AECOM Water

Prepared for:
The City of Gonzales
Gonzales, California

Prepared by:
AECOM
San Luis Obispo, CA
September 2011

City of Gonzales
Wastewater System Conceptual Plan
Draft

Table of Contents

2.0 Introduction 4

2.1 Wastewater Collection System Overview 4

2.2 Purpose and Scope of Work 4

3.0 Wastewater Flow Projections 7

3.1 Current and Future Population and Service Area 7

3.2 Analysis of Current Wastewater Flowrates 8

Flow Parameters and Peaking Factors 8

Current Wastewater Loading 11

3.3 Future Wastewater Production 11

Residential Wastewater Factors 11

Non-Residential Wastewater Use Factors 12

Wastewater Generation at Buildout 13

Comparison to Previous Projections 14

4.0 Wastewater Treatment Capacity Evaluation 15

4.1 Current Headworks Capacity 16

4.2 Current Treatment and Disposal Capacity 16

4.3 Available Capacity to Support Infill and New Development 18

Available Capacity to Support Build-out of the Existing City 18

Increasing Capacity for Future Service 19

Growth Beyond the 2035 Urban Growth Area 20

5.0 Evaluation of Treatment and Disposal Alternatives 20

5.1 Expanded Facultative Pond System 21

5.2 System Upgrade to Activated Sludge Process 22

Biolac EAAS Pond Conversion 23

Oxidation Ditch EAAS System 26

5.3 Comparison and Feasibility of Biological Treatment Alternatives................................ 28

5.4 Effluent Disposal.. 30

Infiltration Basins 30

Agricultural Irrigation 31

Satellite Treatment Plant 32

6.0 Recommended Alternatives 34

6.1 Headworks.. 35

6.2 Influent Pumping... 35

6.3 Sludge Management.. 36

6.4 Effluent Disposal and Reuse 36

7.0 Cost Estimates for Recommended Alternatives 37

9.0 Collection System Capacity Evaluation... 37

9.1 Existing Collection System 37

9.2 Conceptual Plan for Serving the Urban Growth Area.................................... 41

Service Zones 41

Flow Assignment from Urban Growth Area Properties 42

Buildout Deficiencies and Recommended Improvements 42

9.3 Recommended Improvements.. 44

9.4 Urban Growth Area Project Costs... 45

Exhibits

Figure 1 – Modeling of Existing System

Figure 2 – Existing Deficiencies

Figure 3 –Conceptual Improvements and Upgrades for Serving the Urban Growth Area

2.0 INTRODUCTION

2.1 Wastewater Collection System Overview

The City's wastewater collection system serves the existing service area west of Highway 101 with gravity sewer, and relies on lift stations and force mains to convey flow from areas east of Highway 101, across the highway at three locations. City staff has suggested that new gravity sewer crossings of Highway 101 may be needed to relieve demand on some existing lift stations and force mains. In particular, the most recent Master Plan (2001) recommends installing a new gravity sewer crossing at Highway 101 and Gloria Road, and increasing capacity of the existing South Alta Street trunk main.

Due to the configuration of existing sewer system and the topography of the existing City area and the expansion area, a network of new gravity sewer mains, new lift stations, and lift station upgrades will be required to serve the Urban Growth Area. The City prefers to reduce reliance on new and existing lift stations in the future by installing gravity sewer mains to the extent practicable.

Wastewater Treatment System

The existing wastewater treatment plant, located approximately two miles west of the intersection of South Alta Road and Gonzales River Road, at the end of Short Road, operates under Waste Discharge Requirements (WDR) Order R3-2006-0005 and currently provides biological treatment within six facultative aerated ponds and two polishing/ oxidation ponds, operated in two parallel trains. Polishing pond effluent is disposed via evaporation and percolation in one of three 7-acre disposal fields (approximately 21 acres total, alternating in operation). Designs for the most recent plant upgrade were completed in 2006 and the City has made improvements to the headworks and aerated facultative ponds. The expansion strategy presented in the 2001 Waste Wastewater System Master Plan included plans for expanding the plant to 1.0 MGD ADF (Phase 1), and provided additional guidance on further upgrades, which are discussed in Section 5. According to Table 3-5 of the Master Plan, the Phase 1 expansion would accommodate approximately 13,000 customers. Alternatives for upgrading treatment processes to provide service for future service areas are assessed in this document.

2.2 Purpose and Scope of Work

The City of Gonzales is developing a Plan for Public Services as part their current General Plan Update. This portion of the Plan for Public Services will provide conceptual plans for major wastewater infrastructure improvements. A summary of the scope of work for this Wastewater

System Conceptual Plan follows. The collection and treatment portions of the wastewater system are addressed separately, as outlined below.

Wastewater Treatment Scope of Work

Assessment of Existing Conditions and Issues

Collect and review available data pertaining to operation of the wastewater treatment plant. Identify and document known deficiencies and/or areas necessary improvements at the plant.

Wastewater Flow Projections

Evaluate wastewater flow for projected new growth within the General Plan Update study area by considering proposed land use designations and estimating appropriate sewage generation or flow factors. Compare these projections to previous wastewater flow projections and loading found in available documents, existing flow records, and new estimates for future flows by the City's General Plan consultants or the specific plan proponents. Estimate peak flows for maximum day, maximum month, and peak hour wastewater demands.

Treatment and Disposal Capacity Evaluation

Review available documents in order to identify both existing and potential future wastewater treatment and disposal constraints. Provide an opinion as to the available capacity of existing facilities to support the projected new growth within the Study Area. This evaluation will include a review of previously published wastewater treatment capacity and a comparison to the projected demands from the study area.

Evaluation of Treatment and Disposal Alternatives

Identify and evaluate general alternatives to increase treatment and disposal capacity of the City's Wastewater Treatment Plant. Evaluate the technical and financial feasibility of each general alternative and provide relative cost estimates for comparison purposes. Develop a matrix table to qualitatively rank the alternatives based on; (1) effluent quality, (2) regulatory concerns, (3) increase to treatment and disposal capacity, (4) land cost, (5) design and construction costs, and (6) operation and maintenance requirements.

Recommended Alternatives

Recommend the preferred alternatives for wastewater treatment plant improvements to meet future wastewater demands from the Urban Growth Area. The general facilities required for each recommended alternative, including potential locations, are described.

Cost Estimates for Recommended Alternatives

Provide conceptual level cost estimates for the recommended facility improvement alternatives to increase capacity of the City's wastewater treatment plant.

Wastewater Collection Scope of Work

Assessment of Existing Conditions and Issues

Collect and review available data needed to evaluate the City's wastewater collection system. Identify and document known deficiencies and/or areas requiring improvement.

Collection System Capacity Evaluation

Evaluate the available capacity of existing collection system facilities to support projected growth within the Study Area as follows:

- Estimate average day, maximum day, and peak hour flows for the existing collection system;
- Identify critical sections of sewer between the General Plan expansion area and WWTP;
- Develop a "skeleton" model of critical gravity sewer elements;
- Apply future demands to the skeleton model in order to identify potential impacts to transmission capacity; and
- Identify collection system deficiencies for both existing and future build-out demands.

Recommended Improvements

Recommend improvements to the collection system to meet future wastewater demands from the projected new growth area. Summarize recommended improvements to meet existing and projected build-out demands. The locations of required facilities, including additional lift stations, force mains, and gravity sewer trunks and mains, will be described.

Cost Estimates for Recommended Improvements

Provide conceptual level cost estimates for recommended facility improvements to increase collection system capacity.

Wastewater System Master Plan Update Scope of Work

Outline for Wastewater System Master Plan Update

Outline recommended tasks for further investigation and potential strategies the City may want to employ in performing its forthcoming update to the City's Wastewater System Master Plan.

3.0 WASTEWATER FLOW PROJECTIONS

Wastewater flow projections were developed for the Urban Growth Area to allow evaluation of the existing WWTP and the collection system, and to assist the City in planning for additional infrastructure which will be required to serve future growth. Wastewater production from infill development within the existing City was also estimated and is accounted for in projections of future wastewater flow.

3.1 Current and Future Population and Service Area

The Land Use Element of the General Plan identifies existing and future service areas and the increase in population resulting from development of the Urban Growth Area. Existing developed acreage for various land use categories, and projected acreage at buildout are summarized in Table 3.1. The existing City Area, Urban Growth Area, and Urban Reserve Area are identified in Figure II-4 of the Land Use Element.

Table 3.1: General Plan Land Uses and Densities for Existing City and Buildout Urban Growth Area

	Existing			Total at Buildout		
Use Category	Total Acreage	DUs	Estimated Population	Total Acreage	DUs	Estimated Population
Residential	338	2,067	9,025	1,978	9,767	37,824
Commercial	23	--	--	183	--	930
Manufacturing	159	--	--	489	--	--
Other	458	--	--	2,978	--	--
Total	978	2,067	9,025	5,628	9,767	37,824

Based on the 2010 General Plan Land Use Element Table II-2.

Development of the study area will result in significant increases in service area and population, and will shift the mix of development from existing conditions. Population will increase to 37,824 residents at buildout (currently 9,025 residents). Residential development and population growth, as proposed, will result in an overall reduction in household density (4.37 persons/ DU currently, versus 3.87 persons/ DU at buildout). The proposed buildout scenario also identifies increased commercial development relative to residential, and industrial/ manufacturing uses (acreage basis). These changes are expected to result in a deviation from the current gross per capita wastewater generation. Therefore, wastewater generation is considered on an area-basis to allow shifts in community characteristics and changes in the land use inventory at buildout to be considered. Duty factors for proposed development are discussed in Section 3.

3.2 Analysis of Current Wastewater Flowrates

WWTP monitoring data from the past three years (January 2006 through December 2008) were analyzed to develop wastewater peaking factors. Flow parameters and peaking factors are discussed in the following paragraphs and results are summarized in Table 3.2. Historical data from the 36-month period of record is presented in Table 3.3.

Flow Parameters and Peaking Factors

Average Daily Flow (ADF) is the total wastewater flow received at the WWTP averaged over the number of days per year. Based on WWTP records from January 2006 through December 2008 (Table 3.3), the current ADF is 0.584 MGD.

Average Wet Weather (AWWF) and Dry Weather (ADWF) Flow are the average of daily flow rates experienced during wet and dry weather months, respectively. Consideration of average wet and dry weather flows allows analysis of pond treatment systems at appropriate flow rates and temperatures for the wet and dry seasons. Precipitation of 0.5 inches or more per month has been assumed to identify wet weather months. AWWF and ADWF for the record period were 0.529 MGD and 0.627 MGD respectively. Seasonal wastewater patterns indicated increased loading during the summer months, consistent with the City’s dynamic industrial activity and economy, with greater agricultural processing and production occurring during dry weather months.

