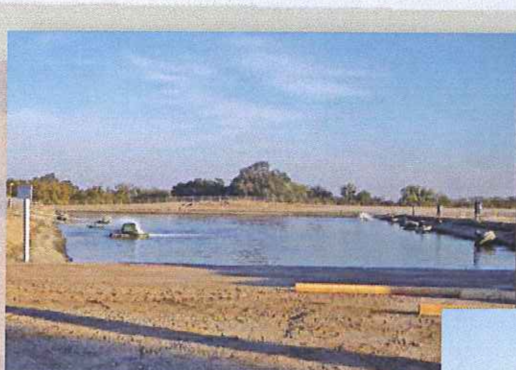
LAND OF PEACHES & CREAM

ESCALON

CITY OF ESCALON

Sewer Master Plan



January 2007

Prepared for
City of Escalon

Prepared by
ECO:LOGIC Engineering

ECO:LOGIC

Consulting Engineers

CITY OF ESCALON

Sewer Master Plan

January 2007

Prepared for:

City of Escalon
1854 Main Street
Escalon, CA 95230-0248

Prepared by:

ECO:LOGIC Engineering
777 N. Pershing Avenue, Suite 2B
Stockton, CA 95203

REGISTERED PROFESSIONAL ENGINEER
NEAL T. COLWELL
NO. C 59437
EXP. 12/31/07
CIVIL
STATE OF CALIFORNIA

Oct. 16, 2007

ECO:LOGIC

Consulting Engineers

Contents

City of Escalon – Sewer Master Plan

Tables

Figures

Appendices

Section 1

Introduction and Background

Section 1

Introduction and Background

1.1 BACKGROUND

The City of Escalon (City) currently provides sewer service to approximately 7,200 people within an area encompassing approximately 1,200 acres. The City has two sewer systems, the domestic sewer system and the industrial sewer system. Wastewater collected in these systems is conveyed separately to two separate treatment and disposal facilities located near McHenry Avenue immediately north of the Stanislaus River. The domestic sewer system provides service to residential, commercial and "dry" industrial users. The purpose of this Master Plan is to update the previous sewer and wastewater facilities master plans and develop a plan for extending sewer to currently unsewered areas within the City limits and to areas within the City's future growth boundaries. The wastewater treatment section of this Master Plan Update is to prepare an improvement plan for incrementally increasing treatment and disposal capacity of the domestic wastewater treatment and disposal facilities, including consideration of likely future permit requirements. This Master Plan Update does not evaluate the industrial sewer or industrial wastewater treatment and disposal facilities, other than with respect to their potential influence on the domestic system.

The projected growth and land uses used to determine the magnitude and location of future wastewater flows is based on the City's updated General Plan that was adopted in June 2005 and prepared by Quad Knopf. The General Plan provides planning for the future growth in the City and encompasses the current City boundary and provides the overall framework for how the City will grow. The General Plan includes estimates about future population and land uses through the year 2035. The General Plan defines three distinct planning boundaries which may be expanded incrementally to the year 2035. The General Plan land uses, population projections, and planning horizons were used in the development of the Sewer Master Plan Update for planning capital improvements.

The objectives of the Sewer Master Plan Update include:

- Review of the City's major sewer facilities; lift stations and collector mains, and a reconnaissance assessment of available capacity in these facilities to convey future wastewater flows.
- Projection of future wastewater flows based on current average wastewater flows and future planned land uses as defined in the General Plan.
- Development of a logical sewer system expansion plan based on the growth projections and land uses that can be phased as growth actually occurs.

- Review of current and anticipated regulatory requirements that will influence requirements for wastewater disposal and assess the possible facilities ramifications of such regulatory requirements.

- Development of a reasonable wastewater treatment and disposal facilities expansion plan, with the goal of maximizing the use of current facilities.

- Provide capital costs for the improvements, which can be used to develop an appropriate sewer facilities expansion or connection charge.

- Provide an implementation plan and recommended sewer capital cost per equivalent dwelling unit.

1.2 SCOPE

The scope of this work consists of the following major elements:

- Review existing reports, drawings, land use and zoning maps, and other relevant drawings;

- Evaluate existing major sewer facilities, in particular regarding excess available capacity to convey future wastewater flows;

- Project future wastewater flows based on land uses defined in the City's General Plan and existing wastewater flow data;

- Review regulatory and permit requirements as they may be applied to the City's wastewater treatment and disposal facilities;

- Evaluate existing wastewater treatment and disposal facilities and their capabilities to serve future growth;

- Develop and describe a recommended plan of expansion including cost and staging;

- Recommend an appropriate capital cost per equivalent dwelling unit for master planned facilities.

1.3 ACKNOWLEDGMENTS

The cooperation, input and support received from Doug Stidham and his staffs are gratefully acknowledged.

Section 2

Executive Summary

Section 2

Executive Summary

2.1 DESIGN CRITERIA AND PROJECTED GROWTH

This Sewer Master Plan Update was based on approved and planned land uses as outlined in the City's 2005 General Plan Update and as included in the City's General Plan Update Background Report (February 2004). For the Liberty Business Park area, additional information was obtained from the Initial Study prepared for the Liberty Business Park. Projected sewer flows for various levels of development were considered from current conditions through the ultimate build out condition in the 2035 growth boundary as defined in the General Plan.

The planning boundaries defined in the General Plan include:

- 2015 General Plan Growth Boundary
- 2025 General Plan Growth Boundary
- 2035 General Plan Growth Boundary

These planning boundaries have been used as a guide in projecting future development within a phased expansion of the City's boundary. The growth boundaries are not associated with an apparent time line as the City plans to develop the area encompassed by each boundary prior to expanding into the subsequent boundary.

Recent Wastewater Treatment Plant influent flow records were used to evaluate current wastewater generation rates as compared to the City current design standard. Based on this information, the sewer generation factors were modified in order to calculate the flows generated from future development within the current City limits and within the City's growth boundaries. This modification was done to better reflect the current average influent flows to the WWTP, with a reasonable factor of safety. Table 2-1 summarizes the wastewater flow factors used to project the wastewater generated from undeveloped areas within the current City limits and areas within the 2015, 2025, and 2035 growth boundaries. It is recommended that the City review the policies contained in this Master Plan Update, including possibly modifying the Improvement Standard No. S2 to reflect actual sewer flows.

Current influent flows to the WWTP are about 600,000 to 700,000 gallons per day on average during the dry summer months (0.6 to 0.7 million gallons per day). Influent flows are anticipated to increase to approximately 2.8 million gallons per day as development occurs within the 2035 growth boundary. Based on the revised unit flow factor per equivalent single family dwelling unit (EDUs), the projected 2.8 million gallons per day future flow represents approximately future 8,400 EDUs to be connected to the sewer system. Currently, there are approximately

3,300 EDUs connected to the system with an average existing unit flow factor of approximately 210 gallons per day per existing EDU.

Table 2-1
Wastewater Flow Factors for Undeveloped and Future Growth Boundaries

Land Use Designation	Description	Flow Factor Units	Unit Flow Factor
Low Density Residential	Single Family Home	gpd/acre/density	250
Medium Density Residential	Multi-family unit	gpd/acre/density	210
High Density Residential	Large Apartment	gpd/acre/density	210
Public Facility[a]	School	NA	Varies
Commercial	Commercial	gpd/acre	1,000
Industrial	Light Industrial, Industrial	gpd/acre	1,000

[a} Flows for public facilities (schools) were assigned based on the number of student capacity of each school and typical wastewater flows per student according Metcalf & Eddy "Wastewater Engineering", Third Edition, Table 2-11

2.2 SEWER SYSTEM EXPANSION ALTERNATIVES

Existing sewer facilities were reviewed and areas of planed expansion identified within the current sewer service area and each growth boundary as shown in Figure 7-1. Current and future sewer service areas were identified by sewer collection areas with defined connection points to the existing or future trunk sewer facilities. Sewer system expansion focused on major system improvements necessary to increase sewer capacity or extend sewer service to currently unsewered areas. Sewer improvements that provide benefit only to individual development projects were not included in this evaluation.

Alternatives to expanding the sewer system were screened based on such criteria as cost, conflicts with other existing utilities, ease of phasing, and general ease of operation and maintenance. Based on this screening a recommended sewer expansion plan was prepared and is outlined in Table 7-1. Sewer improvements have been grouped into four phases as follows. These groupings are based on the relative timing and need for the improvements and are:

- Near-Term improvements to the existing sewer system to provide capacity for immediate development.
- Improvements needed to extend service to the Heritage Park and Liberty Business Park areas.
- Future improvements to the existing system to allow conveyance of future flows through existing facilities (improvements in addition to the “Near-Term improvements”).
- Sewer system expansion improvements, which consist of new sewer lines to extend service to currently unsewered areas within the City’s growth boundaries.

Extension of sewer services to currently unsewered areas will likely follow the pattern of actual development within each growth area, which is currently unknown. Therefore the proposed expansion plan will be used as a guide to developing sewer system expansion plans to future areas as they are developed. In order to better define the location and phasing of improvements, the City can conduct a development forecast for the next ten to twenty years or more.

A major component of the Near-Term Improvements identified in Section 7 is the construction of a new City Main Pump Station or Lift Station (where no pressure force main is included) and new pipeline to the WWTP. Two alternatives are presented in the proposed sewer system expansion plan for these facilities:

1. Construction of the new facilities as a sewer lift station with ultimate capacity of 6.2 million gallons per day to lift into a new 33-inch diameter gravity sewer to the WWTP, or

2. Construction of the new facilities as a sewage pumping station with ultimate capacity of 6.2 million gallons per day to pump into a new 18-inch diameter force main that pumps to the WWTP.

The proposed new City Main Lift Station is located approximately 1,400 feet due west of the current location of the McHenry Lift Station, south of the proposed SR 120 bypass right of way. This lift station would replace capacity in the McHenry Lift Station. An interim project of increasing reliable pumping capacity in the McHenry Lift Station is recommended if the new City Main Lift Station is not scheduled within the next eighteen months.

Three new sewer lift or pump stations are proposed and include a new 2.6 million gallon per day ultimate capacity sewer pump station to serve the Liberty Business Park area and two approximately 60,000 gallon per day lift stations to serve sewer systems on the periphery of the 2035 growth boundary.

2.3 WWTP EXPANSION ALTERNATIVES

Review of expansion alternatives for the WWTP started with review of the existing treatment and disposal facilities and compliance with Waste Discharge Requirements (WDRs) issued by the California Regional Water Quality Control Board (hereafter Regional Board). The Regional Board establishes the wastewater treatment standards necessary for case-specific wastewater disposal methods in order to protect public health and the environment to the level and certainty deemed appropriate by law, regulation, and policy.

The City’s current method of disposing of its treated wastewater (termed “effluent”) is by infiltration into the underlying groundwater resource. This disposal method is called “land application” and is preferred by the Regional Board over direct discharge of effluent to the Stanislaus River. Effluent reuse or reclamation (such as by irrigation of crops or landscaping) is the effluent disposal method most favored by the Regional Board, however this disposal method does not appear to be a feasible for the City at the present time. Therefore, the wastewater facilities planning process commenced by determining if there is anything wrong with the current wastewater treatment and disposal operation such that it can be determined if the current operation 1) needs improvement, 2) could serve as a basis for increasing sewer service capacity, or 3) should not be the basis for increasing sewer service capacity.

Two critical questions to be answered with respect to determining if the current disposal method (and therefore the necessary effluent treatment requirements to comply with current laws, regulations, and policies) can continue as the basis for facilities expansion are:

- Is land application of effluent causing impairment of groundwater i.e., causing an exceedance of an applicable water quality objective (WQO)?
- Is land application of effluent causing groundwater degradation, i.e., causing a deterioration of groundwater quality, but not pollution as defined above?

In order to answer these questions, an assessment of the WWTP site hydrogeology and groundwater monitoring was performed utilizing existing information to determine if impairment or degradation of groundwater appears to be occurring. For contaminants causing or potentially causing pollution or degradation, possible groundwater impact mitigation measures are developed. These mitigation measures include changes in treatment process and/or effluent disposal method.

The existing information suggests that groundwater beneath the WWTP may be impaired with respect to the salinity constituents chloride and sodium. Table 2-2 summarizes the WWTP Groundwater Impact Assessment based on current monitoring data contained in Appendix D. However, it is not clear to what extent the municipal facilities may be influencing underlying groundwater quality as compared to mixing with Stanislaus River water and effluent discharges from the industrial wastewater treatment and disposal facilities.

Table 2-2
WWTP Groundwater Impact Assessment

Parameter	WQO	Range of Median Concentrations from Monitoring Wells		Apparent Potential Impact of the Municipal WWTP Ponds on Groundwater Quality
		Background Wells	Downgradient Wells[a]	
TDS, mg/L	450	570 – 640	484 - 630	No apparent impact
EC, µS/cm	700	769 – 1,015	691 – 1,024	No apparent impact
Nitrate as N, mg/L	10	2.5 – 13.4	1.2 – 6.2	Groundwater improved
Chloride, mg/L	106	21 – 59	62 - 71	Groundwater potentially degraded
Sodium, mg/L	69	24 – 77	57 - 91	Groundwater potentially impaired

[a] Based on Monitoring Wells No. 1, 5, and 8 , immediately adjacent to municipal treatment and disposal ponds.

A comparison of the ranges of median concentrations from the background monitoring wells and downgradient monitoring wells directly downgradient of the municipal treatment and disposal activities suggest that the municipal and industrial WWTP ponds:

- Have no apparent impact on overall groundwater salinity as measured by TDS and EC.
- May reduce groundwater nitrate concentrations.
- Potentially cause degradation with regards to groundwater chloride.
- Potentially cause groundwater impacts with regards to groundwater sodium.

If this preliminary assessment of groundwater impacts are found to be the result of the municipal wastewater facilities, then mitigation measures for potential impacts to groundwater are proposed and include:

- Switch to the SSJID water supply to the maximum extent feasible to reduce water supply (and therefore effluent) sodium and chloride concentrations to the extent feasible.
- Monitor the residential, commercial, and industrial wastewater flows to determine if there is an elevated source of sodium (or chloride) in the community that could be reduced by feasible pretreatment, source reduction, or other means.
- Continue with public education to try to reduce wastewater sodium and chloride concentrations to the degree feasible.
- Propose that the point of compliance with groundwater limitations be MW-5 and MW-6 in light of the foregoing measures.

Two alternatives for wastewater treatment were evaluated, given the potential for groundwater impacts and uncertainty in the groundwater impact assessment. It was assumed that the current method of effluent disposal would remain viable for future flows, with likely future measures to decrease groundwater impacts being associated with changes in source water quality and wastewater treatment.

2.3.1 Alternative 1 – Secondary Pond Treatment and Land Disposal Capacity Upgrade

The first alternative reviewed for expanding treatment was continuing with the general plan as outlined in the 1990 Master Plan. This alternative consists of expanding pond treatment as it is currently employed, likely with lining of the treatment ponds in order to mitigate the potential for groundwater impacts from these facilities. A phased approach to expanding the pond treatment system was proposed with concurrent expansion of the disposal ponds on land to be acquired by the City.

It is recommended that the City evaluate groundwater quality beneath and in the vicinity of the WWTP prior to permitting of the first phase expansion. Given the uncertainty in how the City may be regulated in the future and in particular, potential impacts being found to exist, it was recommended that the City continue to maximize the usefulness of existing facilities to the extent practicable. Some risk does exist that this treatment method may not feasibly meet future permit requirements, therefore alternative 2 is a probable backup to the current method.

2.3.2 ALTERNATIVE 2 – SECONDARY TREATMENT WITH ACTIVATED SLUDGE AND LAND DISPOSAL

The alternative of converting the treatment system to the more mechanically intensive activated sludge process was evaluated with three phases provided in Section 8. The first phase of the Activated Sludge alternative would include replacing the existing treatment capacity achieved by the aerated treatment ponds.

Activated sludge treatment was considered the best apparent alternative for the below reasons; however, conversion to this treatment method cannot occur immediately nor is the existing water quality evidence strong enough to warrant the much higher capital and operation and maintenance cost:

1. Activated sludge treatment provide the Best Practicable Treatment and Control (BPTC) for nitrogen, e.g., total nitrogen can feasibly and economically be reduced to below the drinking water maximum contaminant level;
2. Activated sludge in concrete or lined basins provides BPTC for potential pathogen contamination of groundwater;
3. Activated sludge in concrete or lined basins and high effluent quality provides BPTC for potential iron, manganese, and arsenic mobilization through disposal.

During permitting of the first phase improvements for the WWTP, many of the questions regarding potential impacts to groundwater and the need to change from the current pond treatment system to an activated sludge type of system will be reviewed. Although the existing and potential impacts to water quality may not individually lead the City to converting treatment to activated sludge, two or more of these impacts may require that conversion.

If the City has any concerns regarding potential risk with keeping the current pond treatment process, then conversion to an activated sludge treatment process should be considered.

2.4 RECOMMENDED PROJECTS

The recommended expansion and improvement projects are described in Section 7.3 for the Sewer System and Section 8.5 for the wastewater treatment and disposal system. These recommended projects are summarized in Table 2-3 with planning level estimated facilities cost.

Table 2-3
Recommended Improvement Projects Summary

Improvement	When Needed	Estimated Cost
Sewer System Improvements		
Near-Term Improvements	Prior to connecting additional new development	$6,710,000
Improvements for Heritage Park Development	During Heritage Park Phase 1	$976,000
Improvements for Liberty Business Park	During construction of first phase of development	$3,990,000
Improvements for Future Developments	As needed per Table 7-1	$3,710,000
Sewer System Expansion Improvements	As needed per Table 7-1	$8,510,000
	Subtotal	**$23,896,000**
Treatment and Disposal System Improvements		
Phase I, IPS/Headworks, Treatment, and Disposal[a]	When Influent Flow is 1.0 MGD	$8,890,000
Phase II, IPS/Headworks, Treatment, and Disposal	Prior to 1.5 MGD	$6,250,000
Phase III, IPS/Headworks, Treatment, and Disposal	Prior to 2.25 MGD	$2,490,000
	Subtotal	**$17,630,000**
Total Recommended Improvement Projects Cost		**$41,526,000**

[a] – Permitting for Phase I to start at 0.72 MGD.

2.5 IMPLEMENTATION

Implementation of the recommended improvements will include designing, financing, and constructing facilities in anticipation of new development. As each growth boundary is annexed, the City will begin implementation of the recommended sewer improvements in response to the actual rate and location of development. While sewer improvements need to be constructed to provide service to the ultimate flows anticipated in and beyond a development (with facilities phasing available on a limited basis), wastewater treatment and disposal facilities can be constructed as actual influent flows increase. Below phasing considerations are summarized for each area of improvement. If further definition of project phasing is desired, the City can undertake a development projection that projects the probable location and timing of development within the current sewer service area and future growth boundaries (taking into consideration the City's Growth Management Ordinance for residential developments).

2.5.1 PHASING

Table 7-1 lists the general phasing of the sewer system expansion to serve current and future development. These facilities are broken down in Table 7-1 into five areas or phases of system expansion:

1. "Near-Term Improvements" that are necessary to construct reliability in the existing system and to allow additional sewer flows within the current sewer service area. The City should consider scheduling these improvements as soon as possible, based on financing, regulatory, and environmental constraints.

2. Improvements for the Heritage Park Development will be necessary for the first phase of this project. These facilities also provide system benefit in the vicinity of the Heritage Park Development and beyond. The Heritage Park Development would also contribute to the Near-Term Improvements to convey flows to the City wastewater treatment and disposal facilities.

3. Improvements for the Liberty Business Park Development will be necessary to collect wastewater from the first phase of this development and to convey those flows to the City's wastewater treatment and disposal facilities. As with the Heritage Park Development, the Liberty Business Park Development would contribute to the construction of the Near-Term Improvements.

4. Improvements for Future Developments consist of expansion of the existing facilities primarily to serve new flows within the current sewer service area. These improvements will be based on the actual rate of infill development and on conveying flows from future developments through the City's existing facilities.

5. Sewer System Expansion Improvements are necessary to extend the City sewer system to serve currently unsewered land within the City's growth boundaries (excluding the first phase projects required for the Liberty Business Park and Heritage Park developments). The location and timing of new development within these currently unsewered areas is unknown, therefore as each project makes application to the City, it is recommended that a sewer facilities study be prepared based on the project's specific location and phasing, including an evaluation of required sewer facilities to extend service to the development, facilities needs for any flows to be conveyed through the development, and an evaluation of the projects impacts to the City's existing facilities.

For the WWTP expansion, phases will be constructed as actual influent flows increase. Based on the City's anticipated population, the Phase I expansion is not likely to be necessary until after the year 2015, these facilities should be expanded ahead of the actual increase in wastewater flows to the plant. The Phase I project should be initiated prior to the average dry weather flow to the plant reaching 0.9 MGD, which could occur prior to 2015. Re-permitting of the WWTP, as required by the WDRs, is best done in conjunction with the Phase I expansion and should be initiated prior to the influent flow exceeding an average of 720,000 gallons per day during dry periods. This could occur within a year or two based on recent influent flow records.

2.5.2 FUNDING

Project construction costs, including engineering and administration, are summarized in Table 2-3. The total cost of the recommended sewer system expansion and improvement program is $23.9 million. The total cost of the recommended wastewater treatment and disposal improvement plan is $17.6 million. However, total project costs should include the cost of financing if some form of debt is incurred.

The City's current approach for expanding sewer systems for new development is to require that such new development extend the sewer system and to "oversize" the facility to accommodate future flows within or beyond the development. In addition to this mechanism for constructing facilities, the City will collect connection fees and likely need to finance certain project components through long-term debt. The cash pay-as-you go financing basis, whereby connection fees collected by the City are used to construct facilities as they are needed, is not likely to provide sufficient funding for the first improvements.

In addition to financing project costs through the above two methods, the City could consider a number of different long-term debt financing alternatives, including:

- State Revolving Fund Loans,
- State Infrastructure Bank Loans,
- Bonds or Assessment District financing,
- Federal infrastructure financing such as USDA etc.,
- Commercial bank loans.

The cost of the particular financing, including any critical cash flow analysis, must be included in the final calculation of each component of the City's sewer connection fees. Estimated financing costs have been calculated for the master plan facilities with the Near-Term Improvements, Liberty Business Park Improvements, and the Phase I WWTP improvements costs discussed. These costs are summarized in Section 9 and are reflected in the potential range of impact to fees.

In order to better define the likely impact of debt financing on the overall project cost, the City should undertake a financing analysis. This analysis would include information from the development projection. With incorporating project schedule requirements and a cash-flow analysis, the need and timing for debt financing can be estimated. The additional expense that results from this debt financing would then be included in the calculation of the connection fee.

2.5.3 PHASE I RECOMMENDED EXPANSION PROGRAM

In order to accommodate projected development within the current City sewer service area and provide capacity for anticipated growth within the City's growth boundaries, sewer improvements must be constructed. This Master Plan projects wastewater flows within the City's growth boundaries, however the actual rate of development will largely be limited by the GMO. Within a twenty year planning horizon, approximately 1,500 future residential units could connect to the City sewer system with additional future commercial and industrial connections also being developed. In order to accommodate currently planned development, the Phase I recommended expansion program summarized in Table 3-4 would be necessary. This expansion program will allow the City to evaluate potential changes in effluent disposal policies as well as evaluate groundwater quality underlying the existing effluent disposal site. If changes in the required treatment and disposal method are warranted, then a revision to this Master Plan would be initiated.

Table 2-4
Phase I Recommended Expansion Program

Improvement	Approximate Expanded Capacity (gal/d)[a]	Estimated Cost[b]
Gravity sewer 18-inch minimum diameter 1,400-foot length along future HWY 120 bypass east of McHenry Lift Station	2,100,000	$495,000
Construct Phase I of City Main Lift Station to replace existing McHenry Lift Station (Phase I at 3.1 MGD)	330,000	$1,110,000
Construct 9,000-foot 33-inch minimum diameter gravity sewer from new City Main Lift Station to the Escalon WWTP	2,100,000	$5,100,000
Improvements for Heritage Park Development	2,100,000	$976,000
Improvements for Liberty Business Park	2,100,000	$3,990,000
Phase I, IPS/Headworks, Treatment, and Disposal[a]	800,000	$8,890,000
Total		$20,561,000

[a] Compared with ultimate Master Plan additional capacity requirement of additional 2.1 Mgal/d average day flow.

[b] Including contingency, engineering, and administration.

2.5.4 REGULATORY REQUIREMENTS

Regulatory requirements now extend to the City's sewer system as well as the wastewater treatment and disposal facilities. With the adoption of the statewide Waste Discharge Requirements (WDR) for collection systems, the City will be required to perform a detailed analysis of the sewer system, including a capacity analysis that builds on the analysis prepared for this master plan (e.g., sewer system hydraulic model with flow monitoring and calibration).

The City's municipal wastewater treatment and disposal facility is currently regulated by the Regional Board through WDRs. In order to expand treatment and disposal services to ADWF greater than 0.9 MGD, the City will be required to submit a Report of Waste Discharge, including a description of the proposed expansion and an "Antidegradation Analysis" of the process to determine its consistency with State Board Resolution 68-16 (Antidegradation Policy). If the Antidegradation Analysis indicates that an alternative disposal method or different treatment method is warranted, then additional facilities design and construction will be necessary and revision to this master plan would be appropriate.

2.6 POTENTIAL RANGE OF IMPACT TO FEES

Depending on the means of financing capital improvements, the average cost per equivalent single-family dwelling unit (at an average wastewater flow per new EDU of 250 gallons per unit) for the Master Plan facilities is approximately $4,945 to $12,060 per EDU as outlined in Table 2-5. If only the Phase I recommended expansion program is considered as the basis for connection fees, then the potential impact of the cost of these improvements is approximately $4,900 to $11,900. These ranges in fees are based on the assumptions regarding project financing described in Section 9. It is unlikely that all Master Plan facilities will require long-term debt financing. However the Phase I recommended expansion program, including the Near-Term Improvements, Liberty Business Area Improvements, and the Phase I WWTP improvements, is likely to require financing similar to the assumptions in Section 9. In this case, it is appropriate to include the cost of financing in the calculation of the connection fee for these improvements

In considering connection fees, the City should evaluate the means of financing of future projects, and include the cost of such financing in the fee calculation. The potential range of impact to connection fees discussed herein should be considered preliminary with actual cost of long-term debt incorporated into any updated fees once the conditions of the debt are known. Also, the value of existing available capacity in the WWTP should be included in the connection fee.

2.6.1 REVIEW AND INDEXING OF CONNECTION FEES

Sewer connection fees should be reviewed periodically primarily for two purposes:

1. To account for changes in project costs and project financing, and

2. To adjust project costs for inflation and increases in construction costs.

Therefore, it is recommended that the City review the Sewer Master Plan periodically and review project costs at the following milestones:

- Upon completion of a development projection and financing plan.

- Upon obtaining financing for the Near-Term Improvements, Liberty Business Park Improvements, and WWTP Phase I improvements;

- During design and financing of the WWTP Phase II Improvements; and
- During design and financing of the major facilities projects listed in Table 7-1 for Future Development and System Expansion.

It is also recommended that the City provide for annual indexing of the sewer connection fee on an annual basis according to the change in the Engineering News Record Construction Cost Index (ENR CCI). Where the cost of long-term debt for a facility is incorporated into the connection fee, it may not be appropriate to increase that cost on an annual basis as long as the financing has been secured and the facilities have been constructed. The basis for the costs contained in this Master Plan Update are an ENR CCI of 7721 for the mid 2006 level.

Table 2-5
Potential Range of Impact To Sewer Connection Fees for Master Plan Facilities

	Estimated Cost[a]	Estimated Cost w/Financing[b]
Sewer System Improvements	$23,896,000	$ 58,306,000
Expansion Average Day Capacity (gallons/day)	2,100,000	2,100,000
Sewer Cost per Gallon Average Day Capacity	$11.38	$ 27.76
Gallons per Future EDU (average gpd)	250	250
Sewer Expansion Cost per Future EDU	$ 2,845	$ 6,940
Treatment and Disposal (T&D) Improvements	$17,630,000	$ 43,017,000
Expansion Average Day Capacity (gallons/day)	2,100,000	2,100,000
T & D Cost per Gallon Average Day Capacity	$8.39	$20.48
Gallons per Future EDU	250	250
T&D Cost per Future EDU	$2,100	$ 5,120
Total Cost per Future EDU	$4,945	$ 12,060

[a] – Not including cost for financing of system expansion.
[b] – Includes estimated financing cost as calculated in example form in table 9-2.

Section 3

Planning Criteria

Section 3

Planning Criteria

This section discusses the planning criteria contained in the City's recently adopted General Plan Update and how that criteria was used to estimate future wastewater flows and plan future sewer facilities and wastewater treatment and disposal facilities. Additional information was generated by ECO:LOGIC based on the San Joaquin County Assessors Office databases and from other documents prepared for the City of Escalon, including the EIR prepared for the City of Escalon General Plan 2005-2035 and the Initial Study prepared for the Liberty Business Park.

Land use projections were based primarily on the County Assessors Office database with the 2035 totals compared to the Escalon General Plan. Future land uses to be provided sewer service were determined using that information within the 2015, 2025, and 2035 boundaries as defined in the Land Use and Urban Boundaries map.

3.1 EXISTING SERVICE AREA CHARACTERISTICS

The sewer service area (area currently sewered or to be sewered through main line extensions) within the City limits encompass approximately 1,130 acres of land with the majority of the land use in the low to high density residential category, approximately 42 percent in these categories. The remaining areas are divided among industrial, commercial, park and open space, and public facilities, with industrial representing the second highest land use at approximately 34 percent of the total.

Wastewater originating in the current service area is primarily of domestic origins (including residential, commercial, and public facilities), as the major industrial areas discharge to a separate industrial sewer system. Industrial dischargers to the "domestic" sewer system are of the "dry" industry type and exclude such industries as food processing.

Review of the industrial sewer system and industrial wastewater treatment and disposal facilities is not within the scope of this Sewer Master Plan Update.

Table 3-1 summarizes the current and future proposed incremental increase in developable land use until year 2035 as depicted in the City's current General Plan Map. Current sewer service area land uses are based on information regarding the planned 2035 sewer service area and the estimated incremental increase in the service area through the three planned growth boundaries and exclude right-of-way and agricultural uses within the current City limits.

Table 3-1
General Plan Land Use By Planning Boundary

General Plan Land Use Category	Current[a]	Acres Added per Growth Boundary			Total
		2015	2025	2035	
Commercial	175	22	23	25	245
Industrial	370	102	0	90	562
Low Density Residential	413	254	242	317	1226
Medium Density Residential	29	10	0	6	45
High Density Residential	29	17	4	0	50
Park/Open Space	42	10	52	33	137
Public Facilities	72	0	0	0	72
Total	**1,130**	**415**	**321**	**471**	**2,337**

[a] Approximate current sewer service area, acres.

