effluent	Once per week	Total Arsenic	500 mL/ HDPE	None	HNO_3 to pH < 2	180 days	EPA 200.8
	Once per week	Dissolved Arsenic	500 mL/ HDPE	Field filter after completion of composite collection and prior to preservation with HNO_3. Filter with a 0.45-µm filter and label samples as "Field filtered" for the lab.	Prior to filtration, keep cool at ≤ 4°C. Following filtration, preserve with HNO_3 to pH < 2.	180 days	EPA 200.8

Evaluation of Monitoring Results

The monitoring data were first reviewed for conformance to the monitoring plan and adequacy of QA/QC data; the sampling and laboratory analyses proceeded according to the plan. Once the data were found to be adequately representative, the ratio of dissolved to total arsenic was evaluated, based on average values, for the three different monitoring locations. Averages, maximums, minimums, and standard deviations for each location were also evaluated, and these results are presented in Table 7.

Table 7. Descriptive Statistics of Three-Week Arsenic Monitoring Data for City of Galt Wastewater Treatment Plant and Reclamation Facility

Location	Parameter	Average µg/L	Maximum µg/L	Minimum µg/L	Standard Deviation µg/L	No. of Samples
Live Oak P.S. Influent	Arsenic, Total	11.3	13.0	9.4	1.6	6
	Arsenic, Dissolved	8.0	8.5	7.1	0.6	6
	Percent Dissolved	70%				
WWTP Influent	Arsenic, Total	16.5	26.0	11.0	7.0	6
	Arsenic, Dissolved	10.8	15.0	8.4	2.9	6
	Percent Dissolved	65%				
WWTP Effluent	Arsenic, Total	15.3	16.0	15.0	0.5	6
	Arsenic, Dissolved	15.3	16.0	14.0	0.8	6
	Percent Dissolved	100%				

As indicated by the results in Table 7, the partitioning of arsenic does not appreciably change from the collection system to the WWTP influent. The observed partitioning of 70 percent confirms that most of the arsenic is entering the WWTP in a dissolved form, which would not be readily removed by the WWTP's filtration facilities. In addition, the partitioning in the effluent is estimated to be 100 percent, which is not surprising given the use of filtration at the WWTP.

Action Plan

The alternatives discussed in this TM each involve specific assumptions that need to be verified through a series of actions. The recommended next steps for each alternative are summarized as follows:

- Alternative 3 requires a certain level of arsenic sequestration that, based on the monitoring results discussed above, does not appear to be feasible. Therefore, Alternative 3 should not be considered a viable alternative.

- Given that both Alternatives 1 and 2 rely on the City's ability to capture and remove settled solids from wellhead backwash water storage tanks, the City should proceed with modifications to the backwash storage tanks at two wells with relatively high arsenic levels and flows (Carillion and Industrial Park Wells). These wells are also the most suitable locations for backwash dewatering equipment, if such equipment is ultimately called for. The tank modifications should include the addition of an overflow outlet and pipe discharging to the sanitary sewer (or recycle system if available), plus modifications necessary to connect a Vactor truck or other liquid waste hauling truck to remove settled solids. Backflow potential and protection should be considered.
- Evaluate the feasibility of Alternative 1 by experimenting with hauling of settled backwash solids to the WWTP. A more detailed work plan will be needed, but the basic elements should include periodic removal of solids from the backwash storage tanks and hauling them to the sludge storage lagoons, and possibly the headworks. WWTP influent arsenic monitoring should continue, and careful records of the hauling activities (including the labor and equipment hours expended) should be kept.
- Consider meeting with the Regional Board to discuss the regulatory feasibility for hauling the backwash water directly to the Storage Reservoir for subsequent land application. (The long-term viability of this option would need to be considered further, considering the potential for accumulation of arsenic solids in the reservoir over time, and impacts on the land application area.)
- Even with liquid hauling under Alternative 1, or dewatering facilities and solids hauling under Alternative 2, compliance with the arsenic effluent limitation may be unreliable. Under either alternative, preferential use of the deep Golden Heights well, and possibly a second deep aquifer well on Kost Road should also be implemented. To further evaluate potential benefits:
  - Evaluate arsenic levels in the Golden Heights well and update the solids balance evaluation using actual arsenic concentrations produced by the well.
  - Confirm the ability of the City's water distribution system to operate effectively with maximized use of the Golden Heights well in summer and winter demand conditions. A water system optimization study could help quantify how much of the low-arsenic water could be used.
  - The City has constructed a test well for a second deep well (Kost Well), which is also expected to also have lower arsenic levels than some of the existing wells. A production well at that site is planned for construction by 2015 pending the availability of adequate funding. Evaluate the ability of the City's water distribution system to operate effectively with maximized use of both the Golden Heights and future Kost well in summer and winter demand conditions.

- If hauling liquid backwash water to the WWTP proves viable and successful in controlling effluent arsenic levels in summer and winter conditions, perform a cost comparison of Alternatives 1 and 2 to determine whether or not backwash dewatering facilities should be constructed in lieu of continuing the liquid hauling operation. The City's previous study prepared during the design of the arsenic treatment systems (ECO:LOGIC, 2008) included a preliminary cost and feasibility analysis of constructing and operating dewatering facilities at the groundwater well sites. The analysis should be updated using actual water treatment system use and backwash water production rates at the well sites, and then compared to the cost of the liquid hauling operations.
- Assuming a 12-month construction period and nine months for design, bidding and award, a decision whether to proceed with Alternative 2 must be made by about November 2013, in order to complete backwash dewatering facilities at two sites by August 2015. If the viability of Alternative 1 has not been confirmed by December 2013, proceed with Alternative 2.
- For the purposes of the WWTP Facilities Master Plan, it will be assumed that backwash dewatering equipment will be installed at two well sites: Carillion and Industrial Park.

Schedule

A recommended schedule for the Arsenic Management Plan has been developed based on our understanding of the City's resources and the requirements in the WDRs and TSO. The proposed schedule for implementing the Arsenic Management Plan is summarized in Table 8.

Table 8. Proposed Schedule for City of Galt Wastewater Treatment Plant and Reclamation Facility Arsenic Management Plan

Task	Timeline
Modify the backwash storage tanks at Carillion and Industrial Park wells to facilitate overflow and solids removal. Develop a work plan for evaluating feasibility and effectiveness of liquid backwash hauling.	January – March 2013
Meet with Regional Board staff to discuss the possibility of hauling backwash directly to the Storage Reservoir for subsequent land application.	March 2013
Proceed with trials of liquid backwash hauling from the wells to the WWTP.	April – September 2013
If discharge to the Storage Reservoir is viable, evaluate long-term implications for the reservoir and land application area.	April – September 2013
Further evaluate potential benefits of preferentially using the Golden Heights and future Kost Road deep wells: • Evaluate arsenic levels in the Golden Heights well and update the solids balance. • Confirm the ability of the City's water distribution system to operate effectively with maximized use of the Golden Heights well and/or future Kost Road well.	April – September 2013
Determine whether or not liquid hauling is viable, and if possible, whether or not Alternative 1 will ensure reliable compliance	September 2013
If liquid hauling is viable and effective, compare cost of liquid hauling vs. dewatering and solids hauling using collected data.	September – October 2013
If Alternative 1 is most cost effective, plan and implement long-term liquid hauling operations.	October 2013 – August 2015
If necessary, select the design team for dewatering facilities (Alternative 2).	October – November 2013
Pilot test, design, bid and award backwash solids dewatering facilities, if necessary.	December 2013 – August 2014
Construct dewatering facilities, if necessary.	September 2014 – August 2015
Arsenic Compliance Deadline	September 2015

REFERENCES

ECO:LOGIC Consulting Engineers, 2008, *Technical Memorandum Arsenic Treatment Residuals Disposal Assessment*, Prepared for City of Galt, April 2008.

Masscheleyn, Patrick H., Ronald D. Delaune, and William H. Patrick, Jr., 1991, "Effect of Redox Potential and pH on Arsenic Speciation and Solubility in a Contaminated Soil," *Environmental Science and Technology* 25 (8), 1414-1419, August 1, 1991.

USEPA, 2012, "Basic Information about the Arsenic Rule." http://water.epa.gov/lawsregs/rulesregs/sdwa/arsenic/Basic-Information.cfm, Site last updated on March 6, 2012. Accessed on July 31, 2012.

West Yost Associates, 2008, *Wastewater Treatment Plant Pollution Prevention Plan*, Prepared for City of Galt, April 2008.

Figure 1. Arsenic Influent and Effluent Levels at City of Galt WWTP

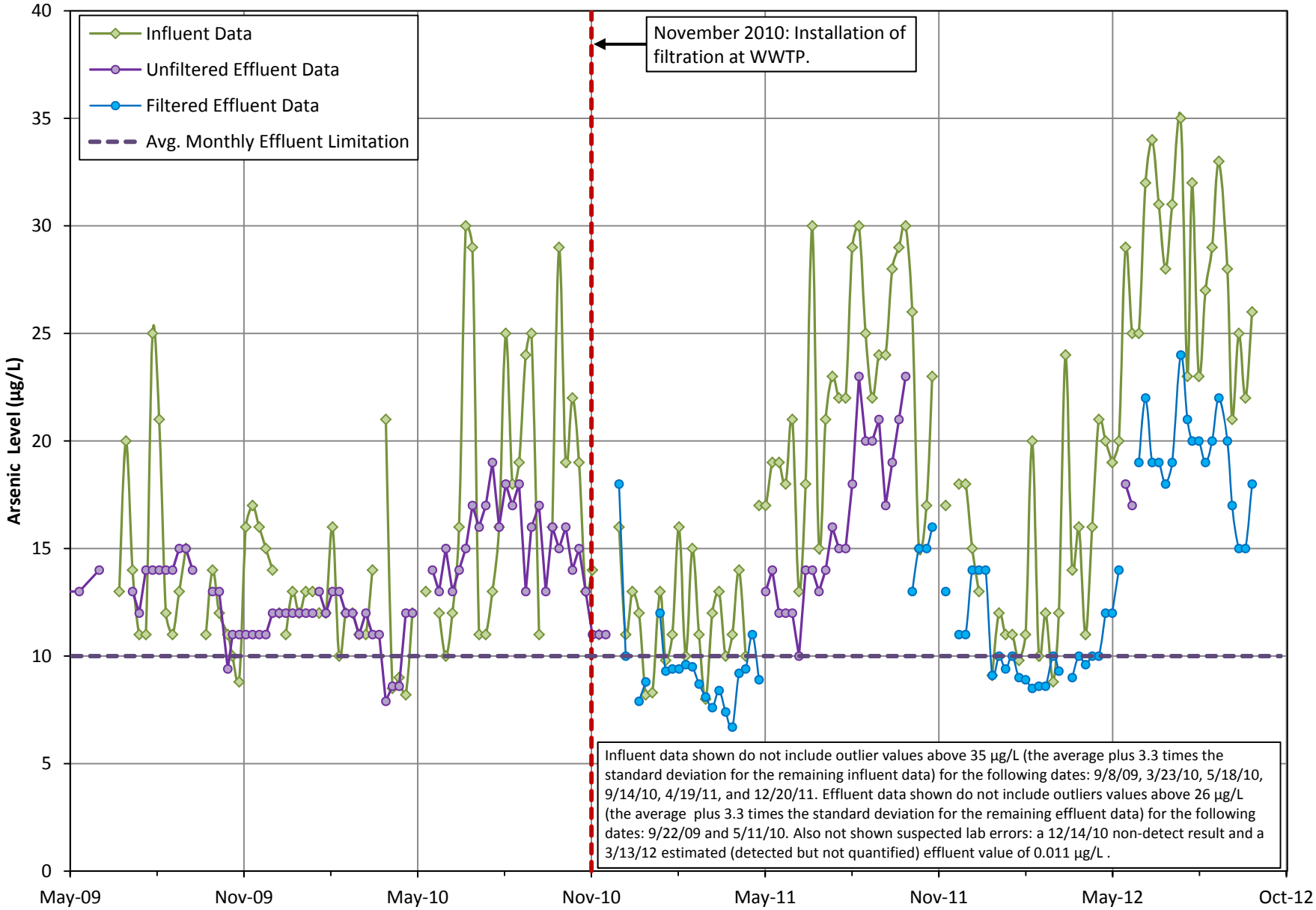


Figure 2. Arsenic Mass Balance for City of Galt - Existing Conditions (Based on May-Oct. 2011 data)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash
Carillion	22%	1.40	0.01	11.0	53%	5.2	0.13	0.06	0.07
Creekside	0%	0.00	-	-	0%	-	-	-	-
Fumasi	8%	0.53	0.01	14.0	68%	4.5	0.06	0.02	0.04
Gateway	10%	0.64	0.01	8.5	7%	7.9	0.05	0.04	0.00
Gold. Hts.	0%	0.00	-	-	0%	-	-	-	-
Industrial	22%	1.44	0.01	12.0	39%	7.3	0.14	0.09	0.06
Monterey	18%	1.13	-	9.9	0%	9.9	0.09	0.09	0.00
River Oaks	20%	1.29	-	13.0	0%	13.0	0.14	0.14	0.00
Total	100%	6.43	0.04	11.4	27%	8.3	0.61	0.44	0.17

Note:
Filtration Backwash flow (mgd) assumed to be 1% of treated groundwater flow.

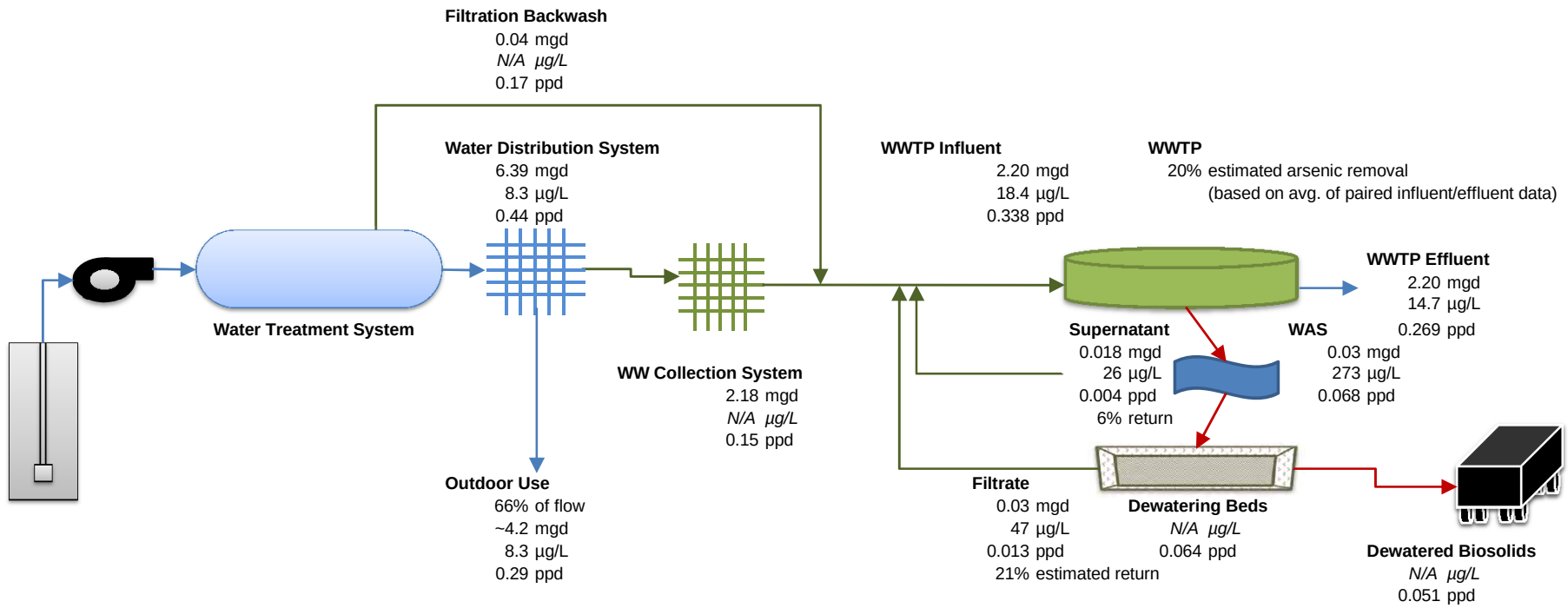


Figure 3. Arsenic Mass Balance for City of Galt - Existing Conditions (Based on Nov. 2010-Apr. 2011 data)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash
Carillion	4%	0.11	0.00	5.4	21%	4.2	0.00	0.00	0.00
Creekside	0%	0.00	-	-	0%	-	-	-	-
Fumasi	1%	0.02	0.00	13.0	34%	8.6	0.00	0.00	0.00
Gateway	24%	0.65	0.01	8.8	7%	8.2	0.05	0.04	0.00
Gold. Hts.	1%	0.02	-	14.0	0%	14.0	0.00	0.00	0.00
Industrial	7%	0.19	0.00	9.1	50%	4.5	0.01	0.01	0.01
Monterey	37%	1.02	-	10.0	0%	10.0	0.09	0.09	0.00
River Oaks	27%	0.73	-	12.5	0%	12.5	0.08	0.08	0.00
Total	100%	2.75	0.01	10.2	6%	9.6	0.23	0.22	0.01

Note:
Filtration Backwash (mgd) assumed to be 1 % of total volume of groundwater pumped.