Maximum Month Flow (MMF) is the average daily flow during the month with the maximum cumulative flow. MMF is often the regulated flow parameter for a WWTP’s Discharge Permit. Flow records indicate a MMF of 721,733 gpd (0.722 MGD) over the past 3 years. The current waste discharge requirements for the City’s WWTP, as specified in Regional Water Quality Control Board (RWQCB) Order No.R3-2006-0005, limit plant effluent to a maximum average monthly flow of 1.3 MGD for Phase 1 improvements described in the Permit.

Peak Day Flow (PDF) is the maximum daily flow rate experienced at the WWTP and is used to design or evaluate hydraulic retention times for certain treatment processes. According to WWTP flow records, PDF between January 2006 and December 2008 was 875,000 gpd, occurring in October of 2006. According to CIMIS data, no precipitation was experienced in October 2006, thus, the PDF corresponds to a dry month.

Peak Hour Flow (PHF) is the maximum one-hour flow experienced by the system, and is typically used for sizing flow meters, interceptors, and headworks systems. Peak hour flow can be derived from detailed WWTP records, flow monitoring, or empirical equations used to estimate PHF based on service area population. Hourly wastewater flow data were not available for this study, therefore, empirical methods were used to estimate the current peak hour flow based on current population information and ADF. A peaking factor of 3.0 was calculated for the current population of 9,025 residents, resulting in a PHF of 1.75 MGD.

Current wastewater flow parameters and peaking factors are summarized below, in Table 3.2. Monthly flow data, average and maximum day flows, and precipitation data for the period of record are summarized in Table 3.3.

Table 3.2: Current Wastewater Flow Parameters and Peaking Factors

Flow Parameter	**Flowrate**	**Peaking Factor** [a]
Average Daily Flow (ADF):	0.58 MGD	na
Average Wet Weather (AWWF):	0.53 MGD	0.92
Average Dry Weather (ADWF):	0.63 MGD	1.07
Maximum Month Flow (MMF):	0.72 MGD	1.24
Peak Day Flow (PDF):	0.88 MGD	1.50
Peak Hour Flow (PHF):	1.75 MGD	3.00 [b]

a. Peaking factors calculated relative to ADF

b. PHF peaking factor calculated according to population using the Ten States Standards for Wastewater Facilities. Although the PHF peaking factor decreases with increased population, peaking factors calculated for current community populations are typically applied for projections.

Table 3.3: Historic WWTP Flow Records and Precipitation Data (CIMIS)

Month	Month ADF (gpd)	Max Day Flow (gpd)	Precipitation (in)
January 2006	436,096	601,000	3.08
February 2006	438,142	582,000	2.00
March 2006	479,838	526,000	7.42
April 2006	534,000	574,000	2.76
May 2006	556,580	631,000	*0.00*
June 2006	548,933	609,000	*0.00*
July 2006	547,290	653,000	*0.02*
August 2006	543,000	636,000	*0.00*
September 2006	549,166	624,000	*0.00*
October 2006	656,000	**875,000**	*0.00*
November 2006	606,000	814,000	*0.01*
December 2006	468,000	539,000	2.46
January 2007	431,290	516,000	0.83
February 2007	452,785	496,000	1.43
March 2007	510,290	617,000	0.56
April 2007	600,100	770,000	0.87
May 2007	720,322	800,000	*0.16*
June 2007	**721,733**	817,000	*0.00*
July 2007	704,361	773,221	*0.00*
August 2007	702,057	793,097	*0.00*
September 2007	708,112	863,366	*0.00*
October 2007	665,646	764,736	1.02
November 2007	609,437	716,260	0.33
December 2007	532,204	571,688	0.24
January 2008	531,824	716,266	1.57
February 2008	551,810	620,259	2.27
March 2008	568,545	635,554	0.65
April 2008	613,329	711,183	*0.10*
May 2008	663,307	722,691	*0.01*
June 2008	677,621	738,358	*0.01*
July 2008	643,843	703,769	*0.49*
August 2008	618,735	675,367	*0.00*
September 2008	618,904	691,265	*0.00*
October 2008	668,911	731,770	1.02
November 2008	609,746	689,391	1.35
December 2008	524,287	597,124	2.20

Current Wastewater Loading

Current organics and solids loading were estimated using flowrates and influent water quality parameters reported in the City's Annual Reports to the Regional Water Quality Control Board for the period of record (January 2006 through December 2008). Based on these records, organic and suspended solids loading are within the typical ranges for municipal wastewater. Average influent organics and suspended solids loading are summarized in Table 3.4.

Table 3.4: Current wastewater constituent loading

Average daily flow, gpd	Average Influent BOD_5			Average Influent TSS		
	mg/ L	lb/ day	lb/ cap.day	mg/ L	lb/ day	lb/ cap.day
584,000	257	1,200	0.142	204	973	0.111

Based on WWTP influent water quality data January 2006 – December 2008.

Per capita loading parameters calculated using the average population between 2006 and 2008.

3.3 Future Wastewater Production

To allow changes in the land use inventory at buildout to be accounted for in wastewater projections, future wastewater production from development of the Urban Growth Area was estimated using wastewater duty factors. Generation from infill development within the existing City was accounted for using an overall percentage-increase from current flow conditions. The development of wastewater use factors and wastewater projections is discussed in the following paragraphs, and projected flows are compared to previously developed wastewater projections.

Residential Wastewater Factors

Population growth and acreages of residential uses proposed in the Land Use Element of the General Plan, information on composition of the *Neighborhood* land use provided by the City, and an assumption of per capita wastewater generation (80 gpcd[1]) were used to approximate gross wastewater generation per acre of residential development for the *Neighborhood Residential* (NR) designation. Moreover, an average wastewater generation of 1,600 gpd/ gross acre was developed according to the proposed NR acreage (1,490 acres) and 25,400 projected residents. This methodology was also used to project wastewater generation per gross acre for traditional residential designations (i.e. *Low, Medium, and High Density Residential*). Residential

[1] The estimated per capita flow factor of 80 gpcd does not account for wastewater generation from non-residential uses.

use factors for new development are summarized in Table 3.5 and additional notes and assumptions are included as footnotes.

Non-Residential Wastewater Use Factors

The greatest increase in non-residential development (land area basis) is proposed for the the *Heavy Industrial* land use designation. The City has indicated agricultural processing facilities may represent a significant portion of industrial development. Based on current potable water demand (approximately 2,200 gpd/ acre) and the assumption that agriculture packaging and processing facilities will have high return-to-sewer, a gross wastewater usage of 2,200 gpd/ acre has been assumed for the *Heavy Industrial* designation. For *Light Industrial* development, a gross wastewater usage of 1,000 gpd/ acre was assumed. Use factors for commercial designations and other non-residential use categories are based on conventional duty factors.

Non-residential use factors for new development are summarized in Table 3.5 and additional notes and assumptions are included as footnotes.

Table 3.5: Use Factors for New Development

Designation	Population added	Residential wastewater generation, gpcd	Wastewater duty factor, gpd/acre
Neighborhood Residential	25,400	80	1,600
Low Density Res.	2,600		1,600
Medium Density Res.	400		3,300
High Density Res.	400		3,900
Community and Neighborhood Comm.	--	--	1,600
Highway Commercial	--	--	2,000
Downtown Mixed Use	--	--	2,000
Heavy Industrial/ Manufacturing	--	--	2,200
Light Industrial	--	--	1,000
Public/Quasi-Public	--	--	1,000
Parks & Open Space	--	--	300
Urban Reserve [1]	--	--	1,600

1. Parks and Open Space use factor assumes average 60 visitors/ acre /day and 5 gallons generated per visitor.
2. Open space assumes average 30 visitors/ acre day and 5 gal/ visitor to public lavatory.
3. Urban reserve wastewater generation assumes future development of UR overlay with neighborhood development, per LUE, with similar inventory.

Wastewater Generation at Buildout

Total wastewater generation at buildout was estimated by applying wastewater generation factors summarized in Table 3.5 to proposed acreage of new development within the Study Area and adding increased flow from infill development in the City. The General Plan indicates few opportunities are available for development within the existing City, therefore, a factor of 25% has been applied to current average flow (ADF) to account for the possibility of infill development contributing additional flow to the City's collection and treatment system at buildout. The resulting wastewater flow rates are tabulated in Table 3.6. Projected flow from the Urban Reserve area is accounted for separately at the bottom of the table and is calculated assuming average gross wastewater generation for Neighborhood designations, consistent with the Land Use Element of the General Plan.

Table 3.6: Future Wastewater Production

Designation	Acreage Added at Buildout, acres	Wastewater duty factor, gpd/acre	Wastewater Production ADF, gpd
Neighborhood Residential	1,490	1,600	2,384,000
Low Density Res.	130	1,600	208,000
Medium Density Res.	10	3,300	33,000
High Density Res.	10	3,900	39,000
Community & Neighborhood Comm.	90	1,600	144,000
Highway Commercial	70	2,000	140,000
Downtown Mixed Use	--	2,000	--
Heavy Industrial/ Manufacturing	310	2,200	682,000
Light Industrial	20	1,000	20,000
Public/Quasi-Public	320	1,000	320,800
Parks & Open Space	155	300	46,500
Subtotal			4,016,500
Existing City at Build-out*			730,000
Total			**4,746,500**
Urban Reserve Area*	*2,128*	*1,600*	*3,408,000*

Production from existing City area assumes 25% increase in wastewater generation from infill development. Wastewater from the Urban Reserve Area is accounted for separately and assumes gross usage per acre consistent with the "Neighborhood" designation, as outlined in the LUE. Wastewater demand from the Urban Reserve area is not included in the overall total.

Since gross wastewater generation (per-acre basis) will vary considerably between communities and with varying development density, more detailed evaluations of wastewater generation from specific projects are recommended as large residential, industrial and commercial projects are planned within the Study Area.

Comparison to Previous Projections

Because the service area considered in the 2001 Wastewater Master Plan was significantly smaller than the Urban Growth Area considered in this study, comparison of current flow projections with previous projects is made on a per unit basis. Use factors assumed in the 2001 Wastewater Master Plan are presented in Table 3.7 for comparison.

Table 3.7: Use factors assumed for the 2001 Gonzales Wastewater System Master Plan

Type of Development Average Daily Wastewater Flow	Gross Wastewater Usage, gpd/acre
Commercial and Light Industrial	2,000
Industrial	3,000
Low Density Residential	1,500
Medium Density Residential	2,000
High Density Residential	3,000
Institutional/Public Facilities	1,500
Non-zoned Areas	1,500

Using the assumed duty factors and proposed acreage for respective land uses (Table 3.6), gross per capita usage for *Neighborhood* designations is approximately 95 gpcd. With the inclusion of traditional residential uses and non-residential uses, gross per capita wastewater generation is approximately 139 gpcd. This higher gross per capita generation is due largely to the amount of non-residential uses. Additionally, gross per capita duty factors generally increase as communities grow and become more prosperous. In contrast, the effects of implementing water conservation measures would be expected to decrease both water consumption and wastewater generation rates. These effects would be difficult to project without detailed analysis of potable water consumption rates, conservation measures implemented, and flow metering.