Source: = EIR for City of Escalon General Plan 2005-2035, Quad Knopf, April 2005, and ECO:LOGIC Engineering 2006 Land Use Survey.

Of the approximately 1,130 acres within the current sewer service area, approximately 70% (780 acres) is currently developed, therefore a roughly 30% increase in sewer flows could result from build out within the current City limits.

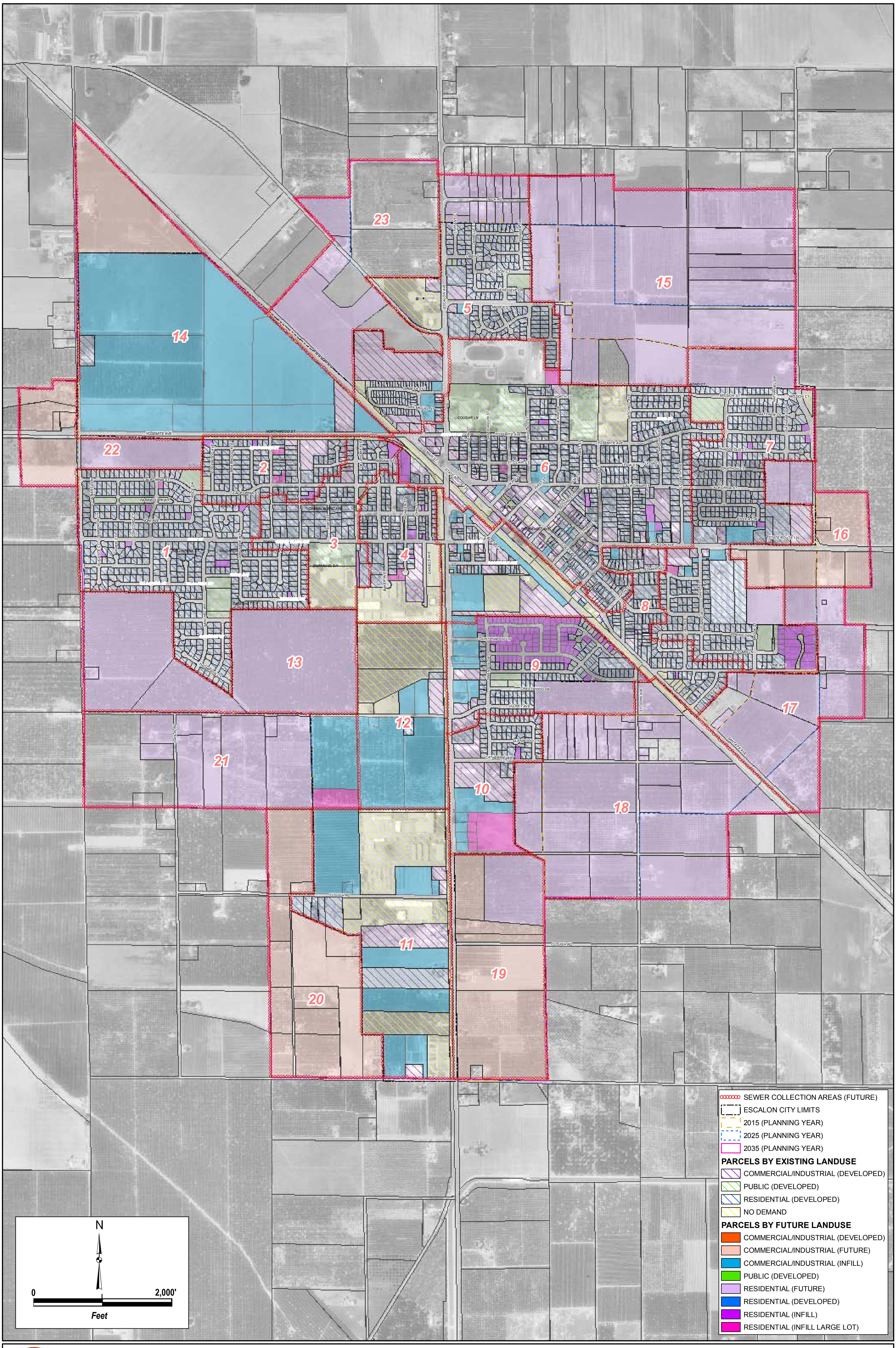
3.2 CITY GROWTH PROJECTIONS

Projections of sewer service areas and wastewater flow rates, including the magnitude and location, are necessary to plan future sewer system improvements. Sewer flows are developed in Section 4 using approved land uses included in the City's General Plan Update Background Report (February 2004) and the Initial Study prepared for the Liberty Business Park area. Projected sewer flows for various levels of development were considered from current conditions through the ultimate build out condition in 2035 as defined in the General Plan.

The planning boundaries defined in the General Plan include:

- 2015 General Plan Growth Boundary
- 2025 General Plan Growth Boundary
- 2035 General Plan Growth Boundary

Although each planning boundary is associated with an apparent time line, the City generally plans to develop the area encompassed by each boundary prior to expanding into the subsequent boundary. As an example the land within the 2015 boundary could develop sooner or later than 2015, and will depend on a number of factors out of the City's control including the economy, interest by the development community to construct in Escalon, public support/opposition of new projects, etc. The City planning staff will monitor growth and determine the appropriate point to expand the City boundary for development based on actual development.



U:project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/]Figure_03_01_City Land Use Designations.ai 8/15/06 cms

Figure 3-1
City Land Use Designations and Growth Boundaries

The City's General Plan estimates that the total acreage within the 2035 growth boundary will encompass approximately 2,020 acres. Figure 3-1 represents the current City boundary, its projected 2035 boundary, and the proposed wastewater drainage basins at 2035 conditions.

The objective of this Sewer Master Plan Update is to develop the basis for sewer and wastewater treatment and disposal requirements and an infrastructure phasing plan with planning level cost for extending wastewater service to the remaining undeveloped and unsewered land within the 2035 growth boundary. A unique aspect of planning for extending sewer service within the proposed City of Escalon growth boundaries is the existence of the City's Growth Management Ordinance (GMO). The current GMO limits the number of new residential construction permits to 75 per year, therefore residential growth will not exceed this rate unless the City were to change this ordinance. The result of this fact is evident when the projected population is compared to the General Plan proposed additional acres for residential, commercial, and industrial land uses. Table 3-2 compares the proposed General Plan increase in land use for residential, commercial, and industrial land uses with the calculated developed land uses and is taken from the City's General Plan Update background Report.

Table 3-2
2005-2025 Population and Land Needs Comparison With Planned Sewer Service Area

Year	Projected Population[a]	Residential (Total Acres)		Commercial (Total Acres)		Industrial (Total Acres)	
		Needed[b]	Proposed[d]	Needed[c]	Proposed[d]	Needed[c]	Proposed[d]
2005	7,150	474	n/a	71	n/a	88	n/a
2015	9,550	627	752	78	197	117	472
2025	11,950	779	998	85	220	147	472

[a] Projection based on 2000 U.S. Census, Department of Finance.

[b] Growth needs based on current GMO allocation formula allocation added to existing per Table 3-1.

[c] Includes additional land demand per Table 1.1-4, *General Plan Update Background Report*, Quad Knopf, February 2004 added to existing per Table 3-1.

[d] Cumulative based on General Plan growth boundaries.

Source: Adapted from Quad Knopf, Inc., February 2004.

3.3 FACILITIES PLANNING CRITERIA

As each area develops and sewer service is extended to serve each new growth area, sewer collection facilities need to be constructed to serve the anticipated maximum land use in the area. In some instances facilities can be phased, such as main trunk lines where parallel lines can be planned for future uses or pumping facilities where mechanical and electrical improvements can be made to serve initial phases while the overall structure is designed to accommodate future flows.

In the case of the sewer system, this Sewer Master Plan Update has estimated peak flows based on complete infill within each growth boundary and facilities have been sized to accommodate those anticipated peak flows. Where facilities phasing is feasible, such phasing has been briefly described. Additional phasing may be feasible and can be addressed on a project-by-project basis. The City currently does not know the location or timing of development within the various growth boundaries, with the exception of limited information on the Liberty Business Park and the proposed Heritage Park Subdivision. Since the location and timing of development within each area is not know, master plan facilities have been sized to accommodate ultimate anticipated wastewater flows from each sewer drainage area.

With the City's current GMO limits on new residential units to no more than 75 homes per year, by the year 2015 an estimated 675 additional homes could be constructed, and by the year 2025 an estimated 1,425 additional homes could be constructed. The City population projections contained in the City's General Plan Update Background Report are consistent with the GMO, however the areas available for development within each growth boundary is about fifty percent larger than the area than would likely be developed within the ten year timeframe designations given to the boundaries.

The current growth ordinance does not limit the development of industrial or commercial uses, and therefore it is possible that future commercial and industrial development within the City of Escalon could exceed the residential growth rate.

Regardless of when individual residential connections are made, facilities must be planned and constructed in anticipation of new development being constructed.

Wastewater treatment and disposal is needed in anticipation of wastewater flows to the treatment facilities. Unlike sewer systems, where the most economical approach is to construct underground facilities sized to accommodate ultimate flows, wastewater treatment and disposal facilities are typically expanded in phases ahead of the actual pace of growth in the City. This phased approach to wastewater treatment and disposal expansion also allows the City to respond to changes in regulations and policies.

Section 4

Projected Flows and Loads

Section 4

Projected Flows and Loads

4.1 WASTEWATER INFLUENT FLOWS

This section discussed current wastewater flows consisting of influent flow to the City's domestic Wastewater Treatment Plant (WWTP) with an assessment of estimated wastewater flows per unit. Future wastewater flows are projected based on future land uses as described in Section 3 based on the flow generation factors developed in this section.

4.1.1 EXISTING INFLUENT FLOWS AND I/I ANALYSIS

Daily influent records for 2005 (a wetter than average year) have been evaluated in this Master Plan Update in order to estimate average wastewater flows and peak WWTP influent flows. Available historic influent flow data was available for the WWTP for 2001, 2002, and 2005. During 2005, the average dry weather flow (ADWF) was approximately 0.60 MGD (which occurred during the months of May through August). Current, 2005, peak daily influent flows can be as high as 1.2 MGD. Peak month influent flows during 2005 were about 0.70 MGD and occurred during October.

Influent flow records for 2001 indicate that the ADWF has decreased by approximately 0.14 MGD, however approximately 30% of this difference is due to influent flows during July 2001 being significantly greater than typical dry weather flows indicated by other summer months in 2001 and 2005. Peak month influent flows in 2001 were recorded at 0.85 MGD during October. ADWF in 2002 was approximately 0.74 MGD with the peak month influent flow recorded at 0.78 MGD during October. The apparent decrease in influent wastewater flows is not known, but could be attributable to influent flow meter calibration and flow measurement errors, or could be the combined result of changes in collection system O&M and general reductions in wastewater generation due to conservation efforts. However, during that same period, City records indicate that water production rose from 509 Mgal in 2001 to 529 Mgal in 2002 and 557 Mgal in 2005.

The City has not yet initiated extensive sewer inspection of the system; therefore, the physical condition of the existing collection system is not well known. However, comparison of the average day monthly wastewater inflow into the WWTP with monthly precipitation for the year 2005 (as measured at the City of Modesto) shows that the system does not seem to experience excessive inflow and infiltration (I/I), e.g., there is little apparent response to influent flow during periods of high precipitation. A graphical comparison of the two parameters is shown in Figure 4-1. Likewise, it is known that groundwater in the vicinity of Escalon tends to be at a depth greater than the existing sewer facilities; therefore there is little potential for groundwater infiltration into the system.

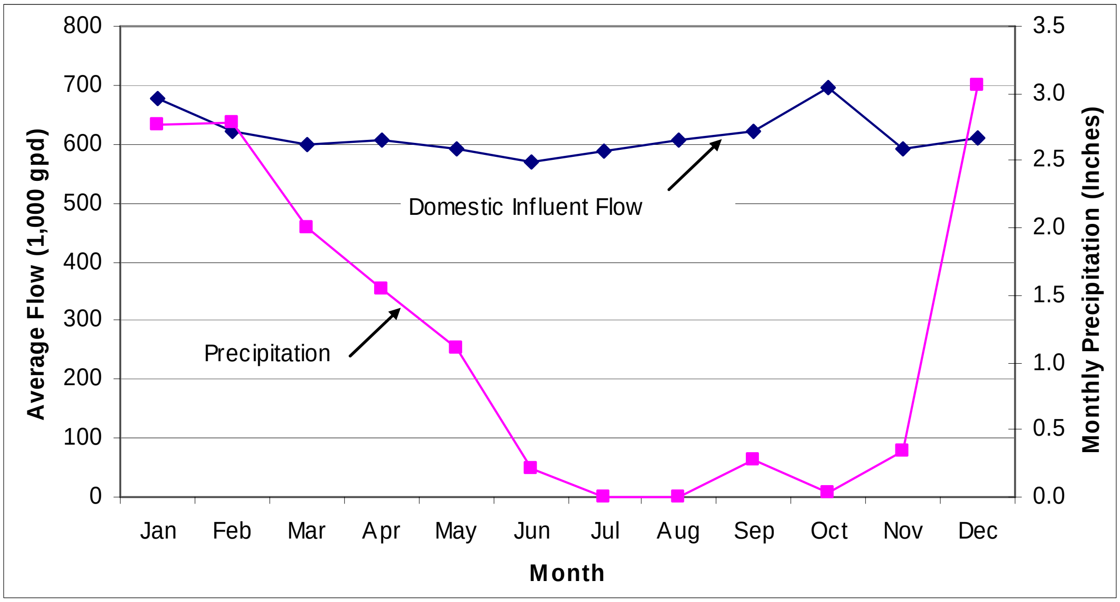


Figure 4-1
Comparison of 2005 Monthly Precipitation and its effect on Average Day Monthly Flows at the City of Escalon

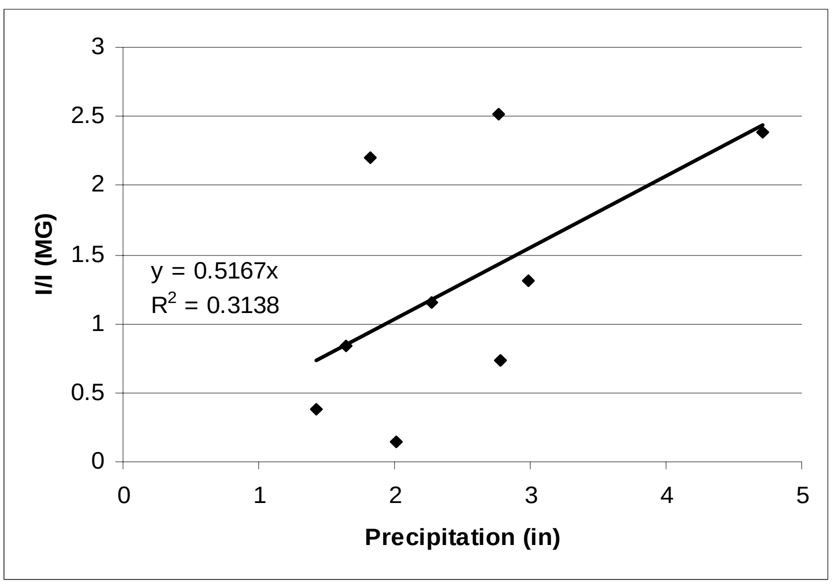


Figure 4-2
Corrected Comparison of 2001 and 2005 Infiltration and Inflow as a Factor of Monthly Precipitation for the City of Escalon WWTP

The actual monthly flows for 2001 and 2005 were compared to expected monthly flows based on ADWF for each respective year to estimate the I/I contribution. These values were plotted against monthly precipitation and no clear relationship was found using all data. Removing summer and fall months, negative values in the winter provides a slightly stronger relationship that corresponds to 3 to 5 percent I/I in average and 1 in 100 year precipitation years (Figure 4-2).

4.1.2 DEVELOPMENT OF UNIT WASTEWATER FLOW FACTORS

Projected wastewater flows have been calculated by assigning unit wastewater flow factors to each land use designation within the existing City limits and calibrating the flow model by comparing the results obtained for calculated influent flows of existing developed lands with the measured flows at the WWTP. Unit flow factors have then been adjusted for each land use until degree of correlation has been achieved and the flow model can reasonably accurately estimate the current conditions and hence will yield a more accurate projection of the future flows as long as assumptions made in the development of the model hold true in the future.

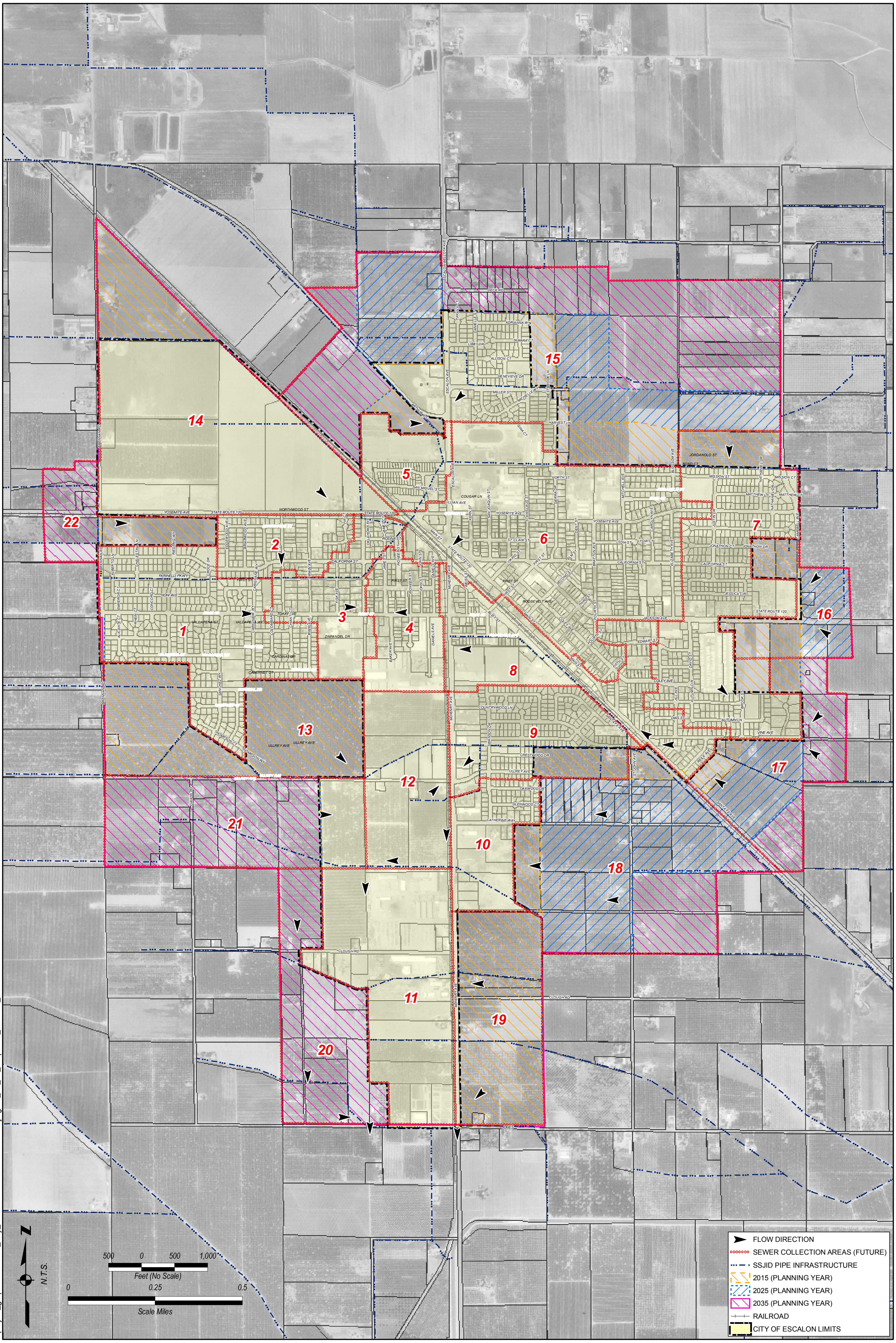
Unit flow factors used in the analysis were initially based on the City of Escalon's Sanitary Sewer Design Data Improvement Standard No. S2. Table 4-1 summarizes the unit flow factors according to land use designation based on Improvement Standard No. S2.

Table 4-1
Escalon Wastewater Standards Design Criteria

Land Use Designation	Description	Flow Factor Units	Unit Flow Factor
Low Density Residential	Single Family Home	gpd/parcel	300
Medium Density Residential	Multi-family unit	gpd/parcel	2,600
High Density Residential	Large Apartment	gpd/parcel	4,100
Public Facility[a]	School	NA	Varies
Commercial	Commercial	gpd/acre	2,500
Industrial	Light Industrial, Industrial	gpd/acre	3,500

[a] Flows for public facilities (schools) were assigned flow factors based on the number of student capacity of each school and typical wastewater flows per student according Metcalf & Eddy "Wastewater Engineering", Third Edition, Table 2-11. School population information was obtained from the California Department of Education.

Unit flow factors were assigned based on land use designation as obtained from the San Joaquin County Parcel Assessor data. For large undeveloped areas within the current City limits and within the City growth boundaries, unit flow factors were assigned to net acres rather than gross acres for each land use. Net acreage was use instead of gross acreage because large, undeveloped parcels must allow for the construction of roads and right-of-way from which there will be no wastewater flows generated. The calculated net acreage used was about 93 and 85 percent of the gross acreage of industrial/commercial and residential land use designation respectively.



U:project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/Figure_04_03_Proposed_Buildout_New.ai 10/2/06 cms

Figure 4-3
Proposed Build-Out Wastewater Collection System Drainage Bains

Based on the Table 4-1 unit flow factors, the average flow contributed by each zoning classification for currently developed land connected to the City sewer system is as shown in Table 4-2.

Table 4-2
Calculated Current Wastewater Flow Utilizing City Standard S2

General Plan Land Use Description		Average Day Flow (MGD)
Residential	LD	630,000
	MD	70,000
	HD	50,000
Commercial		220,000
Industrial		150,000
Light Industrial		10,000
Public Facilities		70,000
Total		**1,200,000**

As shown in Table 4-2, the calculated average day flow for current development is 1.20 MGD, when using the City's existing standards. The measured average day flow at the WWTP for the years 2001, 2002, and 2005 were 0.72, 0.74, and 0.62 MGD (ADWF of 0.60 MGD for May through August) respectively, indicating that the measured flows are considerably lower than the calculated flows using the City's design standards. Such a difference in the measured flows and a particular design standard are not uncommon, however a difference of almost a factor of two is significant.

Due to the significant difference in flows calculated using the current City design standards and measured flows, an adjustment to the unit flow factors was performed in order to more accurately model the actual wastewater flow generation at the City of Escalon and in order to more accurately size future infrastructure required. The adjustment was performed by first estimating the per capita wastewater flow generated. Currently the population in Escalon is approximately 7,200 people. Given an annual average daily wastewater flow to the wastewater treatment plant for 2005 of 620,000 gallons per day, the approximate wastewater flow per capita per day (gpcpd) would be 86 gpcpd, if no industries or commercial dischargers were present in the City.

Given that both industrial and commercial dischargers do, in fact, contribute to the total wastewater flow it is inferred that the per capita flow must be less than 86 gpcpd. After some iterations of flow factor proportioning to residential, commercial and industrial dischargers, the final per capita flow was assumed to be 70 gpcpd, which is also the reported per capita wastewater for a typical home, as reported by Metcalf and Eddy Wastewater Engineering. Considering that the average residential occupancy in Escalon is 3 persons per home, the average day flow from a single family residential unit is estimated at 210 gallons per day (gpd).

The adjustment to the flow generated from the medium and high density residential units was done by first comparing the average day wastewater flow with the total water demand within the City during the 2005. The percentage of the average day wastewater generated to the average day water demand in the City of Escalon was approximately 40 percent. Therefore, the final adjustment for wastewater generated from medium and high density residential units was also estimated to be 40 percent of the water consumed.

The City of Escalon currently collects wet industrial flow in a separate collection system. It is anticipated that this practice will continue for flows from future wet industries and that future industrial discharges will be from “dry industries”. Therefore, it is assumed that expansion of the collection system should consider that only dry industries (such us warehouses) will be allowed to connect to the domestic collection system. Flows from dry industries can be expected to be similar to those of commercial dischargers; therefore, for estimating the flow factors for industrial and commercial dischargers, a common factor of 900 gpd per acre was utilized. This value was obtained by gradually adjusting the flow factor in order to make up the difference between the measured average day flow at the WWTP during 2005 and the calculated flow obtained after assigning modified flow factors to the residential land use sectors.

Table 4-3 summarizes the adjusted wastewater factors developed for modeling the current flow conditions at Escalon. Table 4-4 shows the calculated wastewater flows for the current level of development within the City sewer service area based on the adjusted wastewater flow factors.

Table 4-3
Revised Current Wastewater Flow Factors

Land Use Designation	Description	Flow Factor Units	Unit Flow Factor
Low Density Residential	Single Family Home	gpd/parcel	210
Medium Density Residential	Multi-family unit	gpd/parcel	860
High Density Residential	Large Apartment	gpd/parcel	4,100
Public Facility[a]	School	NA	Varies
Commercial	Commercial	gpd/acre	900
Industrial	Light Industrial, Industrial	gpd/acre	900

[a] Flows for public facilities (schools) were assigned based on the number of student capacity of each school and typical wastewater flows per student according Metcalf & Eddy "Wastewater Engineering", Third Edition, Table 2-11. School population information was obtained from the California Department of Education.

The unit flow factors shown in Table 4-3 are further supported by the results of calculated total flows shown in Table 4-4 based on 2001 and 2002 average WWTP influent flows of approximately 0.72 to 0.74 MGD. Therefore, the Table 4-3 flow factors appear to be representative of potential wastewater generation rates for current land uses.

Table 4-4
Calculated Current Wastewater Flow Utilizing Adjusted Flow Factors

General Plan Land Use Description		Flows (MGD) Average Day
Residential	LD	430,000
	MD	40,000
	HD	40,000
Commercial		90,000
Industrial		40,000
Light Industrial		10,000
Public Facilities		60,000
Total		**710,000**

The factors in Table 4-3 were additionally modified upward in order to calculate the flows generated from future development within the current City limits and within the City's growth boundaries. This was done in order to account for the possible increase in the per capita flow generated as the City continues to expand and to provide for an additional factor of safety in the sizing of infrastructure to serve future growth given that the actual land usage may change in the future. Table 4-5 summarizes the wastewater flow factors used to project the wastewater generated from undeveloped areas within the current City limits and areas within the 2015, 2025, and 2035 growth boundaries.

Peak flows from each land use designation were developed based on the peak flow graph found in the City of Escalon's Sanitary Sewer Design Data Improvement Standard No. S2. Weighted values were used for each sewer drainage basin depending on the total upstream acreage by land use designation.

Table 4-5
Wastewater Flow Factors for Undeveloped and Future Growth Boundaries

Land Use Designation	Description	Flow Factor Units	Unit Flow Factor
Low Density Residential	Single Family Home	gpd/acre/density	250
Medium Density Residential	Multi-family unit	gpd/acre/density	210
High Density Residential	Large Apartment	gpd/acre/density	210
Public Facility[a]	School	NA	Varies
Commercial	Commercial	gpd/acre	1,000
Industrial	Light Industrial, Industrial	gpd/acre	1,000

[a} Flows for public facilities (schools) were assigned based on the number of student capacity of each school and typical wastewater flows per student according Metcalf & Eddy "Wastewater Engineering", Third Edition, Table 2-11. School population information was obtained from the California Department of Education

It is recommended that the City review the policies contained in this Master Plan Update, including possibly modifying the Improvement Standard No. S2 to reflect actual sewer flows plus a reasonable factor of safety. Any design standard used by the City should be conservative enough to allow for changes that could occur in the future, however such standard also must reasonably reflect actual wastewater flows that could be generated from a particular sewer drainage area. Although not performed as part of this work, the City could undertake a sewer flow study that would include measuring average and peak sewer flows form different sectors of the City (e.g., residential, commercial, and industrial). This data would be used to confirm or modify the current Improvement Standard S2 or the flow factors presented in Table 4-5.

4.1.3 PROJECTED INFLUENT FLOWS

The development of sewer drainage basins is a critical part of the planning and development of future conveyance facilities within the existing and future collection system. Figure 4-3 depicts a breakdown of the proposed future wastewater drainage basins and sub-basins. Assigning appropriate unit development and flow generation factors based on the City of Escalon's General Plan Area results in the projected average dry weather flows presented in Table 4-6 for build out within the year 2035 growth boundary. Projections are made based on net area for each land use category times the unit flow factor times planned density, consistent with the calculation used to estimate wastewater flow for the current condition presented in Table 4-4. Peak flow factors were obtained according to the methodology described above, using Improvement Standard No. S2 peaking curves.

The projected wastewater flows presented in Table 4-6 are contingent on maintaining the current land use designation for each area and assuming 100 percent occupancy ratio in each drainage basin within the 2035 growth boundary. These flow projections are not meant to accurately predict actual flows from each drainage basin by year 2035 but are provided in order to serve as a guide for planning of future City infrastructure considering the potential ultimate wastewater flows from each sewer drainage basin, assuming current development land use plans have not changed. In addition, it is recommended that the flow projections be updated along with future general plan updates and whenever major modifications to the general plan land use occur.

4.2 BIOLOGICAL LOADING AND OTHER CONSTITUENTS OF CONCERN

Influent monitoring for the municipal wastewater consists of monthly grab samples for biochemical oxygen demand (BOD), nitrate, total dissolved solids (TDS), and electrical conductivity (EC). The range of results of this monitoring for 2002 and 2005 are presented in Table 6-5 of Section 6.

4.2.1 BIOCHEMICAL OXYGEN DEMAND (BOD) LOADS

Based on the monthly grab samples, the BOD of the influent ranges from 112 to 307 mg/L, equivalent to 580 to 1,600 lbs per day, and averages 172 mg/L. This average BOD concentration is at the low end of medium strength wastewater. The use of monthly grab samples does not accurately characterize the variability of BOD loading. For planning and evaluation purposes the average BOD load was estimated to be 200 mg/L and a peaking factor of 1.3 was used to estimate peak month loads. Per the 1990 Master Plan, the maximum capacity of the proposed treatment facilities was 2,900 lbs BOD per day, equivalent to 1.74 MGD at 200 mg/L. The maximum recorded load is about 55 percent of this maximum load (based on the limited grab sampling). Future development may contribute a higher per capita BOD load due to increased numbers of garbage disposals and water conservation measures.