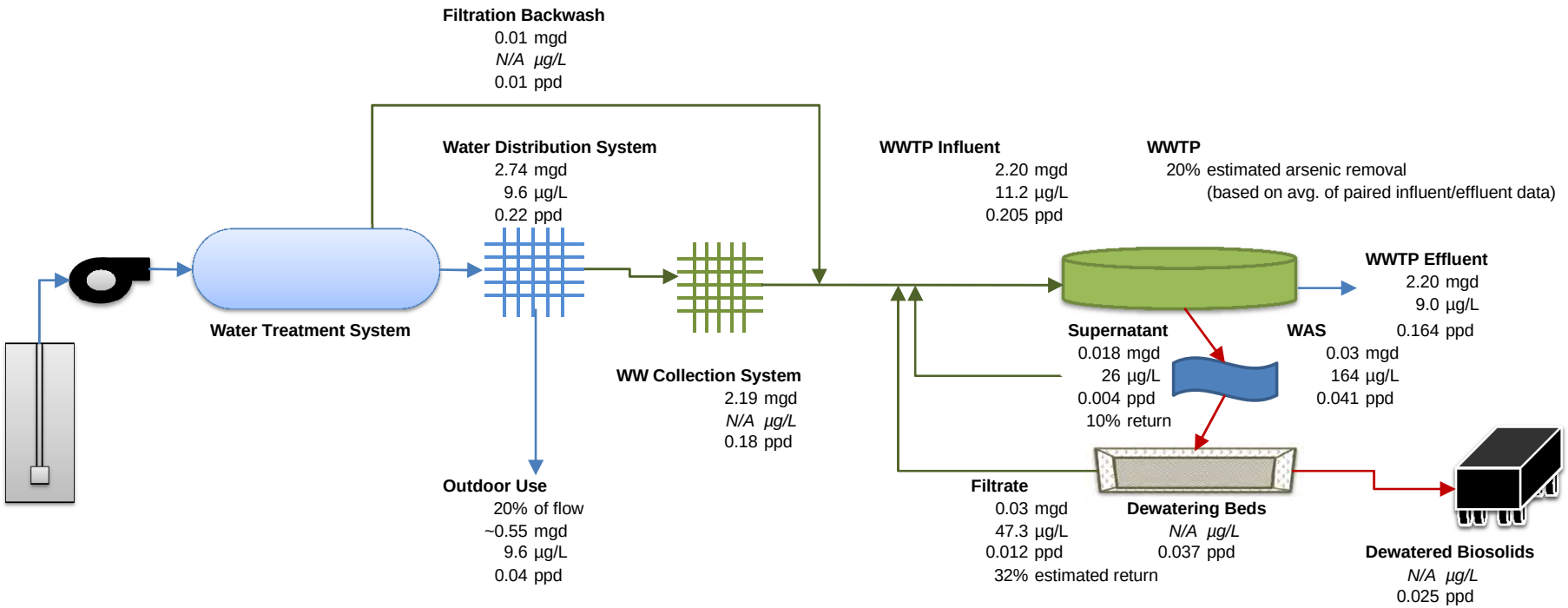


Figure 4. Arsenic Mass Balance for City of Galt - Existing Conditions with Preferential Use of Golden Heights Well (May-Oct.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash
Carillion	22%	1.40	0.01	11.0	53%	5.2	0.13	0.06	0.07
Creekside	0%	0.00	-	-	0%	-	-	-	-
Fumasi	0%	0.00	0.00	14.0	68%	4.5	0.00	0.00	0.00
Gateway	10%	0.64	0.01	8.5	7%	7.9	0.05	0.04	0.00
Gold. Hts.	37%	2.38	-	5.0	0%	5.0	0.10	0.10	0.00
Industrial	14%	0.88	0.01	12.0	39%	7.3	0.09	0.05	0.03
Monterey	18%	1.13	-	9.9	0%	9.9	0.09	0.09	0.00
River Oaks	0%	0.00	-	13.0	0%	13.0	0.00	0.00	0.00
Total	100.0%	6.43	0.03	8.5	18%	6.5	0.45	0.35	0.11

Note:
WWTP removal assumed to be 1 %, filtrate and supernatant return of arsenic are assumed to equal existing condtiions

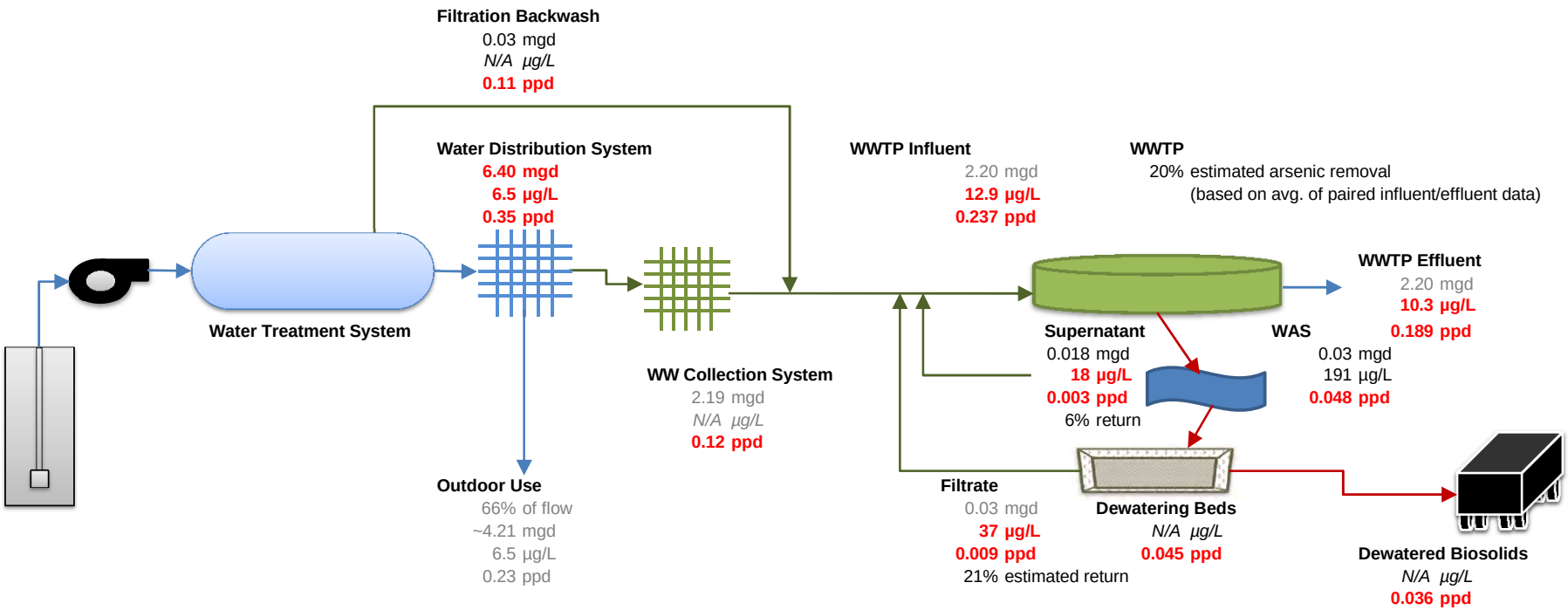


Figure 5. Arsenic Mass Balance for City of Galt - Existing Conditions with Preferential Use of Golden Heights Well (Nov.-Apr.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash
Carillion	0%	0.00	0.00	5.4	21%	4.2	0.00	0.00	0.00
Creekside	0%	0.00	-	-	0%	-	-	-	-
Fumasi	0%	0.00	0.00	13.0	34%	8.6	0.00	0.00	0.00
Gateway	14%	0.37	0.00	8.8	7%	8.2	0.03	0.03	0.00
Gold. Hts.	87%	2.38	-	5.0	0%	5.0	0.10	0.10	0.00
Industrial	0%	0.00	0.00	9.1	50%	4.5	0.00	0.00	0.00
Monterey	0%	0.00	-	10.0	0%	10.0	0.00	0.00	0.00
River Oaks	0%	0.00	-	12.5	0%	12.5	0.00	0.00	0.00
Total	100.0%	2.75	0.00	5.5	1%	5.4	0.13	0.12	0.00

Note:
WWTP removal assumed to be 1%, filtrate and supernatant return of arsenic are assumed to equal existing condtiions

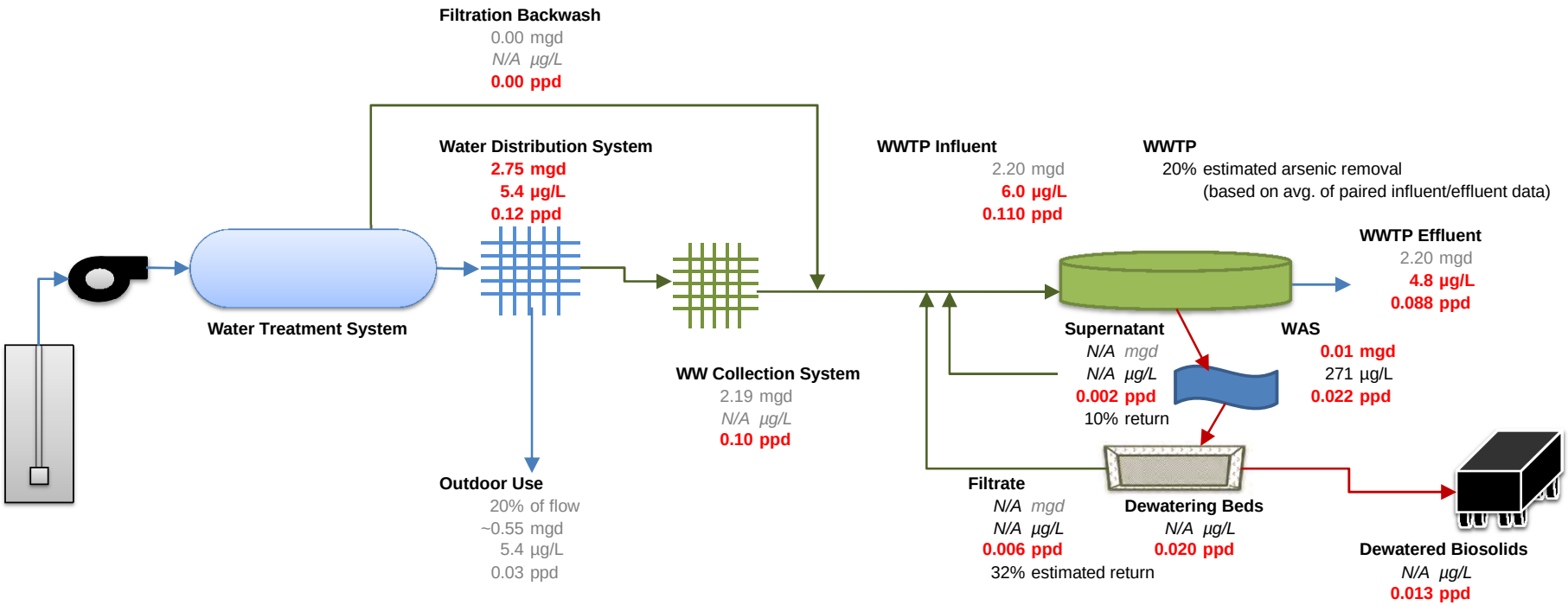


Figure 6. Arsenic Mass Balance for City of Galt - Alternative 1: Bypass of Backwash Water (May-Oct.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash	
Carillion	22%	1.40	0.01	11.0	53%	5.2	0.13	0.06	0.07	Backwash bypassed
Creekside	0%	0.00	-	-	0%	-	-	-	-	
Fumasi	8%	0.53	0.01	14.0	68%	4.5	0.06	0.02	0.04	Backwash bypassed
Gateway	10%	0.64	0.01	8.5	7%	7.9	0.05	0.04	0.00	Backwash not bypassed
Gold. Hts.	0%	0.00	-	-	0%	-	-	-	-	
Industrial	22%	1.44	0.01	12.0	39%	7.3	0.14	0.09	0.06	Backwash bypassed
Monterey	18%	1.13	-	9.9	0%	9.9	0.09	0.09	0.00	
River Oaks	20%	1.29	-	13.0	0%	13.0	0.14	0.14	0.00	
Total	100%	6.43	0.04	11.4	27%	8.3	0.61	0.44	0.17	

Note:
WWTP removal assumed to be 1%, filtrate and supernatant return of arsenic are assumed to equal existing condtiions

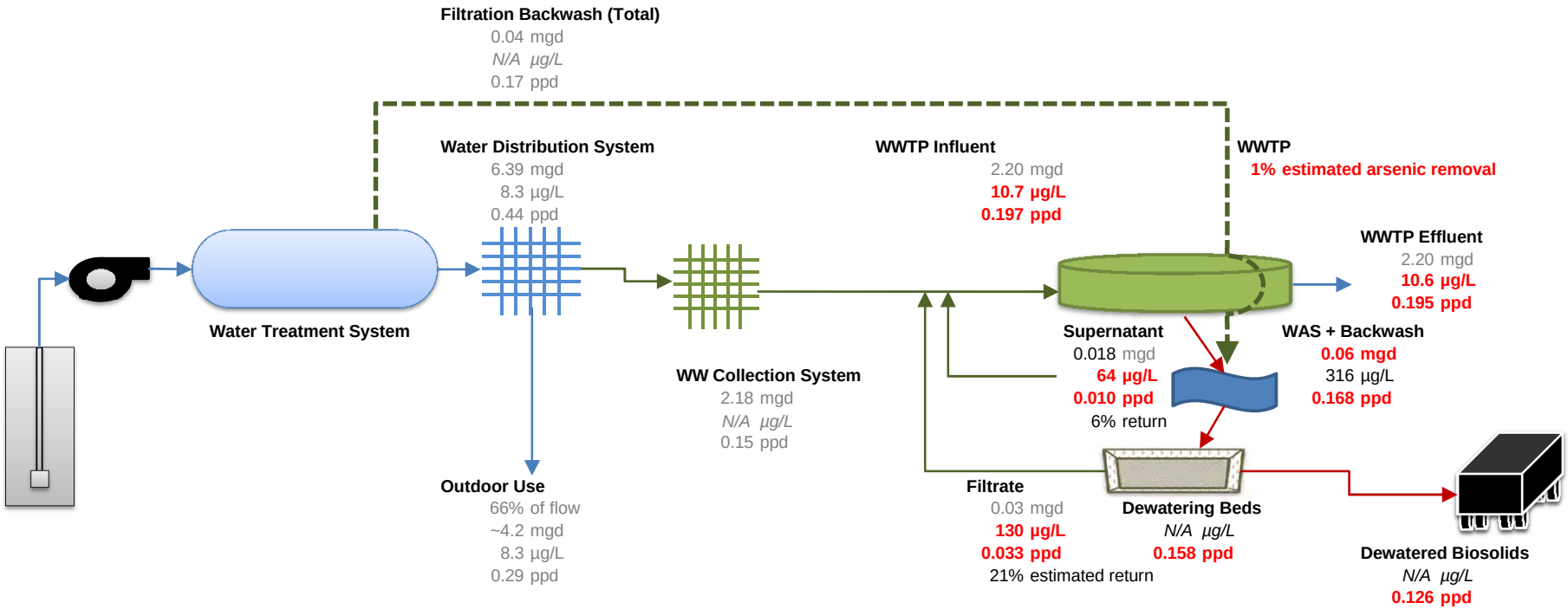


Figure 7. Arsenic Mass Balance for City of Galt - Alternative 1: Bypass of Backwash Water (Nov.-Apr.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Treated	Backwash	
Carillion	4%	0.11	0.00	5.4	21%	4.2	0.00	0.00	0.00	Backwash bypassed
Creekside	0%	0.00	-	-	0%	-	-	-	-	
Fumasi	1%	0.02	0.00	13.0	34%	8.6	0.00	0.00	0.00	Backwash bypassed
Gateway	24%	0.65	0.01	8.8	7%	8.2	0.05	0.04	0.00	Backwash not bypassed
Gold. Hts.	1%	0.02	-	14.0	0%	14.0	0.00	0.00	0.00	
Industrial	7%	0.19	0.00	9.1	50%	4.5	0.01	0.01	0.01	Backwash bypassed
Monterey	37%	1.02	-	10.0	0%	10.0	0.09	0.09	0.00	
River Oaks	27%	0.73	-	12.5	0%	12.5	0.08	0.08	0.00	
Total	100%	2.75	0.01	10.2	6%	9.6	0.23	0.22	0.01	

Note:
WWTP removal assumed to be 1%, filtrate and supernatant return of arsenic are assumed to equal existing condtiions

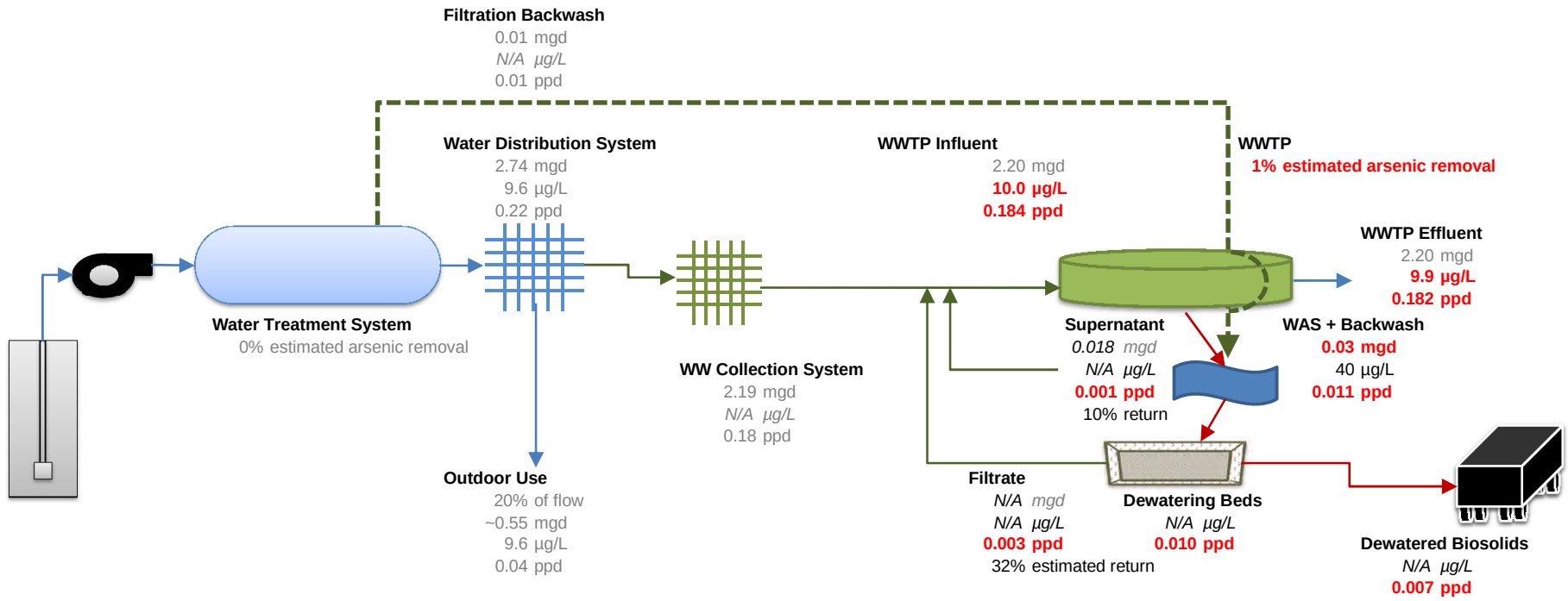


Figure 8. Arsenic Mass Balance for City of Galt - Alternative 1 with Preferential Use of Golden Heights Well (May-Oct.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash	
Carillion	22%	1.40	0.01	11.0	53%	5.2	0.13	0.06	0.07	Backwash bypassed
Creekside	0%	0.00	-	-	0%	-	-	-	-	
Fumasi	0%	0.00	0.00	14.0	68%	4.5	0.00	0.00	0.00	Backwash bypassed
Gateway	10%	0.64	0.01	8.5	7%	7.9	0.05	0.04	0.00	Backwash not bypassed
Gold. Hts.	37%	2.38	-	5.0	0%	5.0	0.10	0.10	0.00	
Industrial	14%	0.88	0.01	12.0	39%	7.3	0.09	0.05	0.03	Backwash bypassed
Monterey	18%	1.13	-	9.9	0%	9.9	0.09	0.09	0.00	
River Oaks	0%	0.00	-	13.0	0%	13.0	0.00	0.00	0.00	
Total	100.0%	6.43	0.03	8.5	18%	6.5	0.45	0.35	0.11	