Current gross per capita wastewater flow factors were estimated using population estimates and reported wastewater flowrates between 2006 and 2008 (presented in the following section). The resulting gross per capita flow factors for the period of record is are summarized in Table 3.8.

Table 3.7: Gross per capita Wastewater Flow Factors

Year	Population [1]	Wastewater ADF [2], MGD	Gross per capita Wastewater flow factor, gpcd
2006	8,495	530,254	62
2007	8,737	613,195	70
2008	8,995	607,572	68
Average	**8,742**	**583,673**	**67**

1. 2006 and 2007 population from the California DOF. 2008 population was provided by the City.

2. Based on wastewater treatment plant records

These recent gross per capita wastewater flow factors are consistent with historical values presented in the 2001 Wastewater Master Plan (Table 3.8), and lower typical assumptions for projecting gross wastewater generation for developing communities. The 2001 Master Plan assumed 100 gpcd and use-specific duty factors for flow projections.

Table 3.8: 2001 Wastewater Master Plan per capita flow factors

Period of record	Wastewater flow factor, gpcd
1984 – 1989	88
1996	61
1998 – 1999	78 - 110
2000	64
2001 Wastewater Master Plan assumption	100

4.0 WASTEWATER TREATMENT CAPACITY EVALUATION

Available documentation and design drawings for the existing WWTP were reviewed to assess current design capacity and provisions for increasing capacity in the future. This assessment allows a comparison of the capacities of existing treatment plant components (based on available documentation and conceptual calculations) to projected future wastewater demands developed in Section 3. A detailed analysis of plant hydraulics, treatment capacity of existing pond system, and verification of capacities of individual components of the headworks and plant piping will be necessary for planning future WWTP improvements. Available information on the existing wastewater treatment plant was limited to the following sources provided by the City:

- 2001 Waste Water System Master Plan

- 2006 WWTP Expansion Phase 1 Design Plans
- RWQCB Waste Discharge Requirements Order No. R3-2006-0005

4.1 Current Headworks Capacity

The 2001 Wastewater System Master Plan proposed several stages of improvements to increase treatment plant capacity. Phase 1 improvements consisted of upgrades to the WWTP headworks, additional aeration in the facultative ponds, and modifications to allow operation of two parallel treatment trains (each consisting of three aerated ponds and one stabilization pond in series).

Design plans for Phase 1 improvements were completed in 2006 and the majority of upgrades have since been constructed. Rated capacities of headworks components constructed as part of the 2006 Phase 1 WWTP Expansion were determined from design plans and are summarized in Table 4.1.

Table 4.1: Headworks Equipment Capacities

Component	Description of Equipment	Peak Capacity
Influent Grinders	Two JWC Channel Monster grinders (5hp), 2.4 mgd each	4.8 mgd
Grit Removal	Smith and Loveless Pista Model Vortex Grit System	4.0 mgd
Flow Measurement	9-inch Parshall Flume	5.7 mgd
Influent Pump Station	Smith and Loveless Triplex Discharge Pump Station, 15-hp close-coupled motor driven vacuum primed non-clog centrifugal pumps in 96" ID wet well	3.8 mgd

Capacities are based on review of available drawings and have not been verified.
Flume Capacity estimated for typical 9-inch Parshall flume.

4.2 Current Treatment and Disposal Capacity

Phase 1 improvements for increasing aeration in the facultative ponds have also been completed. Pond volumes, surface areas, aerator horsepower, and pre-improvement configuration, as well as information on the existing infiltration basins is provided in the 2001 Wastewater System Master Plan. These data and the Phase 1 aeration improvements are summarized in Table 4.2.

Table 4.2 Lagoon System and Disposal Data

Component	Physical Description	Mechanical Aeration Pre-Expansion	Mechanical Aeration Phase 1 Expansion
Aerated Facultative Lagoons	Six 2-acre aerated facultative lagoons operated in parallel, approx. 6 ft operating depth, approx. 3.67 MG operating volume per pond	One 7.5-hp aerator per pond (#1 - 6) 45 hp total	Two 15-hp aerators per pond (#1 & 2) Two 7.5-hp aerators per pond (#3 & 4) One 7.5-hp aerator per pond (#5 & 6) 105 hp total
Stabilization Ponds	Two 5-acre stabilization ponds, approx. 7 ft operating depth	none	none
Infiltration Basins	Three 7-acre basins, alternating operation (one basin online)	na	na

According to the Master Plan, recommended Phase 1 improvements to aeration capacity and modification of the process flow would increase capacity to approximately 1.0 MGD (ADF). Based conceptual calculations, the Phase 1 improvements are sufficient for the increased average daily flow rate (1.0 MGD) and the permitted Phase 1 capacity of 1.3 MGD MMF. Capacities of the Phase 1 pond system and estimated disposal field capacity are summarized in Table 4.3.

Table 4.3 Aerated Facultative Treatment System and Infiltration Basin Capacities

	Pre-Expansion (MGD)	Phase 1 Expansion (MGD)
Estimated treatment system capacity (ADF)	--	1.00
2006 Waste Discharge Permit (MMF basis)	0.763	1.30

Neither percolation test results nor hydrogeologic studies were available to assess of disposal capacity of the existing infiltration basins, therefore, an evaluation was performed based on current loading, observations provided by the City, and information in the Wastewater System Master Plan. It is assumed the current infiltration basins are sufficient for serving current flowrates; however, due to evaporation and infiltration occurring in the existing unlined treatment and stabilization ponds, actual application to the infiltration basins is currently expected to be lower than overall influent flow rate of 0.584 MGD. Based on published recommended application rates for infiltration systems, a moderately conservative application limit of 1 MGD per 20 - 30 acres of infiltration basin is assumed for this capacity evaluation and will be used as the basis for estimating additional infiltration area required to serve future demand. Applying this assumption, the capacity of the existing infiltration basins may range from 0.7 to 1.0 MGD. This assumption coincides with the permitted Phase 1 discharge capacity of 1.3 MGD MMF via the existing treatment and disposal system. It is recommended that percolation testing be performed to allow the City to better assess the limits of the existing system and to identify potential areas for expansion of percolation basins in the future.

4.3 Available Capacity to Support Infill and New Development

Projected demands from development of the Urban Growth Area and build out of the existing City are compared with the Phase 1 WWTP capacity to estimate the additional treatment capacity (MGD) needed to serve the future service area (excludes the urban reserve area) at buildout. Capacities and estimated deficiencies are summarized in Table 4.4.

Table 4.4: Additional Treatment Capacity Required to Serve Urban Growth Area

Flow	Phase 1 Aerated Facultative Pond Capacity, MGD	Required Capacity at Buildout, MGD *			Capacity Deficiency at Buildout, MGD *
		Exist. City	Urban Growth Area	Total	
ADF	1.0	0.7	4.0	4.7	3.7
MMF	1.3	0.9	5.0	5.9	4.6

* Includes estimated flow from buildout of the existing City and the planning.
* Capacity Deficiency calculated relative to Phase 1 Aerated Pond Capacity.

The Phase 1 expanded WWTP will not be capable of providing wastewater treatment for the Urban Growth Area without significant improvements. Since development of the Urban Growth Area and infill development within the existing City will occur over many years, a phased approach to expansion of the WWTP is recommended. Expansion alternatives are considered in Section 5.

Available Capacity to Support Build-out of the Existing City

The capacities of the pre-expansion and Phase 1 facility are compared to current and projected demand from the City at build-out in Table 4.5. The headworks and the pond treatment system capacities are evaluated on a PHF and MMF basis, respectively, and remaining capacity is calculated in terms of ADF using the peaking factors developed in Section 3.

Table 4.5: Available WWTP Capacity at Current Flow and Projected Buildout of the City

WWTP Component	Basis for Evaluation	Capacity (MGD)	Current Conditions			B/O of Existing City		
			Flow (MGD)	Remaining Capacity		Flow (MGD)	Remaining Capacity	
				(MGD)	ADF* (MGD)		(MGD)	ADF* (MGD)
Pre-expansion Pond System	MMF	0.76	0.72	0.04	0.03	0.90	-0.14	-0.11
Headworks (Improved)	PHF	3.80	1.75	2.05	0.68	2.19	1.61	0.54
Phase 1 Pond System	MMF	1.30	0.72	0.58	0.47	0.90	0.40	0.32

*Equivalent ADF calculated using peaking factors of 3.0x and 1.24x for PHF and MMF, respectively (Section 3).
* Negative values for 'Remaining Capacity' represent deficiencies for serving projected growth.

Before Phase 1 improvements to the treatment ponds, minimal reserve wastewater treatment capacity was available. Many agencies plan for wastewater treatment improvements once a treatment facility has reached 80-85% of the rated capacity. Appropriately, the City has already begun improvements to increase the plant's capacity. According to the assumed City-buildout scenario (approximately 25% increase in flow from infill development) and assumptions described in this section, the completed Phase 1 WWTP would have sufficient treatment capacity to serve the City at buildout while maintaining a reserve capacity of approximately 0.32 MGD (Table 4.5). This remaining capacity exceeds the industrial reserve capacity historically maintained by the City (0.25 MGD), however, it is recommended the City plan for further expansion of the WWTP once monthly average flow to the Phase 1 plant exceeds approximately 1.0 MGD (or simultaneously with planning development which would result in exceedance of this planning threshold).

Increasing Capacity for Future Service

As shown in Table 4.5, the Phase 1 plant has sufficient capacity to serve buildout of the existing City area; however, significant improvements will be necessary to serve projected demand from the Urban Growth Area. Based on recent flow records, the WWTP is nearing the recommended planning threshold of 80%. Since development of the Urban Growth Area is expected to begin in the near future and will occur simultaneously with infill development of the existing community, the City should plan for additional improvements in the forthcoming Wastewater Facilities Master Plan. Alternatives for increasing the WWTP treatment capacity to accommodate future growth are evaluated in Section 5.

Growth Beyond the 2035 Urban Growth Area

Based on wastewater projections developed in Section 3, development of the Urban Reserve area could result in additional wastewater generation of approximately 3.4 MGD and would nearly double the projected demand from buildout of the existing City and the Urban Growth Area. Design life of a wastewater treatment facility is typically 20 - 30 years. Additionally, since actual growth rates may vary and operational issues, changes in regulatory requirements, and advancement of treatment technologies cannot be predicted, it is recommended evaluation of treatment needs for serving the Urban Reserve Area (planned for development beyond the year 2035 be conducted at a later planning phase. Future wastewater planning is recommended once observed flowrates have reached 80% of the expanded capacity.