Table 4-6
Summary of Flows for Each Drainage Basin[a] Projected Wastewater Flows (gallons per day)

Sewer Basin	Projected Wastewater Flows (gallons per day)									
	LD-Res	**MD-Res**	**HD-Res**	**C**	**I**	**LI**	**PF**	**Avg. Total Per Basin**	**PF**	**Peak Flow per Basin**
1	100,000	-	-	-	-	-	-	100,000	2.50	250,000
2	31,000	4,000	-	-	-	-	-	35,000	2.50	87,500
3	30,000	-	15,000	-	-	-	20,000	65,000	2.50	162,500
4	20,000	-	20,000	4,000	-	-	-	44,000	2.60	114,400
5	60,000	30,000	5,000	10,000	-	-	-	105,000	2.60	273,000
6	96,000	-	16,000	41,000	-	3,000	41,000	197,000	2.70	531,900
7	125,000	-	-	11,000	-	-	-	136,000	2.60	353,600
8	40,000	-	5,000	40,000	-	-	-	85,000	2.80	238,000
9	72,000	-	-	12,000	-	-	-	84,000	2.70	226,800
10	26,000	-	-	17,000	-	-	-	43,000	2.80	120,400
11		-	-	-	83,000	-	-	83,000	2.50	207,500
12		-	-	-	42,000	-	-	42,000	2.60	109,200
13	160,000	-	-	-	-	-	-	160,000	2.50	400,000
14[b]		-	-	130,000	-	345,000	-	475,000	1.70	807,500
15	260,000	-	50,000	-	-	-	-	310,000	2.50	775,000
16	13,000	-	-	34,000	-	-	-	47,000	2.90	136,300
17	100,000	-	-	-	-	-	-	100,000	2.50	250,000
18	204,000	-	52,000	-	-	-	-	256,000	2.50	640,000
19	25,000	-	-	9,000	60,000	-	-	94,000	2.60	244,400
20		-	-	-	84,000	-	-	84,000	2.50	210,000
21	85,000	-	24,000	-	15,000	-	-	124,000	2.50	310,000
22	23,000	-	-	24,000	-	-	-	47,000	2.80	131,600
23	50,000	34,000	-	-	-	-	-	84,000	2.50	210,000
Total	**1,520,000**	**68,000**	**187,000**	**332,000**	**284,000**	**348,000**	**61,000**	**2,800,000**		**6,160,000**

[a] LD-Res, MD-Res, HD-Res = low density, medium density and high density residential respectively. C = commercial. I and LI = industrial and light industrial respectively. PF = public facilities. [b] – Flow factors used for the Liberty Business Park drainage basin were based on previously developed flow factors by O'Dell Engineering.

4.2.2 NITROGEN

Monitoring of influent for nitrogen is limited to nitrate, and the results are below the detection limit. This is to be expected as the majority of nitrogen in domestic wastewater is in organic and ammonic forms. Using a typical range of 13.3 g/capita/d for total Kjeldahl nitrogen (TKN), the nitrogen loading from the wastewater is about 174 lbs/d with a concentration of 35 mg/L.

Section 5

Existing Collection, Treatment, and Disposal Facilities

Section 5

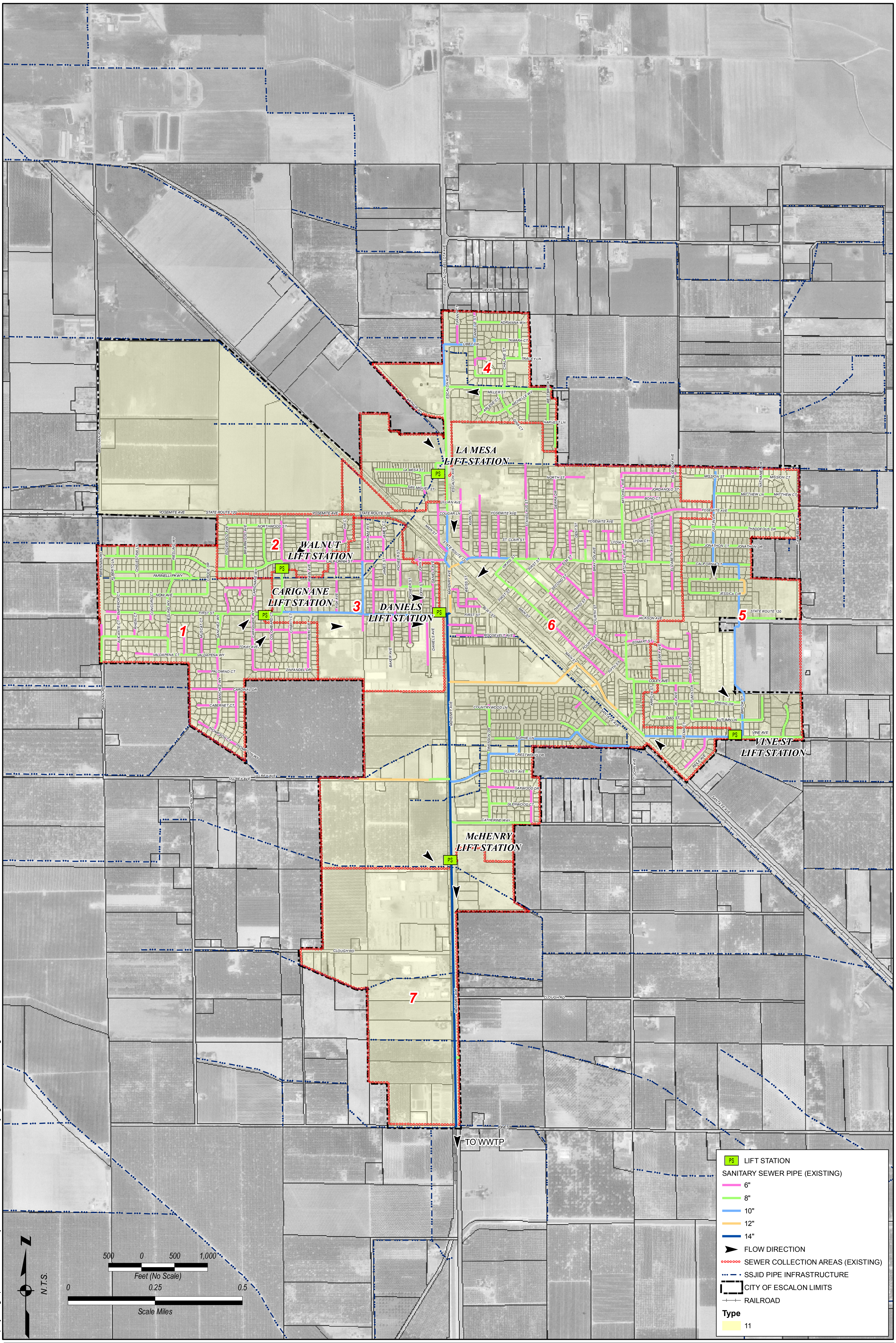
Existing Collection, Treatment, and Disposal Facilities

5.1 EXISTING COLLECTION SYSTEM

The City of Escalon is located in south eastern San Joaquin County within a high-level agricultural area. Given the City's topography it has been necessary to construct a network of gravity sewers constructed at or near minimum slopes with many lift stations to convey wastewater to the wastewater treatment plant located south of the City limits. The City maintains two collection systems; 1) the domestic sanitary sewer system, which consists of seven sub-basins and six lift stations, and 2) the industrial sewer system which serves several industrial dischargers along McHenry Ave., and which also serves as a storm drainage system during the non industrial discharge season. Both systems currently convey flows to the McHenry Ave. pump station where they are separately pumped to the domestic and industrial wastewater treatment plants respectively. The two flows do not mix and are conveyed through separate pipelines. Domestic wastewater is pumped into a 14-inch vitrified clay gravity sewer line that conveys flows to the domestic wastewater treatment plant (WWTP) influent pump station. Industrial wastewater and stormwater are pumped into and conveyed by gravity via an 18-inch pressure pipeline. This Master Plan Update does not include any review or assessment of the industrial sewer system or industrial wastewater treatment plant.

5.1.1 EXISTING DOMESTIC SEWER SYSTEM

The current total population of the City is about 7,200 people and the backbone of the City's sewer system was constructed prior 1950 (14-in McHenry Ave. sewer trunk main). Because of this, existing sewer diameters are not large (14-in currently being the largest diameter). The majority of the City's sewer system is eight inches in diameter or less. These smaller diameter sewers require steeper slopes to maintain adequate flow velocities to keep the sewers self-cleansing. The steeper pipe slopes, in conjunction with the existing level topography, have resulted in the City's system consisting of a network of sewer lines constructed near minimum slopes with many lift stations that pump flows onto shallow manholes (four feet deep) in order to keep sewer lines at reasonable depths. The sewer system layout, sewer drainage basins and network of lift stations are depicted in Figure 5-1. There are approximately 137,230 lineal feet (26.0 miles) of gravity and pressure sewer within the City's domestic sewer system. Sewer sizes range from 6-inch to 14-inch diameter and are composed of vitrified clay, PVC, and asbestos cement pipe.



U:project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/Figure_05_01_Existing_Collection_nEW.ai 10/2/06 cms

Figure 5-1

Existing Wastewater Collection Systems and Drainage Basins

5.1.2 EXISTING SEWER LIFT STATIONS

There are currently six sewer lift stations within the City of Escalon's domestic sewer system. The locations of these lift stations and the sewer drainage basins for each lift station are shown in Figure 5-1.

Table 5-1 lists the lift stations and range of potential influent flow rates to each lift station. In general, both the Walnut and Carignane lift stations lift wastewater from their respective sub-basins into the Daniels lift station sub-basin, while the Daniels, La Mesa and Vine Street lift stations lift wastewater into the McHenry sub basin. As mentioned previously, the McHenry Lift Station is the southern most and last lift station after which all flow from the City is conveyed by gravity through the 14-inch sewer main to the WWTP.

Table 5-1
City of Escalon Sewer Lift Stations

Name	No. Pumps Duty/SB	Capacity[a] (gpm)	Capacity (gpd)	Make/Model/HP	Age/Condition
McHenry P.S[b]	1/1	580	835,200	Fairbanks Morse/B5441/7.5 HP	Installed 7-04
Daniels P.S[c]	1/1	246	354,240	Paco/78-49531-34631 0-01/5HP	Running for over 5 years, One pump rusting and need to replace 4 inch valves
Walnut P.S[c]	1/1	106	152,640	Flygt/One-CP3085MT,One-3085.092-9021/3,2.2	Installed 3/18/2004
Carignane P.S[c]	1/1	209	300,960	Hydr-O-Matic/40MP/NA	Pump # 1 is 3.5 years old, Pump # 2 is 3 years old
La Mesa P.S[d]	1/1	100	144,000	Pea Body Barnes/4SCY/NA	One pump was rebuilt 9/2005, the second is scheduled to be overhauled soon
Vine P.S[e]	1/1	350	504,000	Hydr-O-Matic/40MP/NA	Both pumps 2.5 years old

[a] Detail construction drawings for each lift station were not available therefore lift station capacities were estimated from previous planning level reports and worksheets provided by the City.

[b] Estimated pumping capacity based of analysis of manufacturer provided pump curve and assumed lifting head requirements.

[c] Information on pumping capacity was obtained from the Northwest Industrial Area Alternative Sewer Collection Analysis Technical Memorandum, prepared by ECO:LOGIC Engineering, February 22, 2006.

[d] Information on approximate capacity was obtained from worksheet calculations prepared for the previous Sanitary Sewer Master Plan.

[e] Information on approximate capacity was obtained from a letter to the City from Nolte and Associates regarding the Sanitary Sewer Trunk Extension – Brentwood Estates Unit IV, V, VI dated March 2, 1994.

The City lift stations are currently equipped with telemetry, which allows the City to collect information on real time wet well levels (based on float positions) and pump runtimes. Dedicated backup power is not provided in any of the lift stations, however a portable generator capable of running any of the lift stations is available during power outage. As most of the lift stations, with the exception of the McHenry Lift Station, are located within residential neighborhoods, there is limited space available for above ground equipment.

Currently, the City uses the portable generator to operate its lift stations during power outages, essentially chasing the flows from lift station to lift station, thereby utilizing each station's storage capacity to contain flows. While this practice has proven effective given the current system size, an efficient plan for the design of future lift station facilities should include the installation of permanent backup generators within the lift station site or the addition of overflow lines connecting wet wells with downstream manholes wherever site conditions and upstream sewer and service elevations allow.

Outside of regular equipment maintenance, all lift stations are in proper working condition and appear to provide sufficient capacity to pump existing influent flows. However, increasing flows throughout the City have increased the pump run times of all lift stations, in particular, the McHenry lift station, which based on recorded influent flows at the WWTP, requires that both lead and stand-by pumps operate during peak flow periods. As continued growth throughout the City results in increased wastewater flows to each sub-basin, the pump runtimes of the remaining lift stations will continue to increase requiring more frequent maintenance of equipment and, in some cases, replacement of existing equipment with higher capacity pumps and appurtenances.

All City lift stations should be capable of pumping current peak flows reliably (with the largest pump out of service). Based on operating records and current operator experience, the McHenry lift station does not have sufficient reliable capacity to pump current peak flows, e.g., frequently both pumps are running at this station to convey peak flows.

5.1.3 COLLECTION SYSTEM ANALYSIS

Currently all wastewater is conveyed to the 14-inch gravity sewer in McHenry Ave, and either flows by gravity or is pumped to the WWTP by the McHenry Lift Station located approximately 600 feet south of the intersection of McHenry Avenue and Catherine Way. Both the gravity main and lift station serve as the central hub where virtually all of the City's domestic wastewater is collected and conveyed to the WWTP. Currently, these facilities are reported to be in proper operating condition and are capable of conveying existing peak flows; however, an analysis of both the capacity of the 14-inch sewer and pumping of the McHenry Lift Station indicate that these facilities could be operating at or above their minimum level of service. For instance, the McHenry Lift Station currently conveys peak flows but often with both pumps running, thereby no meeting the City's reliability standard. Also the calculated peak flows through the 14-in sewer would indicate that it is flowing with a slightly surcharged condition. This has not been observed by City staff which could indicate that the pipe slope is somewhat greater than recorded or other factors are causing the actual pipe capacity to slightly exceed the calculated capacity.

The maximum capacity of the 14-inch line between the McHenry Avenue and First Street intersection and the McHenry Lift Station is calculated to be approximately 0.78 million gallons per day (MGD), and the capacity of the sewer between the McHenry Lift Station to the WWTP is calculated to be approximately 0.75 MGD. The current average day flow to the WWTP is approximately 0.62 MGD with a recorded peak day flow of 1.14 MGD in January 2005. Therefore it is possible that during peak flow periods some surcharging within the 14-inch line is occurring, though this has not been observed. Increased development within and around the City will increase the peak flows conveyed throughout these facilities, potentially exceeding the absolute peak capabilities of these facilities to convey wastewater without spilling.

The recently adopted collection system permit will require the City to set up a program to upgrade and replace their aging collection system. On May 2, 2006, the State Water Resources Control Board (SWRCB) adopted a statewide Waste Discharge Requirement (WDR) for collection systems that has taken effect this year. The WDR will require all Cities and Agencies to complete the development and implementation of a Sanitary Sewer Management Program (SSMP). One component of the SSMP is to evaluate the condition and complete a capacity assessment of the existing collection system. A timeline to complete all elements of the SSMP will be given to the City. It is expected that the City will have approximately four years to complete the SSMP.

5.2 EXISTING TREATMENT AND DISPOSAL SYSTEMS

The City of Escalon currently treats an average dry weather flow (ADWF, May through August) of approximately 0.60 MGD wastewater through the use of aerated ponds and disposes of the effluent in percolation ponds. The current City municipal WWTP Waste Discharge Requirements permit the treatment and disposal of up to 0.90 MGD ADWF. This section describes the existing WWTP facilities, compares these facilities to the 1990 Master Plan recommended facilities, and identifies facilities which currently require upgrading. The layout of the existing facilities is depicted in Figure 5-2.

5.2.1 HEADWORKS AND INFLUENT PUMP STATION

Currently, the plant handles gross influent solids by using a Muffin Monster™ inline grinder situated at the McHenry Lift Station. There currently is no grit removal or screening of the raw wastewater prior to the treatment ponds. The accumulation of rags and inert solids has the potential to foul pumps and aerators creating an increased maintenance cost to the operation of the treatment ponds. At a minimum, a bar screen should be installed to remove a portion of the gross solids, consistent with the 1990 Master Plan. Ideally, the bar screen would be used as a backup to a mechanical screen to more efficiently remove influent solids. The impact of the grit is minimal on the gravity fed pond system.

The influent pump station consists of two ABS submersible pumps (9.4 hp) in a 20 ft x 30 ft wet well installed in 1996. The design flow of each pump is 1,100 gpm, giving the influent pump station a reliable capacity of 1,100 GPM (1.58 MGD peak hour flow), with one pump out of service. Based on 2005 flow data, this is just enough capacity to handle current peak hour flows. An additional influent pump or larger capacity pumps should be installed to provide reliable capacity greater than projected peak hour flows.

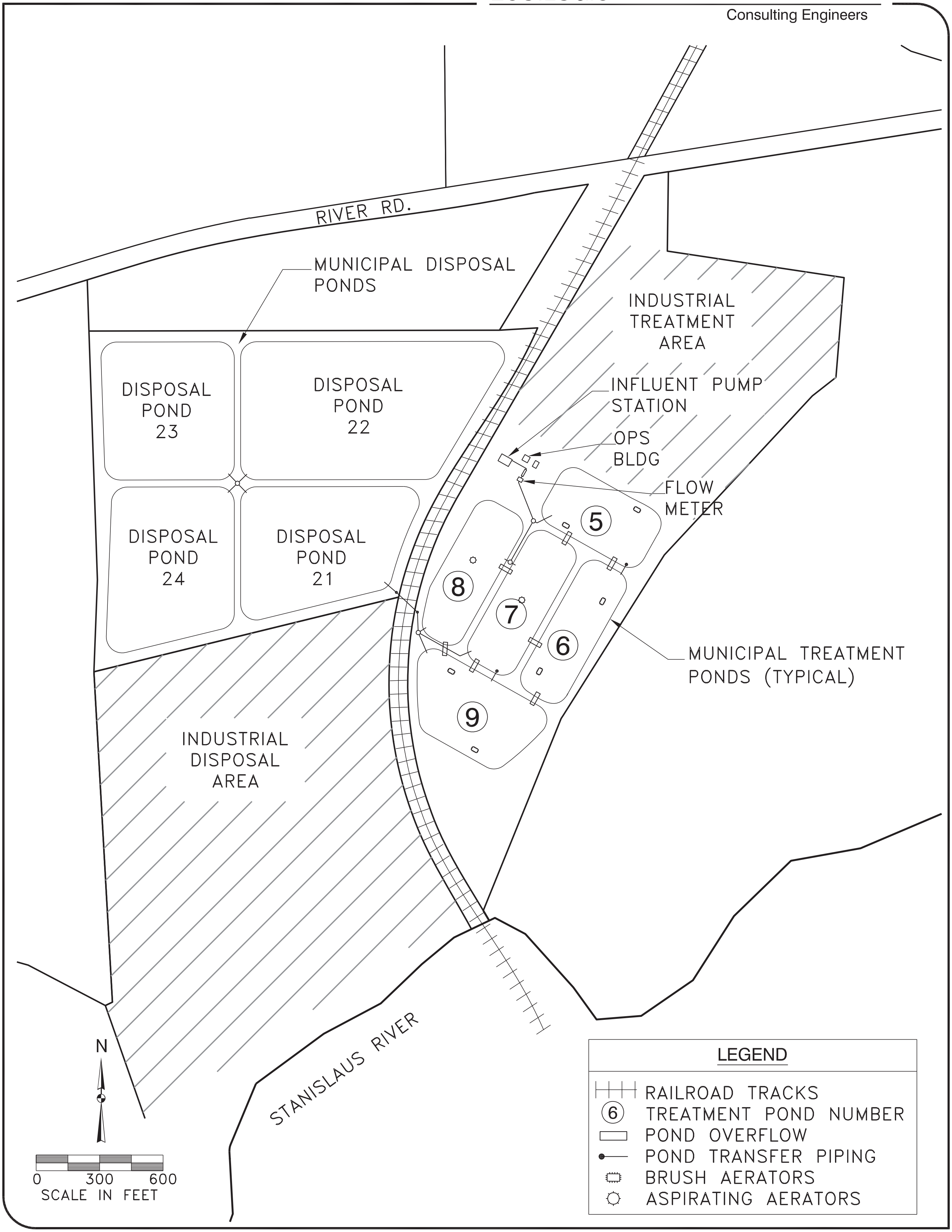


Figure 5-2
Existing Municipal Facilities Layout

The influent is conveyed from the lift station to the overflow box of an abandoned Imhoff tank. The influent continues from the Imhoff tank through a six (6) inch Parshall flume. The current headworks was configured to utilize the flow measurement of the existing Imhoff tank as described in the 1990 Master Plan. Flow metering is accomplished through the use of an ISCO 3010 Ultrasonic Flow Transmitter situated over the 6-inch Parshall flume, which is capable of measuring a maximum flow of 2.5 MGD (Plasti-Fab, Inc.). Flow measurements are continuous with total flow recorded daily, but the existing system requires manual recording. Future flow metering should be updated to provide continuous electronically recorded flow readings or a chart recorder.

5.2.2 TREATMENT PONDS

Wastewater treatment is accomplished in five aerated treatment ponds (numbered 5 through 9). The effective area (surface area at 2 ft freeboard) and volume of the treatment ponds is presented in Table 5-2. The ponds are currently operated in series with wastewater entering pond 5 and flowing into pond 6 then through ponds 9, 8, and 7 in descending order. The first 3 ponds (ponds 5, 6, and 9) have two 15 Hp brush type aerators each and ponds 7 and 8 have one centrally located 10 Hp aspirating aerator each (Figure 5-2). The combined total aerator horsepower is 110 Hp.

Table 5-2
Effective Area and Volume of Existing Ponds.

Pond Type Designation	Effective Area Surface[a] Acres	Volume MG
Treatment		
Pond 5	1.1	3.0
Pond 6	1.3	3.2
Pond 7	1.5	4.3
Pond 8	1.6	5.4
Pond 9	1.6	6.1
Disposal		
Pond 21	3.8	9.8
Pond 22	6.3	6.2
Pond 23	3.4	3.3
Pond 24	3.9	6.3

[a] Pond area at 2 feet of free board

In 2001, BOD reduction was measured by monthly grab samples and ranged from 72 to 99 percent of influent BOD. The average BOD reduction was 82 %. During this same period, BOD loading ranged from 530 to 1,120 lbs per day. Per the 1990 Master Plan, the treatment system is designed to handle 2,900 lbs BOD per day. In 2005, the lowest recorded dissolved oxygen (DO) concentration was 1.2 mg/L, which occurred in pond 5 during July. Average DO ranged from 2.9 to 13.8 mg/L in all ponds during 2005.

The existing treatment ponds are unlined and currently provide incidental disposal. There has been an observed decline in disposal rates over time, likely due to the build up of sludge in the ponds. Previously, the Escalon treatment ponds were managed to facilitate additional disposal through periodically emptying, drying, and disking the ponds. This practice has not continued and therefore any disposal accomplished thought the treatment ponds is incidental. The treatment facilities have incorporated the Phase 1 pond expansions and aerator requirements as detailed in the 1990 Master Plan. The total aerator horsepower exceeds the requirements of Phase 2. Rather than constructing pond 22 first, the City chose to construct pond 21 which was constructed for use as a future aerated treatment pond.

Based on review of sludge profiling conducted by Probiotic Solution on September 21, 2005, 20 to 30 % of the volume of ponds 5 and 6 consists of sludge, which is likely reducing treatment and disposal capacity of the pond significantly. The sludge should be removed from the ponds on a regular basis to maximize treatment volume and disposal.

5.2.3 DISPOSAL PONDS AND EXISTING DISPOSAL CAPACITY

There are four percolation ponds available for municipal effluent disposal (numbered 21 through 24, Figure 5-2). The City consistently uses only two of these ponds (21 and 24) for disposal. Pond 23 is used as capacity is required or when maintenance of Pond 21 or 24 is required. Pond 22 is currently being deepened to provide increased storage capacity. Treated effluent is conveyed by gravity to pond 21 and through pond 21 to a central distribution box where flow can be directed to any of the other three percolation ponds. These ponds were designed assuming a reliable long term percolation disposal capacity of 66,000 gallons per acre per day. Disposal is accomplished primarily by rapid infiltration, however some evaporation does occur. Based on this design disposal capacity, approximately 1.15 MGD of flow can be disposed of in the existing disposal ponds. An emergency treated effluent bypass exists, where the effluent can be diverted to the industrial disposal ponds. This bypass would likely be used only in the winter when industrial flows are minimal to non existent and pond capacity is reduced by incident precipitation, or when emergency maintenance is necessary.

Currently, the disposal ponds are operated with little to no maintenance to maintain percolation rates. The deposition of algae and growth of weeds can significantly decrease the percolation rates of these ponds over time. Operating the ponds independently with frequent wet/dry periods and occasional deep ripping could increase the disposal capacity of the existing disposal ponds and would help maintain the high percolation rates experienced in Escalon. To facilitate this, the distribution of flow to Pond 22 should be modified to allow flow to go to any of the existing disposal ponds independently.

Per the 1990 Master Plan, the existing disposal ponds have been constructed to provide the disposal capacity through Phase 4. Operationally, the ponds are functioning at the Phase 2 level, with one of the ponds being held in reserve.

5.2.4 FLOW DISTRIBUTION

The influent is transferred from the headworks to a valved distribution box where it can be directed to pond 5 or through a pipeline to a distribution box that can feed pond 8 and/or 7. Gravity overflow piping allows for transfer between ponds except there is no connection between pond 5 and 8 or between pond 6 and 8. The overflow piping consists of 15 inch pipe set near the top of the pond levees. The overflow piping is fixed with a blind flange on one side. Lower inter pond piping is known to exist between pond 5 and 6 at an unknown depth and between pond 7 and 9 at about 3.5 feet below the top of the pond levee. Other lower inter pond piping may be present but the locations are unknown.

The pond effluent can be transferred from ponds 7, 8, and 9 through a 15 in PVC pipe to a distribution box that feeds pond 21. Over flow from pond 21 feeds a distribution box where the over flow can be directed to ponds 22, 23 and/or 24.

5.2.5 OPERATIONS/ LABORATORY BUILDING

The current operations building was constructed in 1971, and is used primarily for operations and secure equipment storage. The total area of the floor plan is 350 square feet. There is a lab/ operations work room, with a counter and a sink, desk and a small area for records storage. There is an equipment storage room and a restroom without a shower.

A new operations and laboratory building was to be constructed as a part of Phase 2 of the 1990 Master Plan. No new buildings have been constructed and minimal upgrades have been conducted to the existing building. Two sheds are located behind the operations building for storage. A secure equipment storage building with a work area should be constructed, if a larger operations building is not constructed, to maximize use of the current operations building.

5.2.6 PLANT ACCESS AND SECURITY

A six foot fence has been constructed surrounding the facility. Access is controlled by an electric gate operated by a numeric pass code installed in early 2006. Both sides of the railroad tracks are fenced to prevent unauthorized plant access from the railroad tracks. Currently, the only access between the treatment and disposal portions of the plants is at the far south end of the plant.

5.2.7 OLD EQUIPMENT/PROCESSES

There are two abandoned Imhoff tanks at the WWTP. One has been filled with soil while its over flow box has continued to be used to facilitate influent flow metering. The second Imhoff Tank remains intact and is empty. These tanks will likely need to be demolished prior to use of the area adjacent to them for future improvements.

5.3 PLANT ELECTRICAL AND CONTROLS

The existing municipal treatment plant is almost entirely operated manually. Existing power is provided by a 220 amp service to operate the influent pump station and aerators. The influent pumps are initiated by float switches in the lift station. The aerators are manually operated using on/off switches. Currently, no dedicated back up power supply is present, however the City's portable backup generator can be connected to the system during periods of power failure. At minimum, approximately 120 KVa back up power supply should be available to operate the influent pump station and aerators in the first three ponds during extended power outages.

Section 6

Waste Discharge Requirements and Regulatory Issues

Section 6

Waste Discharge Requirements and Regulatory Issues

6.1 INTRODUCTION

The key to wastewater treatment and disposal facilities planning is compliance with Waste Discharge Requirements (WDRs) issued by the California Regional Water Quality Control Board (hereafter Regional Board). The Regional Board establishes the wastewater treatment standards necessary for case and site-specific wastewater disposal methods in order to protect public health and the environment to the level and certainty deemed appropriate by law, regulation, and policy. Recently the State Water Resources Control Board has added wastewater collection under the regulation of WDRs by issuance of Order No. 2006-0003-DWQ regarding Statewide General Waste Discharge Requirements for Sanitary Sewer Systems.

The City's current method of disposing of its treated wastewater (termed "effluent") is by infiltration into the underlying groundwater resource which flows to the Stanislaus River (as will be discussed). This disposal method is called "land application" and is preferred by the Regional Board over direct discharge of effluent to the Stanislaus River. Effluent reuse or reclamation (such as by irrigation of crops or landscaping) is the effluent disposal method most favored by the Regional Board. However, there is no known interest in effluent reclamation in the greater Escalon area because other sources of water are readily available in the area, and because effluent reclamation can cause groundwater degradation (as will be discussed). Consequently, effluent reclamation does not appear to be a feasible effluent disposal method for the City at the present time. Therefore, it is appropriate to begin the wastewater facilities planning process by determining if there is anything wrong with the current wastewater treatment and disposal operation such that it can be determined if the current operation 1) needs improvement, 2) could serve as a basis for increasing sewer service capacity, or 3) should not be the basis for increasing sewer service capacity. These determinations have significant bearing on the nature and cost of facilities to serve new growth, as well as on costs for continuing lawful sewer service to existing City residents and businesses.

With the City currently disposing of effluent by land application, the critical questions that determine the necessary effluent quality requirements (and therefore the necessary effluent treatment requirements to comply with current laws, regulations, and policies) are:

- Is land application of effluent causing groundwater pollution, i.e., causing an exceedance of an applicable water quality objective (WQO)?
- Is land application of effluent causing groundwater degradation, i.e., causing a deterioration of groundwater quality, but not pollution as defined above?

Effluent quality requirements in WDRs are established such that groundwater pollution does not occur. However, effluent quality requirements in WDRs may be established such that some groundwater degradation is allowed to occur under State Water Resources Control Board Resolution No. 68-16, as long as the degradation is determined to be consistent with maximum benefit to the people of California.

With the foregoing introduction to the regulatory basis for wastewater facilities planning, the remainder of this section assesses whether there is evidence that the existing wastewater treatment and disposal facilities are causing or have a reasonable potential to cause either groundwater degradation or pollution; and if so, what wastewater facilities planning is necessary to address the degradation or pollution. This assessment begins with a review of wastewater treatment plant (WWTP) site hydrogeology and groundwater monitoring to determine if groundwater pollution or degradation appears to be occurring. For contaminants causing or potentially causing pollution or degradation, groundwater impact mitigation measures are developed. These mitigation measures include changes in treatment process and/or effluent disposal method.