Note:
WWTP removal assumed to be 1 %, filtrate and supernatant return of arsenic are assumed to equal existing condtiions

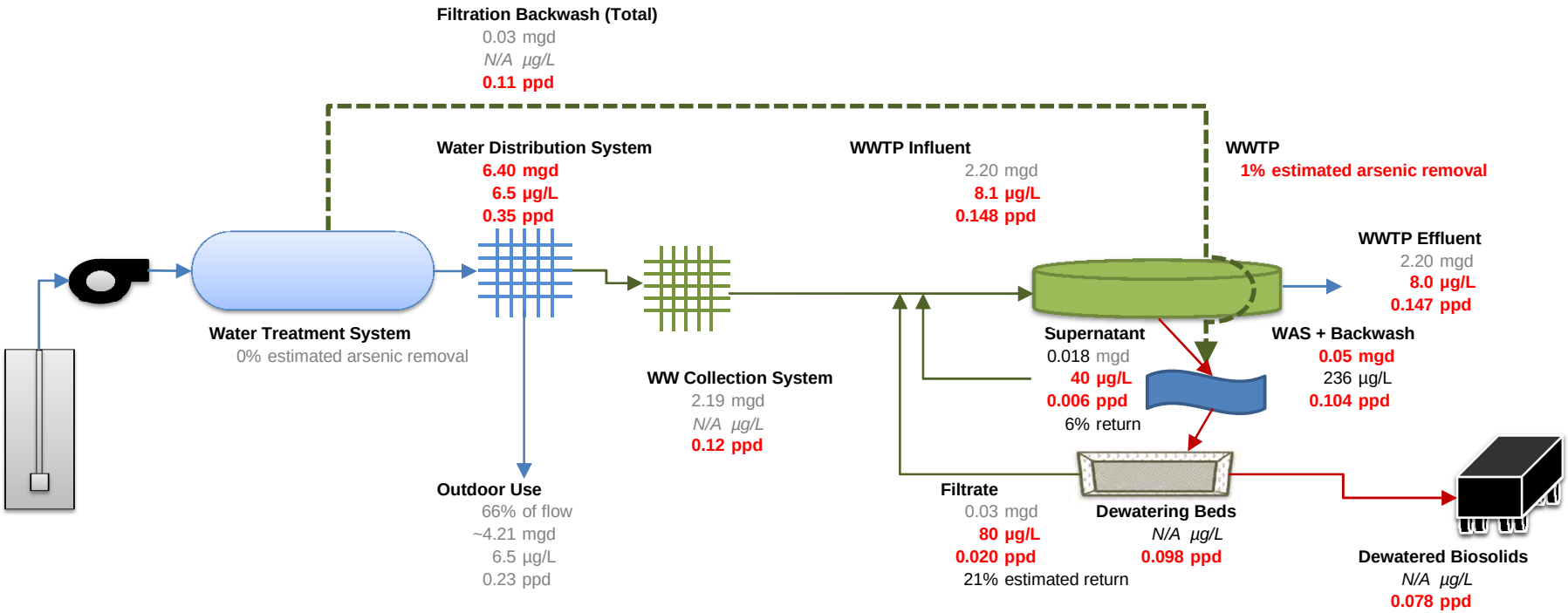


Figure 9. Arsenic Mass Balance for City of Galt - Alternative 2: Dispose of Backwash Water Offsite (May-Oct.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash	
Carillion	22%	1.40	0.01	11.0	53%	5.2	0.13	0.06	0.07	Backwash disposed offsite
Creekside	0%	0.00	-	-	0%	-	-	-	-	
Fumasi	8%	0.53	0.01	14.0	68%	4.5	0.06	0.02	0.04	Backwash disposed offsite
Gateway	10%	0.64	0.01	8.5	7%	7.9	0.05	0.04	0.00	Backwash not disposed offsite
Gold. Hts.	0%	0.00	-	-	0%	-	-	-	-	
Industrial	22%	1.44	0.01	12.0	39%	7.3	0.14	0.09	0.06	Backwash disposed offsite
Monterey	18%	1.13	-	9.9	0%	9.9	0.09	0.09	0.00	
River Oaks	20%	1.29	-	13.0	0%	13.0	0.14	0.14	0.00	
Total	100%	6.43	0.04	11.4	27%	8.3	0.61	0.44	0.17	

Note:
WWTP removal assumed to be 1%, filtrate and supernatant return of arsenic are assumed to equal existing condtiions

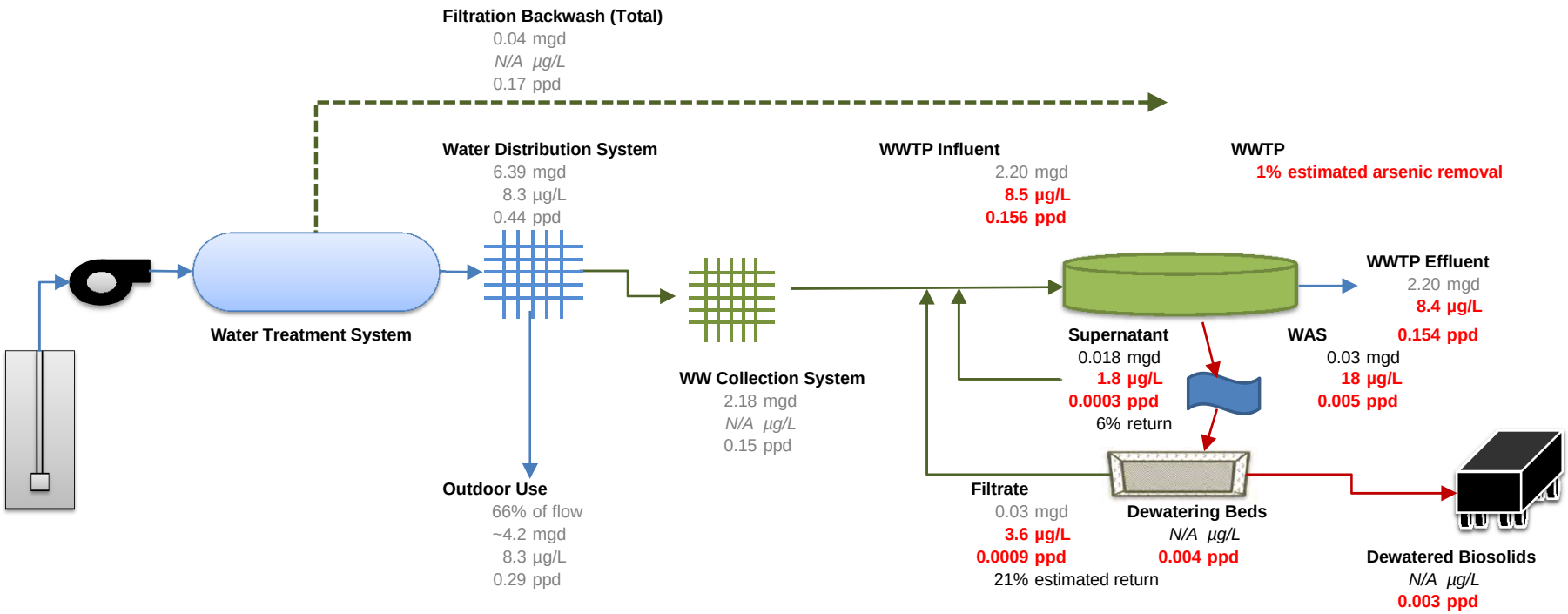


Figure 10. Arsenic Mass Balance for City of Galt - Alternative 2: Dispose of Backwash Water Offsite (Nov.-Apr.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash	
Carillion	4%	0.11	0.00	5.4	21%	4.2	0.00	0.00	0.00	Backwash disposed offsite
Creekside	0%	0.00	-	-	0%	-	-	-	-	
Fumasi	1%	0.02	0.00	13.0	34%	8.6	0.00	0.00	0.00	Backwash disposed offsite
Gateway	24%	0.65	0.01	8.8	7%	8.2	0.05	0.04	0.00	Backwash not disposed offsite
Gold. Hts.	1%	0.02	-	14.0	0%	14.0	0.00	0.00	0.00	
Industrial	7%	0.19	0.00	9.1	50%	4.5	0.01	0.01	0.01	Backwash disposed offsite
Monterey	37%	1.02	-	10.0	0%	10.0	0.09	0.09	0.00	
River Oaks	27%	0.73	-	12.5	0%	12.5	0.08	0.08	0.00	
Total	100%	2.75	0.01	10.2	6%	9.6	0.23	0.22	0.01	

Note:
WWTP removal assumed to be 1%, filtrate and supernatant return of arsenic are assumed to equal existing condtiions

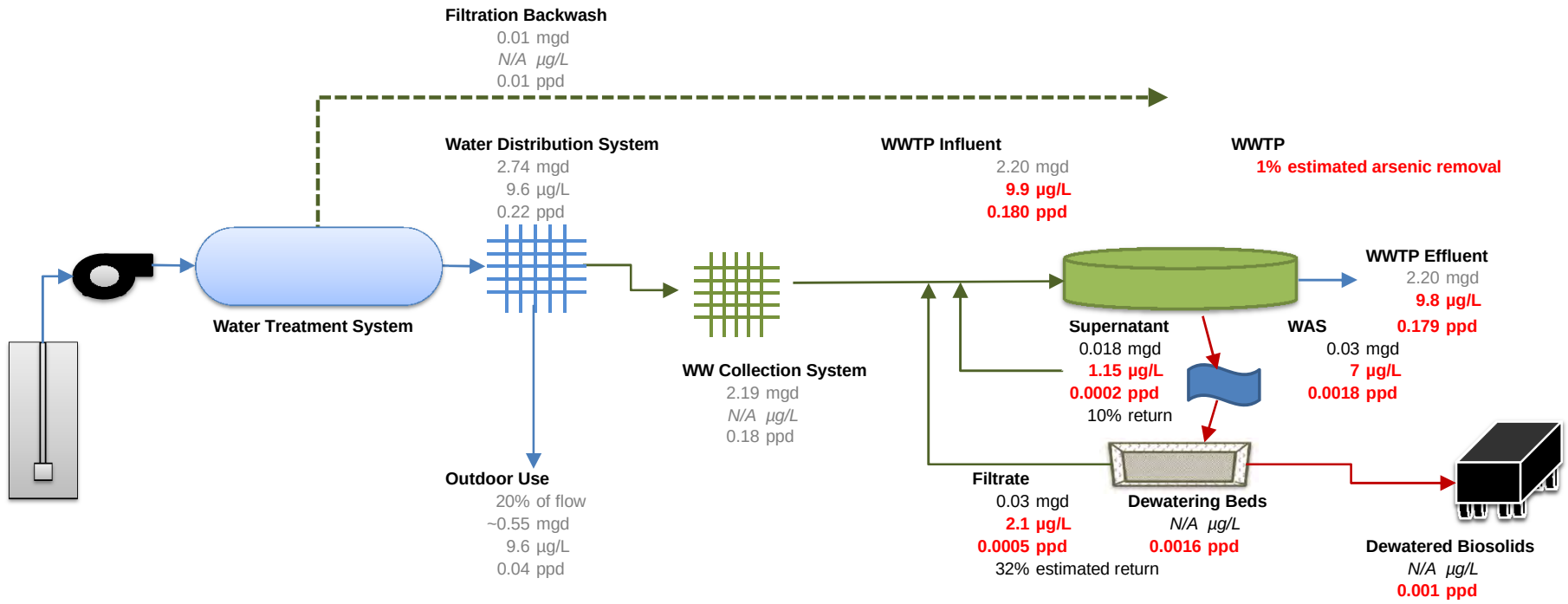


Figure 11. Arsenic Mass Balance for City of Galt - Alternative 2 with Preferential Use of Golden Heights Well (May-Oct.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash	
Carillion	22%	1.40	0.01	11.0	53%	5.2	0.13	0.06	0.07	Backwash disposed offsite
Creekside	0%	0.00	-	-	0%	-	-	-	-	
Fumasi	0%	0.00	0.00	14.0	68%	4.5	0.00	0.00	0.00	(No backwash)
Gateway	10%	0.64	0.01	8.5	7%	7.9	0.05	0.04	0.00	Backwash not disposed offsite
Gold. Hts.	37%	2.38	-	5.0	0%	5.0	0.10	0.10	0.00	
Industrial	14%	0.88	0.01	12.0	39%	7.3	0.09	0.05	0.03	Backwash disposed offsite
Monterey	18%	1.13	-	9.9	0%	9.9	0.09	0.09	0.00	
River Oaks	0%	0.00	-	13.0	0%	13.0	0.00	0.00	0.00	
Total	100%	6.43	0.03	8.5	18%	6.5	0.45	0.35	0.11	

Note:
WWTP removal assumed to be 1%, filtrate and supernatant return of arsenic are assumed to equal existing condtiions

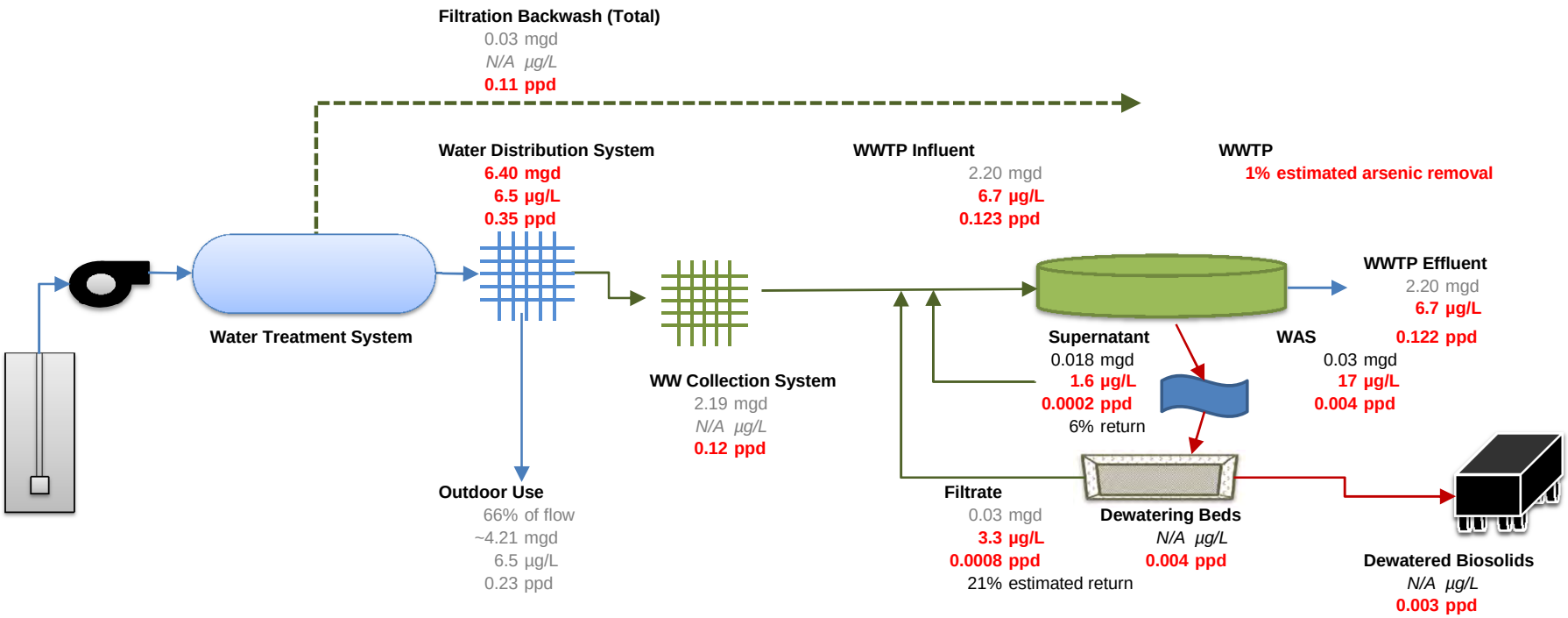


Figure 12. Arsenic Mass Balance for City of Galt - Alternative 3: Sequester Arsenic from Backwash Water (May-Oct.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Loads, ppd Treated	Loads, ppd Backwash
Carillion	22%	1.40	0.01	11.0	53%	5.2	0.13	0.06	0.07
Creekside	0%	0.00	-	-	0%	-	-	-	-
Fumasi	8%	0.53	0.01	14.0	68%	4.5	0.06	0.02	0.04
Gateway	10%	0.64	0.01	8.5	7%	7.9	0.05	0.04	0.00
Gold. Hts.	0%	0.00	-	-	0%	-	-	-	-
Industrial	22%	1.44	0.01	12.0	39%	7.3	0.14	0.09	0.06
Monterey	18%	1.13	-	9.9	0%	9.9	0.09	0.09	0.00
River Oaks	20%	1.29	-	13.0	0%	13.0	0.14	0.14	0.00
Total	100%	6.43	0.04	11.4	27%	8.3	0.61	0.44	0.17

Note:
Biosolids filtrate return (%) and supernatant % are assumed equal to existing conditions.
WWTP removal increases because all arsenic in backwash is sequestered in particulate form.