5.0 EVALUATION OF TREATMENT AND DISPOSAL ALTERNATIVES

The increase in demand for wastewater treatment resulting from development of the Urban Growth Area will necessitate expansion of wastewater treatment facilities beyond the Phase 1 improvements outlined in the 2001 Wastewater System Master Plan. Recommendations in the Master Plan proposed expansion beyond Phase 1 with either (a) conversion of the pond system to an Advanced Integrated Pond System, (b) construction of primary clarification facilities followed by the existing pond system, or (c) construction of a sequencing batch reactor process and using existing lagoons for effluent polishing.

Two general alternatives to these expansion options are evaluated in this section: expanding the Phase 1 aerated facultative pond system with additional ponds, or upgrading the level of treatment to an extended aeration activated sludge process (Biolac pond system or oxidation ditch technology). Conceptual-level costs have been prepared for these alternatives to assist the City in evaluating each biological wastewater treatment alternative. These conceptual estimates, provided in Table 5.2, are expected to be accurate within -30% to +50%.

Alternatives for disposal of effluent from the existing WWTP plant are also evaluated in this section and an alternative for treating wastewater at one or more satellite facilities near the Urban Growth Area is considered. Additional WWTP improvements which will be required to serve projected buildout demand are presented in Section 6.0.

5.1 Expanded Facultative Pond System

Phase 1 Improvement plans outlined in the 2001 Wastewater System Master Plan recommend modifying process flow in the pre-expansion pond system to allow parallel operation of two treatment trains (each consisting of 3 aerated facultative ponds and one facultative stabilization pond in series), and adding additional aerators to increase treatment capacity to 1.0 MGD (ADF). A comparison with typical aerated facultative pond design parameters indicates the Phase 1 plant improvements result in high utilization of the existing pond system. This alternative for increasing treatment capacity to 4.7 MGD uses current performance of the Phase 1 aerated facultative pond system as the basis for future expansion.

Expansion of the aerated facultative pond system would require approximately 82 additional acres of treatment ponds and additional mechanical surface aerators totaling approximately 400 hp. This expansion alternative would likely include the following project components:

- New lined treatment ponds in the vicinity of the existing treatment plant
- Larger distribution box and piping to each of the ponds
- Effluent piping and junctions from each of the ponds
- Installation of additional surface aerators for new ponds
- Electrical supply, monitoring, and control conduit to each pond
- Construction of access roads

The approximate area required for an expanded facultative pond system, including 20% additional area for ancillary facilities such as distribution structures and access roads, is shown in Figure 5.1, adjacent to the existing ponds, to demonstrate scale. Approximately 100 acres of additional facultative ponds and associated facilities would be required.

At this scale, the usual benefits of facultative pond treatment such as simplicity of operation, and limited mechanical equipment become less prevalent. Conceptual construction costs for the expanded pond alternative are also high, primarily due to excavation required for the additional treatment ponds. Routing long sections of yard piping and conduit for electrical supply and controls also becomes a significant project component. Beyond these issues, perhaps the most significant limitation of an expanded pond system is the inability to meet higher treatment standards. Facultative and aerated treatment pond effluent is generally not considered suitable for efficient production of recycled water for irrigation due to typically high effluent suspended solids concentrations, nitrogen content, and inconsistent effluent quality. Although suitable for current discharge limits, additional treatment may be necessary in the future to improve effluent water quality and address constituents such as nitrogen, or to provide for reuse opportunities.

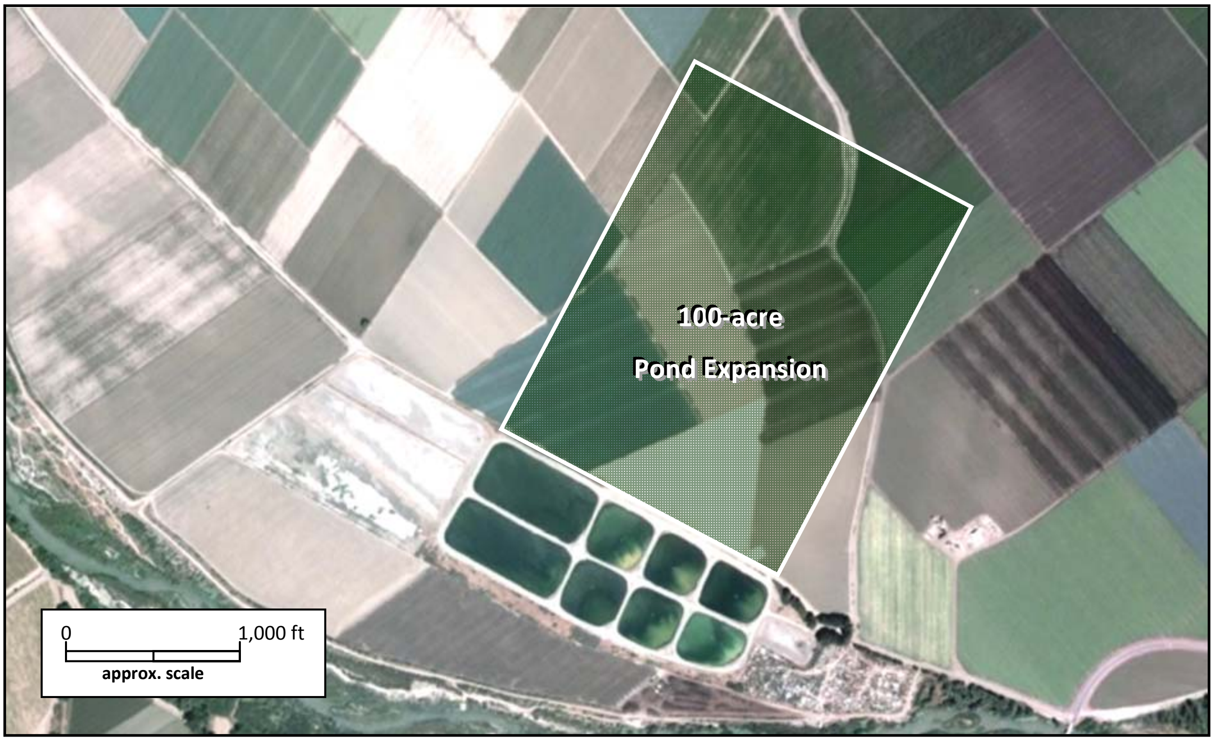

Figure 5.1: Estimated Area required for an expanded facultative pond treatment system at buildout.

5.2 System Upgrade to Activated Sludge Process

Activated sludge treatment processes are constructed in a variety of configurations which including plug-flow, complete-mix, and sequential batch reactors. Activated sludge processes utilize mixing, aeration, and a return-flow of activated floc for biological treatment and secondary clarification for separation of suspended biological floc and effluent. Additionally, an initial primary settling step is typically used to remove settleable solids before biological treatment. In general, activated sludge systems are capable of providing a high degree of organics and solids removal and can be designed and operated for nitrification and denitrification. Significant process observation and control is generally required for operation of a conventional activated sludge plant, however, variations on the activated sludge process which incorporate larger aeration vessels with longer hydraulic and solids retention times (generally referred to as extended aeration activated sludge processes) provide more stable operation and allow some conventional process elements such as primary settling to be eliminated. The extended aeration activated sludge (EAAS) processes are generally favorable for communities with limited operations staff and where land availability is of lesser concern. However, an

EAAS plant would require Class II certification for the chief plant operator and a conventional activated sludge plant would require Class III certification.[2]

Two EAAS alternatives are reviewed and discussed in the following sub-sections: conversion of the existing pond system to a Biolac Wave Oxidation system, and construction of an oxidation ditch facility and abandoning the existing ponds. Although not evaluated in this Conceptual Plan, investigation of other activated sludge processes may be considered in future studies and wastewater treatment facility planning efforts. Unlike the Biolac pond conversion, the majority of activated sludge processes would not directly utilize the existing ponds.

Biolac EAAS Pond Conversion

The Biolac Wave Oxidation EAAS system uses suspended fine-bubble diffusers for aeration and mixing in large treatment ponds. Air is supplied to the system on a cycle which creates alternating aerobic and anoxic zones, promoting nitrification and denitrification called Wave Oxidation. Floating aeration chains are installed at the pond surface, supplying pressurized air to multiple submerged diffuser assemblies which are suspended from the aeration chains (see Figure 5.2). Air flow to the diffusers results in a swinging motion which maintains the required mixing and suspension of active floc in the pond system. As with the majority of activated sludge processes, secondary clarification is required for floc settling, typically accomplished either with clarifiers integrated in the ponds or conventional clarifiers constructed externally, and sludge processing facilities such as drying beds will be required to process sludge settled in the clarifiers.

Utilizing ponds as reactors, Biolac systems are well suited for pond retrofits. Existing ponds are typically emptied of process water and accumulated sludge and, if not already lined, lined with HDPE or another impermeable material. If necessary, inlet and outlet structures are modified, and ponds can be regraded to 2:1 side slopes and deepened to optimize performance. Enclosures for blowers are constructed in a central location and manifolds are installed to supply air to each Biolac pond. Piping is constructed to convey water to and from the ponds and conduit for electrical supply, process monitoring and control is also routed to each pond. If space is available, rectangular integrated secondary clarifiers can be constructed adjacent to retrofitted ponds, otherwise, traditional circular clarifiers would be constructed onsite, near Biolac ponds. Unused ponds could be converted to sludge drying beds for processing sludge from the secondary clarifiers.

---

[2] For activated sludge plants treating more than 5.0 MGD, additional operator certification is required.

Conceptual-level calculations have been performed to determine pond volume and equipment required to serve buildout demand. Four of the existing 2-acre ponds would be taken offline, in phases, and converted to Biolac ponds which would be operated in parallel. It was also assumed pond depth would be increased approximately four feet and side-walls would be modified from 3:1 to 2:1 side-slopes to optimize Biolac operation, and HDPE liners would be installed. These modifications are recommended to optimize operation and eliminate seepage from the unlined ponds.

In summary, a Biolac Wave Oxidation system retrofit serving buildout demand would include the following project components:

- Retrofitting four of the existing six 2-acre ponds, including increasing depth to 10 feet; modifying banks to 2:1 slope, installing HDPE lining;
- Installing air headers on pond banks, installing aeration chains and diffusers in ponds;
- Construction of a blower and controls building;
- Construction of aeration piping to each of the Biolac Ponds;
- Construction of a new distribution box and effluent junction box;
- Routing conduit to each converted pond for electrical supply, monitoring, and controls;
- Construction of secondary clarifiers and piping;
- Conversion of existing stabilization pond areas to sludge drying lagoons; and
- Construction of sludge piping.