Requirements of the Statewide General Waste Discharge Requirements for Sanitary Sewer Systems is discussed in Section 5 with the collection system analysis.

6.2 HYDROGEOLOGY AND BACKGROUND GROUNDWATER MONITORING

The City has studied hydrogeology (i.e., groundwater movement and water quality) at the WWTP site since 2000 via a set of ten groundwater monitoring wells at the locations shown in Figure 6-1. Also shown in Figure 6-1 are the municipal WWTP ponds (the subject of this facilities planning document), the industrial WWTP ponds, adjacent lands and uses, and the Stanislaus River to the south, east, and west of the WWTP site.

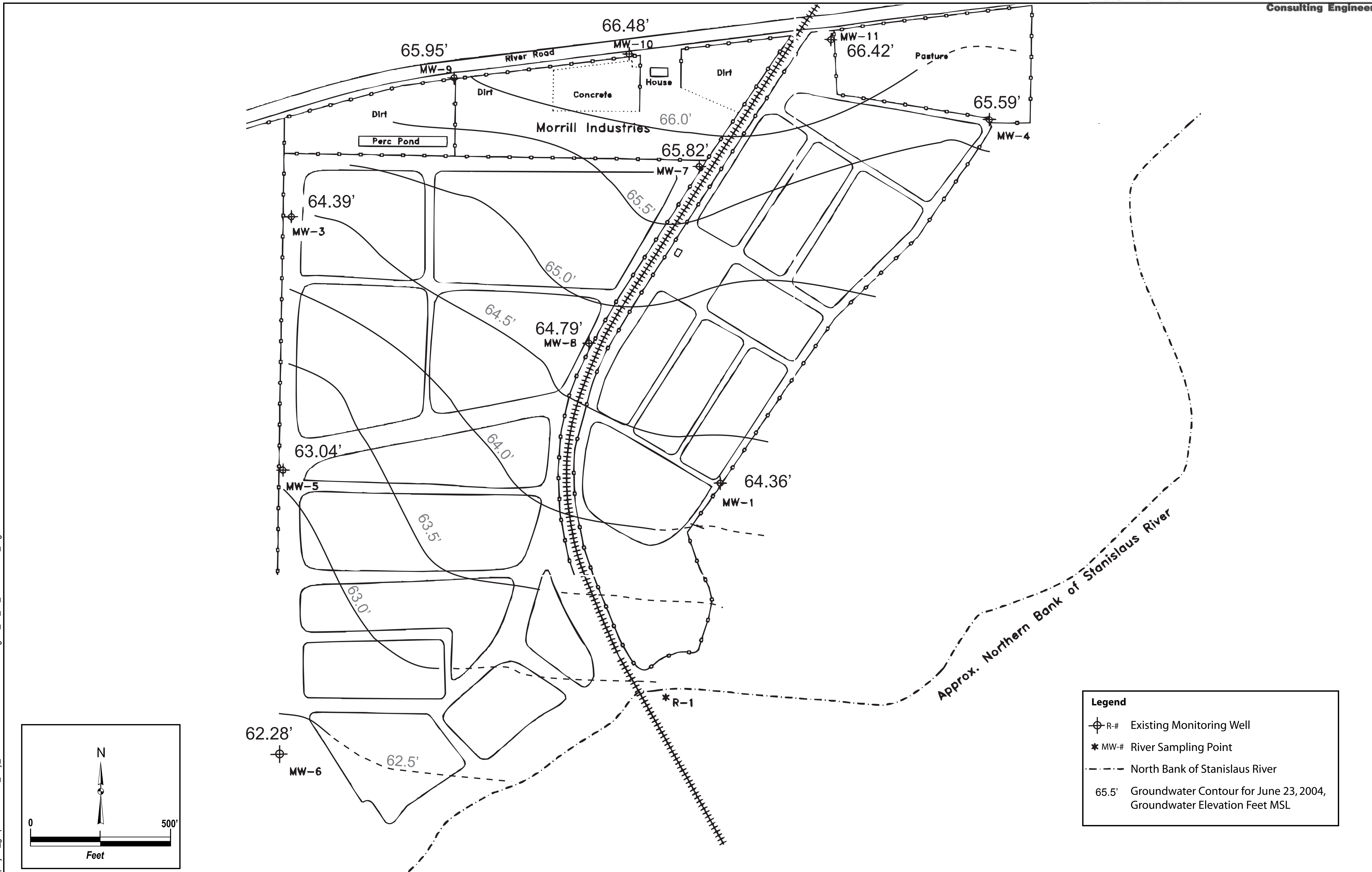
6.2.1 GROUNDWATER FLOW DIRECTION

Typical groundwater flow direction projections, for the WWTP site under seasonally high and low groundwater tables, based on water levels in the City's monitoring wells, are shown in Figure 6-2 for June 2004, and Figure 6-3 for November 2005. As shown, groundwater appears to be flowing to the south-southwest, and therefore into the Stanislaus River under most conditions. This assessment is in general agreement with regional groundwater elevation maps prepared by the Department of Water Resources. Under high river stage conditions, river water may exfiltrate and cause a short-term, localized, groundwater flow gradient to the north-northwest.

6.2.2 BACKGROUND GROUNDWATER QUALITY

A necessary element of an assessment of whether the existing municipal WWTP treatment and/or disposal operation is causing pollution or degradation of groundwater is the determination of what constitutes baseline or "background" groundwater quality. The Regional Board requires that this determination be made based on first recoverable groundwater quality.

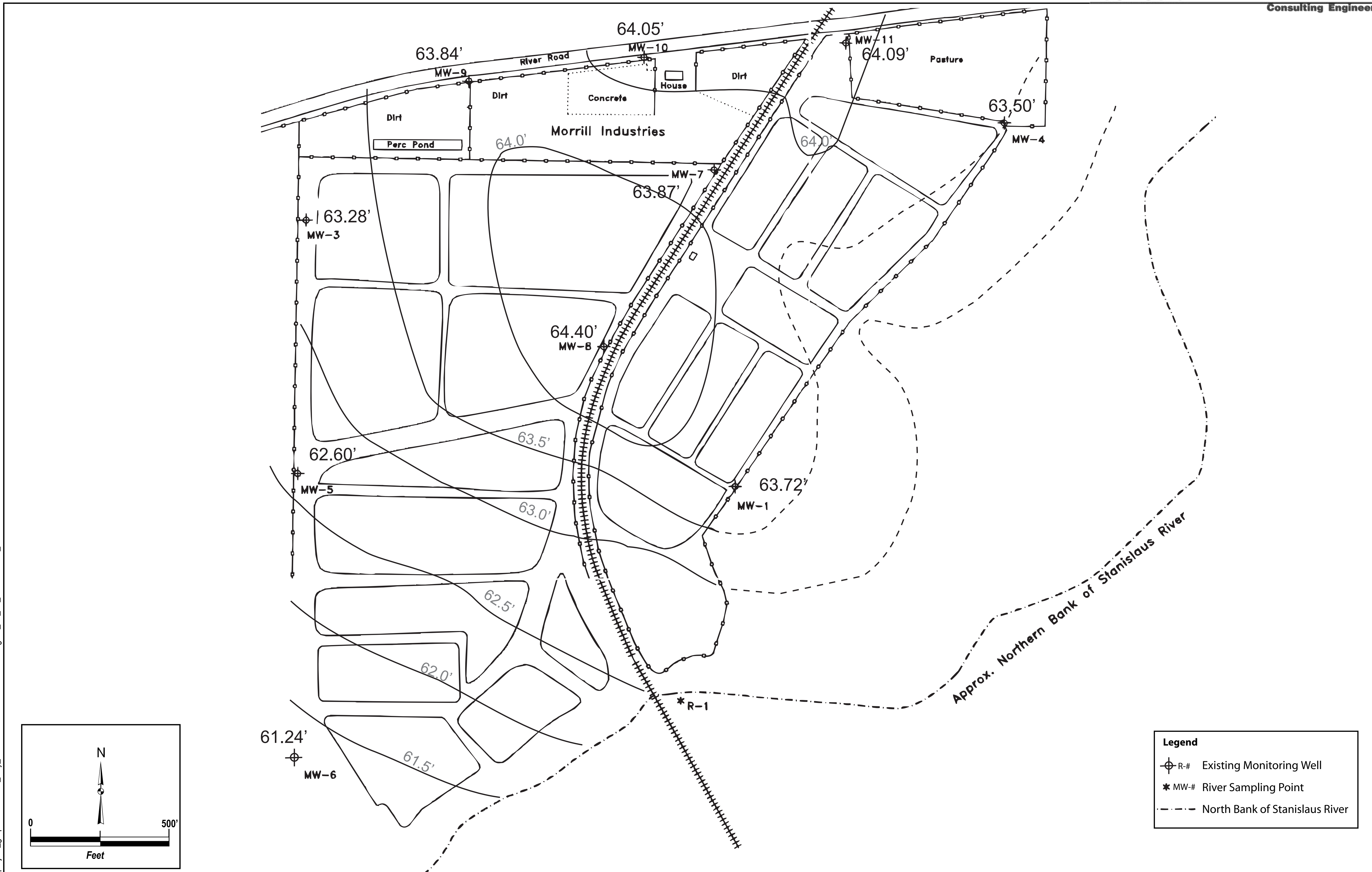
Figure 6-1
Groundwater Monitoring Well Locations



u:/project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/Figure_06_02_Groundwater_High.ai 8/14/06 cms

Source: Geological Technis, Inc., 2005

Figure 6-2
Groundwater Contours During Seasonally High Period, 2004



u:/project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/Figure_06_02_Groundwater_Low.ai 8/14/06 cms

Figure 6-3

Groundwater Contours During Seasonally Low Period, 2005

A problem with this requirement is that first recoverable groundwater quality varies significantly both spatially and temporally as a result of very localized spatial and temporal variations in overlying land use. The baseline/background groundwater quality prior to land application of effluent studied by the Mountain House Community Services District is a dramatic example of the spatial and temporal variation of first recoverable groundwater from western San Joaquin County. Variations in background groundwater quality at the Escalon site do not appear to be as extreme as at Mountain House (as will be discussed), but there is variation. Because of this variation, there is no single value for "background" water quality that becomes the basis for groundwater pollution and degradation analyses; but rather, there is a range of "background" groundwater quality that is used for groundwater pollution and degradation analyses.

Based on the Figure 6-2 and Figure 6-3 groundwater flow direction projections, monitoring wells MW-9, MW-10, and MW-11 along River Road are the best wells for estimating background groundwater quality at the WWTP site as they are up-gradient of the facility. A summary of water quality data from these apparent "background" monitoring wells is presented in Table 6-1. From these data, the long-term, median background concentrations for the Table 6-1 parameters are:

Table 6-1
Median Background Water Quality for Wells MW-9, MW-10, and MW-11

Parameter	Range in Medians of Background Concentrations	WQO
TDS, mg/L	570 – 640	450
EC, μS/cm	769 – 1,015	700
Nitrate, mg/L as N	2.5 – 13.4	10
Chloride, mg/L	21 – 59	106
Sodium, mg/L	24 - 77	69

The monitoring for pathogens in groundwater is conducted by analyzing for total coliform on a present/absent basis. There are several present results for the background wells without enumeration of the coliforms, and the percent of samples with total coliform present ranges from 17.6 to 42.1 in background monitoring wells. It is likely that the presence of total coliform in background wells is derived from well construction and/or sampling techniques and handling, rather than from their actual existence in background groundwater, since the limited mobility of coliform bacteria in sediments precludes their presence in groundwater beyond very localized conditions.

Background data for important groundwater quality parameters such as iron, manganese, arsenic, etc, are not available for analyses.

6.3 GROUNDWATER IMPACT ASSESSMENT

The municipal WWTP treatment and disposal system has been operating at the current site for decades; thus, if the operation is impacting groundwater quality, then that impact should be evident in the groundwater quality data from City monitoring wells located downgradient from the municipal WWTP treatment and disposal ponds. Since the City has both industrial and municipal treatment and disposal occurring at the same site, any impacts to groundwater are a combined product of the two disposal activities, and direct impacts from any one use cannot readily be ascertained with existing data. Groundwater quality data collected to date from the monitoring wells is limited with regard to constituents monitored. Further, there is limited monitoring of the effluent and other potential sources of recharge in the immediate area to assess the various contributions to groundwater quality.

6.3.1 DOWNGRADIENT GROUNDWATER QUALITY

Based on the Figure 6-2 and Figure 6-3 groundwater flow direction projections, the monitoring wells most downgradient from the municipal WWTP treatment and disposal ponds (however with the least amount of influence from the industrial WWTP ponds) are MW-1, MW-5, and MW-8, while MW-6 is down gradient of the combined domestic and industrial disposal activities at the site. A summary of water quality data from these "downgradient" monitoring wells is presented in Table 6-2. From these data, the long-term, median downgradient concentrations in monitoring wells directly downgradient of the municipal treatment and disposal ponds for the Table 6-1 parameters are:

Table 6-2
Median Downgradient Water Quality for Wells MW-1, MW-5, and MW-8

Parameter	Range in Medians of Background Concentrations	WQO
TDS, mg/L	484 – 630	450
EC, µS/cm	691 – 1,024	700
Nitrate, mg/L as N	1.2 – 6.2	10
Chloride, mg/L	62 – 71	106
Sodium, mg/L	57 - 91	69

Total coliform monitoring in wells immediately downgradient of municipal treatment and disposal activities, indicates the percent of samples with total coliform present ranges from 4.5 to 31.8. Table 6-4 compares the percent present results for background and downgradient monitoring wells.

Table 6-3
Water Quality Data Summary for Background Monitoring Wells MW-9, MW-10, MW-11

Parameter	MW-9	MW-10	MW-11
Years of data	4	4	4
Sampling frequency	Quarterly	Quarterly	Quarterly
TDS: WQO = 450 mg/L			
Number of analyses	17	17	17
Range of results, mg/L	190-1,355	388-890	160-730
Median of results, mg/L	576	570	640
EC: WQO = 700 µS/cm			
Number of analyses	17	17	17
Range of results, µS/cm	488-1,207	545-1,283	677-1,517
Median of results, µS/cm	769	804	1,015
Nitrate (as N): WQO = 10 mg/L			
Number of analyses	17	17	17
Range of results, mg/L	10.8-38.0	8.6-22.8	<0.5-13.0
Median of results, mg/L	13.4	13.0	2.5
Chloride: WQO = 106 mg/L			
Number of analyses	17	17	17
Range of results, mg/L	12-53	16-55	44-83
Median of results, mg/L	25	21	59
Sodium: WQO = 69 mg/L			
Number of analyses	17	17	17
Range of results, mg/L	18-41	28-48	68-92
Median of results, mg/L	24	35	77

Table 6-4
Water Quality Data Summary for
Downgradient Monitoring Wells MW-1, MW-5, MW-8

Parameter	MW-1	MW-5	MW-6*	MW-8
Years of data	5	5	5	5
Sampling frequency	Quarterly	Quarterly	Quarterly	Quarterly
TDS: WQO = 450 mg/L				
Number of analyses	23	23	23	23
Range of results, mg/L	398-760	173-1,050	360-965	330-740
Median of results, mg/L	490	630	570	484
EC: WQO = 700 μS/cm				
Number of analyses	23	23	23	23
Range of results, μS/cm	588-1,183	534-1,600	496-1,302	475-1,163
Median of results, μS/cm	745	1,024	874	691
Nitrate (as N): WQO = 10 mg/L				
Number of analyses	23	23	23	23
Range of results, mg/L	<0.11-2.0	<1.1-30.9	<0.11-17.0	<0.5-18.0
Median of results, mg/L	1.2	6.2	1.2	3.6
Chloride: WQO = 106 mg/L				
Number of analyses	23	23	23	23
Range of results, mg/L	41-92	39-100	23-112	37-98
Median of results, mg/L	65	62	54	71
Sodium: WQO = 69 mg/L				
Number of analyses	23	23	23	23
Range of results, mg/L	76-94	46-75	44-86	70-100
Median of results, mg/L	84	57	56	91

(*) MW-6 is not immediately downgradient of the municipal treatment and disposal ponds, but is downgradient of the combined municipal and industrial treatment and disposal ponds.

6.3.2 Apparent Potential Impact of Municipal WWTP on Groundwater Quality

A comparison of the ranges of median concentrations from the background monitoring wells and downgradient monitoring wells directly downgradient of the municipal treatment and disposal activities is presented in Table 6-5. As shown and noted in Table 6-5, from available data it appears that the municipal WWTP ponds:

- Have no apparent impact on overall groundwater salinity as measured by TDS and EC.
- May reduce groundwater nitrate concentrations.
- Potentially cause degradation with regards to groundwater chloride.
- Potentially cause groundwater impacts with regards to groundwater sodium.

Table 6-5
WWTP Groundwater Impact Assessment

Parameter	WQO	Range of Median Concentrations from Monitoring Wells		Apparent Potential Impact of the Municipal WWTP Ponds on Groundwater Quality
		Background Wells	Downgradient Wells[a]	
TDS, mg/L	450	570 – 640	484 - 630	No apparent impact
EC, μS/cm	700	769 – 1,015	691 – 1,024	No apparent impact
Nitrate as N, mg/L	10	2.5 – 13.4	1.2 – 6.2	Groundwater improved
Chloride, mg/L	106	21 – 59	62 - 71	Groundwater potentially degraded
Sodium, mg/L	69	24 – 77	57 - 91	Groundwater potentially impaired

[a] Based on Monitoring Wells No. 1, 5, and 8 , immediately adjacent to municipal treatment and disposal ponds.

The qualitative nature of the total coliform data does not allow for numeric comparison of coliform data. A comparison of the percent present total coliform samples, as presented in Table 6-6, indicates there is no apparent impact on groundwater quality with respect to total coliforms, e.g., in most cases the majority of groundwater samples resulted in the absence of total coliform organisms. The data shows that the presence of total coliform in downgradient wells is comparable to slightly less than that of the background wells.

Table 6-6
WWTP Groundwater Total Coliform Monitoring Results for Background and Downgradient Monitoring Wells

Monitoring Well	Designation	Present Samples %
MW-1	Downgradient	4.5
MW-5	Downgradient	27.3
MW-8	Downgradient	31.8
MW-9	Background	41.2
MW-10	Background	29.4
MW-11	Background	17.6

There are no data to assess other possible impacts of the municipal WWTP ponds on area groundwater quality, such as with regards to iron, manganese, arsenic, etc.

Although groundwater quality leaving the site, as characterized by MW-5 and MW-6 appears to have nitrate concentrations less than what is encountered in the background monitoring wells, other forms of nitrogen are not characterized. Pond and rapid infiltration disposal facilities are known to have only limited capabilities to reliably reduce nitrogen concentrations to less than the WQO for nitrate (10 mg/L as N). Therefore any future expansion of the existing effluent disposal facility is anticipated to include a comprehensive study of the soil capabilities to reduce effluent nitrogen to less than 10 mg/L.

6.3.3 A Check on the Credibility of the Apparent Impact Assessment

With any groundwater impact assessment, it is important to ascertain whether the apparent impact assessment is credible based on the quality of the effluent being disposed of, and how the municipal WWTP treatment and disposal ponds are operated. What is known about the quality of the City's wastewater is presented in Table 6-7. There are no sodium or chloride data for either the influent wastewater or the effluent.

Net evaporation from the ponds (i.e., after consideration of rainfall into the ponds) is expected to cause concentrations of relatively conservative contaminants such as TDS, sodium, and chloride to increase somewhat. The elevated pH of pond water (resulting from algal photosynthesis) may cause some precipitates to form in the ponds (such as calcium carbonate) resulting in a limited reduction of TDS. Generally sodium or chloride do not form common pond precipitates, and therefore are not expected to decrease during pond treatment.

Table 6-7
Municipal Wastewater and Effluent Quality Data

Sample	BOD, mg/L	Nitrate, mg/L as N[b]	TDS, mg/L	EC, µS/cm
2002 Wastewater entering the treatment ponds[a]				
Number of samples	12	12	12	0
Range of concentrations	112-260	<0.2-0.4	316-594	NA
Mean of concentrations	171	0.3	462	NA
2005 Wastewater entering the treatment ponds[a]				
Number of samples	12	12	12	12
Range of concentrations	125-307	<0.2	376-496	616-770
Mean of concentrations	172	<0.2	437	703
2002 Effluent entering the disposal ponds[a]				
Number of samples	12	12	12	12
Range of concentrations	10-31	0.3-13	300-447	627-756
Mean of concentrations	17	4.8	393	671

[a] See Figure 6-1 for the locations of the treatment and disposal ponds.

[b] Only nitrate data is available for nitrogen species, which does not characterize the predominant forms of nitrogen likely to be in these facilities.
NA- Not analyzed

The water balance prepared for these facilities (see Chapter 8) suggests that net evaporation from the ponds is about 14 percent of inflow. This should concentrate conservative salinity constituents such as TDS, sodium, and chloride by about 14 percent. If the Table 6-7 influent TDS and EC values are increased by 14 percent to reflect net evaporative concentration, the resulting percolate TDS and EC values would be about 523 mg/L and 817 μS/cm, respectively. These values are within the ranges of median values in downgradient wells as presented in Table 6-5. Thus, the available wastewater TDS and EC data suggest that the assessment of apparent impacts is credible.

Effluent sodium and chloride concentrations are not available to determine the credibility of the critical aspects of the assessment of apparent impacts dealing with possible chloride degradation of groundwater and sodium impact to groundwater. This information is needed prior to designing any new municipal treatment and/or disposal facilities. In the absence of this information, this facilities plan needs to address the possibility that the municipal WWTP may cause localized chloride degradation of groundwater and localized sodium impact to groundwater at the municipal WWTP site. This is because:

- A facilities plan needs to address reasonable worst-case scenarios, which in this case includes the possibility of the municipal WWTP site causing at least localized chloride degradation and sodium impact to shallow groundwater underlying the municipal WWTP site.
- The TDS/EC data suggest the apparent impact assessment is credible.
- Municipal effluent commonly has elevated concentrations of sodium and chloride, particularly when the municipal water supply is groundwater such as in Escalon.

It is recommended that prior to permitting and designing any significant facilities expansion, the City commence an expanded effluent, surface water, and groundwater sampling program to better characterize the various sources of recharge to groundwater including the percolating municipal and industrial effluent.

6.4 GROUNDWATER IMPACT MITIGATION MEASURES

As stated, the key to wastewater treatment and disposal facilities planning is achieving compliance with WDRs. The foregoing groundwater impact assessment suggests that there are at least two effluent constituents of concern from a WDR compliance perspective if disposal continues to occur via land application:

- Chloride, which appears to be potentially degrading groundwater, underlying the municipal WWTP site.

- Sodium, which appears to be potentially impacting groundwater underlying the municipal WWTP site.

The actual source or relative contribution between the municipal and industrial discharges is not known. However, of the two, sodium is the more critical because of the apparent impact to the groundwater must be mitigated. Degradation of groundwater by chloride may be permitted if certain conditions are satisfied. Consequently, this discussion of groundwater impact mitigation measures focuses on sodium mitigation measures with lesser discussion of chloride mitigation measures, though, as will be seen, the mitigations for sodium also tend to mitigate chloride.

Any mitigation measure employed by the City must consider the relative contribution of sodium and chloride to the site from the municipal and industrial discharges. If either one of these discharges contributes a majority of the sodium or chloride, mitigation measures should be focused on the primary source, with appropriate consideration of the other source if it is still significant. The following mitigation measures focus on the municipal WWTP as this is the scope of this Master Plan, however, these measures may also be considered for the industrial system as warranted the results of additional expanded monitoring.

6.4.1 THE RANGE OF MITIGATION MEASURES

Mitigation measures to prevent sodium impacts to groundwater include:

- Removing sodium from the wastewater at the WWTP site.
- Removing sodium from the wastewater at its source, i.e., source control.
- Changing the effluent disposal method to one more tolerant of current effluent sodium concentrations.
- Establishing a point of compliance for groundwater limitations that is protective of water resources, without being overly protective from a cost/benefit perspective.

Each of these mitigation measures is discussed in the following subsections in the context of wastewater treatment and disposal facilities planning.

Removing Sodium at the WWTP Site

Removing sodium at the WWTP site requires a reverse osmosis (RO) treatment process, or one of the related “demineralization” processes such as nanofiltration (NF) or electrodialysis reversed (EDR). To reduce the influent sodium concentration from 75 mg/L (a guess for the purposes of this example based on the Table 6-2 data) down to about 60 mg/L (below the 69 mg/L WQO so as to allow some evaporative concentration of sodium during disposal) may require about 21 percent of the wastewater to pass through the RO unit after first receiving tertiary treatment, water conditioning, and membrane filtration as necessary pretreatments to the RO unit. The cost to build and operate an RO unit (including waste brine disposal) is very high, and therefore is given no further consideration in this report because by inspection more feasible/cost effective mitigation measures are available to the City to resolve the possible localized sodium impacts.

Removing Sodium at the Source(s)

Municipal use of water typically adds 40 to 70 mg/L of sodium (and 20 to 50 mg/L of chloride) in communities such as Escalon that do not have substantial numbers of water softeners. Using a central value for sodium addition of 55 mg/L (35 mg/L for chloride), and a current water supply average sodium concentration of 15 mg/L, influent and effluent wastewater sodium concentrations of about 70 mg/L and 81 mg/L, respectively, would be expected. Effluent sodium concentrations of around 81 mg/L would be close to the center of the range in median sodium concentrations (57 to 91 mg/L) observed in downgradient monitoring wells (see Table 6-2).

The City is in the process of switching a portion of the potable water supply over to South San Joaquin Irrigation District (SSJID) water that has an average sodium concentration of about 3 mg/L (chloride = 1 mg/L). Based on typical municipal values (in the absence of Escalon-specific data), this change should reduce the influent and effluent sodium concentrations down to long-term (i.e., annual) average concentrations of about 58 mg/L and 67 mg/L, respectively. These concentrations are less than the sodium water quality objective of 69 mg/L, and therefore, should eliminate the possibility of the municipal WWTP operation causing shallow groundwater impacts if the water supply was switched mostly to entirely over to the SSJID water supply. Some sodium (and chloride) degradation may still occur, but this may be permitted as discussed earlier.

Maximizing use of the SSJID potable water supply is the recommended mitigation measure to prevent potential municipal WWTP sodium pollution of shallow groundwater, and to minimize sodium and chloride degradation of groundwater. This mitigation measure should be supported by the additional mitigation measures of 1) source control via public education and ordinance, and 2) specification of points of compliance for groundwater limitations.

Changing the Effluent Disposal Method

Credible alternative effluent disposal methods include reclamation, discharge to the Stanislaus River, and moving the land application disposal method to another site that can safely assimilate effluent sodium (e.g., overlying an area already severely impaired with respect to sodium by historic land uses).

Switching effluent disposal to reclamation via irrigation of crops or landscaping is not expected to mitigate impacts to groundwater associated with sodium. This is because vegetation evapotranspiratively concentrates conservative water constituents such as sodium and take up relatively little sodium into the harvested portion of the vegetation. Consequently, effluent disposal by reclamation would be expected to exacerbate any soil and groundwater sodium problem, not reduce it.

Discharging effluent to the Stanislaus River is possible, though unlikely. The estimated minimum release from New Melones Dam is 240 MGD; thus, an effluent discharge from the City of up to 12 MGD is feasible without exceeding Department of Health Services guidelines that potable water supplies not exceed 5 percent effluent, regardless of the level of wastewater treatment. Limiting the discharge such that it is only 5 percent of the resulting downstream river flow should also result in compliance with all water quality requirements for the Stanislaus River based on the Basin Plan and the California Toxics Rule.

The problem with discharging effluent to the Stanislaus river is that it flows to the Delta which is Clean Water Act 303(d) listed as being water quality impaired for salinity, mercury, and other constituents. Adding salinity and other constituents of concern to the Stanislaus River via a direct effluent discharge may exacerbate already troubled water quality conditions in the Delta, thus, the unlikely nature of being permitted to discharge effluent to the Stanislaus River, except as a possible alternative to the City undertaking a major RO treatment and brine disposal project. The fact that City effluent salinity is already flowing subsurface to the Stanislaus River and Delta does not change materially the foregoing assessment.

Point(s) of Compliance

A fourth mitigation measure would be to establish a point (or points) of compliance for groundwater limitations. Generally, the Regional Board is not looking for compliance with groundwater limitations on an any time, any place, any where basis because of the natural variability in shallow groundwater quality (as seen in the three background monitoring wells at Escalon). The Regional Board is generally more concerned about groundwater quality leaving a WWTP site rather than shallow groundwater immediately under a WWTP site where natural soil treatment and attenuation may not be complete, and the WWTP has control over land uses and groundwater uses.

Table 6-8
WWTP Groundwater Impact Assessment of Groundwater Leaving the Joint Municipal and Industrial Site

Parameter	WQO	Range of Median Concentrations from Monitoring Wells		Apparent Impact of the Municipal WWTP Ponds on Groundwater Quality
		Background Wells	Downgradient Wells	
TDS, mg/L	450	570 – 640	570 – 630	No apparent impact
EC, μS/cm	700	769 – 1,015	874 – 1,024	No apparent impact
Nitrate as N, mg/L	10	2.5 – 13.4	1.2 – 6.2	Groundwater improved
Chloride, mg/L	106	21 – 59	54 – 62	Groundwater potentially degraded
Sodium, mg/L	69	24 – 77	56 – 57	Groundwater potentially degraded

[a] Based on Monitoring Wells No. 5 and 6, representative of groundwater leaving the industrial and municipal treatment and disposal site, e.g., point of compliance wells.

Considering the foregoing, the Regional Board may allow monitoring wells MW-5 and MW-6 to be the points of compliance for groundwater limitations from this complex site incorporating both industrial and municipal wastewater treatment and disposal on a single site with the municipal facilities being located between the industrial treatment ponds and the industrial disposal ponds. Clearly, MW-5 and MW-6 are most downgradient from the joint wastewater treatment and disposal activities occurring on this site as shown in Figures 6-2 and 6-3. Water quality data from these two wells are presented in Table 6-8 along with data from the background wells. As shown in Table 6-8 sodium pollution of groundwater does not appear to be occurring. Potential sodium and chloride degradation does appear to be occurring.

It is recommended that this mitigation measure be included in the overall mitigation plan incorporating the source control measures discussed earlier. Further, it is recommended that the City conduct a comprehensive groundwater quality monitoring program to include analysis of the chemistry of both industrial and municipal influent and effluent, and other surface waters that could potentially be impacting the shallow groundwater. This monitoring program should be broad enough to characterize contributions from surface water to the quality of the underlying and adjacent groundwater.

Recommended Mitigation Measures

If, upon review of expanded monitoring, impacts are found to exist, several mitigation measures can be reviewed and possibly employed to reduce or minimize those impacts.