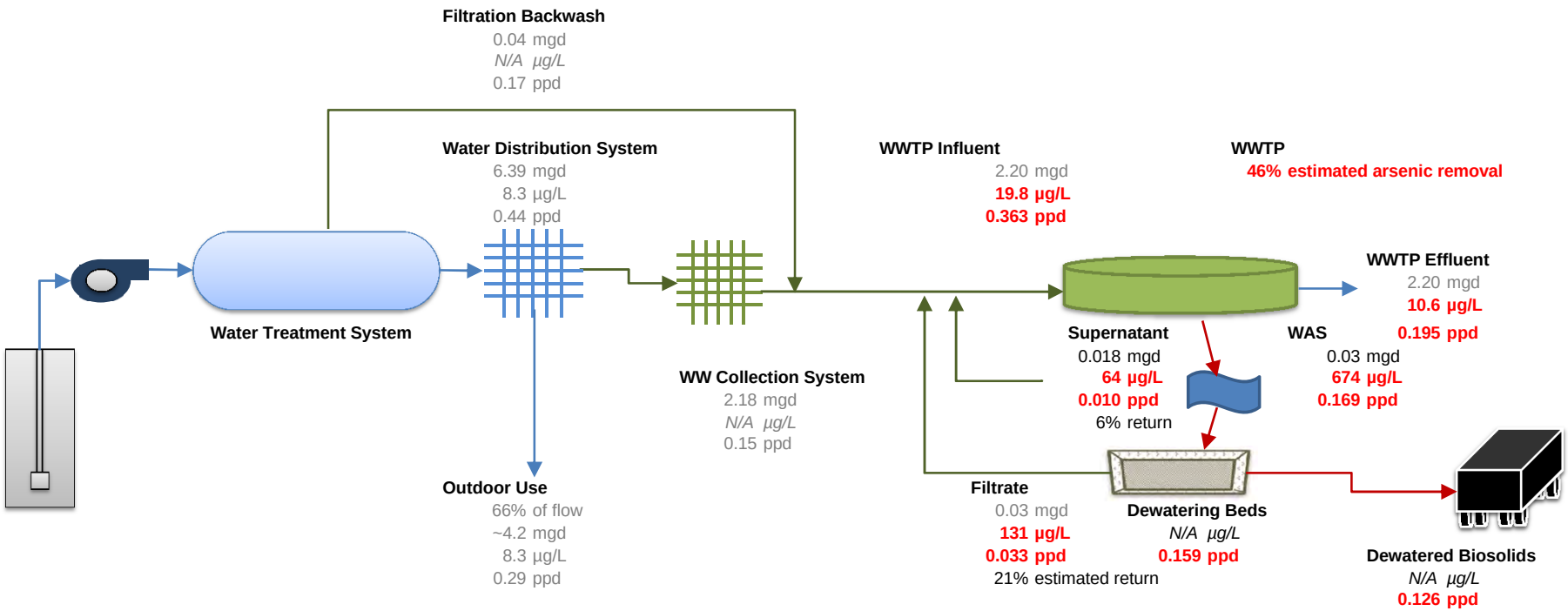


Figure 13. Arsenic Mass Balance for City of Galt - Alternative 3: Sequester Arsenic from Backwash Water (Nov.-Apr.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Treated	Backwash
Carillion	4%	0.11	0.00	5.4	21%	4.2	0.00	0.00	0.00
Creekside	0%	0.00	-	-	0%	-	-	-	-
Fumasi	1%	0.02	0.00	13.0	34%	8.6	0.00	0.00	0.00
Gateway	24%	0.65	0.01	8.8	7%	8.2	0.05	0.04	0.00
Gold. Hts.	1%	0.02	-	14.0	0%	14.0	0.00	0.00	0.00
Industrial	7%	0.19	0.00	9.1	50%	4.5	0.01	0.01	0.01
Monterey	37%	1.02	-	10.0	0%	10.0	0.09	0.09	0.00
River Oaks	27%	0.73	-	12.5	0%	12.5	0.08	0.08	0.00
Total	100%	2.75	0.01	10.2	6%	9.6	0.23	0.22	0.01

Note:
Biosolids filtrate return (%) and supernatant % are assumed equal to existing conditions.
WWTP removal remains the same (arsenic in backwash is sequestered in particulate form).

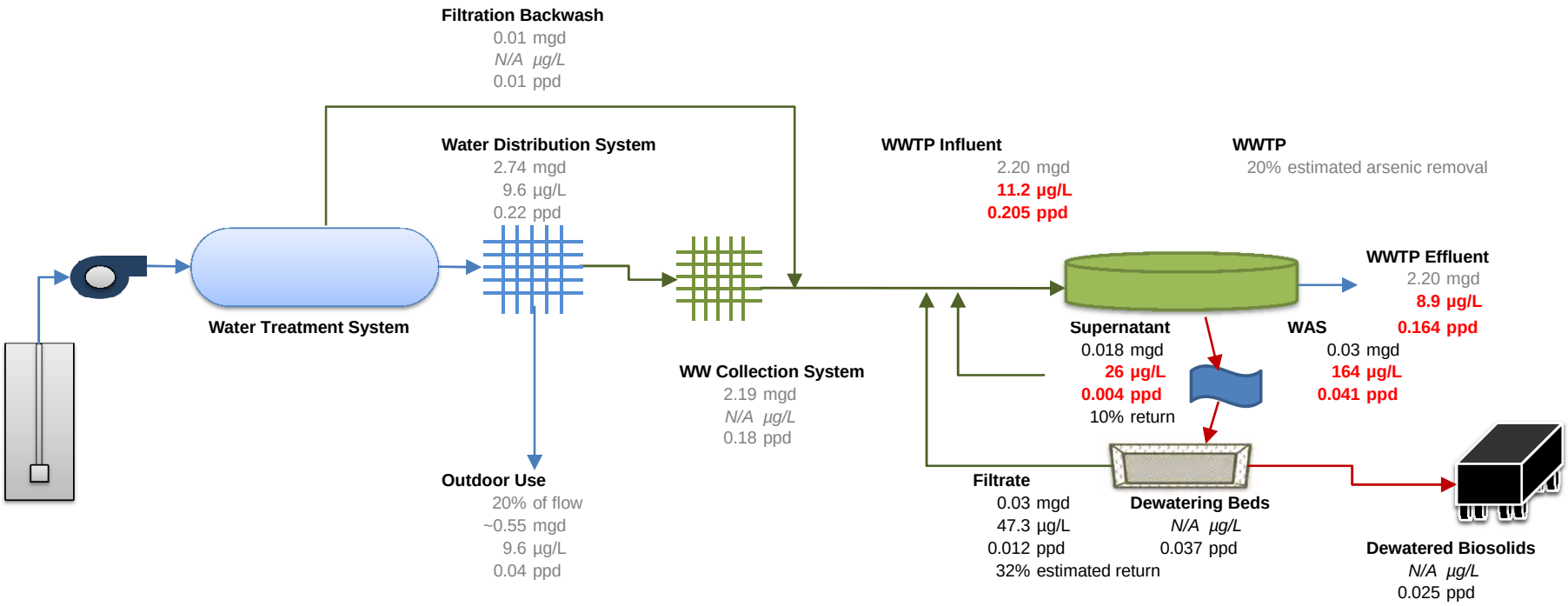
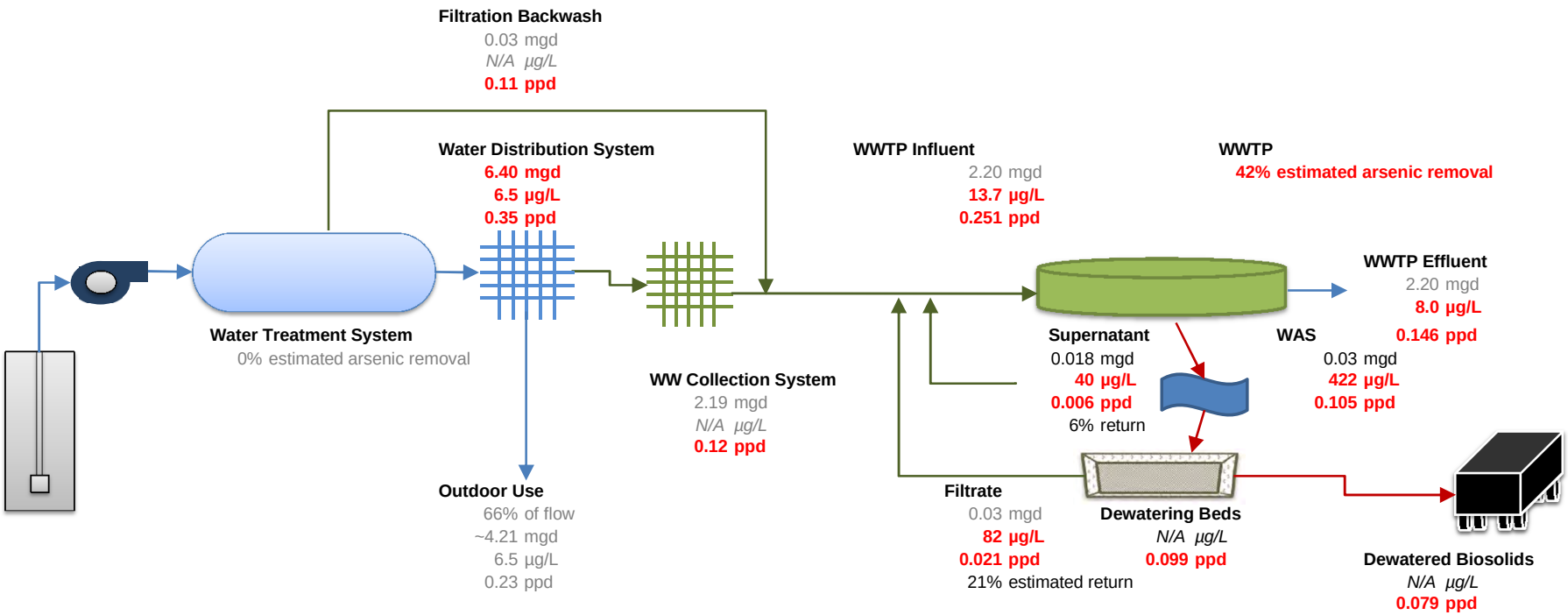


Figure 14. Arsenic Mass Balance for City of Galt - Alternative 3 with Preferential Use of Golden Heights Well (May-Oct.)

Groundwater Wells (supply)

Well	% of Flow	Raw Flow, mgd	Backwash Flow, mgd	Raw Conc., µg/L	Removal	Treated Conc., µg/L	Loads, ppd Raw	Treated	Backwash
Carillion	22%	1.40	0.01	11.0	53%	5.2	0.13	0.06	0.07
Creekside	0%	0.00	-	-	0%	-	-	-	-
Fumasi	**0%**	0.00	0.00	14.0	68%	4.5	0.00	0.00	0.00
Gateway	10%	0.64	0.01	8.5	7%	7.9	0.05	0.04	0.00
Gold. Hts.	**37%**	2.38	-	**5.0**	0%	5.0	0.10	0.10	0.00
Industrial	**14%**	0.88	0.01	12.0	39%	7.3	0.09	0.05	0.03
Monterey	18%	1.13	-	9.9	0%	9.9	0.09	0.09	0.00
River Oaks	**0%**	0.00	-	13.0	0%	13.0	0.00	0.00	0.00
Total	100.0%	6.43	0.03	8.5	18%	6.5	0.45	0.35	0.11

Note:
Biosolids filtrate return (%) and supernatant % are assumed equal to existing conditions.
WWTP removal increases because all arsenic in backwash is sequestered in particulate form.



APPENDIX D

Existing Facilities Design Basis

Table D-1. Liquid Treatment Unit Processes Design Criteria

Unit Process	Criteria
Headworks and Screening	
Mechanical Bar Screen	
Bar Screen Width	4 feet (ft)
Screen Openings	½ inch (in)
Capacity	18 mgd
Manually-Cleaned Bar Screen	
Bar Spacing	1 in
Flow Meter	
Meter Type	Magnetic Flow Meter
Diameter	18 in
Grit Removal	
Grit Chamber	
Capacity	10 mgd
Paddle Motor Size	5 hp
Removal Efficiency	95% U.S. No. 50 sieve 85% U.S. No. 70 sieve 65% U.S. No. 100 sieve
Grit Classifier	
Capacity	432 cubic feet/day (ft^3/d)
Flow	250 gallon per minute (gpm)
Classifier Motor Size	0.5 horsepower (hp)
Oxidation Ditches	
Headworks Splitter Box	
Weir Type	Downward opening slide gate
Weir Length	6 ft
Oxidation Ditches (each)[(a)]	
BOD Loading	12.5 pound per cubic foot (lb/ft^3)
F/M Ratio	0.06
Mean Cell Residence Time	30 days
Sludge Yield	0.7 lb/lb BOD removed
Tank Volume	1.5 million gallons (MG)
Channel Width	22 ft
Maximum Channel Depth	11.1 ft
Effluent Weir Length	20 ft
Number of Aerators	2
Aerator Motor Size, each	60 hp
Secondary Clarifiers	
Secondary Splitter Box	
Weir Type	Downward opening slide gate
No. Weirs	2
Weir Length (Total)	6 ft
Secondary Clarifiers (Each)[(a)]	
Tank Diameter	80 ft
Sidewater Depth	16 ft
Feed Type	Center
Sludge Collection Type	Plows
Overflow rate @ 3.0 mgd	300 gallons per day per square foot (gpd/ft^2)
Overflow rate @ 9.0 mgd	900 gpd/ft^2
Solids Loading Rate @ 3 mgd and 100% RAS	20 lb/day/ft^2
Solids Loading Rate @ 9 mgd and 100% RAS	40 lb/day/ft^2
Clarifier Mechanism Drive Motor	1.0 hp
Polymer Storage and Dosing Facility	
Polymer Storage Box	
Capacity	1,090 gallons (gal)
Neat Polymer Pumps	
Type	Diaphragm Metering
Number	2
Capacity, Each	10 gal per hour (gph)
Maximum Discharge Pressure	58 pound per square inch (psi)
Pump Drive Type	Solenoid

Table D-1. Liquid Treatment Unit Processes Design Criteria

Unit Process	Criteria
Filter Feed and Utility Water Pump Station	
Filter Feed Pumps	
Type	Vertical Turbine
Number	3
Capacity, each (with two pumps operating)	2,250 gpm (3.24 mgd)
Firm Capacity	4,500 gpm (6.48 mgd)
Total Dynamic Head (TDH)	18.3 ft
Speed Control	Variable-frequency drive (VFD)
Motor Size	15 hp
High-Pressure W3 Pumps	
Type	Vertical Turbine
Number	3
Capacity, each (with two pumps operating)	272 gpm
Firm Capacity	545 gpm
TDH	211 ft
Discharge Pressure	80-90 psi
Speed Control	VFD
Motor Size	20 hp
Low-Pressure W3 Pumps	
Type	Vertical Turbine
Number	2
Capacity, each	500 gpm
Firm Capacity	500 gpm
TDH	37.4 feet
Speed Control	Constant
Motor Size	7.5 hp
Tertiary Disc Filters	
Number	3
Filter Media	Cloth Disc
Filter Pore Size	10 micron
Discs (Cartridges) Per Filter	10
Filter Submerged Surface Area, Each	392 square feet (ft^2)
Filtration rate @ 1.5 mgd	2.65 gpm/ft^2
Filtration rate @ 3.0 mgd	5.31 gpm/ft^2
Tertiary Plant Drain Station	
Pump Type	Submersible
Number	2
Capacity, Each	450 gpm
TDH	33 ft
Speed Control	Constant
Motor Size	7.5 hp
UV Disinfection	
UV Lamp Type	Low-pressure, high output
Number of Channels	3
Maximum Flow Per Channel	3.0 mgd
Minimum UV Transmittance (UVT)	55%
Lamp Aging Factor	98%
Lamp Fouling Factor	95%
Minimum UV Dose	100 millijoules per square centimeter (mJ/cm^2)
Number of Active Modules Per Channel	24
Number of Active Lamps Per Channel	192
Total Number of Active Lamps	576
Effluent Diversion Structure	
Weir Length	14 ft
Effluent Flow Metering	
Meter Type	Magnetic Flow Meter
Diameter	16 inches

(a) Listed are original design values, which will be confirmed or updated as part of the Facilities Master Plan work.

Table D-2. Solids Treatment Unit Processes Design Criteria	
Unit Process	**Criteria**
Secondary Sludge Pump Station	
RAS Pumps (Screw Centrifugal)	
Number	2
Capacity, Each	310-1,600 gpm
TDH	25 feet
Motor Speed Control	Constant
Stroke (Capacity) Control	Variable
Motor Size	25 hp
RAS Pumps (Dry Pit Submersible)	
Number	1
Best Efficiency Point (BEP) Capacity	1,350 gpm
BEP TDH	23 feet
Speed Control	VFD
Motor Size	15 hp
WAS Pumps	
Type	Diaphragm
Number	2
Capacity, Each	0-152 gpm
TDH	60 feet
Speed Control	Adjustable
Motor Size	Air-operated
Scum Pumps	
Type	Diaphragm
Number	2
Capacity, Each	0-152 gpm
TDH	60 feet
Speed Control	Adjustable
Motor Size	Air-operated
Compressor Capacity	92 CFM at 125 psig
Sludge Lagoons	
Lagoons	
Number	2
Capacity (each)	0.647 MG
Decant Pump Station	
Pump Type	Submersible
Number	1
Capacity	450 gpm
TDH	29 feet
Speed Control	Constant
Motor Size	0.5 hp
Mixers	
Type	Floating Paddle Mixer/Aerator
Number	2
Motor Size	5 hp
Sludge Feed Pump Station	
Type	Progressing cavity
Number	2
Capacity, Each	250 gpm
Firm Capacity	250 gpm
TDH	57 feet
Speed Control	Constant
Motor Size	20 hp
Biosolids Polymer Facility	
Polymer Blending Unit	
Mixing Type	Mechanical
Dilution Water Range	180-1,800 gph
Polymer Pump Capacity	0.4-8.0 gph
Flocculator/Mixer Device	
Diameter	6 inches
Mixing Type	Static
Hydraulic Capacity	120-1,200 gpm
Number of Injection Ports	4

Table D-2. Solids Treatment Unit Processes Design Criteria

Unit Process	Criteria
Biosolids Dewatering Beds[a]	
Type	Gravity dewatering, granular media
Number	4
Drying Cycle	7 days
Length	100 feet
Width	48 feet
Solids Concentration	2.5%
Filter Loading	2.0 lb/ft^2
Solids Capture	>90%
Filtrate Pump Station	
Pump Type	Submersible
Number	3
Capacity, each (with two pumps operating at max TDH)	260 gpm
Firm Capacity	520 gpm
Max TDH	26.6 feet
Speed Control	VFD
Motor Size	5 hp
Auxiliary Basin Pump Station	
Pump Type	Submersible
Number	2
Pump 1 Capacity	650 gpm
Pump 1 TDH	15 ft
Pump 2 Capacity	100 gpm
Pump 2 TDH	30 ft
Firm Capacity	100 gpm
Speed Control (each)	Constant
Pump 1 Motor Size	5 hp
Pump 2 Motor Size	3 hp

[a] Listed are original equipment proposal values, which will be confirmed or updated as part of the Facilities Master Plan work.