The most prominent advantages of a Biolac conversion relative to other EAAS alternatives result from the system's simple design and ability to utilize existing ponds for the upgraded treatment process, eliminating the need for construction of large reinforced concrete vessels. Large pond volumes provide resistance to shock loading and, compared to traditional pond systems, allows significantly greater treatment capacity within a smaller footprint, yet requires only minimal additional construction for retrofit. The Biolac system can also be designed to allow increased treatment capacity to be achieved from a commissioned pond through subsequent addition of aeration equipment. This installation of additional diffusers can be performed without significant downtime. Disadvantages of the Biolac system include increased power consumption relative to a facultative pond system, additional controls and maintenance requirements, and additional staffing and operator certification requirements. Submerged diffusers will require periodic inspection and replacement; however, individual aeration chains can be isolated to allow replacement of diffusers while the other aeration chains stay online. Diffuser assemblies are designed for near-neutral buoyancy and easy retrieval to simplify maintenance tasks. Blowers will also require routine inspection and preventative maintenance. Figure 5.2 shows aeration components of a Biolac pond system. A conceptual site layout representing estimated footprint and possible locations for process components is provided as Figure 5.3. Process components are shown at approximate scale.

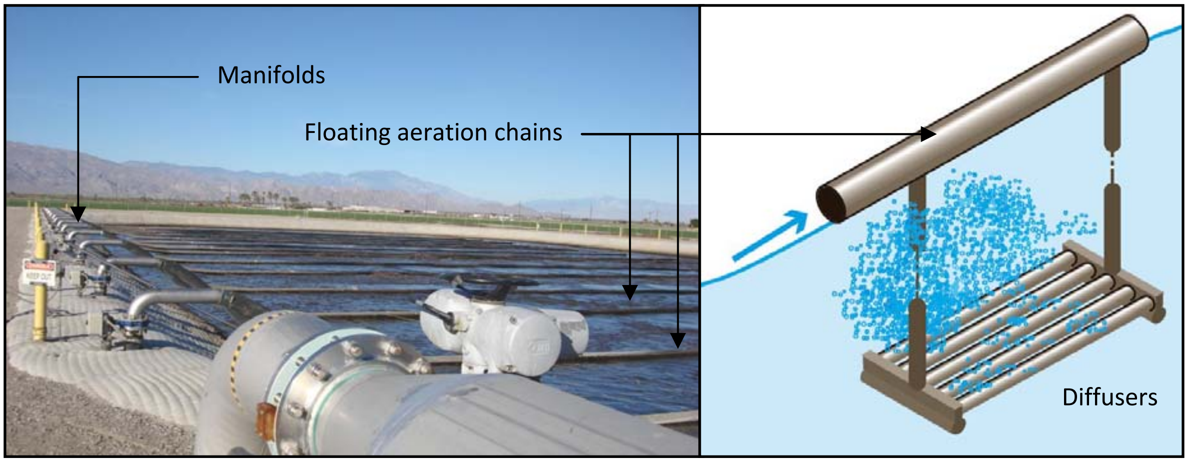

Figure 5.2: Biolac air distribution and diffuser components

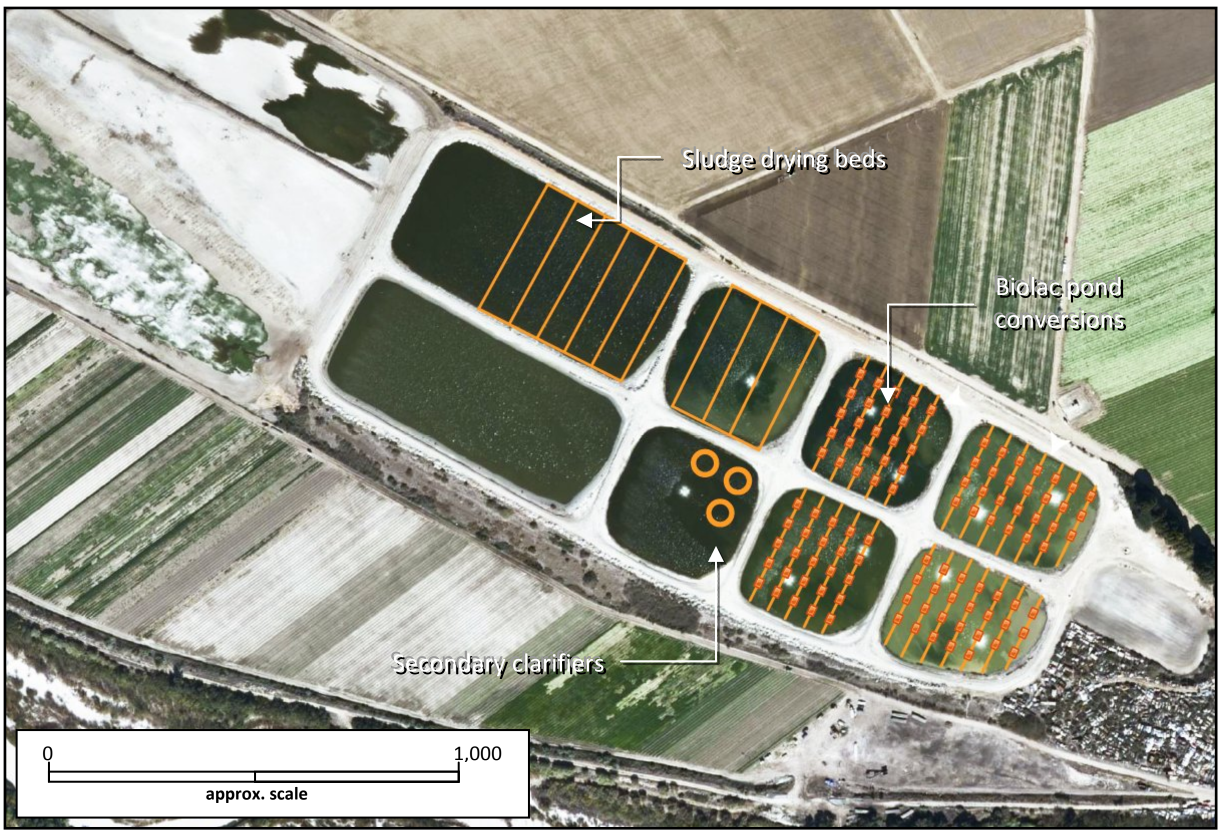

Figure 5.3: Estimated area required for an expanded facultative pond treatment system at buildout.

Oxidation Ditch EAAS System

An oxidation ditch is an EAAS process which utilizes mechanical mixers which induce continuous flow through a large continuous channel. The biological floc suspended through the mixing process is aerated with either mechanical aerators or submerged diffusers. The large volume and extended hydraulic and solids retention times provide stable operation and consistent effluent quality, common to EAAS treatment processes. Some oxidation ditch designs accomplish both nitrification and denitrification by incorporating complete-mix aerobic conditions near the mechanical mixers, typically at either end of the channel, and plug-flow through the straight sections of channel, where decreased oxygen concentrations promote denitrification. Like the Biolac EAAS process and other activated sludge systems, oxidation ditches rely on secondary clarification for floc settling, and separate sludge processing facilities. A recently constructed oxidation ditch system and secondary clarifiers are shown in Figure 5.4.

Figure 5.4: Example oxidation ditches and secondary clarifiers at a 10-MGD plant.

Oxidation ditch technology is a relatively simple activated sludge process and, like the Biolac systems, does not require a high level of operator observation and control for sufficient performance. Unlike the Biolac system, oxidation ditch designs require relatively little air piping. While the Biolac system relies on air delivery to multiple diffuser assemblies in each pond, an oxidation ditch will generally provide aeration to only few locations in the process, if any[3]. Additionally, oxidation ditch designs are slightly more compact than the Biolac pond system; however, subsequent and ancillary facilities such as secondary clarifiers and sludge processing facilities will be similar in size to those of a Biolac system, reducing some advantage of the compact biological process design.

[3] Oxidation ditches designed with mechanical mixing and aeration do not utilize diffusers or require aeration piping or blowers.

The most significant disadvantages of an oxidation ditch system result from the infrastructure-intensive design. Design and construction of a continuous channel (typically reinforced concrete for the subject flow range) is much more costly than a Biolac pond system retrofit sized for the same demand. And although aeration piping is minimal, mixing and aeration will require mechanical components and motors which would need routine inspection and maintenance. Failure of mixer equipment could also result in an entire oxidation ditch being offline until the issue is corrected. Phasing of an oxidation ditch system is also relatively inflexible due to magnitude of the construction effort and high cost.

The required land requirement for an oxidation ditch system sized to serve buildout demand was estimated using conceptual calculations. It was assumed the existing pond system would remain in operation during the construction of the initial oxidation ditch and that subsequent faculties could be construction on the locations of decommissioned ponds.

Construction of an oxidation ditch system serving buildout demand would include the following project components:

- Construction of oxidation ditch near the existing headworks;
- Installing air headers on pond banks, installing aeration chains and diffusers in ponds;
- Construction of a controls building in the vicinity of the oxidation ditch and headworks;
- Construction of a new distribution box and effluent junction;
- Electrical supply, monitoring, and control conduit to the oxidation ditch site;
- Construction of secondary clarifiers and piping;
- Conversion existing stabilization pond areas to sludge drying lagoons; and
- Construction of sludge piping.

5.3 Comparison and Feasibility of Biological Treatment Alternatives

A general comparison of the facultative pond expansion, Biolac pond conversion and oxidation ditch alternatives is provided in Table 5.1.

Table 5.1: Advantages and disadvantages of treatment alternatives

Upgrade Alternative	Advantages	Disadvantages
Facultative Pond Expansion	Simple and stable operation, does not require secondary clarifiers, sludge processing, or additional operator certification	Low treatment level not suitable for reuse, unable to treat nitrogen, large land area required, high cost associated with constructing new ponds for B/O demand
Biolac Wave Oxidation Pond Retrofit	Utilizes existing ponds and does not require additional construction of additional ponds, high level of treatment with stable operation and relatively low operator involvement, fewer mechanical components than aerated ponds	Requires construction of secondary clarifiers; inspection and maintenance requirements for blowers, air headers, and diffusers; additional operator certification requirements
Oxidation Ditch Plant	High level of treatment with stable operation and relatively low operator involvement	Requires construction of secondary clarifiers; inspection and maintenance requirements for motors, mechanical aerators, and possibly diffusers; additional operator certification requirements

Expansion of the facultative pond treatment system would require the City to acquire a significant amount of land in the vicinity of the existing plant which may not be feasible. Similar to the expansion strategies proposed in the Wastewater System Master Plan using AIPS and primary settling with aerated pond alternatives, this alternative would offer little opportunity for improving effluent water quality. Therefore, the facultative pond system expansion alternative is relatively incompatible with production of recycled water. The estimated construction cost for this alternative is very high, especially considering the limited effluent quality the system would produce. Land availability may also constrain this alternative. For these reasons, expansion of the facultative pond system beyond Phase 1 is not recommended as a long-term wastewater treatment strategy.