Considering the cost, efficacies, and regulatory issues associated with the various sodium (and chloride) mitigation measures considered herein, the best apparent plan to prevent potential impacts and minimize degradation is to:

- Switch to the SSJID water supply to the maximum extent feasible to reduce water supply (and therefore effluent) sodium and chloride concentrations to the extent feasible.
- Monitor the residential, commercial, and industrial wastewater flows to determine if there is an elevated source of sodium (or chloride) in the community that could be reduced by feasible pretreatment or other means.
- Continue with public education to try to reduce wastewater sodium and chloride concentrations.
- Propose that the point of compliance with groundwater limitations be MW-5 and MW-6 in light of the foregoing BPTC measures.

6.5 BEST APPARENT FACILITIES PLAN

If groundwater impacts are truly found to exist and if the foregoing mitigation plan is acceptable to the Regional Board as the best practicable plan for achieving compliance with current laws, regulations, and policies (including the Basin Plan and Resolution No. 68-16), the following is the best apparent plan for expanding the municipal wastewater treatment and disposal facilities to serve new growth:

1. Additional wastewater and groundwater monitoring is necessary to determine if there are other constituents of concern from a groundwater pollution or degradation perspective other than sodium and chloride. Particular attention is warranted around the existing

unlined treatment ponds. At the minimum, it is expected that these unlined treatment ponds will potentially cause sodium and chloride degradation of groundwater even with water supply improvements, public education, and feasible source control. Therefore, under Resolution No. 68-16 there would be a basis to require either lining the treatment ponds, or replacing these ponds with an activated sludge process. The problem in such an approach is that neither expensive transformation would reduce the flow of sodium or chloride or any other conservative constituent to the groundwater to any material extent as long as effluent disposal continues to occur via land application. Therefore, lining the treatment ponds or converting to activated sludge for existing development or new growth would need to be driven by factors other than conservative wastewater constituent such as sodium, chloride, or salinity in general.

2. Effluent disposal is always a critical issue. The current disposal method of land application appears to be superior to the alternatives of reclamation (with its evapotranspirative concentration of conservative constituents such as sodium, chloride, and salinity) or river discharge (with the downstream 303(d) listings as well as general Regional Board policy opposition to new river discharges). The existing municipal effluent disposal ponds have sufficient capacity for existing and near-term growth. The existing disposal ponds do not have sufficient capacity for planned growth (as discussed in Chapter 8). This leads the City to four alternatives:

   - Limit growth to what can be served by the existing disposal ponds.
   - Acquire additional lands for construction of additional disposal ponds.
   - Convert wastewater treatment to activated sludge such that the current aerated treatment ponds can be converted into new effluent disposal ponds (noting that disposal ponds have greater disposal capacity than continuously inundated treatment ponds).
   - Move industrial wastewater treatment and disposal to a new site and free up industrial disposal capacity for municipal use.

   Of these, the first is not a consideration of this facilities plan. Implementation of the second and third will depend on several factors:

   - Acquiring more land for additional treatment and disposal ponds is most desirable.
   - However, if land is not available except by condemnation or at very high cost, then converting to activated sludge treatment to free up the existing treatment ponds for disposal purposes may be appropriate.
   - If the Regional Board insists that the existing treatment ponds be lined under Resolution No. 68-16, then, it may be more cost effective to switch to activated sludge treatment and convert the existing treatment ponds into disposal ponds.

Because there is uncertainty in land availability and costs, legal interpretation of what constitutes lawful condemnation of land, shallow groundwater data, and regulatory interpretation of policy, the following facilities plan is based on a phased approach to facilities expansion with opportunity for the City to evaluate alternatives at each stage as more information is available and application of Regional Board policies and requirements are elucidated for this site. It is possible that a substantially more costly conceptual facilities plan will result from the needed groundwater studies, regulatory negotiations, and information on land availability (considering soil type, groundwater quality, cost, and need for condemnation). Based on the foregoing, the best apparent plan for planning purposes entails the following specific elements:

1. Eliminate all regulatory uncertainty as to whether the existing unlined treatment ponds constitute BPTC under Resolution No. 68-16 in light of the probability that the treatment ponds are degrading immediately underlying groundwater to some extent and therefore line the existing treatment ponds for existing residents.

2. Expand treatment in new lined treatment ponds as necessary.

3. Purchase additional lands for additional effluent disposal purposes.

Section 7

Collection System Capital Improvement Alternatives

Section 7

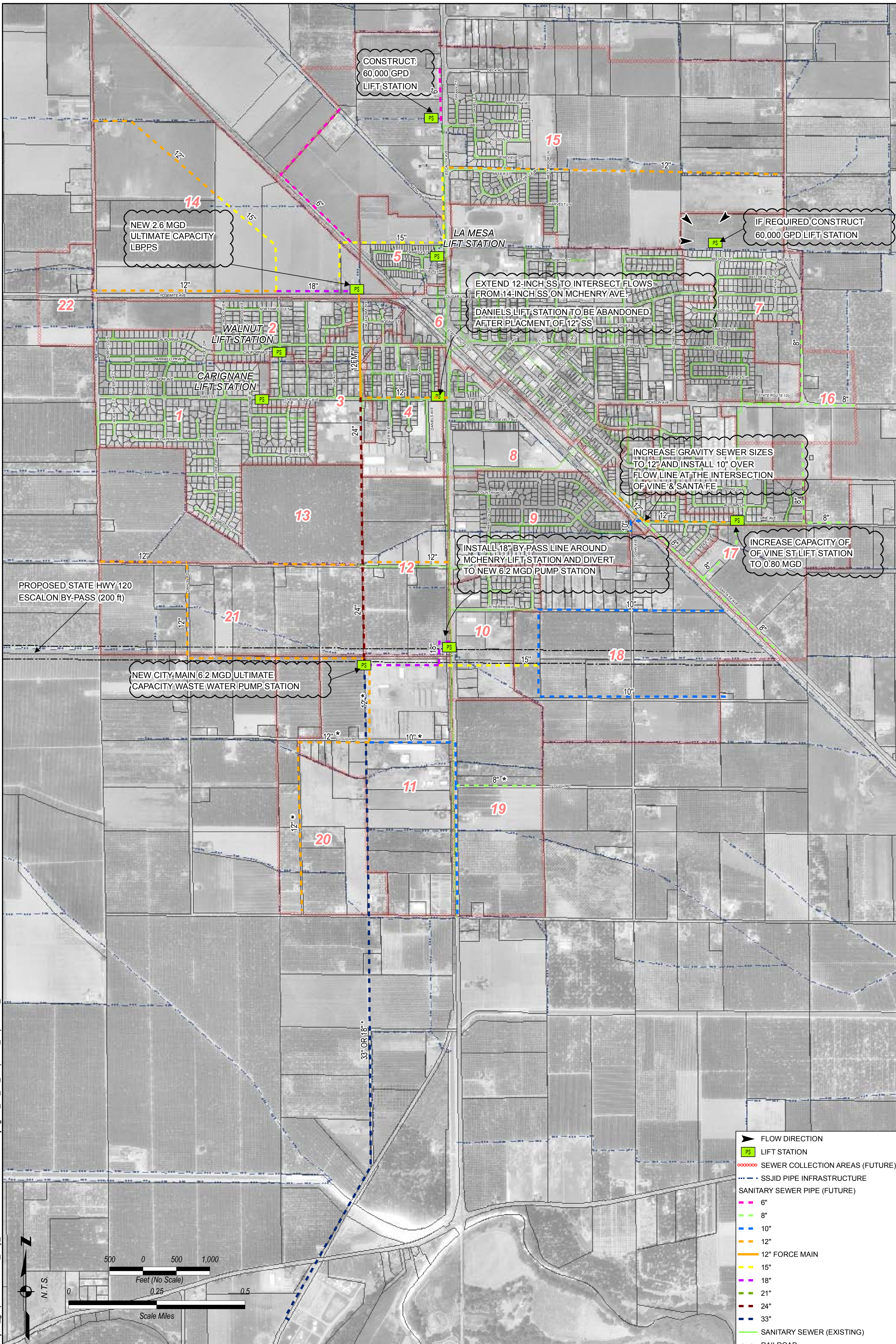
Collection System Capital Improvement Alternatives

The expansion of the existing sewer system to serve currently undeveloped areas can be achieved through various combinations of sewer trunk line extensions, lift stations, pump stations and sewer force mains, and other sewer improvements. The phasing of the required expansion is dependent on the location and timing of future development within future City growth boundaries. Given the location of the existing wastewater treatment plant (WWTP), a large portion of the current and future flow will require conveyance through existing and/or future lift stations and gravity trunks. Wastewater cannot economically be conveyed by gravity from the City to the WWTP due to the flat topography.

The locations and sizes of the required lift stations will determine pipe routing. Within this Master Plan Update, the recommended system expansion has been approached with an attempt to minimize the need for pumping wastewater, instead opting for the alternative to divert flows through gravity trunk lines whenever possible. The analysis relies on USGS digital elevation maps and City records showing existing manhole invert elevations. This approach of conveying wastewater through gravity mains reduces the need for O&M and provides a more reliable system than one with lift stations scattered throughout the City. Each alternative concept developed in this Master Plan Update will require further pre-design and planning based on the results of field surveys of the proposed alignments and incorporating project specific requirements including constructability and possible major utility conflicts. Figure 7-1 depicts the proposed future system improvements to extend service to future sewer drainage basins within the 2035 growth boundary. This section describes the methodology used to develop the proposed improvements. Also, each improvement is described with an opinion of probable cost and recommended phasing.

7.1.1 Near-Term Improvements for Service Within Existing City Limits

The majority of the gravity sewers within the City are adequate to handle the current level of development plus the undeveloped infill within the current City limits. However, after reviewing the Escalon Sanitary District Sewer Plan and Profiles for the 14-inch trunk line on McHenry Avenue, it appears that this facility does not have any excess available capacity. Therefore, additional flows from infill areas within this sewer drainage basin must be conveyed through new facilities. City staff has expressed concern that expanding this interceptor in its current location would be problematic due to the high traffic density along McHenry Avenue in conjunction with the existence of numerous other utility conflicts. Therefore, it is recommended that the City construct the proposed new 24-inch interceptor, 33-inch trunk main and new City Main Lift Station located about 1,400 feet west of the existing 14-inch McHenry Ave. as soon as practicable and gradually divert flows from the existing 14-inch interceptor to these facilities.



U:project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/Figure_07_01_Proposed_Expansion_NEW.ai 1/2/2007 cms

*These lines may need to be rerouted if the option to install a 33" SS to convey flows to the WWTP is used. The alignment depicted is for the option to convey flows through an 18" forcemain.

Figure 7-1
Proposed Wastewater Collection System Expansion

The new proposed interceptor and trunk main would ultimately include approximately 5,000 feet of 24-inch gravity sewer from the intersection of First Street and Oklahoma Avenue to a new "City Main Lift Station" with an ultimate capacity of 6.2 MGD (see Section 7.2.1) and approximately 11,000 feet of 33-inch gravity sewer from the new Main Lift Station to the WWTP influent lift station (see Figure 7-1). The new interceptor would be designed to provide sufficient capacity for the 2035 projected flows and the proposed alignment would allow for its construction at a lesser slope allowing for better balance of sewer depth between its upstream and down stream inverts. When constructed, the proposed sewer interceptor would provide increased flexibility and available capacity for connection of down stream sewer drainage basins without the necessity for additional lift stations.

A phasing plan for construction of the new interceptor is discussed later on in this section. The most critical aspect of the new interceptor involves the immediate construction of the new Main Lift Station, approximately 11,000 feet of 33-inch gravity sewer to the WWTP and the diversion of flows from the McHenry Lift Station to the new Main Lift Station through approximately 1,400 feet of 18-inch gravity sewer. Sections of the interceptor located upstream of new Main Lift Station can be constructed at a later date as additional development occurs (i.e. Heritage Park Subdivision, Liberty Business Park Area).

7.1.2 NORTHERN CITY GROWTH AREA

Increased flows as a result of development of the Liberty Business Park and the rest of sewer drainage basin 14, in addition to flows from sewer drainage basins 5, 6, 15 and 23, could result in surcharge conditions in sections of the 14-inch gravity sewer up gradient of the McHenry Lift Station. Therefore it is proposed that the 24-inch portion of the new gravity sewer interceptor discussed above and shown in Figure 7-1, be constructed in conjunction with the Liberty Business Park development's connection to the City's sewer system.

In addition to the 24-inch section, the connection of the Liberty Business Park will require approximately 600 feet of 21-inch gravity sewer and 1,200 feet of 12-inch force main as part of the construction of a new 2.6 MGD capacity Liberty Business Park Pump Station (LBPPS) which is described below.

As shown in the future sewer drainage basin layout in Figure 4-3, flows from sewer drainage basins 5, 15 and 23 are proposed to converge at the La Mesa Lift Station. Diversion of flow originating from the northern sewer drainage basins to the new gravity interceptor can be achieved by diverting flows from the La Mesa Lift Station through a new 15-inch gravity sewer to the new proposed 2.6 MGD LBPPS (see figure 7-1). By diverting flows away from the La Mesa Lift Station and towards the LBPPS, downstream capacity in the 14-inch sewer main can be made available for development within the sewer drainage basin located east of McHenry Avenue. This will avoid the need to construct a larger capacity sewer through the congested McHenry Avenue/Escalon Avenue intersection with SR 120.

The combined ultimate flow within sewer drainage basins 6 through 10, 16 and 17 is estimated to be 1.90 MGD. As discussed previously in Section 5, the maximum capacity in the 14-inch gravity sewer main between First St. and the McHenry Lift Station is approximately 0.78 MGD. In order to free up capacity in the existing 14-inch gravity sewer upstream of the McHenry Lift Station, two new 12-inch gravity sewer mains are proposed. One 1,400-foot 12-inch section is proposed along First St., which will intercept and divert flows from the 14-inch sewer main to the new 24-inch gravity interceptor along Oklahoma Avenue (Figure 7-1). The second proposed 12-inch gravity sewer main is also approximately 1,400-feet in length an will collect flows from the sewer drainage basin 8 existing 12-inch connection south of its point of connection to the 14-inch McHenry Ave. main at Ullrey Ave. Both sewers constructed at the City Standard minimum slope would have a diversion capacity from the 14-inch line of approximately 1.7 MGD at 70 percent full.

7.1.3 EASTERN CITY GROWTH AREA

Flows from the eastern most sewer drainage basins will be conveyed through the Vine Street Lift Station. Currently, flows from the Vine Street Lift Station, and other gravity sewers, are conveyed to the 14-inch McHenry Ave. interceptor through a combination of 8-, 10- and 12-inch gravity sewer lines long Vine Street and Santa Fe Avenue. The current maximum capacity of these lines is approximately 340,000, 700,000, and 730,000 gpd respectively. Once the eastern most sewer drainage basins have been fully developed, the required sewer capacity will be over 900,000 gpd. Therefore, it is recommended that the City increase the capacity of the sewer in this area to 700,000 gpd by replacing the existing 8- and 10-inch sections with 12-inch sewers or construct parallel sewers to achieve the same capacity, and divert a portion of the flow through the manhole located on the Vine Street and Santa Fe Avenue intersection to a new 10-inch overflow sewer to a manhole located at the Countrywood Lane and St. Johns Street intersection.

Currently, the maximum capacity through the Countrywood/Sophie Lane /Crestwood Drive/Swanson Drive/Ullrey Avenue 10-inch collector is approximately 630,000 gpd. Once all of drainage basin 9 has been developed, approximately 226,800 gpd must be conveyed through this sewer, hence a maximum of about 400,000 gpd could be diverted through this sewer, which is sufficient to meet the additional 200,000 gpd, required at the Vine Street Santa Fe Avenue manhole.

7.1.4 SOUTHERN CITY GROWTH AREAS

Future growth areas in the southern part of the City will be provided sewer service through new facilities constructed to convey flows to the new Main Lift Station or to the proposed 33-inch sewer trunk main. Portions of the existing 14-in may be used to collect wastewater locally, but this line will likely be abandoned as the main trunk line to the WWTP. In order to collect wastewater in these areas new 10-inch and 12-inch sewers would be constructed as shown in Figure 7-1 with connections to the 33-inch sewer or Main Lift Station.

7.2 SEWER LIFT STATION IMPROVEMENT ALTERNATIVES

As mentioned previously, this effort has focused on limiting the number of future lift stations within the City. In concert with this goal, future recommended lift stations have been located to serve future and existing sewer shed areas and, in some cases, replace existing lift stations that, due to their size and location, could not adequately convey future flows. One new major lift station (which replaces an existing lift station) and one major pump station, with an associated force main, are recommended to convey flows from within the proposed City growth boundaries. The locations of both stations are shown in Figure 7-1.

Because the existing lift stations serve to literally 'lift' wastewater to a higher elevation manhole, the potential for sewer overflows due to power outages can be mitigated through the construction of overflow lines which convey the surcharged flows from the lift station wet well to the downstream manhole to which flows are normally pumped. This option is not available for the LBPPS or the Main Lift Station force main alterative, which include long force mains. The option to install overflow lines in lift stations would have to be evaluated for each lift station, including evaluating whether the surcharged lift station would flood upstream homes or businesses.

7.2.1 MCHENRY LIFT STATION AND NEW CITY MAIN LIFT STATION

A new lift station is proposed located approximately 1,400 feet west of the existing McHenry Lift Station and would replace the McHenry Lift Station as the last major lift station within the City prior to conveying flow to the WWTP. According to USGS digital elevation information, the ground elevation of this lift station site would be approximately five feet lower than the McHenry Lift station, and hence, if elevation conditions allow, flow may be diverted from the McHenry Lift station by gravity. The exact location of the recommended lift station will be contingent upon a detailed site survey and analysis of the alignment of the new 33-inch gravity sewer trunk that would convey flow to the WWTP's influent lift station.

An alternative to constructing this facility as a lift station is to construct a sewage pump station with associated force main. This alternative is shown on Figure 7-1 and would consist of constructing the sewer pump station with an 18-in diameter force main to the WWTP. If this alternative is pursued, sewer flow from the southern part of the City (south of the future SR 120 right of way) would have to be conveyed north to the new City Main Pump Station.

7.2.2 LIBERTY BUSINESS PARK PUMP STATION (LBPPS)

A new pump station is proposed located south of the future Liberty Business Park along the northern edge of SR 120. This pump station will convey flows from the Liberty Business Park and, in the future, from diverted flows from sewer drainage basins 15, 23, and parts of 5. As shown in Figure 7-1, flows from the pump station would be conveyed through a 12-inch force main running east along SR 120 and then south along Oklahoma Avenue to a new manhole located just south of the SSJID irrigation lateral. At this point the force main can change to a gravity sewer or continue as a force main to First Street.

7.2.3 ADDITIONAL MINOR LIFT STATIONS

Two additional approximately 60,000 gpd lift stations may be required to convey future flows from specific development areas as shown on Figure 7-1. One of these lift stations may be required for development of the northern most undeveloped parcel on sewer drainage basin 7 in order to convey flow into the existing 10-inch collector sewer along Justin Drive. A second lift station may be required to convey wastewater from development of the northern most parcel of sewer drainage basin 5 to the existing 10-inch collector sewer in Escalon Avenue.

7.2.4 VINE STREET LIFT STATION

The Vine Street Lift Station is currently located near the Vine Street and Brook Street intersection. It is proposed that, in addition to the current sewer drainage basin 7 flows, future development within sewer drainage basins 16 and 17 be discharged into the existing collection system along Vine Street through the Vine Street Lift Station. While sufficient wet well capacity may be available in the lift station, a thorough evaluation of the facility should be performed to determine its potential for flooding within the dry pit, which houses the mechanical equipment. The existing location of the dry pit is adjacent to a residential area, and it is currently susceptible to flooding during rain events and from landscape irrigation. In order to facilitate the O&M of this lift station, it is recommended that the existing dry pit be relocated to the southern edge of Vine Street when future modifications to the wet well are considered. At this time, the City may elect to redesign the Vine Street Lift Station to incorporate submersible sewage pumps with above ground controls, which have the added benefit of allowing for a more compact design and eliminate the added O&M associated with repairing damage due to flooding of the existing dry pit.

7.2.5 EXISTING LIFT STATION DECOMMISSIONING

In addition to conveying additional flows, the recommended new lift stations within the City will also allow for the decommissioning of the Daniels Lift Station, and the McHenry Lift Station.

Based on the USGS digital elevation information, in conjunction with City records of sewer invert elevations, flows from the Daniels Lift Station can be conveyed by gravity to the new Main Lift Station through the proposed First Street 12-inch and 24-inch interceptors. Ultimately, flow going to the McHenry Lift Station site can be conveyed by gravity through a new 18-inch trunk line along the southern edge of the future HWY 120 right of way to the new Main Lift Station located near the intersection of the HWY120 right of way with the Oklahoma Avenue extension. At this point, the domestic portion of the McHenry Lift Station can be decommissioned.

7.2.6 ALTERNATIVE LIFT STATION CONVEYANCE SYSTEM

An alternative for relieving capacity in the 14-inch line on McHenry Avenue, in lieu of constructing a new 12-inch gravity interceptor along First St., would be to modify the Daniels Lift Station to intercept flows from sewer drainage basins 4 and 6 and pump through a new force main west towards the new 24-inch interceptor. Future influent flows to the Daniels Lift Station will require that the current pumps, valves and other appurtenances be up-sized in order to effectively convey these flows.

Similar to the option of conveying Daniels Lift Station flows away from the 14-in interceptor, another option for diverting flows from sewer drainage basins 5, 15, and 23 around the 14-inch line on McHenry Avenue would be to modify the La Mesa Lift Station to intercept flows from these areas and pump them to towards the new LBPPS. The additional influent flows to the La Mesa Lift Station will require that the current pumps, valves and other appurtenances be up sized in order to effectively convey the flows.

These options would allow for the placement of smaller diameter sewers to convey the flows along First Street and from the La Mesa Lift Station to the LBPPS. However, the increase in equipment and O&M costs, in addition to system complexity associated with these options, may offset any benefits these options would provide over the construction of larger diameter sewer interceptors.

7.3 Proposed Sewer System Expansion And Improvement Plan

This section summarizes the proposed sewer improvement plan based on the above analysis of improvement plan options and alternatives. The most significant characteristics reviewed for selection between improvement options were relative construction cost and facilities staging. Other characteristics, including operation and maintenance (O&M) costs, dependability, reliability, and environmental impacts were important as discussed above.

7.3.1 Sewer Collector Main Expansion and Improvement Plan

The recommended sewer improvement plan for increasing sewer system capacity for future development is contained in Table 7-1. Table 7-1 is divided into for areas based on the relative timing and need for the improvements. These areas are:

- Near-Term improvements to the existing sewer system to provide capacity for immediate development.
- Improvements needed to extend service to the Heritage Park and Liberty Business Park areas.
- Future improvements to the existing system to allow conveyance of future flows through existing facilities (improvements in addition to the “Near-Term improvements”).
- Sewer system expansion improvements, which consist of new sewer lines to extend service to currently unsewered areas within the City’s growth boundaries.

Near-Term Improvements: The following improvements listed in Table 7-1 are recommended prior to the connection of additional development to the existing collection system and are sized to serve existing and future drainage basins:

- 18-inch connector from McHenry Lift Station to new Main Lift Station;
- New Main Lift Station Phase I to replace McHenry Lift Station capacity and provide additional capacity for a portion of anticipated future flows;
- 33-inch gravity sewer trunk main to WWTP.

Improvements to be constructed as part of the Heritage Park and Liberty Business Park Developments: Currently the Heritage Park Subdivision and the North Industrial Park developments are the largest approved or upcoming developments to be incorporated in to the City's collection system. Connection of the Liberty Business Park development will require more extensive improvements given the difficulty in conveying flows from its location in the northwestern portion of the City, relative to the location of the WWTP. However, as Figure 7-1 shows, a portion of the necessary improvements are also required for connection of the Heritage Park Subdivision to the collection system. These improvements will ultimately serve drainage basins 1-8, 14-17, 22 and 23.

Improvements within existing City System to Serve Future Developments: These additional improvements are recommended to the existing collection system facilities to increase capacity as currently undeveloped areas within the City limits become developed and as additional sewer drainage basins are connected to the system.

Sewer System Expansion Improvements: These improvements are recommended as a guide for expanding the sewer system to serve areas within the future growth boundaries.

7.3.2 SEWER PUMP STATION IMPROVEMENT PLAN

The recommended improvements plan contained in Table 7-1 includes the abandonment of the domestic portion of the McHenry Lift Station and the construction of a new 6.2 MGD ultimate capacity Main Lift Station located approximately 1,400 feet due west of the current location of the McHenry Lift Station. This lift station would serve all sewer drainage basins currently served by the McHenry Lift Station and all new sewer drainage basins located to the north and east of the City limits. The first phase of the new Main Lift Station is to be incorporated as part of the Near-Term improvements with a capacity of approximately 3.1 MGD. Subsequent phases of this lift station could be constructed for 4.1 MGD and 6.2 MGD ultimate capacities each.

In order to provide sewer service to the Liberty Business Park area, a new 2.6 MGD ultimate capacity sewer pump station is recommended. This pump station is intended to serve as a central collection point for flows originating from sewer drainage basins 5, 14, 15, 22 and 23. It is recommended that this pump station be constructed as part of the North Industrial Park Development, with the pump station structure and force main sized to accommodate up to 2.6 MGD. This pump station can be phased, with the first phase consisting of wet well and piping sized for ultimate capacity, but pumping and electrical equipment provided for 1.3 MGD capacity.

It is recommended that the existing Vine St. Lift Station be upgraded in order to serve drainage basins 7, 16 and 17. Its current location makes its dry pit prone to flooding causing the pump motors to burnout. It is therefore recommended that the lift station be eventually abandoned and replaced in the future with a more compact submersible pump lift station.

Two additional approximately 60,000 gpd lift stations are recommended in order to serve portions of sewer drainage basin 5 and 7 respectively. These lift stations would be constructed to serve new development in these sewer drainage basins.

As shown in Figure 7-1, a new 12-inch gravity sewer is recommended along First St. connecting the 14-inch gravity sewer along McHenry Ave. to the new 24-inch gravity collection main along Oklahoma Ave. As part of the installation of the 12-inch gravity sewer, it is recommended that the City abandon in place the Daniels Lift Station and that the 12-inch line along First St. intercept all flows currently going to the Daniels Lift Station.

Table 7-1 summarizes the phasing plan and estimated cost for implementing the collection system expansion. The costs in Table 7-1 are based on the preferred alternative of constructing the Main Lift Station with a gravity sewer line to the WWTP. The difference in probable costs between this preferred alternative and the force main alternative is approximately $1.1 million, i.e., the pump station and force main capital cost project may be $1.1 million less to construct. However, the operating costs of the force main project are anticipated to exceed the cost of list station option by about $45,000 per year (assuming an electricity cost of $0.10/kWh). The phasing plan is organized according to which options are deemed to be most critical in the expansion of the collection system. The estimated costs are based on 2006 dollars at an ENR CCI for July 2006 of 7721.