Table D-3. Storage and Irrigation Systems	
Unit Process	**Criteria**
Storage Basins	
Auxiliary Basin	
Water Depth	10 ft
Surface Area	1.8 ac
Storage Volume	5.8 MG
Storage Pond #1	
Water Depth	5 ft
Surface Area	5.4 ac
Storage Volume	7.4 MG
Storage Pond #2	
Pond #2 was filled and replaced with a solar array in 2011 and no longer used as a storage basin	
Storage Pond #3	
Water Depth	5 ft
Surface Area	4.5 ac
Storage Volume	6.1 MG
Storage Pond #4	
Water Depth	5 ft
Surface Area	4.6 ac
Storage Volume	6.1 MG
Effluent Storage Reservoir	
Water Depth	9 ft
Surface Area	17.9 ac
Storage Volume	38 MG
Auxiliary Basin Pump Station	
Year of Construction	2003
Pump Type	Submersible
Number	2
Pump 1 Capacity	650 gpm
Pump 1 TDH	15 ft
Pump 2 Capacity	100 gpm
Pump 2 TDH	30 ft
Firm Capacity	100 gpm
Speed Control (each)	Constant
Pump 1 Motor Size	5 hp
Pump 2 Motor Size	3 hp
Irrigation Pump Stations	
Irrigation Pump Station #1 (Dam Pump)	
Year of Construction	1980
Pump Type	Vertical turbine
Number	1
Capacity	2,200 gpm
TDH	35 feet
Speed Control	Constant
Motor Size	40 hp
Irrigation Pump Station #2 (40-Acre Pumps)	
Year of Construction	Prior to 2004
Pump Type	Vertical turbine
Number	1
Capacity	1,780 gpm
TDH	53 feet
Speed Control	Constant
Motor Size	40 hp
Return Pump Station #1	
Year of Construction	1980
Pump Type	Vertical turbine
Number	1
Capacity	300 gpm
TDH	15.5 feet
Speed Control	Constant
Motor Size	40 hp
Return Pump Station #2	
Year of Construction	2003
Pump Type	Submersible
Number	2
Capacity	Unknown
TDH	Unknown
Speed Control	Constant
Motor Size	3 hp

Table D-4. City-Owned Irrigation Fields and Acreages	
Field Number	Acreage
1	8.4
2	8.9
3	7.5
4	8.2
5	8.2
6	8.2
7	8.2
8	8.2
9	8.2
10	8.2
11	24.6
14	8.2
15	8.2
16	12.6
18	50
Total Acreage	185.8

Table D-5. Plant Water Systems	
Unit Process	**Criteria**
Nonpotable Wells and Hydropneumatic Tank	
Well #1	
Year of Construction	1980
Well Depth	225 feet
Pump Type	Submersible
Pump Capacity	Unknown
Design TDH	Unknown
Speed Control	Constant
Motor Size	5 hp
Well #2	
Year of Construction	2010
Well Depth	562 feet
Pump Type	Vertical Turbine
Pump Capacity	500 gpm
Design TDH	300 ft
Speed Control	Constant
Motor Size	50 hp
Hydropneumatic Tank #2	
Type	Horizontal
Nominal Capacity	3,000 gallons
Design Pressure	100 psi
Shell Length	16 feet
Shell Diameter	5.5 feet
Hydropneumatic Tank #1	
Type	Vertical
Capacity	525 gallons
Plant High-Pressure Utility Water (W3) System	
Capacity	472 gpm
Service Pressure	80-90 psi
Plant Nonpotable Water (W2) System	
Capacity	500 gpm
Service Pressure	40-65 psi

Table D-6. Electrical and Control Systems	
System	**Criteria**
Utility Service	
Service Provider	SMUD
Incoming Voltage	12 kV
Transformer Installation Year	2010
Stepdown Voltage	480 V
Service Size	2,000 A
Electrical Switchgear (Main Switchboard)	
Installation Year	2010
Incoming Voltage	480 V
Capacity	2,000 A
Connected Branch Feeders	4
Connected Generator(s)	600 kW
Solar Facility Breaker	1,600 A
Motor Control Centers	
Number	4
Original MCC	
Installation Year	1980
Spare Spaces[a]	(5) 2x
MCC-MCA	
Installation Year	1990
Spare Spaces[a]	(3) 1x, (1) 2x, (1) 3x
MCC-MCE	
Installation Year	1990
Spare Spaces[a]	(1) 1x, (3) 2x, (1) 3x
MCC-TF	
Installation Year	2010
Spare Spaces[a]	(1) 1x, (2) 2x, (4) 3x, (2) 7x
Emergency Generators	
New Generator	
Installation Year	2010
Type	Diesel
Output Power	600 kW
Horsepower at Rated kW	864 hp
Fuel Storage	Steel Sub-Base
Fuel Capacity	1,150 gal
Fuel Consumption	46.2 gal/hr
Enclosure	Sound Attenuating
Old Generator	
Installation Year	1990
Type	Diesel
Output Power	300 kW
Horsepower at Rated kW	~400 hp
Fuel Storage	Above Ground Concrete Vault
Fuel Capacity	1,000 gal
Enclosure	None (inside building)
SCADA System	
Installation Year	2010
Software Manufacturer	Wonderware
Historian Database Size	5,000 tags
Number SCADA Computers at WWTP	2
Screens	119
Monitored Local (WWTP) Tags	5,660
Monitored Remote (MSC) Tags	4,275

[a] Designation of MCC spaces indicates the vertical dimension of the available space, where "x" is equal to 6 vertical inches (*e.g.*, 2x = 12-inch vertical dimension).

APPENDIX E

Existing Facilities Conditions Assessment

Table 1. Conditions Assessment of Liquid Treatment Processes

Unit Process / Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
Headworks and Screening	• ½-inch climber screen is in good condition; however, the screen rake, rake shocks, and rake wiper are worn. The screenings auger motor has been replaced once, and the gear reducer has been repaired once. Staff is planning to change the auger motor orientation on the Headworks deck to improve accessibility for maintenance. Some plastics are able to pass the screens and impact the disc filters. Screenings are not washed and can create messes near the Headworks. • Manual stop gate upstream of the manually cleaned screen is difficult to remove, due to metal-on-metal binding and very tight screen frame tolerances. • Grating on the Headworks and Screening Facility deck is not secured (*i.e.,* can be lifted, tilted or shifted without tools) and is a safety concern. • Minor leaking is present at the screened effluent outlet flange on the north side of the structure.	• The only alarms for the facility include high flow (indicated by high wet well level) and flow transducer failure. • There are no level-based controls for the rake. There is also no alarm to indicate auger failure. • The bar screen control panel was installed in 1990, and its components are fatigued; many of the components can no longer be replaced in kind. City staff has expressed dissatisfaction with the current panel's proximity to the screen, and would prefer a panel located at grade. • The screening system is not monitored or controlled by SCADA.	• Replace existing mechanical climber screen with new screen with ¼-inch opening (actual opening size will be influenced by downstream treatment processes as discussed in later Chapters). • Provide redundant mechanical climber screen and new screenings washer for both screens. • Provide automatically actuated control gates for screening channel. • Replace and relocate the bar screen control panel to grade. • Provide level-based rake controls. • Provide SCADA monitoring and control capabilities for the bar screen rake(s). • Provide an alarm to indicate auger failure. • Secure grating to address safety concerns.
Grit Removal	• The grit tank drain line tends to clog with grit, but lacks a cleanout in either direction for effective maintenance. Therefore, the drain line's 4-inch plug valve must be exercised weekly to ensure grit is not clogging the drain line and/or valve. Due to the tank drain line's location (under the upper "bench" of the grit chamber), the plug valve on the tank drain line is not easily accessible. • The grit chamber scour piping performs inadequately. • The grit chamber lining has disintegrated and partially clogged the tank drain; exposed concrete in the grit chamber has not suffered degradation to date. However, such degradation will occur over time. • Corrosion can be observed on miscellaneous uncoated steel surfaces and steel surfaces with deteriorated coatings (*e.g.,* slide gate actuator, electrical conduit hardware, hose bibs, gate operator pedestal). • The plastic sheath over the slide gate threaded shaft is missing. • Minor, healed cracking can be observed on grit structure exterior.	• None identified.	• Provide two easily accessible cleanouts for the grit tank drain line, one in each direction. • Modify grit chamber scour piping to improve effectiveness. • Remove and replace damaged grit chamber lining. • Miscellaneous coating touchup. • Replace plastic sheath over slide gate shaft.
Oxidation Ditches	• Aerator motors were re-built in 2008, mostly due to previous over-greasing. The motors are currently in good condition, but gear reducers on some of the motors have experienced seal failure. • The aerator paddles have never been checked for wear, but amperage draw from the motors have been within tolerances over the last 2 years. • Foam sprayers are moderately effective, and could likely be improved with different nozzles. • City staff intends to measure grit depths in the ditches during 2012 and prepare cross sections.	• Instrument gauges are weather-beaten. • Prior to the construction of the tertiary facilities in 2010, the plant's original 300 kW generator supplied backup power to the aerators and most of the plant's other loads. During a power outage, the inrush current for the aerators typically exceeded the generator's capacity, and the aerator motors would trip. With the abandonment of the chlorination facilities and the eliminate of related loads, the 300 kW generator now has adequate capacity, however the aerator motors must still be manually reset in the event of a power outage. • Analog meter relays have failed.	• As part of modifications to the system to ensure reliable nitrification and denitrification, address other minor performance issues (e.g., foam control). Alternatives for achieving this objective will be discussed in later Chapters. • Measure grit deposits. • Implement SCADA controls to automatically restart aerator motors in the event of a power outage. • Repair or replace failed meter relays.

Table 1. Conditions Assessment of Liquid Treatment Processes

Unit Process / Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
Secondary Clarifiers	• The Secondary Splitter Box is in good condition. • The clarifier drive mechanisms and gear boxes are in decent condition. • The submerged portions of the clarifier mechanisms have only been inspected once during their life, but the date of the inspection is unknown. The condition of the metal components of the clarifier mechanism is unknown; City staff is unsure whether the components are galvanized or epoxy coated. City staff intends to drain and inspect one clarifier following installation of a new RAS pump, if possible. • The clarifier's drive mechanism torque gauges need to be overhauled. • Skimmer arms need new wipers, and at least one of the arms is slightly bent and rusted. • Globe valves on the utility water spray headers need to be replaced with ball valves. • One of the utility stations serving the clarifiers is out of service and is in need of repair. • Bird droppings along the clarifier bridge are excessive and are hosed down weekly. • Corrosion can be observed on portions of the diamond plate decking near the clarifier drives, where the coating is either missing or cracked.	• None identified.	• Drain and inspect each clarifier, if possible. • Sandblast and paint submerged clarifier mechanisms if warranted by inspection findings. • Overhaul drive mechanism torque gauges. • Replace skimmer arm wipers. • Replace spray header globe valves with ball valves. • Repair out of service utility station(s). • Miscellaneous coating touchup.
Polymer Storage and Dosing Facility	• None identified.	• None identified.	• No improvements are recommended for the Polymer Storage and Dosing Area.
Filter Feed and Utility Water Pump Station	• Filter Feed Pump No. 2 has experienced motor issues that are being addressed under the manufacturer's warranty. • Isolation valves for the Filter Feed pumps were throttled during startup to provide optimal backpressure and remain in the same positions. • The knife gate bypass valves have only been exercised once since installation. • Following construction, some ponding was observed on the pump station top deck; the Contractor drilled several holes in the deck to correct the problem. • The bypass valve vault collects water and needs to be pumped out frequently using a diesel powered portable pump. • Utility stations and seal water assemblies supplied by the high pressure W3 pumps are susceptible to clogging from solids pumped out of the wet well. • City staff would ultimately like to provide solely disinfected tertiary effluent for the W3 system.	• A turbidimeter installed in the tertiary filter influent (*i.e.,* Filter Feed pump discharge) header records periodic spikes in turbidity, typically between 3 and 5 a.m. City staff have been unable to diagnose the cause of the spikes, but suspect they may be partially due to the way in which the turbidimeter probe is installed. • Power is available nearby the bypass vault for a future dedicated sump pump.	• Provide a dedicated bypass vault sump pump and hard piping, with level control by float switch • Change the source of W3 to UV effluent by relocating the high-pressure W3 pumps, or provide strainers on the utility water pump discharge headers to protect both W3 systems from solids overflow until such time as the source of W3 is changed to UV effluent • Provide portable gasoline or electrically powered valve actuator to facilitate exercising of bypass knife gate valves (and other valves throughout the WWTP). • If turbidity spikes continue, coordinate with turbidimeter manufacturer. Modify location or installation of tertiary filter influent turbidimeter if warranted to resolve undiagnosed periodic high turbidity readings. • Add or relocate high-pressure W3 pumps to the UV effluent channel to provided disinfected water to the plant's W3 system.

Table 1. Conditions Assessment of Liquid Treatment Processes

Unit Process / Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
Tertiary Disc Filters	• Prior to a recent solids overflow event at the Secondary Clarifiers, the disc filter panels had never been manually maintained. City staff tried cleaning the panels using chemicals (sodium hypochlorite, per manufacture recommendations, and citric acid), but did not notice significant improvement. Panels were then pressure-washed, which has resulted in a significant decrease in the frequency of backwashing cycles. Because the panels were washed while lying on asphalt, pinholes are now present in most panels. Replacement panels are likely required. • The filter backwash troughs are difficult to clean due to limited access; plastics tend to accumulate on the troughs during backwash cycles, and can block flow from the furthest upstream filter panels without frequent cleaning. • Pressure gauges on the backwash pump systems appear to be of poor quality, and require or have already required glycerin replacement. The braided hose used to bypass the backwash throttling valve on one filter unit failed and has been replaced. Backwash sprayer O-rings have been replaced. Nylon nuts on the filter panels have experienced stripping.	• None identified.	• Provide mesh screen in filter influent chambers to remove plastics and other debris. • Repair or replace backwash piping pressure gauges. • Replace damaged filter panels.
Tertiary Plant Drain Pump Station	• Pressure cleanouts on the drain line from the UV facility need a way to bleed air pressure; when the UV channel mud valves are opened, the resulting compression of air in the line can pop the cleanout lids off.	• City staff would prefer an ultrasonic level indicator with a HIGH-HIGH float switch, rather than a submersible level transmitter with multiple float backups. The submersible transmitter and bottom two floats and their respective cables have experienced problems with debris and grease inside the wet well, requiring frequent removal and cleaning by staff.	• Replace submersible level transmitter (i.e., pressure transducer) and LOW-LOW, LOW, and HIGH float switches with an ultrasonic level sensor; modify HIGH-HIGH float backup control logic to timer draw-down. • Provide air release mechanism for pressure cleanout lids on UV facility drain line.
UV Disinfection	• Approximately 10 UV module ballasts have been replaced. Approximately 20 UV lamps have been replaced, mostly due to breakage. At least one UV lamp fuse blew, which falsely indicated that the bulb had failed. One of the hydraulic service centers leaked, but the manufacturer (Trojan) fixed it during a site visit. • The influent gates have experienced overtorquing and leakage issues. During installation, spacers at the tops of the gate frames were mistakenly discarded, resulting in the initial overtorque problems and subsequent gate failure. This issue was corrected by the installation contractor and the gate manufacturer, Golden Harvest. Subsequent overtorquing issues resulted from previous maintenance. Gate and frame seals have also separated and resulted in leaking. City staff has been dealing directly with Golden Harvest to address the leakage issues, and the gates have recently been leak-free. Weekly greasing of gate shafts is important.	• The ultrasonic level sensor on Channel No.1 had to be repaired by City staff. • Power wires for the hoist came undone, but were repaired by City staff. • There is no stop button on the chemical wash tank agitation air blower; the emergency stop button is used to shut the unit down.	• Evaluate enclosing the UV structure to protect against weather, dust, and debris. • Remove eye-level sunshades at instrument panels. • Add a safety grate to the UV effluent channel access hatch.

Table 1. Conditions Assessment of Liquid Treatment Processes

Unit Process / Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
	• The influent turbidimeter peristaltic sample pump was replaced by City staff after failing; the new Stenner unit has been problem-free • There is no safety grate for the access hatch over the UV effluent channel. Sunshades for several of the instrument panels are at eye/head height, and are potentially dangerous. • Due to the canopy height, the UV facility is vulnerable to dust, debris, and slanting rainfall. Dust and debris have the potential to shield pathogens in the UV channels. City staff is interested in potentially enclosing the structure to avoid such concerns.		
Effluent Diversion Structure	• City staff has generally had issues dealing with gate/actuator system suppliers, as no single entity will claim responsibility for the overall system.	• The sluice gate/hydraulic actuator system has experienced problems and has been modified several times. The gate only provides “open” and “closed” position signals, but the monitoring system has reported both conditions simultaneously (resulting in double illumination of indicator lights). City staff report that “closed” position switch no longer functions. City staff is planning to install a whisker switch on the gate.	• Install whisker switch on hydraulically actuated sluice gate.
Effluent Flow Monitoring	• The mag meter is very close to the vault wall, making the removal and re-installation of the instrument very difficult. The vault does not drain well and tends to collect water. City staff suggests that a small solar-powered sump pump would resolve the drainage issue. • The 30” line between the cascade aerator and the vault upstream of the mag meter is only partially plugged. The 30” line between the Effluent Storage Reservoir and the cascade aerator is plugged with an inflatable plug. A more permanent plug is needed to ensure water from the Effluent Storage Reservoir does not enter the flow meter vault.	• City staff is concerned that the mag meter’s electronic components will be compromised in the event of prolonged submergence. • Power is available within 100 feet for a future dedicated sump pump.	• Enlarge the mag meter vault for improved instrument access and maintenance. • Provide a hard-wired or solar-powered drainage pump in the mag meter vault. • OR Replace existing flow monitoring vault with new vault nearer to Effluent Diversion Structure • Permanently plug the 30” line between the Effluent Storage Reservoir and the cascade aerator.
Chlorine Contact Chamber (Abandoned)	• The Chlorine Contact Chamber is no longer in use. However, the 36-inch pipelines that once connected the structure to the Secondary Clarifiers and the Effluent Diversion Structure have been blocked, so wastewater can no longer enter the structure.	• None identified.	• Demolish the structure, as needed, to accommodate new facilities as defined in the Master Planning effort.

Table 2. Conditions Assessment of Solids Treatment Processes

Unit Process / Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
Secondary Sludge Pump Station	• The City plans to overhaul the Secondary Sludge Pump Station during 2012 and 2013. Improvements will include new VFD-driven RAS and WAS pumps, constant speed scum pumps, flow metering, and isolation valves. The original WAS, RAS and scum pumps are at or near the end of their useful lives, and have inadequate isolation valving.	• The existing solids handling pumps have inadequate flow metering capabilities. • None of the solids handling pumps are currently monitored or controlled by the plant's SCADA or PLC systems.	• Implement planned near-term overhaul (replace pumps and motor controls, provide additional valving, provide additional flow metering). • Provide additional SCADA monitoring and control capabilities.
Sludge Lagoons	• The Hypalon lagoon liner seams are coming apart in some areas and have been patched twice, but the liner is generally in good condition. Seam failures may be responsible for recent elevated nitrate readings in nearby monitoring wells. • The decant pump station level control gates are in good condition. • One of the surface aerators is broken, due to use in very thick sludge. • There is only one vehicle access ramp to the lagoons, at the southeast corner of the south lagoon. There are no pedestrian stairs to the top of the lagoons, requiring staff to either climb the berm or walk up the vehicle ramp.	• The decant pump needs a new control panel; the current panel was fabricated by City staff from spare parts.	• Replace Hypalon liner system in both lagoons. • Install new, robust lagoon mixing equipment for homogenizing thickened (i.e., decanted) sludge prior to dewatering. • Provide an additional vehicle access ramp to the lagoon berm. • Provide pedestrian stairway to the top of the lagoon berm. • Replace decant pump control panel.
Sludge Feed Pump Station	• The rotor on sludge feed pump No.2 is scarred, due to overtightening of the stator during shipment and/or startup. The rotor shows grooving and early signs of fatigue. Performance of the pump has not suffered, however. • There is no dedicated sample port and sludge samples must be obtained by forcing sludge through an air release valve.	• Flow switches on the sludge feed lines do not work properly and the pump protection mechanisms originally installed (flow switch, temperature switch and pressure switch) have all been bypassed, allowing the pumps to operate under dangerous conditions.	• Restore temperature and pressure switches to service to protect sludge feed pumps from overpressurization and run dry conditions. • Provide a dedicated sample port on sludge feed line, either at the Sludge Feed Pump Station, or at the Biosolids Polymer Facility.
Biosolids Polymer Facility	• City staff would like the polymer storage area to include spill containment. • Staff currently use a palette jack for moving polymer totes, but a wheeled gantry crane would likely be required to lift totes into position on or in curbed containment unit. • The polymer dilution water feed needs a pressure reducing valve (PRV); the PRV that was originally installed for this purpose is not available. • The air release valve on the static flocculator/mixer device is currently used to obtain sludge samples. An alternative to this location is recommended. • City staff suggested installing a concrete pad outside the building to prevent palette jack/gantry crane wheels from damaging the asphalt on hot days. • City staff believes that the flocculator/mixer device may be partially clogged.	• None identified.	• Provide a spill containment unit for polymer totes/drums. • Provide a gantry crane or similar for maneuvering totes. • Provide a concrete landing outside building. • Provide a PRV on dilution water line. • Provide an alternative sludge sampling port.