In anticipation of more stringent regulatory requirements in the future, it is important that provisions for improving plant effluent be made. Both the oxidation ditch and Biolac Wave Oxidation upgrades would provide nitrogen removal and produce an effluent which is suitable for some reuse opportunities. The Biolac pond system will occupy more land area than the oxidation ditch; however, since the existing ponds would be available for this retrofit, the Biolac alternative is expected to have a cost advantage over construction of an oxidation ditch facility.

A summary of conceptual estimated land area requirements and construction costs is provided in Table 5.2 and a comparison of alternatives with regard to effluent quality, regulatory concerns, land requirements, design and construction cost, and operations and maintenance requirements is provided in Table 5.3. Additional project costs of approximately 50% should be considered for design, construction administration, and contingency.

Table 5.2: Relative comparison of area and construction costs of treatment alternatives

Treatment Upgrade	Estimated Treatment System Footprint at Buildout [1]	Estimated Land Acquisition	Treatment Component Conceptual Construction Cost [3]
Facultative Pond Expansion	144 acres	100 acres	$20.7.6MM ($5.59/ gal) [4]
Biolac Wave Oxidation Pond Retrofit and Clarifiers	10 acres	1 – 2 acres [2]	$18.3MM ($3.89/ gal) [5]
Oxidation Ditch Plant and Clarifiers	3 acres	1 – 3 acres [2]	$22.9MM ($4.87/ gal) [5]

(1) Estimated footprint for biological treatment system only.

(2) Required land acquisition will depend on feasibility of taking existing aerated facultative ponds offline for construction. Areas shown are for treatment facilities only. It is estimated approximately 80 to 140 additional acres will be required for expanded percolation facilities, as discussed in the Effluent Disposal section below.

(3) Construction costs are for biological treatment components only and do not include land acquisition. Conceptual-level estimates within + 50% to -30% of construction cost.

(4) Per gallon cost on ADF basis for 3.7 MGD of added capacity.

(5) Per gallon costs on ADF basis for 4.7 MGD build-out capacity.

Table 5.3 : Qualitative Ranking of Alternatives (lower is better)

Treatment Alternative	Effluent Quality	Regulatory concerns	Land Acquisition Requirement	Design cost	Construction Cost	O&M Requirements
Expanded pond system	2	2	3	1	3	1
Biolac Wave Ox. EAAS	1	1	2	2	2	2
Oxidation Ditch EAAS	1	1	1	3	2	2

5.4 Effluent Disposal

Two alternatives for increasing effluent disposal capacity at the existing WWTP were considered and are discussed in the following subsections.

Infiltration Basins

Based on assumptions in Section 4, current infiltration basins are insufficient for disposing of effluent from an expanded WWTP serving build out demand. Expansion of the existing system would require additional land in the vicinity of the existing plant to be acquired, grading, construction of piping to convey biological treatment process effluent to new percolation bed locations, and construction of monitoring wells upstream and downstream of new percolation bed locations for assessing impacts to groundwater. Pumping may also be required to convey treated wastewater to the infiltration basins depending on basin location and configuration of biological treatment components. Advantages of percolating effluent include simple design, construction, operation, and maintenance. Possible disadvantages are large land area requirements, potential impacts to groundwater, and maintenance required to mitigate the accumulation of solids which results in clogging of the soil surface. This is not expected to be a significant issue if secondary clarification were included in future upgrades.

Currently one of the three infiltration basins is operated at a time, allowing offline basins to dry and for scarification/ tilling in preparation for future use. This approach is common for managing infiltration systems and often results in improved percolation rates. Cycling ponds also promotes biological activity in the soil column below the percolation beds which may otherwise be inhibited by continuous flooding.

The City has not reported any issues operating the existing beds at the current application rate. However, as noted in Section 4, improvements to the biological treatment processes may result in greater overall hydraulic loading of the infiltration beds.[4] Therefore, it is difficult to anticipate future requirements for infiltration basin area. An analysis of limiting conditions for the disposal fields including soil permeability, and projected nitrogen impacts from the current and improved treatment plant, and consideration of potential regulatory constraints should be undertaken to determine if additional capacity is available from the existing beds and if expansion is feasible from a regulatory perspective. Upgrading to an EAAS plant would improve nitrogen removl and is expected to result in reduced impacts to groundwater.

Based on the evaluation of existing capacity provided Section 4 and assuming future infiltration basins would perform similarly, an estimated 80 to 140 acres of additional percolation beds may be required. Conceptual construction costs for percolation basin expansion range from $1.4 to $2.4 million dollars, not including costs for land acquisition.

Agricultural Irrigation

Nearby agricultural areas may represent an opportunity for beneficial reuse and disposal of treated effluent which would offset some demand for groundwater in agricultural areas near the existing WWTP. Based on flow projections, wastewater flow rate will total 4.7 MGD (14.2 acre-feet per day) at buildout. Assuming the entire flow would be used for irrigation and a year-round application rate of 14,000 gpd/ acre, approximately 340 acres of irrigated area would be required. However, since crops are not typically irrigated year-round, seasonal storage or another means of effluent disposal would be required. Treatment and disinfection requirements for production of recycled water for agricultural irrigation are driven by human exposure risk and will depend on the type of crop which will be irrigated. Depending on the crop recycled water is applied to, agricultural irrigation would may require disinfection of secondary treated effluent or tertiary filtration and disinfection of secondary effluent, therefore, addition of disinfection faculties and possibly a tertiary filtration process would be necessary at the upgraded plant.

In summary, project components may include:

- Disinfection facilities (depending on resuse);
- Tertiary treatment processes (depending on reuse);

---

[4] It is assumed some infiltration through the existing unlined lagoon beds is resulting in overall effluent reduction. Lining ponds or upgrading to an EAAS process would inhibit effluent reduction through the treatment process and result in some increase in effluent requiring disposal.

- Seasonal storage and transmission main;
- Distribution pumping; and
- Recycled water planning including identifying prospective users, water quality requirements, preparation of necessary engineering reports, developing agreements with users, and permitting tasks.

Considering the significant agricultural irrigation demands near the existing treatment plant, it is possible that the entire plant flow could be utilized through irrigation. This opportunity warrants further evaluation of economics and feasibility and is recommended as part of the forthcoming Master Plan.

Satellite Treatment Plant

Historically, projects proposing construction and operation of decentralized plants were met with significant regulatory challenges. In general, regional approaches to wastewater management were favored and decentralized systems were permitted only when specific constraints required that a decentralized system be used. However, since more emphasis is being placed on conservation of water resources, wastewater management strategies which offer opportunities for production and distribution of recycled water have gained favor. Therefore, the feasibility of constructing and operating a satellite wastewater treatment plant is discussed in the following paragraphs. A satellite treatment plant would provide for beneficial reuse while being connected to the existing collection system would allow bypass of the system in the case of significant operational issues or reductions in recycled water demands (such as seasonal demand fluctuations).

It is assumed a satellite treatment plant treating wastewater generated from the planning area would provide for reuses in nearby areas. In addition to proximity to reuse applications, the following should be considered when citing a satellite plant:

- Supply of wastewater
- Potential impacts to the community including noise, odor, aesthetics
- Land availability
- Water quality, treatment and distribution requirements for reuses
- O&M requirements and process reliability

One commonly employed treatment technology for production of recycled water is the membrane bioreactor (MBR). MBR technology requires a relatively small footprint, is fairly reliable, and is capable of providing a high level of treatment, thereby maximizing opportunities for reuse. Due to their compact designs, MBRs are often constructed in enclosures, allowing noise and odor from the process to be easily controlled, although both are typically minimal.

Ideally, an MBR satellite treatment facility serving the Urban Growth Area would be sited downstream of new development producing wastewater in volumes similar those required by specified recycled water users (such as urban irrigation sites), and near points of use to minimize the need for transmission pipelines and pumping of reclaimed water. The treatment facility would recover a portion of wastewater from collection system transmission mains for treatment and return residual solids to the collection system.

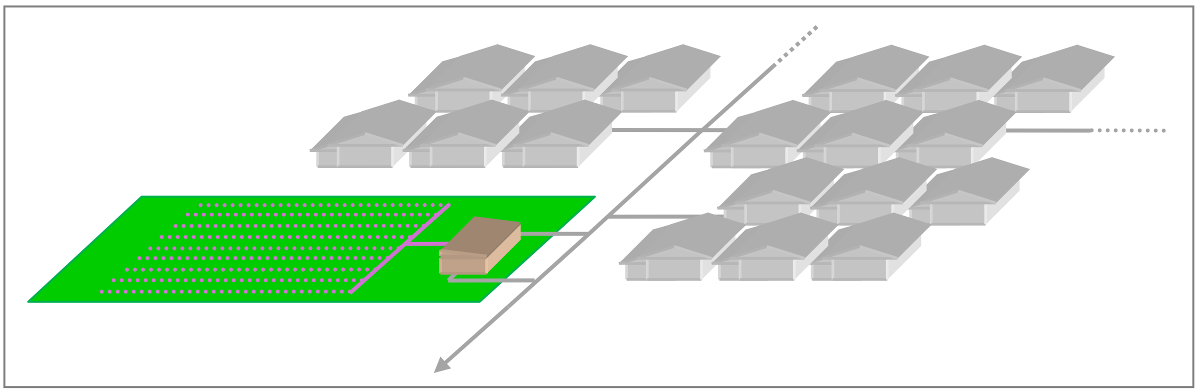

Figure 5.5: Extraction-type satellite facility recovering wastewater from collection system for local reuse

Construction and operation of a satellite facility for production of recycled water will require a reclamation permit, and considerable planning and regulatory involvement. Construction of a satellite MBR facility may require the following components:

- Connection of a flow diversion structure to supply the satellite plant;
- Pretreatment and treatment components including equalization tank, screening and grit removal, biological treatment, and membrane filtration;
- Reclaimed water operational storage;
- Reclaimed water pumping and distribution piping; and
- Residuals return line (connection to collection system).

A satellite MBR facility would require routine observation and maintenance of chemical supplies used in the treatment process. Disadvantages include high capital cost and energy consumption, proprietary membranes are used and are costly to replace, and since MBR technology is relatively new, records and references of long-term performance and operation vary between manufacturers.

Although constructing and operating a satellite treatment plant for recycling a portion of wastewater generated in the Urban Growth Area is feasible, it is unlikely this technique could be used to entirely divert wastewater generated from Urban Growth Area properties for reuse. Therefore, it is recommended this approach be considered as a means of providing recycled water and offsetting water demands rather than a wastewater flow elimination technique. It is recommended satellite treatment be evaluation on a project by project basis to identify opportunities and benefits from this approach.

Since potential reuses have not been identified, conceptual costs for constructing an MBR facility were developed for a range of flow rates. It is expected that actual project costs will vary considerably as information specific to projects is defined. Estimated area requirements are also provided for conceptual planning purposes.