Table 7-1
Proposed Sewer System Improvements

Improvement	When Needed	Estimated Cost
Near-Term Improvements		
Gravity sewer 18-inch minimum diameter 1,400-foot length along future HWY 120 bypass east of McHenry Lift Station	Prior to connecting additional developments to existing collection system	$330,000
Construct Phase I of new 6.2 MGD Ultimate Capacity City Main Lift Station to replace existing McHenry Lift Station (Phase I at 3.1 MGD)	Prior to connecting additional developments to existing collection system	$740,000
Construct 9,000-foot 33-inch minimum diameter gravity sewer from new City Main Lift Station to the Escalon WWTP	Prior to connecting additional developments to existing collection system	$3,400,000
Subtotal		**$4,470,000**
Contingency (25%)		$1,120,000
Engineering/Administration (25%)		$1,120,000
Total		**$6,710,000**

Improvement	When Needed	Estimated Cost
Improvements for Heritage Park Development		
Install 1,400-foot 12-inch minimum gravity sewer west-east along Ullrey Av. and 1,400 24-inch minimum diameter gravity sewer north-south along Oklahoma St. extension towards new 6.2 MGD Main Lift Station	During construction of Heritage Park Phase 1	$650,000
Subtotal		**$650,000**
Contingency (25%)		$163,000
Engineering/Administration (25%)		$163,000
Total		**$976,000**
Improvements for Liberty Business Park Developments		
1,900-foot 12-inch and 1,200 15-inch minimum diameter gravity sewers within the North Industrial Park	During construction of 178 acre development	$520,000
1,200-foot 18-inch gravity sewer along SR 120 south of North Industrial Park development	During construction of 178 acre development	$380,000
Construct Phase I of 2.6 MGD Ultimate Capacity North Industrial Park Pump Station (Phase I at 1.3 MGD)	During construction of 178 acre development	$550,000
Install 1,700-foot 12-inch minimum diameter force main from LBPPS to new manhole located on Oklahoma St. and south of SSJID pipe	During construction of 178 acre development	$340,000
Install 2,500-foot 24-inch minimum diameter gravity sewers south along Oklahoma St. and its extension thru future Heritage Park development to drain at intersection of Oklahoma St. extension and Ullrey Ave.	During construction of 178 acre development	$870,000
Subtotal		**$2,660,000**
Contingency (25%)		$665,000
Engineering/Administration (25%)		$665,000
Total		**$3,990,000**

Improvement	When Needed	Estimated Cost
Improvements for future Developments		
Install 1,400-foot 12-inch minimum gravity sewer along First St. from First/McHenry intersection to First/Oklahoma intersection	When flows in 14-inch McHenry Ave. gravity sewer upstream of First/McHenry intersection are above 0.65 MGD	$320,000
Install 3,000-foot 15-inch minimum gravity sewer from manhole on Escalon Ave/Miller St. intersection west through Hogan Park then south-west through Arthur Rd. and south along western property line of Escalon Covenant Church across the railroad tracks and south towards the LBPPS	When flows in 8-inch gravity sewer on Escalon Ave. south of the La Mesa Lift Station are above 0.38 MGD	$650,000
Replace existing 8-inch gravity sewer on Santa Fe Ave. between Santa Fe/Vine and Santa Fe/Franklin intersections with 12-inch minimum diameter gravity sewer	When flows upstream of manhole on Santa Fe/Franklin intersection are above 0.38 MGD	$90,000
Replace and increase pumping capacity of Vine St. Lift Station to 0.72 MGD	When inflows to lift station exceed current 0.5 MGD pumping capacity	$290,000
Replace existing 10-inch gravity sewer on Vine St. between Vine Lift Station and Santa Fe/Vine intersections with 12-inch minimum diameter gravity sewer	When flows upstream of manhole on Santa Fe/Vine intersection are above 0.65 MGD	$250,000
Install 10-inch overflow line connecting manhole on Santa Fe/Vine intersection with manhole on St John/Countrywood intersection	When flows upstream of manhole on Santa Fe/Vine intersection are above 0.65 MGD	$60,000
Install 1,400-foot 12-inch minimum gravity sewer along Ullrey from McHenry Ave to new 24-inch interceptor	This option is provided in order to divert flows from 14-inch McHenry Ave. gravity sewer	$240,000
Replace existing 8-inch gravity sewer on Miller with 1,900-foot 12-inch minimum diameter gravity sewer	When flows upstream of manhole at Miller/Westbrook intersection are above 0.38 MGD	$330,000
Install 2,100-foot 8-inch minimum gravity sewer from E. Clough/McHenry Ave. intersection north along McHenry Ave. and west along Clough Rd.	As a result of development on Drainage Basin 19	$240,000
Subtotal		**$2,470,000**
Contingency (25%)		$620,000
Engineering/Administration (25%)		$620,000
Total		**$3,710,000**

Improvement	When Needed	Estimated Cost
Sewer System Expansion Improvements		
Construct Phase II of new 6.2 MGD Ultimate Capacity Main Lift Station (Phase II at 4.1 MGD)	When additional flows to the Lift Station will increase the peak flow above 3.1 MGD	$540,000
Installation of 1,300-foot 10-inch minimum diameter gravity sewer, 4,100-foot 12-inch minimum diameter gravity sewer from Brennan Ave. east along Ullrey, south along Dahlin Rd., and east along Clough Rd.	After completion of Heritage Park development Phase 3 or any other development south east of Heritage Park development	$990,000
Installation of 2,700-foot 12-inch minimum diameter gravity sewer from Brennan Ave. east along SR 120 to upstream invert to 18-inch gravity sewer placed as part of the NIP development	As a result of any development south-west of NIP development	$560,000
Phased easterly extension of up to 3,200-foot 12-inch gravity sewer along Miller Ave.	As a result of any development located north-east of existing City limits	$450,000
Construct Phase II of LBPPS 2.6 MGD Ultimate Capacity Pump Station (Phase II at 1.7 MGD)	When additional flows to the Pump Station will increase the peak flow above 1.3 MGD	$380,000
Installation of 1,400-foot 15-inch minimum diameter gravity sewer through drainage basin 10 along the southern edge of the future HWY 120 by-pass and east of McHenry Lift Station.	As a result of development east of Drainage Basin 10	$450,000
Installation of 3,300-foot 10-inch minimum diameter gravity sewer along Catherine way and south towards future HWY 120 by-pass	As a result of development within the northern area of Drainage Basin 18	$460,000
Installation of 3,300-foot 10-inch minimum diameter gravity sewer along Narcissus Rd. and North towards future HWY 120 by-pass	As a result of development within the southern area of Drainage Basin 18	$450,000
Installation of 2,800-foot 8-inch minimum diameter gravity sewer along Main St. and north-west towards Vine Lift Station	As a result of development on the south-east corner of Drainage Basin 17	$340,000
Installation of 1,700-foot 8-inch minimum diameter gravity sewer next to the railroad along the north-east side of the future north industrial park	As a result of development on the western area of Drainage Basin 15	$180,000
Increase Pumping capacity of new City Main Lift Station to 6.2 MGD (Phase III)	When additional flows to the Lift Station will increase the peak flow above 4.1 MGD	$51,000

Improvement	When Needed	Estimated Cost
Installation of 3,600-foot 12-inch minimum diameter gravity sewer from the Clough/Ellis intersection south along Ellis, east along Jones Ave. towards 33-inch SS	As a result of development on drainage basin 20	$590,000
Increase Pumping capacity of LBPPS to 2.6 MGD (Phase III)	When additional flows to the Pump Station will increase the peak flow above 1.7 MGD	$30,000
Construction of new 60,000 gpd lift station and force main to discharge at manhole at intersection of Escalon Ave./Libby Rd intersection	As a result of development of the northern portion of Drainage Basin 5	$200,000
Subtotal		**$5,671,000**
Contingency (25%)		$1,420,000
Engineering/Administration (25%)		$1,420,000
Total		**$8,510,000**

Total Collection System Improvements Capital Costs	**$23,896,000**

Section 8

Wastewater Treatment System Capital Improvement Alternatives

Section 8

Wastewater Treatment System Capital Improvement Alternatives

8.1 INTRODUCTION

Future treatment and disposal method and capacity depend upon Regional Board actions with respect to groundwater quality underlying and down gradient of the site, groundwater quality with regards to constituents not currently analyzed, and methods of future land acquisition, as discussed in Section 6. Two alternatives of treatment are considered in this section, 1) use and expansion of current aerated pond treatment, and 2) activated sludge treatment, both with continued and expanded use of rapid infiltration disposal. These alternatives provide flexibility to plan for future capacity in response to an uncertain regulatory environment where source reduction and localized degradation may be acceptable or could require further mitigation and where land acquisition must be weighed against other treatment alternatives.

Common improvements are required for the headworks and influent pump station under both treatment alternatives. Rather than include in-kind upgrades in each alternative, these improvements will be discussed in Section 8.5 of this chapter. Disinfection has not been included in either alternative, as current requirements for land disposal facilities with adequate separation between the bottom of disposal ponds and seasonal high groundwater do not include disinfection.

In both alternatives, disposal will be accomplished through rapid infiltration. Thus, disposal capacity is dependent upon long term reliable percolation capacity of the disposal ponds. Limited site specific analysis of disposal capacity was conducted and presented in the 1990 Master Plan (Table 5-3), where disposal capacity of the ponds ranged from 46,000 to 180,000 gal/ac/d and a long term average of 66,000 gal/ac/d was used for planning purposes. As a method of maintaining these disposal rates in the treatment ponds, the ponds were to be taken off line each year and cleaned and disked. The long term inundation and sludge build-up in treatment ponds are known to reduce percolation rates to those of synthetically lined ponds (EPA, October 1983, Design Manual: Municipal Wastewater Stabilization Ponds). Based on operations records and observations, it appears the treatment ponds still provide some percolation disposal. If disposal from these facilities is to be included in the disposal program, actual percolation rates of the treatment and disposal ponds should be assessed through a hydrogeologic investigation and used to determine reliable long term disposal capacities.

In lieu of conducting a hydrogeologic investigation of the disposal ponds, records from 2005 (a relatively wet year) were used to create a hydraulic balance to estimate percolation rates of the treatment and percolation ponds (Appendix B). In order to determine percolation rates, it was assumed that all wastewater in the percolation ponds infiltrated during that month, and that the peak monthly influent volumes are representative of the percolation pond capacity during the winter of 2005. The percolation pond weighted average percolation rate for 2005 was 58,750 gal/ac/d. Based on this percolation rate the treatment ponds provided percolation disposal of about 16,830 gal/ac/d. Disposal capacity at various phases was assessed using these percolation rates. A brief summary of the 2005 hydraulic balance and the 1 in 100 year hydraulic balance under current conditions are presented in Table 8-1.

Table 8-1
Summary of Hydraulic Balances for 2005 and the Maximum Flow Under Existing Conditions During a 1- in 100-Year Precipitation Year

Parameter	2005	1- in 100-Year Precipitation
Average Dry Weather Flow (MGD)	0.60	1.16
Percolation Pond, Percolation Rate (gal/ac/d)	58,750	58,750
Treatment Pond, Percolation Rate (gal/ac/d)[a]	16,830	16,830
Annual Inflow (Mgal)		
Wastewater	215	425
Inflow and Infiltration	8	18
Direct Precipitation	6	17
Total	**229**	**460**
Annual Outflow (Mgal)		
Percolation	197	416
Evaporation	32	44
Total	229	460
Inflow – Outflow Balance (Mgal)	**0**	**0**

[a] Based on net disposal for 2005.

Future phases and disposal capacities were assessed using 1-in-100 year precipitation season hydraulic balances (Appendix B). Average precipitation was multiplied by a factor of 1.81 to estimate 1-in-100 year precipitation amounts based on monthly average precipitation. Average evaporation was reduced by a factor of 0.88 in the wet season (October through April) and by 0.95 during the dry season, to account for the depression in evaporation experienced during wet years.

Aerators are known to increase evaporation from treatment ponds, and estimates range from 0.2 to 0.5 equivalent acres of evaporative surface per aerator horsepower. In order to acknowledge this effect, a conservative estimate of 0.1 acres per aerator horsepower was used to estimate enhanced evaporation due to aeration. As a condition of best practicable treatment and control (BPTC), it was assumed that all future treatment ponds would be lined and percolation was assumed to be zero from these facilities in the future. The hydraulic balances assumed no future decline in percolation rates for the disposal ponds, which is based on the assumption that the City will maintain a program of pond cycling, disking, and periodic deep ripping.

8.2 AERATED POND TREATMENT

Continued use of aerated ponds to meet the treatment requirements of future wastewater flows requires that adequate pond volume and aeration are present to reduce influent wastewater loads to a level suitable for rapid infiltration disposal. The treatment capacity of the pond system was evaluated using the Ten States Standard first order kinetic model for microbial metabolism of wastewater derived organic matter (as measured by the 5-day biochemical oxygen demand test, BOD_5). When final effluent BOD_5 was calculated to be less than or equal to 30 mg/L and total BOD_5 reduction was greater than 85 percent, treatment capacity was considered adequate for the existing disposal method. Pond systems are capable of attenuating flows and therefore effluent quality is less influenced by short duration peak flows. As such, the ponds were evaluated on a peak month flow basis using a peaking factor of 1.3. This peaking factor is slightly conservative, as the 2005 data indicates a peak month flow of 1.2 times the average dry weather flow (ADWF). Existing treatment ponds have sufficient volume and aeration to accommodate average dry weather flows up to approximately 1.23 MGD with minor modifications including relocating aeration and redistributing the flows to the ponds in the order listed in Table 8-2. The existing facility is depicted in Figure 5-1.

Table 8-2
Brush Aerator Requirements for Each Treatment Pond, Based on Average Dry Weather Flows

Designation[a]	Aerator Name Plate Horse Power		
	Current[b] (ADWF 1.2 MGD)	Phase I (ADWF 1.8 MGD)	Phase II (ADWF 2.8 MGD)
Pond 9	65	80	80
Pond 6	25	30	30
Pond 5	15	20	25
Pond 7	15	20	30
Pond 8	10	20	30
Pond 21[c]	Disposal	20	35
Pond 22[c]	Disposal	Disposal	35

[a] Ponds are listed in proposed series operation (i.e. Pond 9 is to be the initial pond to discharge into pond 6, etc.)

[b] By redistributing existing aeration and reconfiguring treatment pond order.

[c] Pond 21 and 22 are currently disposal ponds and are proposed to be converted to treatment ponds in order to increase treatment capacity.

The operation of a pond system in series increases the systems treatment capacity to reduce BOD of wastewater origins; however, increased loading in the initial pond will cause increased oxygen requirements. In response to increased loading to the first pond, aeration is added. As the increased aeration has the potential to completely mix the system at about 15 to 30 Hp per million gallon (MG) of volume, the kinetics of the treatment process change creating a system where the mechanical supply of oxygen does not meet the oxygen demand of the completely mixed system. The wastewater treatment plant should be plumbed to split the influent between multiple ponds and/or to direct inflow initially to the largest treatment pond (#9) to minimize problems from future peak loads. The inter pond piping should be modified to allow isolation of single ponds for maintenance without disrupting flow through the remaining ponds. A depiction of the proposed interpond piping is presented in Figure 8-1. Future treatment capacity phases are assessed by having pond 9 as the initial treatment pond. Aeration supply increases will need to occur periodically as flows and loads increase. The variability in flows, loads, seasonal temperature changes, and the effect of mixing with added aeration will affect the oxygen demand of the system and the timing of aerator addition will depend on pond performance under these conditions. Estimated aeration demands using brush style aerators are presented in Table 8-2. Actual requirements will be based on the manufacturer's specifications and actual performance of the models and types of aerators used as well as site specific aeration needs depending on how flows are split.

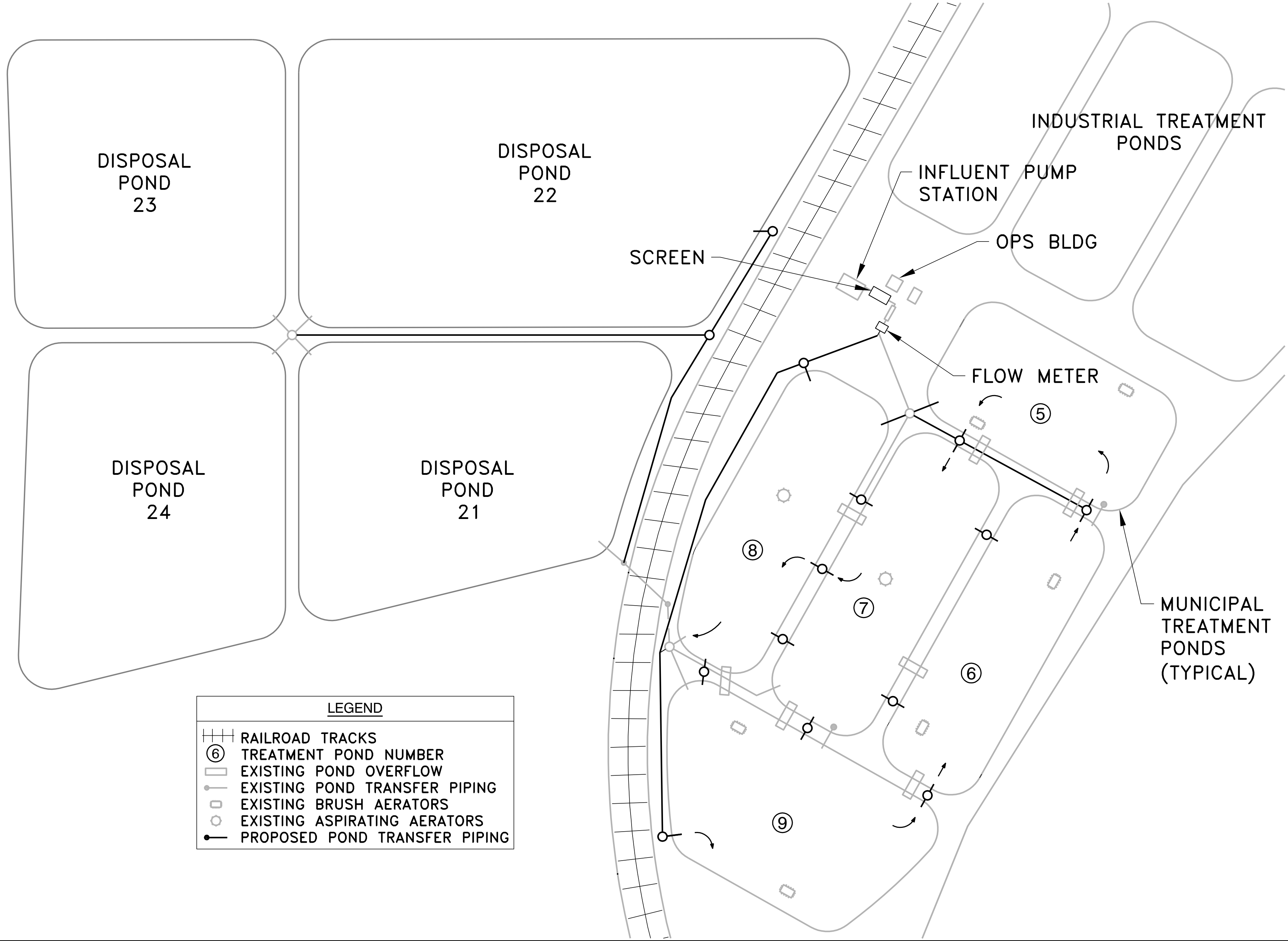
Three major expansion phases based on pond treatment and disposal are proposed to accommodate wastewater treatment and disposal needs to the 2035 growth boundary. The following sections discuss these phases individually and their individual requirements. Treatment and disposal facilities are proposed to increase in even increments based on the base capacity of existing pond structures and through even incremental increases in disposal facilities. The phasing of treatment ponds as described in the 1990 Master Plan was continued in this update to take advantage of previous pond construction and subsequent deepening and to limit the depth of future pond excavations.

8.2.1 PHASE I: 1.8 MGD TREATMENT AND 1.5 MGD DISPOSAL CAPACITY

As shown in Table 8-1, the maximum disposal capacity of the current facility is approximately 1.16 MGD. Further, the treatment capacity using the five treatment ponds (#5 through 9) will not be adequate during the winter once flows reach about 1.23 MGD. Once the plant reaches 0.9 MGD, construction of a new disposal area should be initiated to increase disposal capacity. The required percolating surface, effective area, of the disposal ponds is about 10 acres. The new disposal area should consist of a minimum of two ponds and the ponds should be 5 ft deep to provide an effective depth of 3 ft (allowing for two feet of freeboard). Construction of the new disposal area as multiple ponds provides the benefit of cycling ponds and allows routine maintenance, which maintains and/or improves percolation rates and thus disposal capacity.

Construction of the disposal ponds will provide the necessary additional disposal capacity to allow the conversion of pond 21 to a treatment pond, which will be required prior to the average dry weather flow exceeding 1.23 MGD. The total depth of pond 21 should be a minimum of 11 ft (9 ft of effective depth) to provide adequate volume for treatment. Conversion of pond 21 to a treatment pond will likely require the pond to be lined. Aeration in pond 21 should be staged as flows and loads increase to a total of 20 Hp to allow treatment of up to 1.8 MGD.

ECO:LOGIC
Consulting Engineers



U:project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/Figure_08_01_Proposed_Pond_Treatment_Piping.ai 10/24/06 cms

Figure 8-1
Proposed Pond Treatment Piping and Flow Path

Phase I Aerated Treatment Pond Improvements Consist of the following:

- Inter pond piping modifications for Treatment Ponds 5 through 9.
- Additional aeration in Treatment Ponds 5 through 9.
- Rearrangement of existing aeration in Treatment Ponds 5 through 9.
- Addition of a minimum of 10 acres (effective percolation area) of disposal ponds.
- Conversion of Pond 21 to Treatment Pond 21

Depending on the results of expanded groundwater monitoring and negotiations with the Regional Board during the permitting of this Phase I expansion, lining of the existing treatment ponds will likely be required, which has been included in the facilities cost. The cost to line the existing treatment ponds, excluding sludge removal and disposal, is approximately $1,500,000. If lining of treatment ponds is necessary, the percolation disposal capacity of the treatment ponds will be lost and an additional 0.5 acres of effective disposal area should be added to each new 5 acre disposal pond in each phase.

Further, the City's current Waste Discharge Requirements (Order No. 5-00-142) permit the average dry weather flow (ADWF) of the facility at 0.9 MGD for the domestic WWTP. Permit renewal should be initiated once flows reach 80 percent of permitted capacity (0.72 MGD ADWF), therefore it is expected that pond lining and/or conversion to activated sludge will be pivotal issues in permitting the increased flows from near term City growth.

8.2.2 PHASE II: 2.8 MGD TREATMENT CAPACITY AND 2.4 MGD DISPOSAL CAPACITY

Once the flows to the plant reach 1.5 MGD, the disposal capacity of the Phase I expansion will be approached. Prior to reaching this average flow, new disposal ponds should be constructed consisting of another 10 acres of effective disposal area and a minimum of 3 ft of effective depth. The construction of these ponds will provide the necessary additional disposal capacity to allow the conversion of pond 22 to a treatment pond. Conversion of pond 22 to a treatment pond will be required prior to the average dry weather flow exceeding 1.82 MGD. Pond 22 will need to be lined and should be provided with an effective depth of 8 ft. The aeration of pond 21 and 22 should be increased incrementally up to 35 HP of aeration each over time.

Phase II Aerated Treatment Pond Improvements Consist of the following:

- Addition of a minimum of 10 acres (effective percolation area) of disposal ponds.
- Conversion of Pond 22 to Treatment Pond 22.
- Increasing Aeration in Treatment Pond 21 and all existing treatment ponds (5-9).

8.2.3 PHASE III: 2.8 MGD CAPACITY

Once the plant has reached 2.4 MGD the peak disposal capacity of the Phase II expansion will be reached. An additional 5 acres of effective disposal area will be required to increase disposal capacity for the projected 2035 growth boundary average flows of 2.8 MGD. Additional aeration should be added over time to Treatment Ponds 21 and 22 as flows increase.

Phase III Aerated Treatment Pond Improvements Consist of the following:

- Addition of a minimum of 5 acres (effective percolation area) of disposal ponds.
- Increasing Aeration in Treatment Ponds 5, 7, 8, 21 and 22.

8.3 ACTIVATED SLUDGE TREATMENT

It is possible that impaired groundwater quality and Best Practicable Treatment and Control (BPTC) for such constituents as nitrogen and BOD_5 (with respect to any evidence of mobilization of iron, manganese, or arsenic which is unknown at this time) result in the need for the City to convert to the more mechanically intensive treatment process of activated sludge. A possible program for phasing activated sludge treatment, with biological nitrogen removal, is described below as an alternative to expanding the pond treatment system. During permitting of the first phase of the pond expansion, the City will most likely evaluate the feasibility of this treatment method to continue to meet current wastewater disposal regulations and policies.

The conversion to activated sludge treatment will require an initial investment in new treatment works necessary to treat current flows. Approximately 3.8 acres will be required for the treatment works consisting of 3 to 4 treatment trains to accommodate ultimate flows (three 1.0 MGD trains or four 0.75 MGD trains). Setting the systems up in trains will allow for the phasing of the plant and allow for future development to finance the capacity required for the new development. Activated sludge treatment will require a more intensive sludge handling process as the capacity for long term storage of sludge in aerated treatment ponds would no longer be available.

This analysis is based on using extended aeration consisting of four aeration basins, each with its own clarifier. This alternative is contemplated utilizing existing facilities to the extent practicable. Based on limited influent wastewater characterization, alkalinity may need to be added to the process for proper operation. The use of an activated sludge treatment plant minimizes evaporation during treatment and therefore reduces the concentration of salts compared to pond treatment. As the disposal means would remain land disposal via rapid infiltration, secondary treatment is the appropriate level of treatment. The conversion to activated sludge treatment does not reduce the need for future disposal area expansions.

Based on background groundwater water quality with respect to nitrate, it is likely that the treatment will also require denitrification, e.g., as the Best Practicable Treatment and Control (BPTC) method for nitrogen reduction. Current monitoring of municipal wastewater only analyzes nitrate nitrogen and does not analyze for reduced forms of nitrogen, including total Kjeldahl nitrogen or ammonia nitrogen. The latter reduced forms are the primary nitrogen species in raw wastewater, with organic nitrogen typically being the dominant form in pond effluent. Similarly, the groundwater monitoring does not analyze for reduced forms of nitrogen, and the improved nature of the groundwater with respect to nitrate may not accurately characterize the actual total nitrogen impacts to the groundwater. Moreover, aerobic activated sludge processing is more effective than pond systems are in oxidizing reduced nitrogen forms to nitrate, and since wastewater is relatively elevated in total nitrogen, it is expected that the effluent from the treatment system would contain greater than 10 mg/L nitrate. Given that the existing background groundwater may be impaired with nitrogen and that there is limited knowledge of the soil treatment potential to remove nitrate at the site, it would be impractical to recommend an activated sludge treatment system without denitrification capabilities.

As many variations of the extended aeration activated sludge process exist, including various options to include denitrification, this facilities plan alternative is based on the following concepts:

- Traditional extended aeration activated sludge utilizing lined earthen basins
- Separate circular clarifiers with return and waste activated sludge pumping (RAS/WAS pumping).
- Denitrification provided in a separate earthen basin with mixing and recirculation from the aeration basin.

The conceptual layout of the activated treatment system is depicted in Appendix C. Other processes that could be considered during pre-design if this alternative is pursued could include; 1) Sequencing Batch Reactors (SBR), 2) Oxidation Ditch with de-nitrification, 3) traditional extended aeration in concrete basins. The cost for the proposed type of treatment system is considered to be average to high compared to other alternatives, but this is appropriate for the purpose of financial planning for facilities as many currently unknown factors may cause the cost of facilities to be more than anticipated.

8.3.1 PHASE I: 1.5 MGD ACTIVATED SLUDGE TREATMENT

Similar to aerated pond treatment, Phase I of the Activated Sludge Treatment alternative should be conducted prior to reaching an ADWF of 1.16 MGD. Depending on regulatory actions and consideration of BPTC measures for nitrogen, iron and manganese, and possible regulatory concern over groundwater contamination by pathogens from the unlined treatment ponds, the timing of this first phase may be more rapid than warranted by anticipated growth. The land requirements of the activated sludge treatment system are such that it could be situated to utilize pond 5 and portions of ponds 6, 7 and 8 and the area between pond 5 and the existing operations building. Prior to construction, this phase will require the demolition of both Imhoff tanks and construction of a new headworks facility. Further, the sludge will need to be removed from ponds 5 and 6, and the remaining treatment ponds will need to provide treatment until the new system is operational.

Upon start-up of the new treatment system, the remaining treatment ponds would need to be reconfigured and modified to increase percolation rates in these ponds, including removal of any accumulated sludge. A sludge storage basin will be needed for the waste activated sludge, and a portion of pond 8 could be modified and lined to accommodate this need. The generation of sludge will require sludge handling facilities, where the sludge will be dewatered and removed from the site for ultimate disposal.

The Initial treatment works should be sized for a minimum of 1.5 MGD ADWF. A portion of pond 5 could be converted into two 0.79 MG aeration basins with an approximately 0.40 MG anoxic basin preceding each aeration basin. This will require some excavation of the pond bottom, construction of additional levees and the lining of the basins. Based on economics of scale it may be appropriate to convert pond 5 into three equal basins at once. The two aeration basins will require approximately 90 Hp of blowers for aeration. Since ultimate aeration requirements are about 180 Hp of blowers, two 60 Hp blowers should be installed in Phase I to provide adequate backup capacity. Each treatment train will need its own clarifier, with two 44 ft diameter clarifiers constructed in this first phase. A pump station for return- and waste activated sludge (RAS/WAS) will be required. Rather than building a separate operations and laboratory building, these facilities could be attached to the blower building and the existing operations building could be converted to storage.

Phase I Activated Sludge Treatment Improvements Consist of the following:

- Constructing new headworks facilities.
- Constructing a 1.5 MGD capacity activated sludge treatment system in Pond 5.
- Constructing sludge handling facilities, including sludge storage in a portion of Pond 8.
- Constructing flow equalization.
- Converting Treatment Ponds 6-9 to percolation disposal ponds.

8.3.2 PHASE II: 2.25 MGD ACTIVATED SLUDGE TREATMENT EXPANSION

The disposal capacity of all municipal ponds not converted to activated sludge treatment is estimated to be 1.36 MGD. Phase II will require the construction of additional disposal ponds which would provide a minimum of 15 acres of effective percolation area. A minimum of three disposal ponds should be constructed with a total depth of 5 ft each. An additional 0.75 MGD activated sludge treatment train will need to be constructed to increase treatment capacity before flows reach 1.5 MGD, including the addition of another 60 Hp of blower capacity. This aeration basin will likely require excavation of the levee between pond 5 and ponds 6 and 7 and the reconfiguring of ponds 6 and 7. A third clarifier will be required with additional RAS/WAS pumping.

Phase II Activated Sludge Treatment Improvements Consist of the following:

- Construction of three 5 acre effective area disposal ponds.
- Constructing a 0.75 MGD capacity activated sludge treatment system in Ponds 6 and 7.
- Conversion of the southern portion of Ponds 6, 7 and 8 into a large disposal pond.
- Construction of additional ancillary structures and equipment for the activated sludge treatment including aeration blowers, sludge handling facilities, and RAS/WAS pumping.

8.3.3 PHASE III: 2.8 MGD ACTIVATED SLUDGE TREATMENT EXPANSION

The treatment and disposal capacity of the Phase II Activated Sludge Treatment expansion will be reached at 2.25 MGD. Phase III requires the construction of additional disposal ponds providing a minimum of 10 acres of effective percolation area. There should be a minimum of two disposal ponds providing a total depth of 5 ft each. A fourth 0.75 MGD treatment train will need to be constructed in a portion of pond 6 and 7 to increase treatment capacity to levels exceeding the 2035 growth boundary projected flows and a spare 60 Hp blower will need to be installed. A fourth clarifier and additional RAS/WAS pumping will need to be constructed.

Phase III Activated Sludge Treatment Improvements Consist of the following:

- Construction of two 5 acre effective area disposal ponds.
- Constructing a 0.75 MGD capacity activated sludge treatment system in a portion of Pond 6 and 7.
- Construction of additional ancillary structures and equipment for the activated sludge treatment including aeration blowers and RAS/WAS pumping.

8.4 BEST APPARENT WASTEWATER TREATMENT ALTERNATIVE

With the understanding that financial planning should be conducted for the worst case scenario, the best apparent alternative should reflect that ground water monitoring indicates the potential degradation of groundwater with respect to chloride and possible impairment with respect to sodium, and that unlined treatment ponds have a higher potential to impact shallow groundwater with respect to pathogens of wastewater origins and other constituents (e.g. nitrogen and the mobilization of iron, manganese, and arsenic). Therefore, the best apparent alternative is to convert to activated sludge treatment. The reason for this alternative being selected as the best apparent is based on the current regulatory environment with respect to treatment ponds and our current understanding of groundwater beneath the facility. The following possible and potential water quality impacts are central to this determination:

1. Groundwater potentially degraded with respect to chloride
2. Groundwater potentially impaired with respect to sodium
3. BPTC for nitrogen
4. BPTC for potential pathogen contamination of groundwater
5. BPTC for potential iron, manganese, and arsenic mobilization

Since median background groundwater nitrate levels can be as high as 13.4 mg/L as nitrogen (N), the potential for additional nitrate contamination of shallow groundwater will be very closely reviewed during permitting of the City's first WWTP expansion. Although pond systems have had some success in reducing effluent ammonia and nitrate levels to less than 10 mg/L as N, their reliability in reducing total mobile nitrogen levels to less than 10 mg/L as N is not proven. Activated sludge processes, on the other hand, are typically designed to meet total effluent nitrogen levels less than 10 mg/L.

Current groundwater monitoring data for pathogens or pathogen indicator organisms is limited to total coliform. As discussed in Section 6, total coliform monitoring of groundwater typically does not allow an assessment of the potential migration of pathogens of wastewater origins from untreated or partially treated wastewater from unlined treatment ponds. Current Regional Board and Department of Health Services guidance allows for unlined systems as long as adequate separation is provided between the pond bottoms and seasonally high groundwater. However, the Regional Board is currently contemplating new guidance that would be applicable, initially, to new facilities requiring all treatment systems to be lined and even for disinfection to be provided where five feet or more separation exists to high seasonal groundwater (including mounding that results from the disposal operations). Applying this possibility with the results of the other existing and potential water impacts leads to the requirement of at least lined treatment ponds. The added benefit of activated sludge treatment occurring over a smaller footprint provides for less potential for groundwater contamination from pathogens of wastewater origins.