Table 2. Conditions Assessment of Solids Treatment Processes

Unit Process / Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
Biosolids Dewatering Beds	• There is no way to throttle sludge flow through the two vertical risers in each bed, and staff have taken to manually restricting and/or blocking flow in either riser for periods of a given loading cycle. • Sludge harvesting tractor is not well balanced and the bucket vibrator has broken twice. City staff believes the bucket is too large for the vibrator and tractor. Alternative sludge harvesters with horizontal augers (similar to a snow-blower) are being investigated.	• None identified.	• Provide throttling capability on the sludge feed risers. • Provide modifications to the sludge harvester or provide a more robust harvester.
Filtrate Pump Station	• Pump No.2 had a paintbrush lodged in it at one point, but has been repaired.	• One of the pumps experienced unexplained failures for about a month. The manufacturer was unable to diagnose the issue. The pump is currently operating fine. • City staff would prefer an ultrasonic level indicator with a HIGH-HIGH float switch, rather than a submersible level transmitter with multiple float backups. The submersible transmitter and bottom two floats and their respective cables have experienced problems with debris and grease inside the wet well, requiring frequent removal and cleaning by staff.	• Replace submersible level transmitter (i.e., pressure transducer) and LOW-LOW, LOW, and HIGH float switches with an ultrasonic level sensor; modify HIGH-HIGH backup control logic to timer draw-down.

Table 3. Conditions Assessment of Storage and Irrigation Systems

Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
Storage Basins	• None identified.	• None identified.	• The condition of the storage ponds is adequate for future irrigation water storage and no improvement has been identified. • Evaluate need to provide Auxiliary Basin with floating aerators for treatment of sludge lagoon supernatant before returning to Headworks. Alternatives for achieving this objective will be discussed in later Chapters.
Irrigation Pump Stations	• Fish are able to enter Irrigation Pump Station No. 1, and become trapped in the wet well or become entrained by the pump. Fish caught in the wet well decompose and generate odors. Pieces of fish that enter the pump clog irrigation risers in fields. • The pump in Irrigation Pump Station No. 1 is old but is still operable. It is unlikely that a second (standby) vertical turbine pump would fit in the wet well. • The vertical turbine pump at the Irrigation Pump Station No. 2 is also old, but operates well. Presently there is minimal grating at the top of the pump station; the pump is mounted on two I-beams that straddle the wet well. • The single pump at Return Pump Station No. 1 is relatively new, and operates well. • The pumps at Return Pump Station No. 2 are approximately 8 years old. A drain line from the Return Pump Station No. 2 to Skunk Creek is open during the winter, but is plugged during irrigation season. • Access for pump removal at all pump stations is problematic. Before a tractor is able to remove the pumps, an earthen mound must be constructed near the wet well.	• The control panels for Irrigation Pump Station No. 1 and Return Pump Station No. 1 are old and corroded, and require replacement. The new panels should be weatherproof and should be placed at a higher elevation. • Rodents are able to enter the wet well for Irrigation Pump Station No. 2 and then become trapped. The rodents chew through the wiring for the wet well level pressure transducer, which can cause the pump to fault. An ultrasonic level sensor and/or additional pump run dry protection should be provided.	• Provide fish screens for Irrigation Pump Station No. 1. • Provide grating for Irrigation Pump Station No. 2. • Replace pressure transducer in Irrigation Pump Station No. 2 wet well with ultrasonic level sensor. • Provide a longer service truck boom to facilitate pump removal. • Provide new simplex control panels with local power distribution for Irrigation Pump Station No. 1 and Return Pump Station No. 1. • Stock a complete pump and motor assembly for each irrigation pump station to minimize the time required to return a station to service upon equipment failure. • Stock a complete pump and motor assembly for each return pump station to minimize the time required to return a station to service upon equipment failure,
Irrigation Facilities	• None identified.	• None identified.	• The condition of the irrigation facilities is adequate for future use and no improvement has been identified.

Table 4. Conditions Assessment of Electrical and Control Systems

Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
Utility Service	• None identified.	• The existing service can accommodate a limited expansion of electrical loads (approximately 120 hp). Additional loads exceeding this amount will require a larger service and potentially modifications to power distribution throughout the plant.	• None identified.
Electrical Switchgear	• None identified.	• Equipment located in the old switchboard is old, but could be replaced in kind if necessary.	• None identified.
Standby Generators	• None identified.	• The 600 kW generator experienced issues during scheduled maintenance runs (over-revolution alarms), but the problem was corrected by the generator manufacturer and has functioned reliably since. • The load bank for the 300 kW generator has malfunctioned and is no longer operational. The load bank is normally used to load down the generator during periodic maintenance runs (approximately once annually).	• None identified.
Motor Control Centers	• None identified.	• Some of the equipment located in the original MCC (Control Building), MCC-MCE and MCC-MCA (Chemical / Maintenance Building) is old but can be replaced in kind if necessary. • The power factor correction unit in MCC-MCE is not functional, but is no longer needed.	• None identified.
SCADA System	• None identified.	• During startup, the radio link between the plant and MSC experienced connectivity issues, resulting in problems monitoring and/or controlling facilities remote to the plant. Recent performance has been good, however. • Notification software (SCADAlarm) for the remote facilities has minimum backup power at MSC. • Water section Utilities Division employees do not have a dedicated SCADA computer at the WWTP for monitoring water facilities. • Staff has expressed desire for additional automatic reporting from the SCADA system.	• Provide dedicated Water section SCADA computer at WWTP. • Migrate the MSC SCADA system to the WWTP. • Develop and implement functional SCADA reports for Water and Wastewater section systems as requested by staff.
Solar Facility	• None identified.	• None identified.	• The solar system is new and in good working condition. No improvements are necessary.

Table 5. Conditions Assessment of Plant Water Systems

Facility	Mechanical and Structural Issues	Electrical, Instrumentation and Control Issues	Recommended Activities and Improvements
Plant High-Pressure Utility Water (W3) System	• Some utility stations have experienced ruptures at PVC-galvanized pipe unions.	• None identified.	• Replace PVC nipples with threaded female connections with stainless steel sleeves for additional strength
Plant Nonpotable Water (W2) System	• The sight glass on the side of the new 3,000-gallon hydropneumatic tank has a leak.	• None identified.	• Repair or replace new hydropneumatic tank sight glass.

Table 6. Consolidated Improvement Recommendations and Priority Rankings

Process Area	Improvement No.[a]	Recommended Improvements	Improvement Category and Need (3=Critical Need, 2=Moderate Need, 1=Minor Need)						Priority	Part of Normal O&M Activities
			Performance	Operability & Ease of Maintenance	Condition	Safety	Capacity	Redundancy		
LIQUID TREATMENT PROCESSES										
Headworks / Screening		Secure metal grating				3			HIGH	YES
Headworks / Screening	HW-1	Replace existing climber screen with new mechanical climber screen with 1/4-inch openings			2			2	MODERATE	
Headworks / Screening	HW-2	Provide redundant climber screen with 1/4-inch openings and screenings washer for two screens	2	2				3	HIGH	
Headworks / Screening	HW-3	Provide automatically actuated control gates for the each screening channel		3				3	HIGH	
Grit Removal	GR-1	Provide cleanouts on drain line	2	3					HIGH	
Grit Removal	GR-2	Modify vortex chamber scour piping	3	3					HIGH	
Grit Removal	GR-3	Remove and replace damaged vortex chamber liner			2				LOW	
Grit Removal		Repair coatings, various locations			1				LOW	YES
Grit Removal		Replace slide gate shaft sheath			1				LOW	YES
Oxidation Ditches	OX-1	Modify system to ensure reliable nitrification and dentrification	3				3		HIGH	
Oxidation Ditches	OX-2	Improve foam control sprayer effectiveness		1					LOW	
Oxidation Ditches		Measure grit deposits in ditches	3		1		1		HIGH	YES
Secondary Clarifiers		Drain and inspect clarifiers			3				HIGH	YES
Secondary Clarifiers	SC-1	Sandblast and paint submerged clarifier mechanisms if warranted by inspection findings			2				LOW	YES
Secondary Clarifiers		Replace scum skimmer arm wipers	2		2				MODERATE	YES
Secondary Clarifiers		Repair coatings, various locations			2				LOW	YES
Secondary Clarifiers		Repair out of service utility station(s)		1					LOW	YES
Secondary Clarifiers		Overhaul clarifier torque gauges		1	2				LOW	YES
Secondary Clarifiers		Replace spray header globe valves with ball valves		1					LOW	YES
Filter Feed & Utility Water Pump Station	FF-1	Add strainers to W3 discharge piping while pumps supply secondary effluent		3					HIGH	
Filter Feed & Utility Water Pump Station	FF-2	Relocate high-pressure W3 pumps to pump UV-disinfected effluent	2	2		2			MODERATE	
Filter Feed & Utility Water Pump Station	FF-3	Provide dedicated bypass vault sump pump and hard piping and float control		2		2			MODERATE	
Filter Feed & Utility Water Pump Station		Investigate / modify installation of filter feed turbidimeter	1						LOW	YES
Filter Feed & Utility Water Pump Station		Provide portable valve actuator	1	1					LOW	YES
Tertiary Disc Filters	TF-1	Provide coarse screens to remove plastics	2	2					MODERATE	
Tertiary Disc Filters		Replace backwash piping pressure gauges			1				LOW	YES
Tertiary Disc Filters		Replace compromised disc filter panels	3		2				HIGH	YES
Tertiary Plant Drain Pump Station		Replace lower floats and submersible level transmitter with ultrasonic level sensor		2	2				MODERATE	YES
Tertiary Plant Drain Pump Station	PD-1	Provide air release mechanism on pressure cleanout lids at UV facility		1		2			LOW	
UV Disinfection	UV-1	Evaluate enclosing UV structure		1					LOW	
UV Disinfection		Remove eye-level sunshades at instrument panels				2			LOW	YES
UV Disinfection		Provide safety grate at UV effluent channel access hatch				2			LOW	YES
Effluent Diversion Structure		Install whisker switch on hydraulically actuated sluice gate	2	3					HIGH	YES
Effluent Flow Monitoring	FM-1	Enlarge flow meter vault		2					LOW	
Effluent Flow Monitoring		Provide flow meter vault drain pump	2	2	2				MODERATE	YES
Effluent Flow Monitoring	FM-2	(In lieu of FM-1) Replace existing flow monitoring vault with new vault nearer to Effluent Diversion Structure		2					LOW	
Effluent Flow Monitoring		Permanently plug 30" line from reservoir to cascade aerator	3			1			HIGH	YES
SOLIDS TREATMENT PROCESSES										
Secondary Sludge Pump Station		Replace RAS, WAS and scum pumps, add flow metering	2	2	1				MODERATE	YES
Sludge Lagoons	SL-1	Replace surface aerators with robust mixing equipment	3	2	2				HIGH	
Sludge Lagoons	SL-2	Provide additional truck access ramp to lagoons		1					LOW	
Sludge Lagoons	SL-3	Provide pedestrian stairway to lagoons		2		2			MODERATE	
Sludge Lagoons	SL-4	Replace lagoon liners	2	1	1				MODERATE	
Sludge Feed Pump Station		Restore progressing cavity pump temperature and pressure switches	3	3		3			HIGH	YES
Biosolids Polymer Facility	BP-1	Provide polymer containment unit(s)				1			LOW	
Biosolids Polymer Facility	BP-2	Provide gantry crane or similar for maneuvering totes		2		1			LOW	
Biosolids Polymer Facility	BP-3	Provide concrete landing outside building			1				LOW	
Biosolids Polymer Facility		Add PRV on dilution water feed line	2						LOW	YES
Biosolids Polymer Facility		Provide dedicated sludge sample port		3					HIGH	YES
Biosolids Dewatering Beds		Provide throttling capability on sludge feed risers	3	3					HIGH	YES
Biosolids Dewatering Beds		Modify sludge harvester or provide more robust harvester	3	3			3		HIGH	YES
STORAGE AND IRRIGATION SYSTEMS										
Auxiliary Basin	AB-1	Provide aerators to reduce ammonia load of sludge lagoon supernatant	2				2		MODERATE	
Irrigation Pump Stations		Provide fish screens at Irrigation PS No. 1 (Dam Pump)	2	2					MODERATE	YES
Irrigation Pump Stations		Provide grating at Irrigation PS No. 2 (40-Acre Pumps) wet well		2		3			HIGH	YES
Irrigation Pump Stations		Provide boom arm on service vehicle to facilitate pump removal		2		2			MODERATE	YES
Irrigation Pump Stations		Stock complete spare pump and motor assembly for each irrigation pump station		2				3	MODERATE	YES
Return Pump Stations		Stock complete spare pump and motor assembly for each return pump station		2				3	MODERATE	YES
ELECTRICAL AND CONTROL SYSTEMS										
SCADA and Controls	CO-1	Provide dedicated Water Division SCADA computer and software in O&E Bldg		3					HIGH	
SCADA and Controls	CO-2	Migrate MSC SCADA system to WWTP	2	2				2	MODERATE	
SCADA and Controls	CO-3	Headworks / Screening: Replace and relocate control panel to grade		1	3	1			HIGH	
SCADA and Controls	CO-4	Headworks / Screening: Provide level-based rake controls	2	2					MODERATE	
SCADA and Controls	CO-5	Headworks / Screening: Provide auger failure alarm	2	1		2			MODERATE	
SCADA and Controls	CO-6	Headworks / Screening: Add SCADA monitoring and control capabilities for rake and auger		2					LOW	
SCADA and Controls	CO-7	Oxidation Ditches: Implement SCADA controls to automatically restart aerator motors during power outage						3	HIGH	
SCADA and Controls		Oxidation Ditches: Repair or replace failed meter relays		1	1				LOW	YES
SCADA and Controls	CO-8	Secondary Sludge: Add SCADA monitoring and control capabilities for RAS pumps, scum pump station and WAS pumps		2					LOW	
SCADA and Controls	CO-9	UV Disinfection: Add local OFF switch to chemical wash tank blower		2		3			HIGH	
SCADA and Controls	CO-10	Irrigation Pump Stations: Provide new control panels for Irrigation PS No. 1 and Return PS No. 1			3				HIGH	
SCADA and Controls		Irrigation Pump Stations: Replace pressure transducer at Irrigation PS No. 2 with ultrasonic level sensor and no/low flow indication	2	1	1				MODERATE	YES
SCADA and Controls		Filtrate Pump Station: Replace lower floats and submersible level transmitter with ultrasonic level sensor		2	2				MODERATE	YES
SCADA and Controls		Develop and implement functional SCADA reports for water and wastewater systems		2					LOW	YES
PLANT WATER SYSTEMS										
Nonpotable Wells and Hydropenumatic Tank		Repair or replace hydropneumatic tank sight glass		1					LOW	YES
Plant Washwater System	W2-1	Place W2-supplied utility stations on disinfected W3 system		1			1		LOW	
Plant Washwater System		Replace rupture-prone PVC unions at utility stations		1	1				LOW	YES

[a] Includes only those improvements not assumed to be part of normal O&M activities.

Table 7. Estimated Capital Costs of Recommended Improvements

Improvement No.	Process Area	Description	Priority	Estimated Capital Cost
HW-1	Headworks / Screening	Replace existing climber screen with new mechanical climber screen	Moderate	$247,000
HW-2	Headworks / Screening	Provide redundant climber screen and screenings washer for two screens	High	$594,000
HW-3	Headworks / Screening	Provide automatically actuated control gates for the each screening channel	High	$222,000
GR-1	Grit Removal	Provide cleanouts on drain line	High	$11,000
GR-2	Grit Removal	Modify vortex chamber scour piping	High	$4,000
GR-3	Grit Removal	Remove and replace damaged vortex chamber liner	Low	$7,000
OX-1	Oxidation Ditches	Modify system to ensure reliable nitrification and denitrification addresses other performance issues	High	[a]
SC-1	Secondary Clarifiers	Sandblast and paint submerged clarifier mechanisms if warranted by inspection findings	Low	$202,000
FF-2	Filter Feed & Utility Water Pump Station	Relocate high-pressure W3 pumps to pump UV-disinfected effluent	Moderate	$98,000
FF-3	Filter Feed & Utility Water Pump Station	Provide dedicated bypass vault sump pump and hard piping and float control	Moderate	$7,000
TF-1	Tertiary Disc Filters	Provide coarse screens to remove plastics	Moderate	$12,000
PD-1	Tertiary Plant Drain Pump Station	Provide air release mechanism on pressure cleanout lids at UV facility	Low	$4,000
UV-1	UV Disinfection	Evaluate enclosing UV structure	Low	$288,000
FM-1	Effluent Flow Monitoring	Enlarge flow meter vault	Low	$12,000
FM-2	Effluent Flow Monitoring	(In lieu of FM-1) Replace existing flow monitoring vault with new vault nearer to Effluent Diversion Structure	Low	$41,000
SL-1	Sludge Lagoons	Replace surface aerators with robust mixing equipment	High	[a]
SL-2	Sludge Lagoons	Evaluate need for additional lagoon or mixing day tank	High	[a]
SL-3	Sludge Lagoons	Provide pedestrian stairway to lagoons	Moderate	$31,000
SL-4	Sludge Lagoons	Replace lagoon liners	Moderate	$70,000
BP-1	Biosolids Polymer Facility	Provide polymer containment unit(s)	Low	$6,000
BP-2	Biosolids Polymer Facility	Provide gantry crane or similar for maneuvering totes	Low	$9,000
BP-3	Biosolids Polymer Facility	Provide concrete landing outside building	Low	$2,000
AB-1	Auxiliary Basin	Provide aerators to reduce ammonia load of sludge lagoon supernatant	Moderate	[a]
W2-1	Plant Washwater System	Place W2-supplied utility stations on disinfected W3 system	Low	$7,000
CO-1	SCADA and Controls	Provide dedicated Water Division SCADA computer and software in O&E Bldg	High	$41,000
CO-2	SCADA and Controls	Migrate MSC SCADA system to WWTP	Moderate	$61,000
CO-3	SCADA and Controls	Headworks / Screening: Replace and relocate control panel to grade	High	$31,000
CO-4	SCADA and Controls	Headworks / Screening: Provide level-based rake controls	Moderate	$19,000
CO-5	SCADA and Controls	Headworks / Screening: Provide auger failure alarm	Moderate	$7,000
CO-6	SCADA and Controls	Headworks / Screening: Add SCADA monitoring and control capabilities for rake and auger	Low	$24,000
CO-7	SCADA and Controls	Oxidation Ditches: Implement SCADA controls to automatically restart aerator motors during power outage	High	$7,000
CO-8	SCADA and Controls	Secondary Sludge: Add SCADA monitoring and control capabilities for RAS pumps, scum pump station and WAS pumps	Low	$41,000
CO-9	SCADA and Controls	UV Disinfection: Add local OFF switch to chemical wash tank blower	High	$9,000
CO-10	SCADA and Controls	Irrigation Pump Stations: Provide new control panels for Irrigation PS No. 1 and Return PS No. 1	High	$102,000

[a] The need, scope and estimated costs of these improvements will be evaluated, developed and presented in some detail in later Chapters of the WWTP Facilities Master Plan.