Table 5.4: Conceptual construction costs and estimated area required for MBR satellite treatment facilities

Average Flow, MGD	Conceptual Construction Cost	Approximate footprint, SF
0.1	$3.0MM	3,200
0.5	$11.3MM	6,200
1.0	$20.4MM	8,500
2.0	$25.5MM	11,500
3.0	$34.9MM	15,000

6.0 RECOMMENDED ALTERNATIVES

Based on the evaluation of treatment capacities and upgrade alternatives in Sections 4 and 5, upgrading the current pond treatment system to an extended aeration system is recommended beyond Phase 1. Based on conceptual estimates, costs of an EAAS upgraded treatment plant serving the projected 4.7 MGD buildout flow are expected to range from $19.1 to $23.2 million dollars.

In addition to the biological treatment and effluent disposal alternatives, additional improvements and expansion of WWTP processes will be necessary to treat projected wastewater flow at buildout. A list of recommended improvements follows, with brief discussions of each improvement in the following sub-sections.

- Headworks improvements
- Influent pumping
- Upgraded treatment process (EAAS and secondary clarification)
- Solids treatment/ sludge drying
- Expansion of land disposal and development of reuse applications

For budgetary and planning purposes, conceptual project costs have been developed for recommended improvements for meeting the entire buildout demand and are included with each discussion. Overall costs are summarized in Section 7.

6.1 Headworks

The existing headworks will need to be upgraded to provide increased capacity and improved pretreatment. Screening will be necessary for the EAAS process improvements evaluated in Section 5. The new headworks would include mechanical screening, screenings washing and compaction, grit removal, flow metering, and associated appurtenances. Conceptual construction cost for a new headworks is approximately $2.9MM. Although it is likely an entirely new headworks structure and equipment would eventually be constructed to serve projected buildout flows, an analysis of the existing headworks should be conducted to determine capacities of existing concrete structures and installed equipment such as grit chambers to maximize utilization of existing facilities and to allow for planning of phased improvements.

6.2 Influent Pumping

The existing influent pump station will need to be upgraded to serve projected buildout demand. As indicated in Section 4, the existing pump station is currently rated at 3.8 MGD. Since projected buildout demand is significantly greater than the existing capacity, it is assumed construction of an entirely new pump station would eventually be required to serve buildout demand. Conceptual construction cost for the new influent pumping station is approximately $0.9MM. An analysis of wet well and pumps is recommended to determine the useful life of the existing pump station as designed. This analysis may indicate additional capacity can be achieved from the wet well without constructing new structures by simply replacing impellers in the existing pumps or installing new pumps. In this manner, gradual improvements can be planned to make best use of the existing facilities and equipment, if feasible.

6.3 Sludge Management

Biological activity in activated sludge processes produces greater volumes of sludge than the existing aerated facultative pond system. Waste sludge will require stabilization in preparation for reuse or disposal. Both of the EAAS processes evaluated in this study and recommended in this section are less land intensive than the existing system and would ultimately result in un-used ponds. Since this land area is already owned by the City, it is recommended retired treatment ponds be considered for converted to lined sludge drying lagoons. Sludge transfer piping and control valves would be needed to convey liquid sludge to the drying lagoons and allow decanted liquid to be circulated to the headworks. Concrete sludge lagoon liners are also recommended to prevent potential seepage of liquid sludge and facilitate “working” drying sludge and removal without risk of damaging the lining. Waste activated sludge (WAS) from the secondary clarifiers would be discharged to one of the sludge lagoon at a time and allowed to accumulate to a depth of 2 to 4 feet, at which time the WAS discharge from the clarifiers would be routed an alternate sludge drying lagoon. Once sufficiently dry, the sludge would agitated or “worked’ to increase drying and volume reduction in the lower sludge layers. When a drying cycle is complete, solids can be removed from the drying bed and disposed of or reused.

Approximately 6 - 10 acres of sludge drying beds will be required to continuously process sludge from the EAAS treatment processes at buildout. It is assumed one 2-acre pond and one of the 5-acre ponds would be converted for sludge processing and piping would be constructed to convey sludge to the drying beds. Conceptual construction costs for the sludge drying facilities to serve buildout demand total $1.5MM.

6.4 Effluent Disposal and Reuse

Based on the capacity evaluation in Section 4 and considering the current state of recycled water planning, it is recommended the City investigate opportunities for expansion of the existing infiltration basins. This will include acquiring additional land in the vicinity of the existing basins. An initial NRCS soil database survey indicates areas northwest of the existing basins may have the greatest potential for infiltration basins. Percolation testing and a hydrogeologic study are also recommended to allow a better estimation of existing disposal capacity and determine possible long-term effects of effluent infiltration. Based on the evaluation presented in Section 5, conceptual construction cost for infiltration basins serving the City and Urban Growth Area at buildout is expected to range from $1.4MM to $2.4MM. This range of conceptual construction cost does not include costs for land acquisition or recommended studies.

Additionally, it is recommended the City develop a plan for effluent reuse in the area of the WWTP. This will require identification of prospective applications near the existing treatment plant (minimizing recycled water transmission infrastructure where possible) and evaluation of requirements for providing recycled water to prospective applications. Costs for providing recycled water will vary significantly depending on application, treatment, and volume produced.

7.0 COST ESTIMATES FOR RECOMMENDED ALTERNATIVES

Conceptual cost estimates for the recommended alternatives and project components discussed in Section 6 are presented Table 7.1. The purpose of these estimates is for budgetary planning and actual project costs are expected to vary as designs of project components progress and aspects of the project are better defined. Design, construction administration, and contingency may be calculated as an additional 50% of conceptual construction costs. Cost estimates prepared for this plan are conceptual and are expected to be accurate to within -30% to +50%.

Table 7.1 Conceptual Project Costs

Headworks Screening, Grit Removal, Metering	$ 2.9 MM
Influent Pump Station	$ 0.9 MM
EAAS Biological Treatment* and Secondary Clarifiers	$ 18.3 to 22.9 MM
Sludge Drying Lagoon Retrofit	$ 1.5 MM
Infiltration Basins	$ 1.4 to 2.4 MM
Total Conceptual Construction Cost	**$ 25.0 to 30.6 MM**

*Biolac Wave Oxidation pond conversation and oxidation ditch were evaluated for upgrade to EAAS.

9.0 COLLECTION SYSTEM CAPACITY EVALUATION

9.1 Existing Collection System

The existing sewer collection system has been evaluated based on information provided by the City and flow projections developed in Section 3 of this study. This analysis confirms the feasibility of serving the planning area with an expanded and improved sewer collection system and allows the City to plan for collection system improvements which will be necessary to support infill development within the existing City and new development in the study area.

Wastewater flow rates were estimated for the existing City area according to the estimated number of dwelling units and acreages of land uses within 13 tributaries. Estimated wastewater flow generated from each tributary is summarized in Table 9.1. Although flow metering data was not available to further calibrate the loading scenario, the total flow rate estimated for the existing City using this method was compared to the actual wastewater flow rate experienced at the WWTP. The overall result of this loading scenario is within 10% of actual average flow observed at the WWTP. It is expecting a more refined technique will be used in future master planning efforts; however, this approach is sufficient for conceptual modeling.

Table 9.1 Existing Sewer Collection System Tributaries and estimated average flow rates

Tributary	Estimated Dwelling Units	Other uses	Estimated average wastewater generation, gpd
1	289	Comm./ mixed use	78,182
1A	46	--	12,524
2	262	School, Comm./ mixed use	82,452
3	89	School, Park	39,194
4	146	--	39,494
5	81	--	21,886
6	74	--	19,964
7	290	--	78,616
8	362	--	98,146
9	115	Commercial	34,662
10	314	--	85,064
11	--	Comm./Industrial	24,200
12	--	Comm./Industrial	18,000

A model of existing sewer collection mains and interceptors was creating using the City's Collection System map. Approximate lengths of pipe segments and nominal pipe diameters were available in the Collection System map; however, slopes of gravity sewers and manhole inverts were unavailable. Several parameters are needed to accurately determine gravity sewer capacities, including slopes of pipe segments. Since this data was not available for this study, and the existing City area has minimal terrain relief, minimum pipe slopes capable of half-full velocity of 2 feet per second were assumed for the critical sewer segments modeled. This conservative assumption is the basis for estimating capacities of existing gravity sewers and identifying potential deficiencies within the existing system, and is similar to the method used in the 2001 Wastewater System Master Plan. However, additional information and data on the existing collection system including manhole inverts and pipe slope data should be collected for major trunk mains in preparation for future master planning. These data would allow a more precise representation of existing conditions and actual pipe segment capacities.

The gravity sewer design criteria summarized in Table 9.2 were assumed. Maximum design capacities for gravity sewers calculated using the Manning Equation for gravity flow and assumed slopes, and pipe roughness coefficients of 0.013 and 0.015 for PVC and VCP, respectively, are summarized in Table 9.2. Maximum design capacities for force mains were calculated according to recommended flow velocity. Ideally, force mains would be sized to maintain flow velocities between 3.5 to 5 fps, although maximum velocities of 7 fps may be acceptable. Maximum capacities for force mains are included in Table 9.3.

Table 9.2 Minimum Gravity Sewer Grades and Design Depth Rations

Pipe Diameter, in	Design depth to diameter (d/D)	Minimum Slope, ft/ft
6	0.5	0.0035
8	0.5	0.0035
10	0.5	0.0025
12	0.5	0.0020
15	0.75	0.0015
18	0.75	0.0012
21	0.75	0.0010

Table 9.3 Existing sewer trunk main and force main and estimated capacities

Pipe Segment ID	Diameter	Material	Maximum design capacity [1], gpd
A	21	VCP	2,583,600
B	21	VCP	2,583,600
C	12	PVC	505,800
D	10	PVC	347,700
E	8	PVC	226,800
F	6	PVC FM	630,000 (2)
G	6	PVC FM	630,000 (2)
I	12	PVC	505,800
J	10	PVC	347,700
K	6	PVC FM	630,000 (2)
L	10	VCP	312,900
M	8	VCP	204,300
N	8	VCP	204,300
P	21	PVC	2,870,700
Q	12	PVC	505,800
R	21	PVC	2,870,700
S	18	PVC	2,084,700

(1) Maximum capacity calculated for gravity sewers based on assumptions describe in this section
(2) Maximum capacities for force mains

An analysis of the existing collection system was performed using estimated PHF conditions for tributary areas representing peak loading conditions. Lift station pumping capacities were not available for this study; however, the City has indicated existing lift stations were sized for the current service areas tributary to each lift station without provisions for providing additional capacity. Peaking factors were applied to flows conveyed by lift stations to represent flow conditions when lift stations are operating.[5] The ultimate peak flow condition would most likely occur during daily peaks in diurnal curve when large amounts of wastewater are generated and multiple lift stations operate simultaneously.

[5] For lift station peaking, it was assumed wet wells were designed to cycle four times per hour. An inventory of existing lift station pumping capacities, wet well sizing, and pump runtimes were not available for this study but should be developed for use in future planning efforts.