Similar to possible pathogen contamination of groundwater, existing groundwater monitoring data does not allow for an assessment of the potential mobilization of iron, manganese, or arsenic beneath the treatment ponds. Many unlined pond treatment systems have come under scrutiny by the Regional Board for causing reducing conditions in soils underlying the ponds thereby causing the mobilization of iron, manganese, and arsenic. Although this phenomenon has not been assessed at the Escalon WWTP, if it is found, converting to an activated sludge process (either in smaller lined basins or in concrete structures) would be a likely mitigation measure.

The potential for degradation or impairment of the underlying groundwater with respect to conservative constituents, sodium and chloride, is dependent upon the point of compliance and other factors as discussed in Section 6.

Thus, converting to activated sludge treatment may be required to reduce the evapo-concentration of salts in the treatment process, minimize potential for groundwater degradation from constituents common to wastewater treatment, including sodium, chloride, nitrogen, and pathogens, and mobilization of iron, manganese, and arsenic.

The activated sludge treatment alternative provides benefits beyond the protection of groundwater. The treatment works can be sized to accommodate specific flows and easily be staged on the existing WWTP site. If future regulatory requirements result in an alternative disposal method being employed, a secondary activated sludge treatment system can be easily converted to tertiary treatment with disinfection by adding filters and chlorine facilities or ultraviolet disinfection. Treatment ponds, on the other hand, require additional preliminary treatment prior to filtration and disinfection if tertiary level treatment is required.

8.5 RECOMMENDED WASTEWATER TREATMENT AND DISPOSAL EXPANSION PROGRAM

If financial considerations were minimal and the City wished to minimize the risk of non-compliance in the short-term, then activated sludge treatment is the best apparent alternative for the reasons listed above; however, conversion to this treatment method cannot occur immediately nor is the existing water quality evidence strong enough to warrant the much higher capital and operation and maintenance cost. As recommended in Section 6, the City will be evaluating groundwater quality beneath and in the vicinity of the WWTP. Although the existing and potential impacts to water quality may not individually lead the City to converting treatment to activated sludge, two or more of these impacts may require that conversion. Given the uncertainty in how the City may be regulated in the future and in particular, potential impacts being found to exist, it is recommended that the City continue to maximize the usefulness of existing facilities to the extent practicable and converting to activated sludge treatment only when necessary as determined by facility monitoring. If impacts are found, the earliest the City would need to convert to activated sludge would be by the year 2013. This schedule can change depending on the rate of actual increased influent flows, but is based in the following assumptions:

- The City will have approximately two years to conduct an evaluation of groundwater beneath and beyond the WWTP, including analysis of constituents of concern in the pond effluent and groundwater monitoring wells (likely consisting of general major ions including chlorine and sodium, nitrogen species, iron and manganese, arsenic, total dissolved solids, and oxidation reduction potential).

- Upon completion of the evaluation of groundwater quality and groundwater flow, if degradation is found to be occurring, the Regional Board is likely to give the City approximately five years to plan, finance, design, and construct improvements to mitigate water quality impacts.

The following details the recommended expansion program and the planning level facilities costs associated with the expansions. This expansion program has an unknown potential for non-compliance, which can be mitigated to some extent, however mitigated by deferring the cost of expensive operation and maintenance and acquiring land which may be used for new treatment or disposal facilities. The facilities cost are based on ECO:LOGIC experience with the design, construction, and planning of similar facilities and are presented in 2006 dollars and at an ENR CCI July 2006 of 7721. The facilities costs were prepared based on utilizing or converting existing facilities where possible. Multiple intangible costs are associated with each improvement, and since detailed facilities design and sizing is not within the scope of this Master Plan Update, costs are presented on a lump sum basis per phase with minor facilities breakdown based on reasonably foreseeable facilities components. These facilities costs are intended to be conservative and used for planning purposes.

8.5.1 HEADWORKS AND INFLUENT PUMP STATION

Several upgrades are required to the headworks and influent pump station now and as the influent flows increase. All future upgrades should be accompanied by a backup power supply to provide for proper function of the influent pumping, screening, and flow measurement during power outages. The reliable capacity of the influent pump station is currently 1,100 gpm, which will be exceeded once average dry weather flows exceed 0.72 MGD. The addition of a third 1,100 gpm pump will double reliable capacity and provide enough capacity to handle ADWF of 1.4 MGD. This flow will occur in the early stages of Phase II of the activated sludge treatment alternative and prior to the Phase II pond treatment alternative. As a part of the Phase II expansion, two of the 1,100 gpm pumps should be replaced with 2,500 gpm pumps. The resultant reliable capacity of 3,600 gpm will handle ADWF to 2.3 MGD. Thus, Phase III of the activated sludge treatment alternative or pond treatment alternative will require the replacement of the final 1,100 gpm pump with a 2,500 gpm pump.

As discussed in section 5, the treatment plant has no capacity to remove gross influent solids. In Phase I, a self cleaning fine bar screen or fine rotary screen should be installed in channels designed to handle the 2035 planning boundary projected peak flows, as there would be no cost savings in phasing the screening structure. Equipment, however, can be staged and would consist of first a screening capacity designed to meet peak influent flows of at least 1.5 MGD. The remaining capacity could be split between Phases II and III. The activated sludge treatment system will also require a grit removal system. Since little cost savings can be realized through phasing of grit removal, it should be sized to handle an ADWF of 3.0 MGD.

The existing influent flow meter consists of a 6-inch throat Parshall flume, with an ultrasonic level transmitter. When the influent pumping capacity is increased to 2,200 gpm, the flow capacity of this flume will be exceeded. Prior to modifying the influent pump station, the influent flow meter must be modified. This would likely occur with the construction of screening channels and the installation of a minimum 12-in Parshall flume downstream of the screens or possibly the installation of a magnetic flow meter in the discharge piping from the influent pumps. The flow meter should be capable of measuring peak flows of at least 6.5 MGD, indicate flow rate, and provide recording of flow totals, with chart or chartless recording.

The majority of the improvements to the headworks provide immediate benefit, and the new headworks facilities should be designed to accommodate the conversion to activated sludge treatment in the future, such as locating the influent channel at an elevation that allows gravity flow through the entire plant. The costs for construction of the first phase headworks project include constructing the structure and channels to accommodate future flows and are included in the facilities cost in Table 8-3. Conversion to an activated sludge treatment plant will require additional headworks upgrades, such as grit removal, than would be necessary if the City continues with pond treatment. The total costs associated with the headworks and influent pump station improvements for the current pond treatment alternative and for the activated sludge treatment alternative are detailed in Table 8-4.

8.5.2 TREATMENT

The aerated pond treatment system should continue to be used in the near term, while employing mitigation measures, as described in Chapter 6, to minimize groundwater degradation. Negotiations, specific to treatment and disposal operations, BPTC and point of compliance, should be undertaken with the Regional Board prior to the permitting of the Phase I expansion and prior to the City's influent ADWF reaching 0.72 MGD. The results of the monitoring, success of the mitigation measures, and negotiations will determine the need and/or time frame for converting to activated sludge treatment. Another contributing factor to the length of continued operation of the current system is the availability of and mechanisms for acquiring land for expansion.

Table 8-3
Influent Pump Station and Headworks Improvement

Description	Phase I	Phase II	Phase III[b]
Influent Pumps	$15,000	$40,000	$20,000
Influent Channels	$250,000	-	-
Screening Equipment	$125,000	-	$125,000
Grit Removal[a], [c]	$150,000	-	-
Electrical and Instrumentation[a]	$300,000	$10,000	$20,000
Subtotal	$840,000	$50,000	$165,000
Contingency (25%)	$210,000	$10,000	$40,000
Engineering/Administration (25%)	$210,000	$10,000	$40,000
Total	**$1,260,000**	**$70,000**	**$245,000**
Improvement Total Cost	**$1,575,000**		

[a] Includes cost for flow measurements and recording instrumentation and backup power in Phase I.
[b] Includes pumping, flow measurement, channels to accommodate 6.5 MGD, and screening for 6.5 MGD.
[c] Not necessary for pond treatment.

It is recommended that the City continue, to the extent possible, with the aerated pond treatment following the phasing plan as described in Section 8.2. If the City is required to convert to activated sludge treatment, all phases associated with Section 8.3 will be required to provide for the necessary treatment capacity at such time. The continued use of the treatment ponds will require a sludge management program to maintain and increase treatment capacity in existing ponds.

Probable costs for Phase I through III of the aerated pond treatment alternative are detailed in Table 8-4.

Table 8-4
Treatment Improvement Costs

Description	Phase I (1.5 MGD)	Phase II (2.4 MGD)	Phase III (2.8 MGD)
Modify Existing Treatment Pond Piping	$240,000	-	-
Aeration Equipment and Yard Piping	$190,000	$300,000	$300,000
Lining of Existing Treatment Ponds	$1,100,000	-	-
Convert Disposal Pond to Treatment Pond	$865,000	$1,400,000	-
Electrical and Instrumentation[a]	$555,000	$510,000	$90,000
Subtotal	**$2,950,000**	**$2,210,000**	**$390,000**
Contingency (25%)	$740,000	$550,000	$100,000
Engineering/Administration (25%)	$740,000	$550,000	$100,000
Total	**$4,430,000**	**$3,310,000**	**$590,000**
Improvement Total Cost	**$8,330,000**		

[a] Assumed to be 30% of cost of civil and mechanical improvements for these treatment facilities.

Table 8-5
Expansion Costs for Each Treatment Option

Phasing	Treatment Pond Alternative[a]						Activated Sludge Alternative[a]					
	ADWF Capacity (MGD)	IPS/ Headworks Cost ($M)	Treatment Cost ($M)[a]	Disposal Cost ($M)	Land Cost ($M)	Existing Sludge Removal Cost ($M)	ADWF Capacity (MGD)	IPS/ Headworks Cost ($M)	Treatment Cost ($M)	Disposal Cost ($M)	Land Cost ($M)	Existing Sludge Removal Cost ($M)
Existing	1.1	-	-	-	-	-	0.0	-	$0.4	-	-	-
Phase I	1.5	$1.00	$4.43	$3.04	$0.32	$0.1	1.5	$1.26	$16.4	$1.3	$0.80	$0.25
Phase II	2.45	$0.07	$3.31	$2.45	$0.32	$0.1	2.25	$0.07	$6.9	$4.0	$0	$0.05
Phase III	2.8	$0.24	$0.59	$1.40	$0.16	$0.1	3.0	$0.24	$6.9	$2.0	$0	$0
Subtotal		$1.31	$8.33	$6.89	$0.80	$0.3		$1.57	$30.6	$7.3	$0.80	$0.3
	Phase I Cost		**$8.89**				**Phase I Cost**			**$20.4**		
	Phase II Cost		**$6.25**				**Phase II Cost**			**$11.0**		
	Phase III Cost		**$2.49**				**Phase III Cost**			**$9.1**		
	Total Cost of Alternative		**$17.6**				**Total Cost of Alternative**			**$40.6**		

[a] – Reconnaissance estimate activated sludge treatment costs based on generalized expansion description as seen in Appendix C.

The costs associated with each treatment alternative are presented in Table 8-5. The cost for the activated sludge system are based on an extended aeration system constructed in lined earth basins, including denitrification, treating the wastewater to secondary standards with no tertiary filtration or disinfection. The cost estimate for the aerated pond treatment alternative is based on using existing ponds 21 and 22 for future treatment capacity and equipping these and the existing treatment ponds with geosynthetic liners.

8.5.3 DISPOSAL

The estimated disposal capacity and area requirements are based on 2005 operations data and on analysis and data provided in the 1990 Master Plan. Ultimately, actual effluent disposal capacities should be measured during a hydrogeologic investigation of the existing site to determine future land requirements. Based on topography and soil data from the San Joaquin County Soil Survey, several sites near the existing treatment plant could be used for future disposal expansion. These site and their corresponding areas are presented in Figure 8-2.

Assuming that for each acre of effective percolation area, 1.6 acres of land are necessary (consisting of levees, access roads, setbacks, etc) then a total of approximately 40 acres of additional disposal area is needed to accommodate future wastewater flows. The usable land adjacent to the current municipal treatment and disposal facilities does not provide for 40 contiguous acres, and infrastructure will need to be constructed to convey flows to the new disposal areas. In addition, the inter-pond conveyance system should be upgraded to allow the use of any pond independently. This ability to divert flow to each pond independently will allow the City to cycle the disposal ponds, which we anticipate will help maintain disposal capacity near maximum levels.

Future disposal ponds are divided into effective areas of 5 acres each, which provides a disposal capacity of roughly 0.3 MGD per disposal pond. The construction of these ponds could be phased more often than the three treatment phases to spread costs over longer periods of time. The purchase of future disposal lands should be conducted relatively quickly to minimize the impact of increased property values on the financing of facilities. The wastewater disposal costs in Table 8-6 are phased as described for aerated pond treatment in Section 8.2. The land cost is based on $20,000 per acre. The individual cost for construction of each disposal pond is approximately $1,000,000.

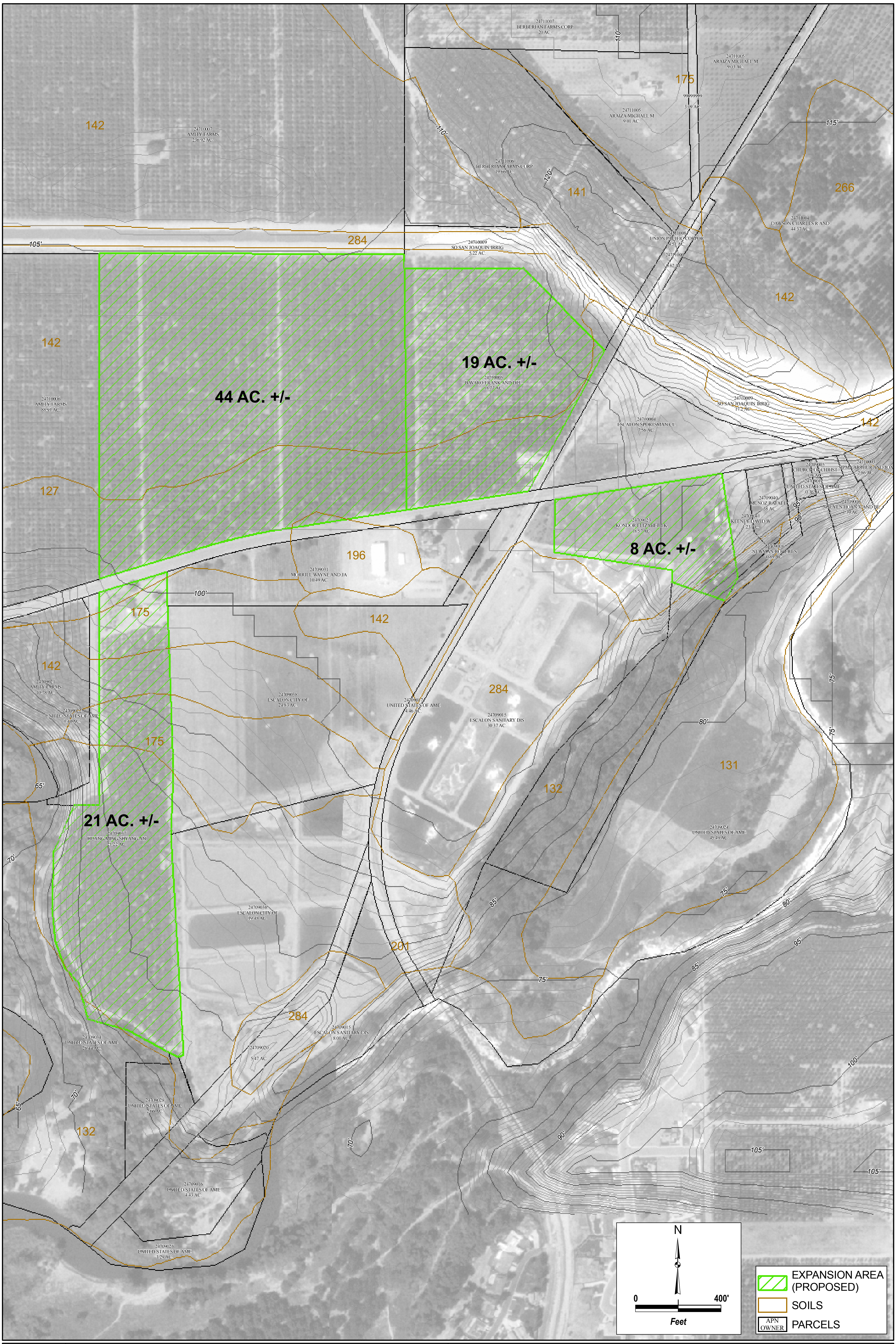


Figure 8-2
Possible Disposal Area Expansion Location

U:project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/Figure_08_02_Disposal_Expansion_Location.ai 8/15/06 cms

Table 8-6
Disposal Improvement Cost

Description	Phase I (1.5 MGD)	Phase II (2.45 MGD)	Phase III (2.8 MGD)
Land Acquisition[a]	$320,000	$320,000	$160,000
Pond Earthwork	$875,000	$875,000	$525,000
Fencing and Site Work	$200,000	$200,000	$120,000
Piping and Flow Distribution	$600,000	$460,000	$240,000
Effluent Pumping[b]	$250,000	-	-
Subtotal	**$2,245,000**	**$1,855,000**	**$1,045,000**
Contingency (25%)	$560,000	$460,000	$260,000
Engineering/Administration (25%)	$560,000	$460,000	$260,000
Total	**$3,365,000**	**$2,775,000**	**$1,565,000**
Improvement Total Cost	**$7,705,000**		

[a] Assuming land cost of $20,000 per acre.

[b] Assuming effluent pumping is needed to convey flows to new disposal areas.

Section 9

Proposed Plan Implementation

Section 9

Proposed Plan Implementation

This proposed implementation plan to the City of Escalon Sewer Master Plan outlines the proposed expansion projects for the sewer system and wastewater treatment and disposal facilities, based on the previous analysis and proposed improvements. Facilities costs and planned capacity are used to estimate the capital cost per equivalent sewer unit irrespective of the means of financing the facilities. Possible means of funding capital projects are explored and a range of potential impacts to sewer connection fees calculated.

Estimated capital cost per equivalent single-family unit for sewer facilities and for wastewater treatment and disposal are calculated in this section. This cost is based on expansion of the system as a whole to provide service to future users and is the basis for nexus between the necessary facilities and the estimated cost for those facilities to be charged in the City's sewer connection fee. Additional cost considerations should be undertaken by the City as the actual means of financing projects is determined and connection fees are established or as project specific fees are calculated during the preparation of any development agreements.

9.1 Recommended Projects

The recommended expansion and improvement projects are described in Section 7.3 for the Sewer System and Section 8.5 for the wastewater treatment and disposal system. These recommended projects and their probable cost are summarized in Table 9-1. The total cost of the recommended sewer system expansion and improvement program is $23.9 million. The total cost of the recommended wastewater treatment and disposal improvement plan is $17.6 million.

Prior to designing any facilities, it is recommended that the City review its current sewer system design standards based on the data evaluated in this plan. The facilities proposed in this master plan are based on a unit flow factor of about 80 to 90 gallons per person per day (e.g., 250 gallons per day per equivalent residential dwelling unit) and sewers designed to convey their design flows at no more than 70% full.

9.2 Project Scheduling

Project scheduling will depend largely on the rate and location of development within the City. Currently planned development in the Liberty Business Park area and the Heritage Park Development will require significant expansion of the sewer system, with wastewater treatment and disposal expansions required as these and other approved developments fill in.

For the purpose of evaluating financing options and the potential impact to project costs due to financing, the sewer improvements associated with the Near-Term Improvements, the Liberty Business Park, and Phase I WWTP improvements are assumed to occur relatively soon and require up-front financing through several possible means, including connection fees. Financing the cost of facilities will have a significant impact on the overall project cost and the resultant connection fee necessary to allocate the cost of facilities to new connections. Therefore it is recommended that after accepting this Sewer Master Plan as the overall plan for facilities, that a development projection be made for the next twenty to twenty five years, including projecting where development may occur within the current City boundaries and within each of the future growth boundaries.

Table 9-1
Recommended Improvement Projects Summary

Improvement	When Needed	Estimated Cost
Sewer System Improvements		
Near-Term Improvements	Prior to connecting additional new development	$6,710,000
Improvements for Heritage Park Development	During Heritage Park Phase 1	$976,000
Improvements for Liberty Business Park	During construction of first phase of development	$3,990,000
Improvements for Future Developments	As needed per Table 7-1	$3,710,000
Sewer System Expansion Improvements	As needed per Table 7-1	$8,510,000
	Subtotal	**$23,896,000**
Treatment and Disposal System Improvements		
Phase I, IPS/Headworks, Treatment, and Disposal[a]	When Influent Flow is 1.0 MGD	$8,890,000
Phase II, IPS/Headworks, Treatment, and Disposal	Prior to 1.5 MGD	$6,250,000
Phase III, IPS/Headworks, Treatment, and Disposal	Prior to 2.25 MGD	$2,490,000
	Subtotal	**$17,630,000**
Total Recommended Improvement Projects Cost		**$41,526,000**

[a] – Permitting for Phase I to start at 0.72 MGD.

9.2.1 Sewer System Expansion

Table 7-1 lists the general phasing of the sewer system expansion required to serve future development. These facilities provide system benefit, e.g., provide service for multiple projects in the vicinity of or beyond the expansion area, and exclude project specific sewer facilities that would typically be constructed and financed solely by the development project. Based on the facilities breakdown in Table 7-1, there are five areas or phases of sewer system expansion:

1. "Near-Term Improvements" are necessary to construct reliability in the existing system and increase capacity to allow additional sewer flows within the current sewer service area. The City should consider scheduling these improvements as soon as possible, based on financing, regulatory, and environmental constraints.

2. Improvements for the Heritage Park Development will be necessary for the first phase of this project. These facilities also provide system benefit in the vicinity of the Heritage Park Development and beyond. The Heritage Park Development would also contribute to the Near-Term Improvements to convey flows to the City wastewater treatment and disposal facilities.

3. Improvements for the Liberty Business Park Development will be necessary to collect wastewater from the first phase of this development and to convey those flows to the City's wastewater treatment and disposal facilities. As with the Heritage Park Development, the Liberty Business Park Development would contribute to the construction of the Near-Term Improvements.

4. Improvements for Future Developments consist of expansion of the existing facilities primarily to serve new flows within the current sewer service area. These improvements will be needed based on the actual rate of infill development and on conveying flows from future developments through the City's existing facilities.

5. Sewer System Expansion Improvements are necessary to extend the City sewer system to serve currently unsewered land within the City's growth boundaries (excluding the first phase projects required for the Liberty Business Park and Heritage Park developments). The location and timing of new development within these currently unsewered areas is unknown, therefore as each project makes application to the City, it is recommended that a sewer facilities study be prepared based on the project's specific location and phasing, including an evaluation and confirmation of required sewer facilities to extend service to the development, facilities needs for any flows to be conveyed through the development, and an evaluation of the projects impacts to the City's existing facilities.

9.2.2 Regulatory

Regulatory requirements extend to the City's sewer system and wastewater treatment and disposal facilities. With the adoption of the statewide Waste Discharge Requirements (WDR) for collection systems, the City will be required to perform a detailed analysis of the sewer system, including a capacity analysis that builds on the analysis prepared for this master plan (e.g., preparation of a sewer system hydraulic model with flow monitoring and calibration). This capacity analysis would be a major part of an overall sewer system management program or Sanitary Sewer Management Program (SSMP) that would include preventative maintenance, record keeping, and reporting. Based on the size of the City, it is estimated that the SSMP can be developed over the next four years.

The City's municipal wastewater treatment and disposal facility is currently regulated by the Regional Board through WDRs. In order to expand treatment and disposal services to ADWF greater than 0.9 MGD, the City will be required to submit a Report of Waste Discharge, including a description of the proposed expansion and an "Antidegradation Analysis" of the process to determine its consistency with State Board Resolution 68-16 (Antidegradation Policy). The initiation of this permitting process customarily occurs once flows reach 80 percent of permitted flows (i.e. 0.72 MGD according to the City's current permit). During this permitting process, many of the regulatory issues identified in Section 6 will be evaluated. It is anticipated that the City will have approximately two to three years to evaluate effluent impacts to shallow groundwater and initiate the Report of Waste Discharge Process. If the Antidegradation Analysis indicates that an alternative disposal method or different treatment method is warranted, then additional facilities design and construction will be necessary and revisions to this master plan would be appropriate. Land acquisition, facilities design, and construction may require an additional three to four years. This master plan has assumed that the current method of land disposal will remain practicable.

9.2.3 Implementation Schedule

Sewer system expansion should occur as new development occurs. Table 7-1 lists the triggers for extension and expansion of the sewer system to serve new development. In order to better estimate the timing and location of new development, the City can undertake a development projection working with current land owners and developers as well as other parties to predict the probable timing and location of new commercial, industrial, and residential development. This projection would take into account the City's current Growth Management Ordinance (GMO), which limits the number of new residential connections to 75 units per year. The GMO does not limit the rate of future commercial or industrial development, therefore the rate of this development is currently unknown and will likely follow local and regional economic development trends.

Based on the City's anticipated population, expansion of the wastewater treatment and disposal facilities (Phase I) is not likely to be necessary until after the year 2015. However, wastewater treatment and disposal facilities should be expanded ahead of the actual increase in wastewater flows to the plant. For example, the Phase I project should be initiated prior to the average dry weather flow to the plant reaching 0.9 MGD, which could occur prior to 2015. Based on the past five years of influent flow data, it is possible that the City influent flows could exceed 0.72 MGD within a year or two, necessitating the permitting of the Phase I improvements.

In order to achieve pumping reliability, it is recommended that the City plan for an interim project at the McHenry Lift Station consisting of replacing the existing pumps with higher capacity pumps. These pumps should be capable of pumping at least 830 gallons per minute each. In order to achieve this, the electrical systems may need to be upgraded as well. The cost for this interim project has not been included in the projected improvement project costs.

9.3 Funding Needs and Financing Assumptions

The City's current approach for expanding sewer systems for new development is to require that such new development extend the sewer system and to "oversize" the facility to accommodate future flows within or beyond the development. This approach places the risk of constructing facilities in anticipation of sewer connections (i.e., the financing of facilities based on future connections) on new development as opposed to current users. It is recommended that the City continue this approach wherever feasible. In the future, the City may need to consider other financing or cost sharing possibilities where new facilities will also benefit future users.

Alternatively, the City could finance facilities on a cash pay-as-you go basis, whereby connection fees collected by the City are used to construct facilities as they are needed. In order for the City to use a cash pay-as-you go financing, there needs to be sufficient available capacity in the existing systems to allow growth to occur until connection fee revenues are sufficient to expand the facility and the connection fee must take into account the cash flow requirements to construct facilities before capacity is needed. This may be possible only for smaller expansions or improvements.

In addition to financing project costs through the above two methods, the City could consider a number of different long-term debt financing alternatives, including:

- State Revolving Fund Loans,
- State Infrastructure Bank Loans,
- Bonds or Assessment District financing,
- Federal infrastructure financing such as USDA etc.,
- Commercial bank loans.

The cost of the particular financing, including any critical cash flow analysis, must be included in the final calculation of each component of the City's sewer connection fees. As the timing and need for project financing is currently unknown, a range of potential impacts to the City's connection fee are estimated from a simple cash pay-as-you go to some sort of financing of all future projects, with City involvement through bonds, assessment district, or other means. Due to the timing and cost of the Near-Term, Heritage Park Development, and Liberty Business Park sewer improvements, and the Phase I WWTP improvements, it is likely that these facilities costs will have be financed through some form of long-term debt.

9.3.1 Project Phasing

Project phasing is the most critical aspect of assessing the funding needs for future facilities. Three major projects areas have critical phasing components to be considered during calculation of funding needs. These projects are:

1. Improvements for the Heritage Park Development and contribution to the Near-Term Improvements,
2. Improvements for the Liberty Business Park Development and contribution to the Near-Term Improvements, and
3. Phase I of the Wastewater Treatment Plan expansion.

Near-Term Improvements

The Near-Term Improvements to the sewer system consist of facilities necessary to continue to provide service to the current system users, growth within the current City limits and sewer service areas, and to provide service for the Heritage Park Development, Liberty Business Park Development, and future development within the City's growth boundaries. The cost for the Near-Term Improvements is approximately $6,710,000. Due to the critical timing of these improvements, it is unlikely that the City will be able to generate connection fee revenue to fund these projects; therefore some form of financing will likely be necessary. In order to assess the potential impact to the overall project cost due to financing, Table 9-2 calculates the probable financing cost of the Sewer master Plan facilitates utilizing a loan program such as the I-Bank or a commercial bank loan with an assumed interest rate of 6.00 percent per year. It is critical to point out that such loan mechanisms will likely have to be backed by a sewer user fee pledge for debt service, versus a pledge of connection fees. Other financing mechanisms are available to the City and if the overall project cost is significantly more than shown in Table 9-2, connection fee calculations should be revised.

If the City were to finance the Near-Term Improvements through a loan program for the approximate $6,710,000 project cost, about $7,515,000 would have to be borrowed and upon completion of the 30-year term, the total cost of the facility including financing costs would be about $16,380,000.

Table 9-2
Facilities Financing Assumptions and Example Calculated Debt Service

Item	Assumption	Amount
Bond/Loan Proceeds[a]		**$41,526,000**
Issuance Costs	1%	$415,260
Capitalized Interest	$0	$0
Debt Service Reserve Fund	1 year	$3,319,000
Rounding Amount		$1,188,740
Total Bond/Loan Amount		$46,509,000
Calculated Debt Service		$3,378,828
Debt Service – Rounded [b]		$3,379,000
Assumptions		
Interest Rate		6.00%
Term		30 years
Bond/Loan Factor[c]		1.12
Total Facilities Cost with Financing [d]		$101,370,000
Financing Cost Factor[e]		2.44

[a] – Assumed financing of all Sewer Master Plan facilities.

[b] – Estimated annual debt service.

[c] – Assumed average issuance cost.

[d] – Total cost for thirty years of debt service.

[e] – Ratio of current probable cost to total facilities cost with financing.

Heritage Park Development

Approximately $976,000 for sewer improvements is necessary to extending service to the first phase of the Heritage Park Development. The cost for these facilities, and contribution to other facilities necessary to convey wastewater to the WWTP, should be considered during review of this project for approval.

Liberty Business Park Development

Significant sewer improvements are necessary to extend service to the first phase of the Liberty Business Park Development, including the Phase I LBPPS, force main and 24-in sewer towards the new City Main Lift Station. The total estimated cost of these facilities is about $3,990,000. Similar to the Near-Term Improvements, if these facilities were to be financed, the total loan amount and financed cost would be approximately $4,469,000 and $9,736,000 respectively. If facilities were to be financed through a Mello-Roos Bond, Assessment District or other bond financing, bond issuance could be in the order of 15% of the project cost.