APPENDIX F

Pond Use Analysis

WEST YOST ASSOCIATES
Consulting Engineers

TECHNICAL MEMORANDUM

DATE: December 21, 2012 Project No.: 175-00-12-36

TO: Mark Clarkson, City of Galt

FROM: Kathryn Gies, R.C.E. #65022

REVIEWED BY: Jeff Pelz, R.C.E. #46088

SUBJECT: Analysis of Potential WWTP Storage Pond Uses

This Technical Memorandum (TM) presents an evaluation of potential uses and management considerations for the following available storage facilities at the City of Galt Wastewater Treatment Plant (WWTP): Auxiliary Basin, Effluent Storage Reservoir, and Storages Ponds 1, 3 and 4.

A detailed description of the storage pond facilities is provided in Chapter 3 of the Facilities Master Plan. In total, the storage facilities provide approximately 62.4 million gallons (Mgal) of storage capacity However, the 5.8 Mgal Auxiliary Basin has been retrofitted to provide emergency storage of influent wastewater, and generally operates independently from the remaining ponds.

Also noted in Chapter 3 of the Facilities Master Plan, the storage capacity was historically needed because discharges to Skunk Creek were not allowed between April 1 and October 31 of each year. However, since October 2011, the City has been permitted to discharge effluent to Skunk Creek year-round. Therefore, the need to store treated effluent over the dry months has been eliminated, creating a large amount of available storage capacity. The newly available storage capacity should be used to the maximum benefit of the City.

The following specific uses of the City's storage capacity are considered in this TM:

- **Influent Equalization:** With some minor modifications at the WWTP headworks, the existing Auxiliary Basin could be used to store any influent wastewater that exceeds a set flow amount. This added flexibility in plant operations increases the level of reliability for the WWTP as a whole, and allows the City to consider providing a lower firm peak flow handling capacity for each individual unit process downstream, thereby resulting in significant savings.
- **Irrigation Water Storage**: During the summer irrigation season, the City uses a portion of the storage capacity provided by the Effluent Storage Reservoir to hold recycled water that is used for irrigation of the City-owned agricultural property surrounding the WWTP.
- **Emergency Storage**: The Effluent Storage Reservoir and three storage ponds can be used to store final effluent in the event of an unexpected operational condition that could impair effluent water quality.

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- **Secondary Effluent Diversions**: The tertiary system was designed to allow for diversions of secondary effluent to the Effluent Storage Reservoir and three storage ponds if flow exceeds the available tertiary treatment system capacity. (This effluent is subsequently used for irrigation on the City-owned fields, or returned to the headworks.) Similar to influent equalization described above, this added flexibility allows the City to provide a lower firm peak flow capacity for each individual unit process downstream. In the recent WWTP upgrade project, this added feature allowed the City to reduce the total number of tertiary filtration and ultraviolet (UV) system treatment units that would otherwise be required, thereby resulting in significant savings.
- **Recycled Water Storage for Offsite Use**: If the City expands its irrigated area (or develops other recycled water uses) additional irrigation storage capacity may be needed, thereby potentially offsetting some of the ability to use to ponds for the uses described above.

This TM is structured around each of the above topics. In addition, a Summary and Conclusions section is presented at the end of this TM.

INFLUENT EQUALIZATION

As discussed in Chapter 6 of the Facilities Master Plan, it is envisioned the Auxiliary Basin will be used for influent equalization on rare emergency-type occasions (e.g. if the secondary effluent is exhibiting exceptionally poor settling properties, a primary or secondary treatment unit is unexpectedly offline for repair, etc.). During these conditions, flows through the WWTP would be limited to a set maximum based on what can be handled by the facilities that are online. This added flexibility will allow the City to reduce the number of primary units (i.e. microscreens) or secondary clarifiers that will be needed to ensure reliable treatment under all anticipated conditions.

The analysis presented in this section demonstrates what the flow through the WWTP could be limited to, given the anticipated influent flows at the three design conditions evaluated in the Facilities Master Plan and the capacity of the Auxiliary Basin. The following specific topics are discussed:

- Operational Assumptions
- Flow Assumptions
- Influent Equalization Analysis Results

Operational Assumptions

As described further below, there is limited winter-season flow data currently available for the WWTP, and a number of assumptions must be used in the assessment of equalization storage needs. Therefore, any reliance on equalization requires a design to accommodate a significant margin of error. As such, this equalization analysis considers the storage needed to hold all flow that exceeds a set amount in a worst-case 30 day influent flow period - with no return flow to the treatment process. In practice, at least some of the flows diverted to the Auxiliary Basin could be returned to the treatment plant over such a 30-day period. Therefore, this operational assumption should provide an adequate factor of safety for the equalization analysis.

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It is generally assumed that the primary and secondary treatment facilities will be sized to handle all dry season flow events. However, there is a possibility that the secondary effluent will exhibit poor settling characteristics at the same time a clarifier is offline for an extended planned maintenance period. In this scenario, the City would need to limit dry season peak flows through the plant. As noted above, it is likely the City would be able to return any diverted flows during off-peak periods. However, to be conservative, the analysis presented herein identifies what the peak dry weather flow can be limited to through the plant over a 60-day dry weather period given the size of the Auxiliary Basin *with* the assumption that no flow is returned to the WWTP.

Flow Assumptions

This section presents a summary of the flow assumptions used in developing the equalization analysis. The following specific topics are addressed.

- Synthesized Peak Flow Hydrographs
- Synthesized Dry Season Flow Curves
- Solids Handling Return Flows

Synthesized Peak Flow Hydrographs

While the City has recorded daily actual or estimated influent flow totals at the WWTP for many years, reliable diurnal hourly influent flow data is far more limited, and no such data exists at all for major storm events. Nevertheless, the need exists to approximate diurnal peak flow conditions so that influent flow volumes and diverted flow storage volumes may be estimated.

Although the City does not have an extended record of diurnal influent flow data, such records do exist for other cities in the region. While the magnitude of peak flows will differ from one city to the next, the pattern of peak flow conditions can be reasonably extrapolated from one city to another, if weather and infiltration/inflow (I&I) conditions are similar.

For this analysis, peak flow patterns from the City of Vacaville Easterly WWTP are used as the basis to approximate design peak flow conditions for the Galt WWTP. Specifically, the 1997/1998 wet season (November through April) data is used, as that was the second highest total annual precipitation in the last 50 years. The exact methodology for applying the Vacaville 1997/1998 data to the Galt WWTP design condition is as follows:

1. A diurnal base flow condition for the City of Vacaville is established based on dry season conditions associated with the 1997/1998 season.
2. The diurnal base flow values are subtracted from the 1997/1998 wet season data, with the result being the observed I&I pattern for the 1997/1998 wet season.
3. A diurnal base flow pattern for the Galt WWTP is established using recent dry season data, and then upward adjusted to correspond to the design average dry weather flow (ADWF) condition in question. For this analysis, design ADWF conditions of 3.0 mgd (Current Permitted Design Condition), 4.5 mgd (Future Permitted Design Condition), and 6.0 mgd (Ultimate Buildout Design Condition) are considered.

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4. The City of Vacaville observed I&I pattern is downward adjusted and then superimposed on the Galt design diurnal base flow pattern to achieve peak hour wet weather flow (PHWWF) condition in question. For this analysis, design PHWWF conditions of 9.0 mgd (Current Permitted Design Condition), 13.5 mgd (Future Permitted Design Condition), and 18.0 mgd (Ultimate Buildout Design Condition) are considered.
5. The worst-case 30-day period (February 1 through March 2) is then identified and used to determine the resulting peak flow through the WWTP when influent peak flow diversion is maximized during the wet-season in an emergency situation

The synthesized November through April flow curve for the Current Permitted Design is presented in Figure 1. (Note that the shape of the curve would be the same at the Future Permitted Design Condition and the Ultimate Buildout Design Condition; however, the y-axis would be shifted so that the peak flow conditions corresponds with the anticipated PHWWF for the given design condition.)

Synthesized Dry Season Flow Curves

The development of a synthesized dry season flow curve makes use of Galt WWTP influent flow data for the period of June–August 2011. These data were linearly adjusted to achieve design ADWF conditions of 3.0 mgd (Current Permitted Design Condition), 4.5 mgd (Future Permitted Design Condition), and 6.0 mgd (Ultimate Buildout Design Condition), respectively.

The synthesized design flow curve for Current Permitted Design Condition is presented in Figure 2. Note that the shape of the curve shown in Figure 2 would be the same at the Future Permitted Design Condition and the Ultimate Buildout Design Condition; however, the y-axis would be shifted so that the peak flow conditions corresponds with the anticipated PHDWF for the given design condition.

Solids Handling Return Flows

The Auxiliary Basin will also be used to hold the return flows from the solids handling processes. Therefore the solids processing return flows must be considered as part of the equalization analysis. Estimates of the daily return flow were determined based on the following assumptions:

- The mass of solids sent to the handling processes each day will be equal to the maximum month solids production rate (in dry tons per year) divided by 365 days
- The solids sent to the handling processes will have a 0.8% solids content
- Solids leaving the handling processes will have a 20% solids content
- Approximately 2/3 of the solids generated at the maximum month condition will be processed in the Deskins Bed. The volume of water returned from this process will be approximately double what would otherwise be generated (to account for the “clean” water used in the Deskin process to facilitate dewatering.)

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Figure 1. Synthetic Wet Season Hydrograph (Current Permitted Design Condition)

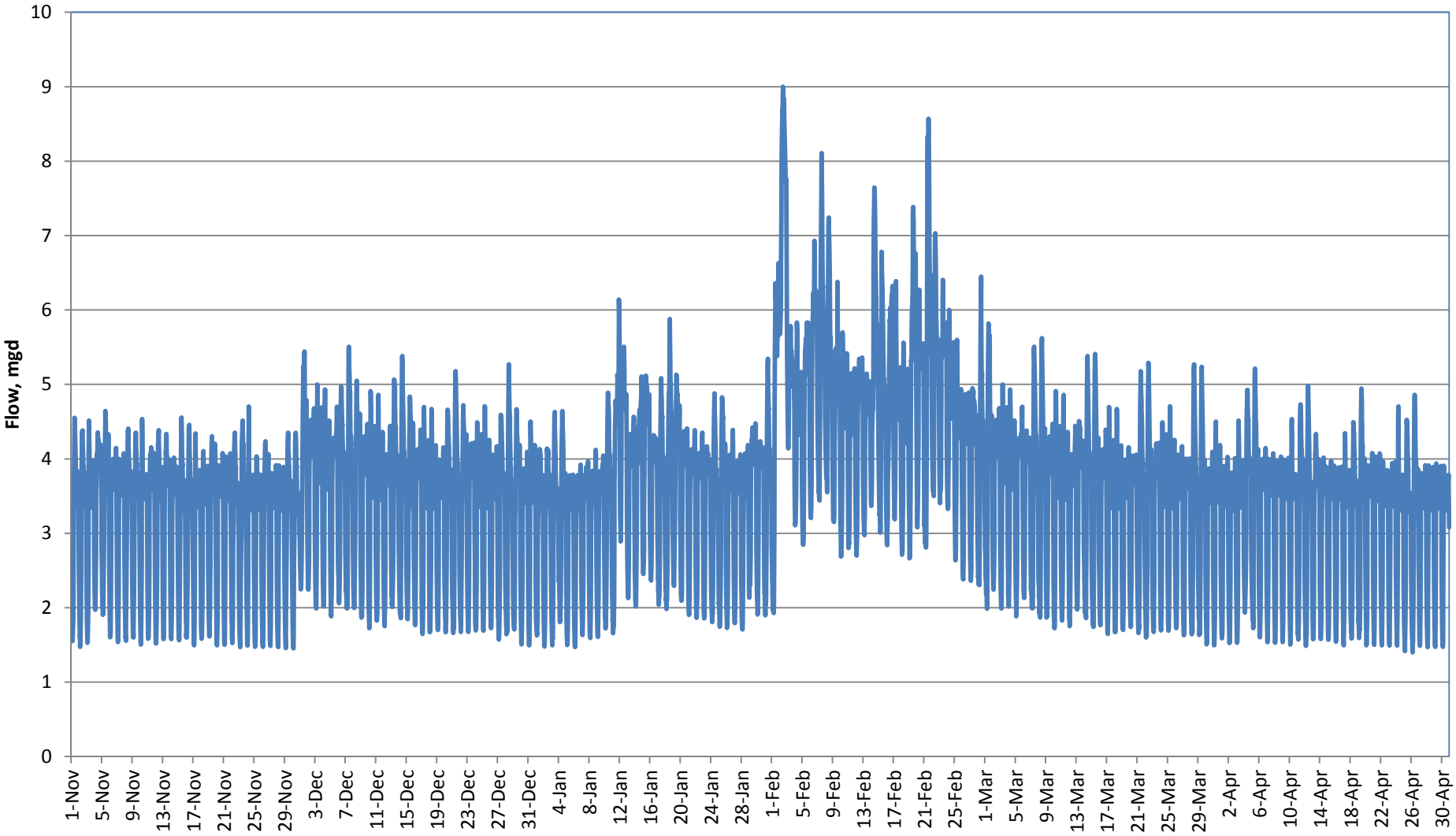
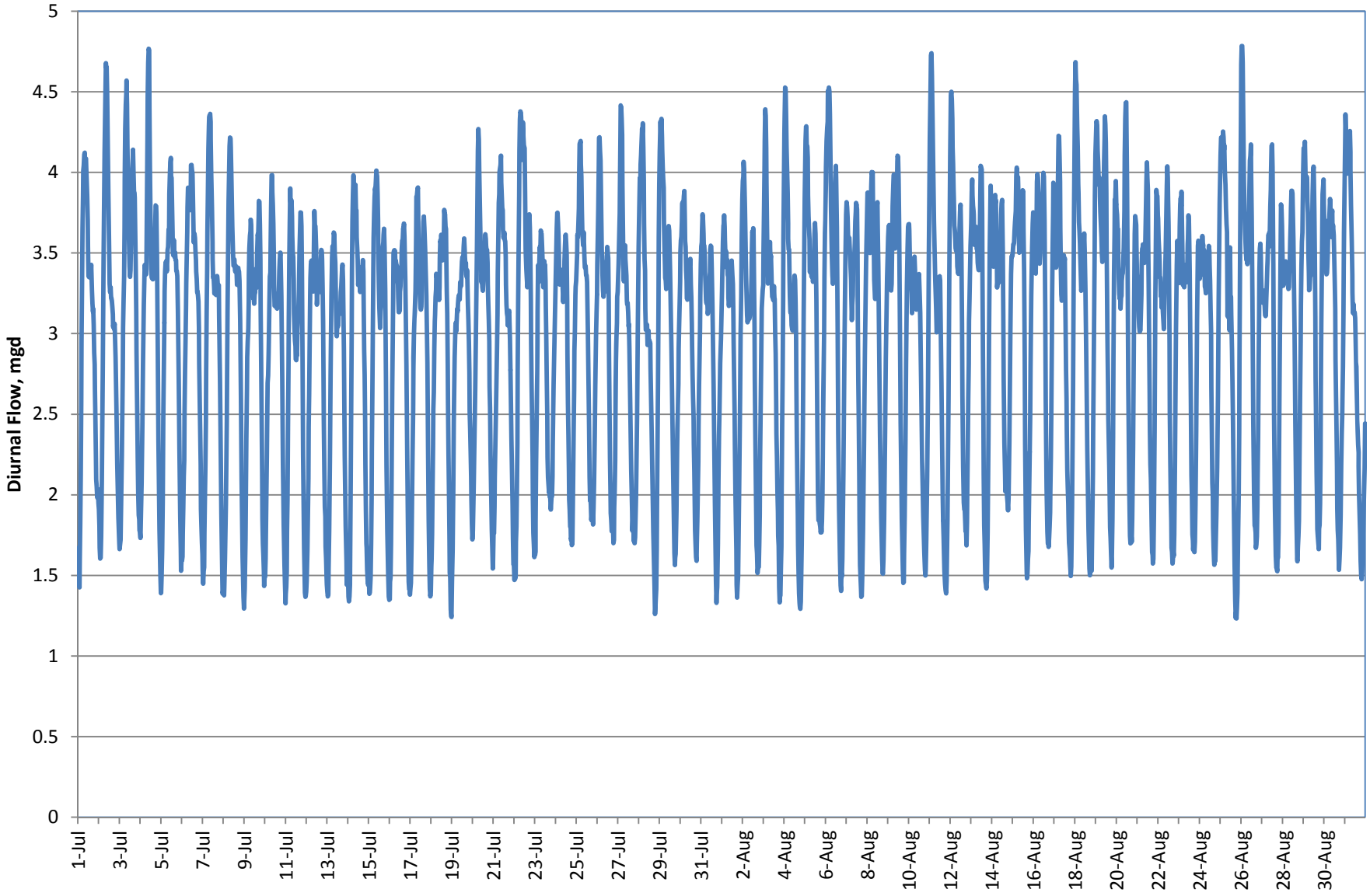


Figure 2. Synthesized Dry Weather Flow Curve (Current Permitted Design Condition)



A summary of the estimated return volumes is shown in Table 1. To simplify the influent equalization analysis, the solids handling return flows are assumed to be introduced back to the plant 7 days a week between the hours of 7 am and 3 pm.