9.2 Conceptual Plan for Serving the Urban Growth Area

A conceptual collection plan for providing service to the Urban Growth Area was developed based on review of the LUE and topographical mapping acquired as part of this project. Recommended approaches for providing sanitary sewer service to the Urban Growth Area were developed for three sewer service zones based on our review of topographical mapping, a map of the existing collection system provided by the City, and land uses proposed in the LUE of the General Plan. Recommended approaches for each zone take into account the following considerations:

- To the extent possible, reliance on pumping and construction of new lift stations should be minimized;
- Creation of new highway crossings should be minimized; and
- To the extent possible, travel time should be minimized.

Service Zones

Zone 1 includes the entire Cielo Grande property, the Pisoni property, and a small northern portion of the D'Arrigo Property. Due to westerly facing topography, the north-western portion of the Cielo Grande area may need to be served by one or more lift stations located in the north-west corner of the property, conveying flow toward a centrally located trunk main. Alternatively, the northern portions of Cielo Grande could be served by constructing an interceptor through the urban reserve area in the west of the Urban Growth Area (Bassi Property), constructing a crossing at Highway 101, and obtaining necessary easements for further construction of a lift station and force main along North Alta road for conveying flow toward the main WWTP interceptor. For the purposes of this study it is assumed one or more lift stations would be constructed within the Ceilo Grande Property and flows will be directed toward the existing Arroyo lift station and existing highway crossing. The Pisoni property would be served independently via new lift station and force main conveying wastewater toward the existing Arroyo lift station.

Zone 2 consists of the northern portion of the D'Arrigo property. It is recommended wastewater collection infrastructure serving Zone 2 be designed to convey flow toward the existing California Breeze lift station and Highway Crossing.

Zone 3 includes a southern portion of the D'Arrigo property, Rincon Village, the Rianda/ Foletta property near Highway 101, the Rianda property, the Jackson property, and the Franscioni property. It is recommended the southern portion of the D'Arrigo property, Rincon Village, and the Rianda/ Foletta property be served by a collector main running parallel to Highway 101 and conveying flow toward a future lift station and highway crossing at Gloria Road, then connecting to the newly upgraded trunk main on South Alta Street. The Jackson and Rianda properties would be served by a trunk main connecting these properties to a future lift station and highway crossing at Gloria Road, and the trunk main on South Alta Street.

Flow Assignment from Urban Growth Area Properties

Loads from new development in the study area were approximated using duty factors developed in Section 3, approximate acreages of Urban Growth Area properties tributary to the proposed tie-in locations, and land uses identified in the land use element. Calculated loads are summarized in Table 9.4 according to the proposed service zones.

Table 9.4: Wastewater Generation from the Future Service Area and downstream infrastructure

Urban Growth Area Collection Zone	Estimated ADF at Buildout (MGD)	Peak load from Urban Growth Area (pumped flow)	Conceptual Tie-in Location	Trunk Main and Downstream Segments
Zone 1	1.16	3,500 gpm	Arroyo	Tenth Street (F, E, D, C, B, A)
Zone 2	0.69	2,250 gpm	California Breeze	Seventh/ Fifth Street (K, J, I, B, A)
Zone 3	2.17	6,100 gpm	Gloria Road (new lift station)	S. Alta Trunk Main (S, R, Q, P, A)

Buildout Deficiencies and Recommended Improvements

The model of existing collection system components was loaded with projected sewer flow rates according to the conceptual tie-in points for each wastewater service area within the Urban Growth Area zone. Deficiencies in the existing system for serving projected buildout flow were identified and improvements for serving future peak loading were developed. Deficiencies and improvements are summarized in Table 9.5.

Table 9.5: Estimated Capacity Requirements and Pipe Diameters for Buildout

	Existing Diameter (inches)	Estimated Existing Capacity (gpd)	Estimated capacity required for Build out* (gpd)	Estimated Diameter Required for Buildout (inches)
Tenth / Elliot Street				
C	12	505,800	5,463,560	30
D	10	347,700	5,312,240	30
E	8	226,800	5,160,920	30
F	6 FM	630,000	5,032,584	16
Seventh/ Fifth Street				
I	12	505,800	3,470,064	24
J	10	347,700	3,320,184	24
K	6 FM	630,000	3,242,904	12 - 16
Gloria Road - S. Alta Trunk Main				
P	21	2,870,700	8,826,250	36
Q	12	505,800	8,826,250	36
R	21	2,870,700	8,758,750	36
S	18	2,084,700	8,758,750	36
WWTP Interceptor				
A	21	2,583,600	18,602,364	48
B	21	2,583,600	9,776,114	36

* Required capacity based on projected PHF loading at buildout and includes lift station peaking for pumped flows

Table 9.6: Estimated Pumping Capacity Required for Buildout

Lift station	Estimated capacity required for Build out* (gpm)
Arroyo	3,500
California Breeze	2,300
Gloria (proposed)	6,100

* Series lift stations in Tributary 8 were not modeled and would not be impacted by proposed Urban Growth Area tie-in locations.

9.3 Recommended Improvements

Recommended improvements for providing sewer service to the Urban Growth Area are described in the following sub-sections.

Tenth Street and Elliot Street Trunk Mains (Segments C, D, E, and F)

Improvements to the Tenth Street and Elliot trunk mains may be necessary to accommodate buildout of the existing City area. Based on modeled conditions, upgrading to 15-inch gravity sewer may be necessary to support future development within the City. It is recommended the City conduct further analysis on this segment of existing sewer in the forthcoming Wastewater Master Plan.

With the proposed tie-in location for future development of the Urban Growth Area properties in Zone 1, upgrading the Tenth Street and Elliot trunk mains to 30-inch PVC gravity sewer (approximately 3,700 LF), increasing capacity of the Arroyo lift station to approximately 3,500 gpm, and upgrading to a 10-inch force main may be necessary to provide service to the Urban Growth Area's Zone 1 properties at buildout.

Seventh Street and Fifth Street Trunk Mains (Segments I, J, and K)

Based on modeled conditions, upgrading segments of the Seventh Street and Fifth Street Mains to 12 and 16-inch gravity sewer may be necessary to accommodate buildout of the existing City area. It is recommended the City conduct further analysis on these existing sewer segments in the forthcoming Water Master Plan.

With the proposed tie-in location for future development of the Urban Growth Area within Zone 2, upgrading the Seventh Street and Tenth Street trunk mains to 24-inch PVC gravity sewer (approximately 3,700 LF), increasing capacity of the California Breeze lift station to approximately 2,300 gpm, and upgrading to a 12 or 16-inch force main may be necessary to provide service to Urban Growth Area properties within Zone 2 at buildout.

Alta Street and First Street Trunk Mains (Segments L and M)

Based on modeled conditions, upgrading the Alta Street and First Street Mains to 15 and 10-inch gravity sewers, respectively, may be necessary to accommodate buildout of the existing City area. It is recommended the City conduct further analysis on these existing sewer segments in the forthcoming Water Master Plan. Based on proposed tie-in locations for Urban Growth Area service zones, no impacts are projected for these segments of existing sewer from future flow rates from the Urban Growth Area.

Gloria Road Lift Station and Highway Crossing (Segments P, Q, R, S, T)

Construction of a highway crossing at Gloria Road will be necessary to convey wastewater generated from Urban Growth Area Zone 3 properties to join the existing trunk on South Alta Street which conveys flows to WWTP interceptor on Femin Lane via Gonzales River Road. Based on modeled conditions and assuming a 6,100 gpm lift station would be constructed at Gloria road, upgrades to the South Alta and Gonzales River Road trunk mains to 36 inches (7,400 LF) would be required to convey projected peak loading from the Gloria Road lift station.

Fermin Lane Interceptor

Based on modeled conditions, the existing 21-inch WWTP sewer interceptors are sufficiently sized to serve demand of the existing City area at buildout but will require upgrades for serving Urban Growth Area properties. This upgrade would consist of increasing the diameter of existing sewer Segment B to 36 inches (approximately 7,900 LF) and increasing the diameter of Segment A to 48 inches (8,300 LF). Since the existing 21-inch sewer may sufficiently serve buildout of the existing City but will require significant upgrade for serving future development, the City should allocate funds for this upgrade as plans for development of the Urban Growth Area properties progress.

9.4 Urban Growth Area Project Costs

Conceptual project costs have been developed based on costs of materials, preparation, earthwork, installation, and roadwork for gravity pipe installation. Cost Criteria are summarized in Table 9.7 and conceptual costs for improvement projects are presented in Tables 9.8 through 9.12. Design, construction administration, and contingency may be calculated as an additional 50% of conceptual construction costs.

Table 9.7: Piping Improvement project cost criteria

Description	Conceptual construction cost
12-inch gravity pipeline	$182/LF
15-inch gravity pipeline	$210/LF
18-inch gravity pipeline	$248/LF
21-inch gravity pipeline	$288/LF
24-inch gravity pipeline	$320/LF
27-inch gravity pipeline	$366/LF
30-inch gravity pipeline	$414/LF
36-inch gravity pipeline	$512/LF
42-inch gravity pipeline	$588/LF
48-inch gravity pipeline	$672/LF
10-inch force main	$185/LF
12-inch force main	$196/LF
16-inch force main	$214/LF

* Required capacity based on projected loading at buildout and includes lift station pumped flow

Table 9.8: Tenth Street and Elliot Street Trunk Mains (Segments C, D, E, and F)

Project Component	Quantity	Unit cost	Total
30-inch gravity sewer	3,700	$414	$1,531,800
16-inch force main	400	$214	$85,600
Lift station upgrade	1	$750,000	$750,000
			$2,367,400

Table 9.9: Seventh Street and Fifth Street Trunk Mains (Segments I, J, and K)

Project Component	Quantity	Unit cost	Total
24-inch gravity sewer	3,700	$320	$1,184,000
16-inch force main	900	$214	$192,600
Lift station upgrade	1	$700,000	$700,000
			$2,076,600

Table 9.10: Alta Street and First Street Trunk Mains (Segments L and M)

Project Component	Quantity	Unit cost	Total
15-inch gravity sewer	2,300	$210	$483,000
10-inch force main	1,300	$142	$174,600
			$657,600

Table 9.11: Gloria Road Lift Station and Highway Crossing and S. Alta Trunk Main (Segments P, Q, R, S, T)

Project Component	Quantity	Unit cost	Total
36-inch gravity sewer	7,400	$521	$3,855,400
Lift station and crossing	1	$1,200,000	$1,200,000
			$5,055,400

Table 9.12: Fermin Lane Interceptor (Segments A and B)

Project Component	Quantity	Unit cost	Total
36-inch gravity sewer	2,600	$521	$1,354,600
48-inch gravity sewer	8,300	$672	$5,577,600
			$6,932,200