Table 1. Estimate of Daily Return Flow Volumes for Influent Equalization Analysis

Parameter	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition
Maximum Month Solids Production Rate[(a)], dry ton per year	940	1,140	1,900
Volume of Solids Sent from the Secondary Process to the Handling Facilities, gallons per day	78,000	118,000	158,000
Volume of Solids Sent from the Handling Facilities to the Storage Area/Land Application, gallons per day	3,000	5,000	6,000
Volume of Solids Filtrate Flow Returned to Auxiliary Basin, gallons per day	75,000	113,000	152,000
Volume of Deskin Bed "Clean" Water Returned to Auxiliary Basin, gallons per day	50,000		
Total Estimated Volume Returned to Auxiliary Basin, gallons per day	**125,000**	**163,000**	**202,000**

[(a)] From Table 5-1 (Chapter 5 of the Facilities Master Plan).

Influent Equalization Analysis Results

The treatment flow thresholds determined using the equalization analysis for each of the design conditions evaluated in WWTP Facilities Master Plan are presented in Table 2. As indicated in the table, a significant reduction in peak flow can be achieved through the use of the Auxiliary Storage Basin. An analysis of the required treatment facilities given the ability to divert primary flows in emergency conditions is presented in Chapter 6.

Table 2. Influent Equalization Analysis Results

	Design Flow Conditions			Maximum WWTP Flow with Equalization, mgd		Max Cumulative Storage Required, Mgal	
Design Condition	ADWF	PHDWF	PHWWF	Winter	Summer	Winter	Summer
Current Permitted Design Condition	3.0	5.0	9.0	5.9	3.8	5.7	5.0
Future Permitted Design Condition	4.5	7.5	13.5	8.8	5.8	5.7	5.4
Ultimate Buildout Design Condition	6.0	10.0	18.0	12.1	7.9	5.6	5.2

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IRRIGATION WATER STORAGE

The City utilizes the storage ponds to hold irrigation water that is used for the 186 acres of irrigated agricultural properties that surround the WWTP. On average, the City applies approximately 4 feet per acre per year, totaling approximately 740 acre-feet (or 242 Mgal). While this is significantly less volume than the total amount of recycled water that can be generated by the WWTP in a given year, the short-term irrigation demands can be as high as 5.2 mgd. Because the peak irrigation demands can significantly exceed the instantaneous flow being discharged from WWTP, the Storage Reservoir is used to equalize demands (i.e., treated effluent is stored during periods when irrigation demands are low so that water is available to meet peak irrigation demands).

The volume of water that needs to be stored to ensure the peak irrigation demands can be satisfied will vary throughout the year: from zero during the non-irrigation season (approximately October through March) to a maximum amount when irrigation demands peak in June and July. During these peak months, the effective irrigation demand is approximately 9.5 inches per month. Therefore, for the entire 186-acre irrigation area, approximately 48 Mgal of irrigation water is needed. Conservatively assuming this volume of irrigation water would be applied at the peak rate of the irrigation pumps, it would be applied over an 8 day period. Table 2 summarizes the peak amount of volume that would need to be stored for the months of June and July based on the conservative assumption that 48 Mgal will be applied over an 8 day period. As shown, the storage needs are less than the total available volume of 56.6 Mgal, and the irrigation storage needs will decrease as effluent flow increases.

Table 3. Storage Volume Requirements at Peak Irrigation Months (June and July)

Design Condition	Design ADWF	Volume of Recycled Water Generated in 8 Days, Mgal	Total 8-Day Irrigation Demand, Mgal	Minimum Storage Needed to Meet Peak Day Irrigation Demands, Mgal	Remaining Storage Volume, Mgal
Current Permitted Design Condition	3.0	24	48	24	32
Future Permitted Design Condition	4.5	36		12	44
Ultimate Buildout Design Condition	6.0	48		0	56

EMERGENCY STORAGE

Certain emergency conditions may require the City to divert all or a portion of WWTP effluent to the storage ponds for prolonged (i.e., multi-day) periods. Examples of such emergency conditions could include equipment failures or biological process upsets. (Note that the City also has the ability to divert influent to the Auxiliary Storage Basin if additional emergency storage capacity is needed.) The avoided costs associated with this storage approach include potential fines and penalties that may be levied for discharge permit violations during emergencies.

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The City has indicated a desire to maintain the ability to store up to 14 days of the entire WWTP effluent flow for the purposes of emergency storage. Table 4 summarizes the total storage volume needed under the different design conditions being considered in this Facilities Master Plan assuming both the annual average and maximum month flow rates. Also shown is the number of days of available storage based on annual average and maximum month flow rates.

Table 4. Storage Needs for 14 Days of Emergency Diversions

Average Dry Weather Flow, mgd	Storage Needs Based on Annual Average Flows, Mgal	Days of Available Storage at Annual Average Flows	Storage Needs Based on Maximum Month Wet Weather Flows, Mgal	Days of Available Storage at Maximum Month Wet Weather Flows
3.0	43	18	54	14
4.5	***64***	***12***	***81***	***10***
6.0	***86***	***9***	***109***	***7***

Notes: Bold, italicized numbers indicate values that exceed the available 56.6 Mgal of storage volume.

A review of Table 3 and Table 4 indicates that a 14-day emergency storage requirement exceeds the available storage during the peak irrigation months. However, under many circumstances the water diverted to the ponds in an emergency can be used for irrigation on the City's fields (i.e. effluent is diverted to the ponds in the event of a tertiary or UV system failure and/or if there is a secondary process upset that does not lead to an exceedance of the standards applicable to discharges to land).Therefore, storage for irrigation needs and storage for emergency purposes are not mutually exclusive.

Table 4 also shows that the 14-day emergency storage requirement will exceed the available pond volume of 56.6 Mgal as flows exceed 3.0 mgd. If 14 days of emergency storage is still desired as flows increase, the City could consider increasing the storage volume by constructing more storage facilities. Alternatively, the City could consider operations with lower emergency storage volume availability. In a worst-case scenario, flows exceeding the storage volume would need to be discharged to the land application area, regardless of whether irrigation demands dictated such a need. While this is not a preferred approach, discharge to the land application site would be a preferred approach to handling excess flows that do not meet surface water discharge limits in an emergency situation.

SECONDARY EFFLUENT DIVERSIONS

Flows exceeding the tertiary treatment system capacity may be diverted and stored on-site, and the stored secondary effluent can be used for irrigation on the City-owned properties surrounding the WWTP. Having this operational flexibility reduces the need for redundancy in the tertiary system process design. The avoided costs associated with this approach include the capital costs of designing and constructing additional tertiary treatment equipment and structures, as well as the recurring costs to operate and maintain the additional equipment.

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For each of the tertiary processes, the peak equipment unit capacity is 3.0 mgd. If no secondary effluent diversion were allowed, the *firm capacity* (i.e. the capacity with the largest unit out of service) of each of the tertiary systems would need to be equal to the design PHWWF (or 9.0 mgd at the Current Permitted Design Condition). The current tertiary system design included allowances for tertiary diversions. Specifically, each of the tertiary process has three treatment units, resulting in a firm peak flow capacity of 6.0 mgd and a total capacity of 9.0 mgd.

This section presents the results of a water balance analysis that is used to demonstrate how much flow would need to be diverted to the storage ponds (and subsequently discharged to the land application site) given a range of assumed number of tertiary units constructed at each of the design conditions evaluated in the WWTP Facilities Master Plan. The following specific topics are discussed:

- Operational Assumptions
- Flow Assumptions
- Water Balance Assumptions
- Secondary Diversion Analysis Results

Operational Assumptions

Secondary effluent diversion volumes are assumed to equal the difference between the influent flows exceeding the tertiary system capacity and the available tertiary capacity. Similar to the influent flow equalization analysis, the secondary diversion analysis considers the worst-case flow conditions as follows:

- A tertiary unit may be out of service for a 30-day period during the worst case wet season month, which is defined as the 30 day period where the WWTP would experience the highest amount of I&I as defined by the synthesized peak wet weather flow hydrograph discussed previously in this TM.
- In the same year that a tertiary unit is out of service during the worst case wet season month, one tertiary unit will also be out of service for a 60 day period during the dry season.
- During the remainder of the year, all units will be available.

It is further assumed that there is no opportunity to for reuse (i.e., land application) during the November through March period, and (conservatively) no opportunity at all to return wastewater to the plant for treatment. Thus, the diverted flows would remain in storage until the irrigation season commences.

Flow Assumptions

For the period of November through April, this analysis makes use of the same synthetic hydrograph used for the equalization analysis above (see Synthesized Peak Flow Hydrographs).

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For the period of May through October, this analysis makes use of observed dry weather diurnal flow patterns upward adjusted to the design ADWF and then further adjusted to account for observed month-to-month variation.

Water Balance Assumptions

Consideration must also be given to evaporation, percolation, and direct rainfall in and out of the storage ponds. For direct rainfall, 1-in-100 rainfall year values are assumed. Evaporation rates are average annual values taken from the Lodi West CIMIS Station (ID 166). Finally, a percolation rate of 2 inches per month is assumed, as per previous water balance analyses developed for the City.

Secondary Diversion Analysis Results

Table 5 shows the calculated incremental volume of water that will need to be diverted to the storage facilities under the three design conditions evaluated in the Facilities Master Plan given a certain number of tertiary units to be constructed. Table 6 shows the cumulative storage requirements associated with the same combinations of design conditions and sizing configurations. The following conclusions are drawn from the results presented in Table 6:

- A 3-unit configuration (as previously constructed) is adequate for the Current Permitted Design Condition
- A 4-unit configuration is adequate for the Future Permitted Design Condition
- A 5-unit configuration is adequate for the Ultimate Buildout Design Condition

In each case, the maximum storage requirements are well within the storage capacity of the ponds and should have little, if any, impacts on the other storage pond uses contemplated.

RECYCLED WATER STORAGE FOR OFFSITE USE

Reclamation alternatives were evaluated in the City's NPDES Permit Compliance Action Plan (West Yost, 2005), and reuse of tertiary disinfected recycled water was not found to be cost-effective at that time. Nevertheless, the City may decide to pursue offsite water recycling in the future if market demands for tertiary disinfected recycled water change. Potential uses for tertiary disinfected effluent may include agricultural irrigation of crops intended for human consumption, unrestricted landscape irrigation, and industrial process or cooling water, among others.

Similar to the existing irrigation uses, some storage would likely be necessary to ensure a reliable supply of recycled water during peak demand periods. However, disinfected tertiary effluent destined for offsite reuse would need to be stored separately from secondary effluent used for land application on City-owned agricultural property. Therefore, if offsite recycled water is pursued in the future; the use of onsite storage to support the recycled water project must be considered within the context of other uses discussed in this TM.

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Table 5. Estimated Monthly Secondary Effluent Diversion Volumes Given an Assumed Number of Tertiary System Units, Mgal

	3 Units (Current Capacity)			4 Units Assumed			5 Units Assumed			6 Units Assumed			7 Units Assumed		
Month	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition
January	-	-	2.7	-	-	-	-	-	-	-	-	-	-	-	-
February	3.7	42.3	102.1	-	5.5	37.1	-	0.4	7.3	-	-	1.2	-	-	-
March	-	1.1	5.8	-	-	0.6	-	-	-	-	-	-	-	-	-
April	-	-	0.4	-	-	-	-	-	-	-	-	-	-	-	-
May	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
June	-	0.0	24.4	-	-	-	-	-	-	-	-	-	-	-	-
July	-	0.0	24.4	-	-	-	-	-	-	-	-	-	-	-	-
August	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
September	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
October	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
November	-	-	0.0	-	-	-	-	-	-	-	-	-	-	-	-
December	-	-	1.8	-	-	-	-	-	-	-	-	-	-	-	-
Total, Mgal	**3.7**	**43.4**	**161.5**	**-**	**5.5**	**37.7**	**-**	**0.4**	**7.3**	**-**	**-**	**1.2**	**-**	**-**	**-**
Total, acre-feet	**11.2**	**133.3**	**495.8**	**-**	**16.9**	**115.7**	**-**	**1.2**	**22.5**	**-**	**-**	**3.8**	**-**	**-**	**-**

Note: Worst-case 30-day and 60 day periods are indicated by grey highlighting.

Table 6. Estimated Cumulative Monthly Storage Needs Given an Assumed Number of Tertiary System Units , acre-feet

	3 Units (Current Capacity)			4 Units Assumed			5 Units Assumed			6 Units Assumed			7 Units Assumed		
Month	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition	Current Permitted Design Condition	Future Permitted Design Condition	Ultimate Buildout Design Condition
January	-	-	8.2	-	-	-	-	-	-	-	-	-	-	-	-
February	11.2	129.9	327.3	-	16.9	114.0	-	1.2	22.5	-	-	3.8	-	-	-
March	12.8	134.8	346.6	-	18.5	117.3	-	2.8	24.1	-	-	5.4	-	-	-
April	7.4	129.4	342.4	-	13.0	111.8	-	-	18.6	-	-	-	-	-	-
May	-	118.8	331.8	-	2.4	101.2	-	-	8.0	-	-	-	-	-	-
June	-	105.5	393.3	-	-	88.0	-	-	-	-	-	-	-	-	-
July	-	91.5	480.2	-	-	73.8	-	-	-	-	-	-	-	-	-
August	-	78.9	458.7	-	-	61.3	-	-	-	-	-	-	-	-	-
September	-	69.8	465.3	-	-	52.2	-	-	-	-	-	-	-	-	-
October	-	65.5	464.5	-	-	47.8	-	-	-	-	-	-	-	-	-
November	-	-	452.8	-	-	-	-	-	-	-	-	-	-	-	-
December	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Total, acre-feet	**12.8**	**134.8**	**480.2**	**-**	**18.5**	**117.3**	**-**	**2.8**	**24.1**	**-**	**-**	**5.4**	**-**	**-**	**-**
Total, Mggal	**4.2**	**43.9**	**156.4**	**-**	**6.0**	**38.2**	**-**	**0.9**	**7.8**	**-**	**-**	**1.7**	**-**	**-**	**-**

Note: Recommended scenarios are highlighted in yellow.

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SUMMARY AND CONCLUSIONS

The major conclusions from this TM are summarized as follows:

1. The City can use the Auxiliary Storage Basin to equalize peak flows through the WWTP in emergency conditions. Given the minor improvements needed to allow for equalization, it is recommended that the City utilize equalization, as appropriate, to allow for some reduction in the firm peak flow requirement applied in the design of the primary and secondary treatment processes.
2. The City has more than adequate effluent storage capacity right now to meet irrigation demands for the foreseeable future.
3. In general, it is expected that irrigation water storage capacity can also be used to meet the City's stated desire for 14 days of emergency flow storage. As flows increase in the future, the existing pond volume may not be adequate to meet a full 14 days of emergency storage, at which time the City may either want to expand its storage capacity or relax the 14-day criterion.
4. The City can use the existing secondary effluent diversion facilities to shave peak flows through the WWTP. Given that no additional improvements are needed to allow for this practice, it is recommended that the City utilize the secondary diversion facilities to allow for a reduction in the total number of tertiary treatment processes trains that would otherwise be necessary.
5. Additional storage and/or treatment would likely be needed in the event that the City were to pursue export of recycled water for offsite use. However, offsite recycled water usage does not appear to be a cost-effective means of disposing of WWTP effluent at the present time.

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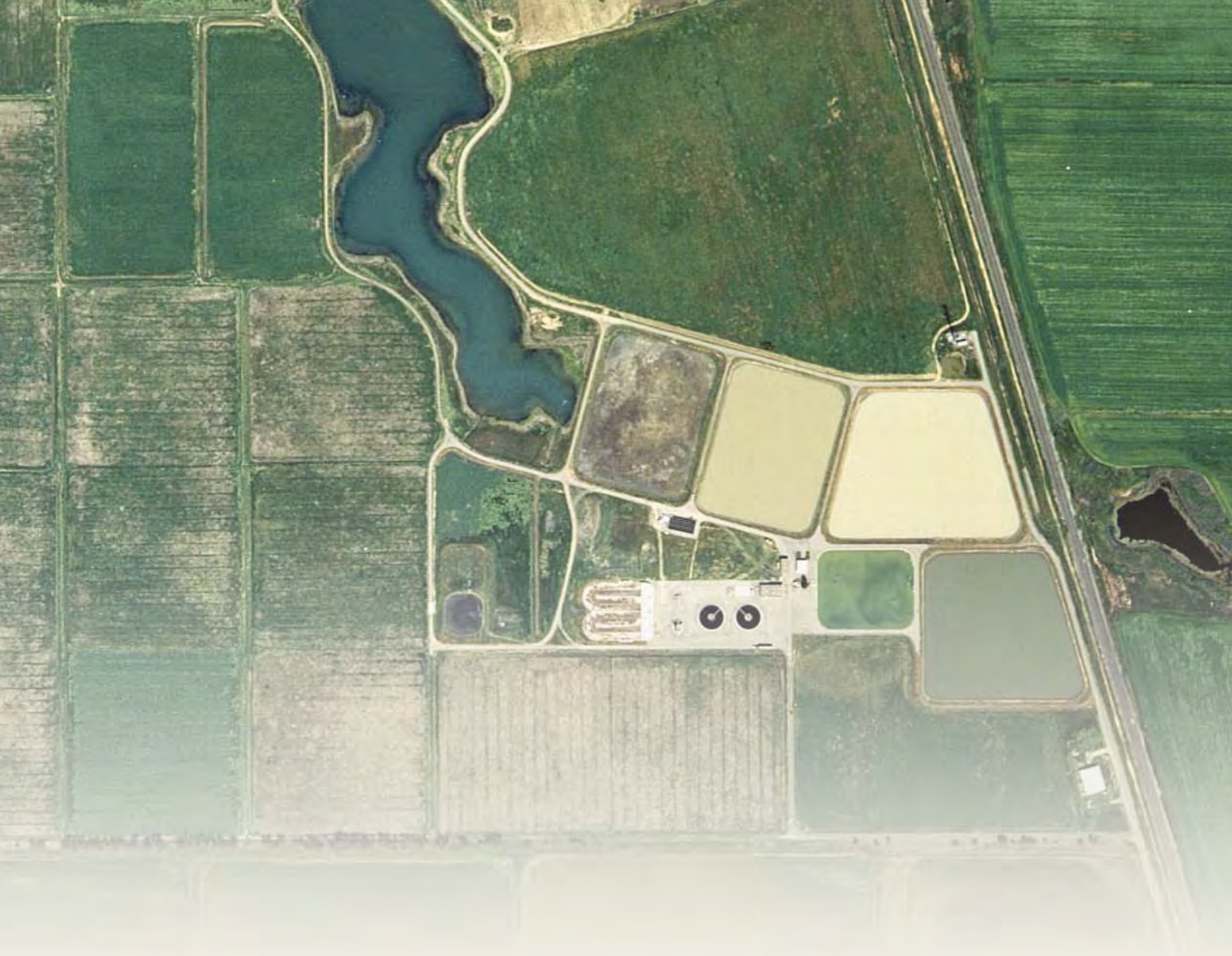

WEST YOST ASSOCIATES

City of GALT