Phase I Wastewater Treatment Plan Expansion

The existing wastewater treatment and disposal facilities have a limiting capacity of about 1.19 MGD (based on disposal capacity). Assuming that the Phase I WWTP improvements are to be constructed by the time influent flows reach about 1.0 MGD, then about 0.3 MGD of future flow can be accommodate prior to the construction of the Phase I project. Assuming a unit wastewater generation factor of 250 gallons per day per Equivalent Dwelling Unit (EDU), then approximately 1,200 additional EDUs can be connected to the system. Therefore the City would have to collect approximately $7,400 per EDU from those 1,200 EDUs to finance this expansion ($8,890,000/1,200) on a cash pay as you go basis. This illustrates the likely need for the City to finance at least a portion of the Phase I WWTP expansion with long-term debt. If similar financing of the Phase I improvements is utilized as evaluated for the sewer system improvements, the Phase I improvements total loan amount and financed cost could be approximately $9,957,000 and $ 21,692,000 respectively.

Likewise, the City should consider the cost of existing facilities that have remaining capacity for use by future connection, as described below.

9.3.2 Estimated Capital Cost Per Sewer Unit

Future average flows are calculated to be 2.1 MGD for approximately 8,400 EDUs. On an overall basis the average capital cost per equivalent sewer unit (at 250 gallons per unit per day) is approximately:

Sewer Improvements: $2,800/unit

Wastewater Treatment Improvements: $1,000/unit

Wastewater Disposal Improvements: $1,000/unit

Total average cost of sewer and wastewater facilities is approximately $4,800/unit. However including the possible cost for financing of master plan facilities (resulting in a total sewer system expansion cost of about $100,290,000), the average cost for Sewer Improvements would be approximately $6,800 per EDU, and $5,200 per EDU for wastewater treatment and disposal, resulting in a total average cost for wastewater facilities of $12,000. The potential range of impact to sewer connection fees is discussed below in Seciton 9.4.

As the City proceeds with each project, the means of financing improvements should be reviewed with financing costs incorporated into revisions to the connection fee.

9.4 POTENTIAL RANGE OF IMPACT TO SEWER CONNECTION FEES

This section explores the potential range of impact to the sewer connection fees based on two scenarios 1) cost of all Master Plan facilities to provide service to ultimate development within the planning boundaries, and 2) cost of initial facilities phases required to serve anticipated growth within the next twenty years.

9.4.1 Potential Fees for Master Plan Facilities

Future master plan facilities are anticipated to provide additional capacity for approximately 8,400 future equivalent single-family dwelling units. Depending on the means of financing capital improvements, the average cost per equivalent single-family dwelling unit (at an average wastewater flow per new EDU of 250 gallons per unit) is approximately $4,910 to $12,060 per EDU as outlined in Table 9-3.

Table 9-3
Potential Fees for Master Plan Facilities

	Estimated Cost[a]	Estimated Cost w/Financing[b]
Sewer System Improvements	$23,896,000	$ 58,306,000
Expansion Average Day Capacity (gallons/day)	2,100,000	2,100,000
Sewer Cost per Gallon Average Day Capacity	$11.38	$ 27.76
Gallons per Future EDU (average gpd)	250	250
Sewer Expansion Cost per Future EDU	$ 2,845	$ 6,940
Treatment and Disposal (T&D) Improvements	$17,630,000	$ 43,017,000
Expansion Average Day Capacity (gallons/day)	2,100,000	2,100,000
T & D Cost per Gallon Average Day Capacity	$8.39	$20.48
Gallons per Future EDU	250	250
T&D Cost per Future EDU	$2,100	$ 5,120
Total Cost per Future EDU	$4,945	$ 12,060

[a] – Not including cost for financing of system expansion.
[b] – Includes estimated financing cost as calculated in example form in table 9-2.

9.4.2 Potential Fees for Phase I Recommended Expansion Program

Existing facilities need to be expanded to accommodate projected development within the current City sewer service area and provide capacity for anticipated growth within the City's growth boundaries. The rate of this development will largely be limited by the GMO. Based on a twenty year planning horizon, approximately 1,500 future residential units could connect to the City sewer system. In addition, future commercial and industrial connections are anticipated, but the rate of growth of these sectors is unknown. In order to assess the potential impact to connection fees for facilities likely needed within the next twenty years, the estimated cost per EDU for the following master plan projects is summarized in Table

9-4. The projects listed in Table 9-4 are the Recommended Phase I Expansion Program. This expansion program will allow the City to evaluate potential changes in effluent disposal policies as well as evaluate groundwater quality underlying the existing effluent disposal site. If changes in the required treatment and disposal method are warranted, then a revision to this Master Plan would be initiated.

Table 9-4
Estimated Cost per EDU of Phase I Recommended Expansion Program

Improvement	Approximate Expanded Capacity (gal/d)[a]	Estimated Cost[b]	Estimated Cost per Gallon Capacity
Gravity sewer 18-inch minimum diameter 1,400-foot length along future HWY 120 bypass east of McHenry Lift Station	2,100,000	$495,000	$0.24
Construct Phase I of City Main Lift Station to replace existing McHenry Lift Station (Phase I at 3.1 MGD)	330,000	$1,110,000	$3.36
Construct 9,000-foot 33-inch minimum diameter gravity sewer from new City Main Lift Station to the Escalon WWTP	2,100,000	$5,100,000	$2.43
Improvements for Heritage Park Development	2,100,000	$976,000	$0.46
Improvements for Liberty Business Park	2,100,000	$3,990,000	$1.90
Phase I, IPS/Headworks, Treatment, and Disposal[a]	800,000	$8,890,000	$11.11
Total		$20,561,000	$19.50
Flow per Future EDU (gpd)			250
Cost per EDU			$4,875

[a] Compared with ultimate Master Plan additional capacity requirement of additional 2.1 Mgal/d average day flow.

[b] Including contingency, engineering, and administration.

The estimated cost per EDU for the Recommended Phase I Expansion program is approximately $4,900. It is likely that these facilities will need to be financed through some form of debt, therefore the actual cost of these facilities will be greater than shown in Section 7 or 8 and in Table 9-4. If the cost of these facilities were to be financed through long-term debt as shown in Table 9-2, then the cost per EDU could be $11,900.

9.5 BUY-IN TO EXISTING FACILITIES

Existing wastewater treatment and disposal facilities have the following approximate available capacities:

- Wastewater Treatment Ponds: 0.53 MGD or approximately 2,100 EDUs.
- Wastewater Disposal Ponds: 0.49 MGD or approximately 2,000 EDUs.

As these facilities will continue to provide benefit to new connections, and will provide benefit in to the future where existing land, treatment ponds, and disposal ponds continue to be utilized, the average cost to replace these facilities should be assigned to the connection fee in the following percentages:

- Land Cost at 75%;
- Treatment Ponds at 43% of replacement value; and
- Disposal Ponds at 41% of replacement value.

After reviewing the source of finds for the existing facilities, the City can allocate the value of the existing facilities to future development based in the above percentages utilizing one of the following; 1) calculated replacement cost of the remaining capacity, or 2) fixed asset value of existing system.

9.6 REVIEW OF CONNECTION FEES

Connection fees should be reviewed by the City periodically, including adjustments where appropriate. Such adjustments can be routine adjustments associated with rising costs of construction, such as adjusting the fee associates with an index as described below. Also, as each major project proceeds and costs are known with more certainty, including the means and cost of financing improvements, connection fees should be reviewed and adjusted.

9.6.1 Indexing of Fees

Indexing is used to provide for automatic adjustment of fees to account for inflationary cost increases. The connection fee enabling ordinance can provide for an automatic fee adjustment on a prescribed date each year, or every other year or third year, etc. Annual indexing revisions are generally preferred over less frequent adjustments; to minimize the magnitude of the change and insure that revenue more closely follows expenses. One approach involves adjustment based on an accepted cost indicator such as the CPI (Consumer Price Index) or the Engineering News Record (ENR) Construction Cost Index. The latter is preferred since it more closely reflects cost changes in the construction industry, which are used as the basis for computing City connection fees. This approach provides the most accurate adjustment, although the incremental change (increase or decrease) is not known beyond the current year.

An alternative approach is to have the fee increased a fixed amount each adjustment period. The incremental amount is set to approximate the inflationary adjustment expected in the next several years. The advantage to this approach is that the adjustment amount is known in advance and can be set to a round number, simplifying accounting.

The preferred approach for the City is an annual adjustment based on the ENR Construction Cost Index, with the based Index value of 7721 (mid 2006) for the costs calculated in this Master Plan. If a financing factor is included in the cost of facilities, based on actual long-term debt to finance facilities, then indexing of the connection fee for that portion of the cost may not be appropriate.

9.6.2 Review of Costs and Adjustment of Fees

The discussion regarding project phasing and possible cost associated with project financing illustrates the degree to which such financing can impact the average cost of facilities, and therefore the connection fee. Most sewer improvements and wastewater treatment and disposal facilities expansion will likely require some sort of long-term debt financing, with connection fee revenues providing initial capital or contributing to debt service as they are available. Therefore, as the City proceeds with each major improvement project described in this Master Plan, the cost of the facilities should be updated and the connection fee calculation revised. This review is of particular importance where most financing mechanisms will likely have to be backed by a sewer user fee pledge for debt service as mentioned above. At a minimum, it is recommended that the City review the connection fee at the following milestones:

- Upon completion of a development projection and facilities financing plan consistent with that projection and the City's current financial capabilities.
- Upon obtaining financing for the Near-Term Improvements, Liberty Business Park Improvements, and WWTP Phase I improvements;
- During design and financing of the WWTP Phase II Improvements; and
- During design and financing of the major facilities projects listed in Table 7-1 for Future Development and System Expansion.

Appendix A

Wastewater Permit-Waste Discharge Requirement Order No. 5-00-142

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL VALLEY REGION

ORDER NO. 5-00-142

WASTE DISCHARGE REQUIREMENTS
FOR
CITY OF ESCALON
ESCALON WASTEWATER TREATMENT PLANT
SAN JOAQUIN COUNTY

The California Regional Water Quality Control Board, Central Valley Region (hereafter Board), finds that:

1. The City of Escalon (hereafter Discharger) submitted a Report of Waste Discharge, dated 29 February 2000, and supplemental information dated 30 March 2000, for an expansion of the industrial portion of its wastewater treatment plant (WWTP).

2. The wastewater treatment facility is at 25100 West River Road, Escalon, in Sections 17 and 20, T2S, R9E, MDB&M, as shown in Attachment A, which is attached hereto and made part of this Order by reference. The facility is on Assessor's Parcel Numbers 247-090-36 and 247-090-38, which are both owned by the City of Escalon.

3. Waste Discharge Requirements Order No. 95-170, adopted by the Board on 23 June 1995, prescribes requirements for discharge of treated effluent from the City of Escalon WWTP to evaporation and percolation ponds on the property. Because of increased flows to the WWTP, and facility improvements at the site, Order No. 95-170 is not adequate and requires amendment.

4. The WWTP treats industrial wastewater generated by vegetable processing industries and domestic wastewater generated by the City of Escalon. The treatment process consists of screening and discharge to mechanically aerated treatment ponds, followed by discharge to evaporation/percolation ponds. The ponds, and proposed improvements, are shown in Attachment B, which is attached hereto and made part of this Order by reference.

5. The industrial and domestic wastewater flows are delivered to the plant in separate pipes, are treated in separate ponds, and are discharged to separate evaporation/percolation ponds. Stormwater from a limited portion of the City of Escalon is piped to the facility and is discharged to the industrial ponds.

6. The Discharger treats domestic wastewater in five treatment ponds, all of which are mechanically aerated. The domestic wastewater is disposed in two evaporation/percolation ponds (Nos. 21 and 24). Pond No. 21 provides 4.4 acres of surface area and 11.3 million gallons of storage capacity. Pond No. 24 provides 4.7 acres of surface area and 11.1 million gallons of storage capacity.

7. Prior to recent upgrades, the treatment plant was designed to treat domestic wastewater average dry weather flows of up to 0.55 million gallons per day (mgd) and peak wet weather flows of 1.0 mgd.

8. A report prepared by Dewante and Stowell titled, "*Wastewater Treatment Facilities Improvement and Expansion*," dated April 1990, identified improvements at the WWTP that could increase capacity and treatment. Improvements that were implemented included construction of Pond Nos. 21 and 24, addition of mechanical aeration, and piping improvements. The improvements resulted in additional storage capacity of 22.4 million gallons and 9.1 acres of surface area to the existing domestic wastewater treatment system. Based on the information in the Dewante and Stowell report, the domestic wastewater treatment facility is designed to treat 0.90 million gallons a day.

9. The Discharger does not disinfect effluent prior to discharge to the evaporation/percolation ponds.

10. The Discharger treats industrial wastewater in four treatment ponds, all of which are mechanically aerated. The industrial wastewater is then discharged to ten evaporation/percolation ponds (Nos. 10 through 19). The approximate surface area and storage capacity is presented below:

Pond Number	Surface Area (acres)	Storage Capacity (millions of gallons)
1	2.8	5.4
2	1.7	1.7
3	1.8	1.8
4	2.3	3.0
10	1.1	2.2
11	2.1	6.1
12	1.8	4.7
13	1.8	2.8
14	1.3	2.0
15	1.0	1.5
16	2.1	2.7
17	2.9	3.7
18	2.7	3.3
19	2.7	3.3
Totals	28.1	44.2

Data from City of Escalon, *Treatment Plant Storage Map*, dated 1/17/97.

11. The treatment plant was designed to treat industrial wastewater average dry weather flows up to 2.0 mgd and peak wet weather flows of 3.0 mgd. The monthly average dry weather maximum industrial flow rate was exceeded in August, 1999; the maximum daily industrial flow rate was exceeded in September and October, 1999.

12. The Discharger allows the industrial wastewater ponds to dry completely prior to initiation of the industrial wastewater generation season. The ponds are disced and ripped annually to maintain percolation rates.

13. Industrial dischargers consist of Escalon Premier Brand (tomato canner) and Eckert Cold Storage (frozen pepper processor). The industrial dischargers' processing season is from approximately May through December. The industrial dischargers presently screen their wastewater to remove solids prior to discharge to the WWTP.

14. Industrial wastewater is characterized by high concentrations of biochemical oxygen demand. The Discharger has not been previously required to analyze for total dissolved solids (TDS) concentrations. Escalon Premier Brands added tomato peeling equipment in 1999 which resulted in additional flow and increased biochemical oxygen demand.

15. Based on the Discharger's self-monitoring data for 1999, the flow rate and biochemical oxygen demand of influent at the WWTP headworks is presented below:

	Domestic Wastewater System		Industrial Wastewater System	
Month	Flow (mg)	BOD[1] (mg/l)	Flow (mg)	BOD[1] (mg/l)
January 1999	21.2	282	0.75	NA
February 1999	19.2	106	0.21	NA
March 1999	20.1	215	0.25	NA
April 1999	19.5	114	0.15	NA
May 1999	19.4	102	0.60	NA
June 1999	20.4	147	7.54	712
July 1999	20.4	181	42.6	505
August 1999	21.3	105	79.7	610
September 1999	18.6	110	93.3	1,380
October 1999	21.9	131	98.7	750
November 1999	21.2	71	32.4	316
December 1999	21.3	75	5.2	NA

[1] 5-day Biochemical Oxygen Demand.
NA denotes Not Analyzed.
Data from City of Escalon, "*1999 Annual Wastewater Treatment Report*," February 2000.

16. Both industrial dischargers are required to sample their wastewater individually. Flow rates and biochemical oxygen demand data for the 1999 processing season are presented below:

Constituent	Units	Escalon Premier Brand Average/peak	Eckert Cold Storage Average/Peak
BOD_5	mg/l	1,151/1,900	1,214/3,000
Flow Rates			
July 1999	mgd	1.8/2.0	0.5/1.0
August 1999	mgd	2.2/2.3	0.4/0.5
September 1999	mgd	2.5/3.0	0.7/0.9
October 1999	mgd	2.3/2.7	0.7/0.9
November 1999	mgd	2.1/2.3	0.7/0.8

Data from Nolte Report, "*Wastewater Treatment Facilities Improvements*," March 2000.

17. A hydraulic analysis of the industrial wastewater portion of the WWTP presented in the RWD indicates that industrial wastewater flows have reached the WWTP's hydraulic capacity. Reclamation is not presently performed; however, the Discharger is investigating reclamation alternatives.

18. The Discharger is in the process of improving the industrial WWTP. The improvements will provide more capacity and better treatment. Improvements include deepening the existing treatment ponds, three new evaporation/percolation ponds (Nos. 20, 22, and 24), piping and pond configuration improvements, addition of mechanical aerators, and reconfiguration of existing aerators. Pond No. 22 will provide 5.6 acres of surface area and 5.1 million gallons of storage capacity. Pond No. 23 will provide 5.0 acres of surface area and 4.7 million gallons of storage capacity. Pond No. 20 will be constructed as an enlargement of Pond No. 12; Pond No. 12/20 will provide 4.0 acres of surface area and 11.7 million gallons of storage capacity. The construction activities are scheduled to be completed by July 2000, in time for the beginning of the industrial discharger's processing season. A summary of the pond configuration improvements is presented below:

	Pond Area	Pond Volume	Hydraulic Capacity	
Condition/Improvement	Acres	MG	MGD	MG/year
Existing Conditions	19.5	32.3	2.0	221
Recommended Improvements				
Deepen Treatment Ponds	NA	3.1	NR	NR
Expand Pond 12 to Incorporate Pond 20	2.2	7.0	0.27	29
Remove Berm between Ponds 13 and 16	0.3	0.5	0.04	3
Remove Berm between Ponds 14 and 17	0.3	0.5	0.04	3
Remove Berm between Ponds 15 and 18	0.3	0.5	0.04	3
Construct Pond 22	5.6	5.1	0.54	58
Construct Pond 23	5.0	4.7	0.48	52
Subtotal of Improvements	13.7	18.3	1.41	148
Total (Ext. + Rec. Improvements)	33.2	50.6	3.41	370

Data from Nolte Report, "*Wastewater Treatment Facilities Improvements*," March 2000.
MG denotes million gallons. MGD denotes million gallons per day.
NA denotes not applicable. NR denotes not reported.
Ext. + Rec. denotes the sum of existing and recommended improvements.

19. Escalon Premier Brands is in the process of retaining a wastewater engineering firm to identify means to reduce the BOD concentrations of its wastewater and to investigate opportunities for recycling of wastewater at the plant.

20. Three groundwater monitoring wells exist at the facility. The wells are screened in the first saturated interval and are currently sampled annually. Well W-1 is upgradient of the

wastewater treatment ponds; Wells W-2 and W-3 are adjacent to the future Ponds 22 and 23. None of the wells are directly downgradient of the treatment or percolation/evaporation ponds. Based on groundwater samples which were collected from April 1997 to January 2000, average characterization of the water quality is:

Well	Electrical Conductivity	Nitrate (as nitrate)
W-1	780 mg/l	5.4 mg/l[1]
W-2	566.7 mg/l	20.8 mg/l
W-3	785 mg/l	43.1 mg/l

Data from Discharger's self-monitoring reports.
[1] Analytical detection limit used to calculate average value.

21. Based on the groundwater monitoring performed at the site, groundwater exists approximately 37 feet below ground surface. Groundwater flows to the west-southwest.

22. In order to determine compliance with the groundwater and surface water limitations contained herein, these WDRs contain a time schedule for preparation of a groundwater evaluation report.

23. Objectionable odor complaints related to the facility were documented in October 1996, November 1997, and September 1999. All the complaints were related to the industrial wastewater system.

24. The WWTP is on the north side of the Stanislaus River. Surrounding land uses are primarily agricultural. A golf country club and private residences exist south of the Stanislaus River.

25. The Board adopted a Water Quality Control Plan, Fourth Edition, for the Sacramento River and San Joaquin River Basins (hereafter Basin Plan), which designates beneficial uses and water quality objectives for waters of the Basins. These requirements implement the Basin Plan.

26. The site lies within the San Joaquin Valley Floor Hydrologic Unit No. 535.10, as depicted on interagency hydrologic maps prepared by the Department of Water Resources in August 1986.

27. Surface water drainage is to the Stanislaus River. The beneficial uses of the Stanislaus River are municipal, agricultural water supply for irrigation and stock watering; contact recreation, canoeing, and non-contact recreation; warm and cold freshwater habitat; cold water migration; warm and cold water spawning; and wildlife habitat. The potential beneficial uses of the Stanislaus River are municipal and domestic water supply.

28. The beneficial uses of the underlying groundwater are municipal, domestic, and industrial supply.

29. The Board has considered anti-degradation pursuant to State Board Resolution No 68-16 and finds that not enough data exists to determine whether this discharge is consistent with those provisions. Therefore, this Order provides a timeline for data collection to determine whether the discharge will cause an increase in groundwater constituents above that of background levels. If the discharge is causing such an increase, then the Discharger may be required to cease the discharge, line the ponds, implement source control, change the method of disposal, or take other action to prevent groundwater degradation.

30. The Discharger prepared a Negative Declaration in accordance with the California Environmental Quality Act (Public Resource Code Section 21000 et seq.) and the State CEQA Guidelines. The CEQA document was approved on 9 May 2000 by the City of Escalon City Council.

31. This discharge is exempt from the requirements of *Consolidated Regulations for Treatment, Storage, Processing, or Disposal of Solid Waste,* as set forth in Title 27, CCR, Division 2, Subdivision 1, Section 2005, et seq., (hereafter Title 27). The exemption pursuant to Section 20090(b), is based on the following:

    a. The Board is issuing waste discharge requirements,
    b. The discharge complies with the Basin Plan, and
    c. The wastewater does not need to be managed according to Title 22 CCR, Division 4.5, and Chapter 11, as a designated or hazardous waste.

32. Section 13267(b) of California Water Code provides that: "In conducting an investigation specified in subdivision (a), the regional board may require that any person who has discharged, discharges, or is suspected of discharging, or who proposes to discharge within its region, or any citizen or domiciliary, or political agency or entity of this state who has discharged, discharges, or is suspected of discharging, or who proposes to discharge waste outside of its region that could affect the quality of the waters of the state within its region shall furnish, under penalty of perjury, technical or monitoring program reports which the board requires. The burden, including costs of these reports, shall bear a reasonable relationship to the need for the reports and the benefits to be obtained from the reports."

33. The Board has notified the Discharger and interested agencies and persons of its intent to prescribe waste discharge requirements for this discharge and has provided them with an opportunity for a public hearing and an opportunity to submit their written views and recommendations.

34. The Board, in a public meeting, heard and considered all comments pertaining to the discharge.

IT IS HEREBY ORDERED that Order No. 95-170 is rescinded and the City of Escalon, its agents, successors, and assigns, in order to meet the provisions contained in Division 7 of the California Water Code and regulations adopted thereunder, shall comply with the following:

A. Prohibitions:

1. Discharge of wastes to surface waters or surface water drainage courses is prohibited.

2. Bypass or overflow of untreated or partially treated waste is prohibited.

3. Neither the treatment nor the discharge shall cause a nuisance or condition of pollution as defined by the California Water Code, Section 13050.

4. The discharge shall not cause the degradation of any water supply.

5. Discharge of waste classified as hazardous, as defined in Sections 2521(a) of Title 23, CCR, Section 2510, et seq., (hereafter Chapter 15), or 'designated', as defined in Section 13173 of the California Water Code, is prohibited.

B. Discharge Specifications:

1. For the domestic WWTP, the monthly average dry weather flow shall not exceed 0.90 mgd. The maximum daily flow shall not exceed 1.0 mgd.

2. For the industrial WWTP, the monthly average dry weather flow shall not exceed 2.0 mgd. The maximum daily flow shall not exceed 3.0 mgd. These industrial flow limits will remain in effect until the industrial wastewater treatment facilities improvements are completed, and a technical report (as described in Provision F.1) has been submitted and approved by the Executive Officer. Upon approval of the technical report, the Discharger's flow limits will be allowed as presented in Discharge Specification No. B.3.

3. Upon approval of the technical report required by Provision F.1, flow rates at the industrial WWTP may increase as follows: the monthly average dry weather industrial discharge flow to the industrial WWTP shall not exceed a maximum average daily flow over any 30-day period of 3.4 mgd, and the total industrial discharge over the processing season shall not exceed 370 million gallons.

4. The effluent discharged to the domestic evaporation/percolation ponds shall not exceed the following limits.

Constituent	Units	Monthly Average	Daily Maximum
BOD_5	mg/l	40[1]	80
Settleable Matter	ml/l	0.2[1]	--

[1] If the reported concentration is greater than the criterion presented, a confirmation sample shall be collected for analysis within seven days of receiving the analytical data.

5. The effluent contained in the industrial evaporation/percolation ponds shall not exceed the following limits.

Constituent	Units	Monthly Average
BOD_5	mg/l	150

6. Objectionable odors originating at this facility shall not be perceivable beyond the limits of the wastewater treatment facility.

7. As a means of discerning compliance with Discharge Specification No. B.6, the dissolved oxygen content shall not be less than 1.0 mg/l in any pond at any time, as measured at a point at as far as practical from the pond inlet and within one foot of the water surface.

8. The discharge to conveyance systems, settling basins, ponds, or land disposal areas not adequately maintained to prevent off-site odor nuisance, fly breeding, or mosquito breeding is prohibited.

9. The treatment facilities shall be designed, constructed, operated, and maintained to prevent inundation or washout due to floods with a 100-year return frequency.

10. The pond system shall have sufficient capacity to accommodate allowable wastewater flow, design seasonal precipitation, seasonal ancillary inflow, and infiltration during the wet season. Design seasonal precipitation shall be based on total annual precipitation using a return of 100 years, distributed monthly in accordance with historical rainfall patterns.

11. Pond freeboard shall never be less than two feet in any pond as measured vertically from the water surface to the upper surface of the lowest adjacent dike or levee.

12. On or about **1 November** each year, available pond storage capacity shall at least equal the volume necessary to comply with Discharge Specifications No. B.10 and B.11.

C. Solids Disposal Requirements:

1. Collected screenings, sludge, and other solids removed from liquid wastes shall be disposed of in a manner approved by the Executive Officer, and consistent with *Consolidated Regulations for Treatment, Storage, Processing, or Disposal of Solid Waste,* as set forth in Title 27, CCR, Division 2, Subdivision 1, Section 20005, et seq.

2. Any proposed change in sludge use or disposal practice from a previously approved practice shall be reported to the Executive Officer and U.S. Environmental Protection Agency (EPA) Regional Administrator at least 90 days in advance of the change.

3. Use and disposal of sewage sludge shall comply with existing Federal, State, and local laws and regulations, including permitting requirements and technical standards included in 40 CFR 503.

4. If the State Water Resources Control Board and the Regional Water Resources Control Board are given the authority to implement regulations contained in 40 CFR 503, this Order may be reopened to incorporate appropriate time schedules and technical standards. The Discharger shall comply with the standards and time schedules contained in 40 CFR 503 whether or not they have been incorporated into this Order.

D. Groundwater Limitations:

The discharge, in combination with other sources, shall not cause underlying groundwater to contain waste constituents in concentrations statistically greater than background water quality, except for coliform. For coliform, increases shall not cause the most probable number of total coliform organisms to exceed 2.2/100 ml over any 7-day period.

E. Surface Water Limitations:

The Discharger shall not cause the Stanislaus River downstream of the evaporation/percolation ponds to contain waste constituents in concentrations statistically greater than background (upstream) surface water quality. Background surface water quality shall be determined when the required monitoring program provides sufficient data.

F. Provisions:

1. As described in Discharge Specification B.2, the Discharger shall submit a technical report which describes the implemented WWTP improvements. The report is required at least **15 days prior** to the Discharger's desire to increase the flow rates as described in Discharge Specification B.3.

2. By **1 January 2001**, the Discharger shall submit a technical report, prepared under the supervision of a registered engineer, which identifies opportunities to reduce, reuse, and recycle wastewater. The report shall identify sources of TDS in industrial wastewater, and recommend ways to reduce TDS. The report shall also evaluate reclamation alternatives both at the industrial discharger facilities and the WWTP. The report shall present a tabulation of all data used in evaluating the industrial wastewater flow.

3. By **1 January 2001** Discharger shall submit a Sludge Management Plan which describes the annual volume of sludge generated by the facility and specifies the proposed testing and disposal practices.

4. By **31 May 2001**, the Discharger shall submit a report, pursuant to Section 13267 of the California Water Code, indicating whether the discharge to the ponds has caused, or is

likely to cause, constituent concentrations in groundwater to exceed background concentrations. The report shall be prepared under the supervision of a Registered Geologist or Registered Engineer. If monitoring data indicates wastewater has degraded groundwater quality beyond background concentrations, the Discharger shall submit a Wastewater Disposal Mitigation Plan (WDMP) within 90 days of request by the Executive Officer. The WDMP shall address the magnitude and extent of groundwater contaminants, evaluate contaminant control alternatives, evaluate appropriate effluent limits, and select a preferred contaminant control alternative. The selected contaminant control alternative must comply with State Water Resources Control Board Resolution No. 68-16, Title 27 CCR, and the most recent Basin Plan. The WDMP shall include a proposed project schedule for design and construction.

5. The Discharger shall comply with Monitoring and Reporting Program No. 5-00-142, which is part of this Order, and any revisions thereto, as ordered by the Executive Officer.

6. The Discharger shall comply with the "Standard Provisions and Reporting Requirements for Waste Discharge Requirements", dated 1 March 1991, which is attached hereto and made part of this Order by reference. This attachment and its individual paragraphs are commonly referenced as "Standard Provision(s)".

7. If the Discharger wishes to increase daily or monthly flow limits at the industrial WWTP above what is allowed in this Order, then it must submit an RWD. A complete RWD shall be submitted at least 120 days prior to the anticipated increase in discharge.

8. The Discharger shall use the best practicable cost-effective control technique(s) currently available to comply with discharge limits specified in this order.

9. The Discharger shall report promptly to the Board any material change or proposed change in the character, location, or volume of the discharge.

10. In the event of any change in control or ownership of land or waste discharge facilities presently owned or controlled by the Discharger, the Discharger shall notify the succeeding owner or operator of the existence of this Order by letter, a copy of which shall be forwarded to this office.

11. The Discharger shall submit to the Board on or before each compliance report due date, the specified document or, if appropriate, a written report detailing compliance or noncompliance with the specific schedule date and task. If noncompliance is being reported, then the Discharger shall state the reasons for such noncompliance and provide an estimate of the date when the Discharger will be in compliance. The Discharger shall notify the Board in writing when it returns to compliance with the time schedule.

12. The Discharger shall comply with all conditions of this Order, including timely submittal of technical and monitoring reports as directed by the Executive Officer.

Violations may result in enforcement action, including Regional Board or court orders requiring corrective action or imposing civil monetary liability, or in revision or rescission of this Order.

13. A copy of this Order shall be kept at the discharge facility for operating personnel. Key operating personnel shall be familiar with its contents.

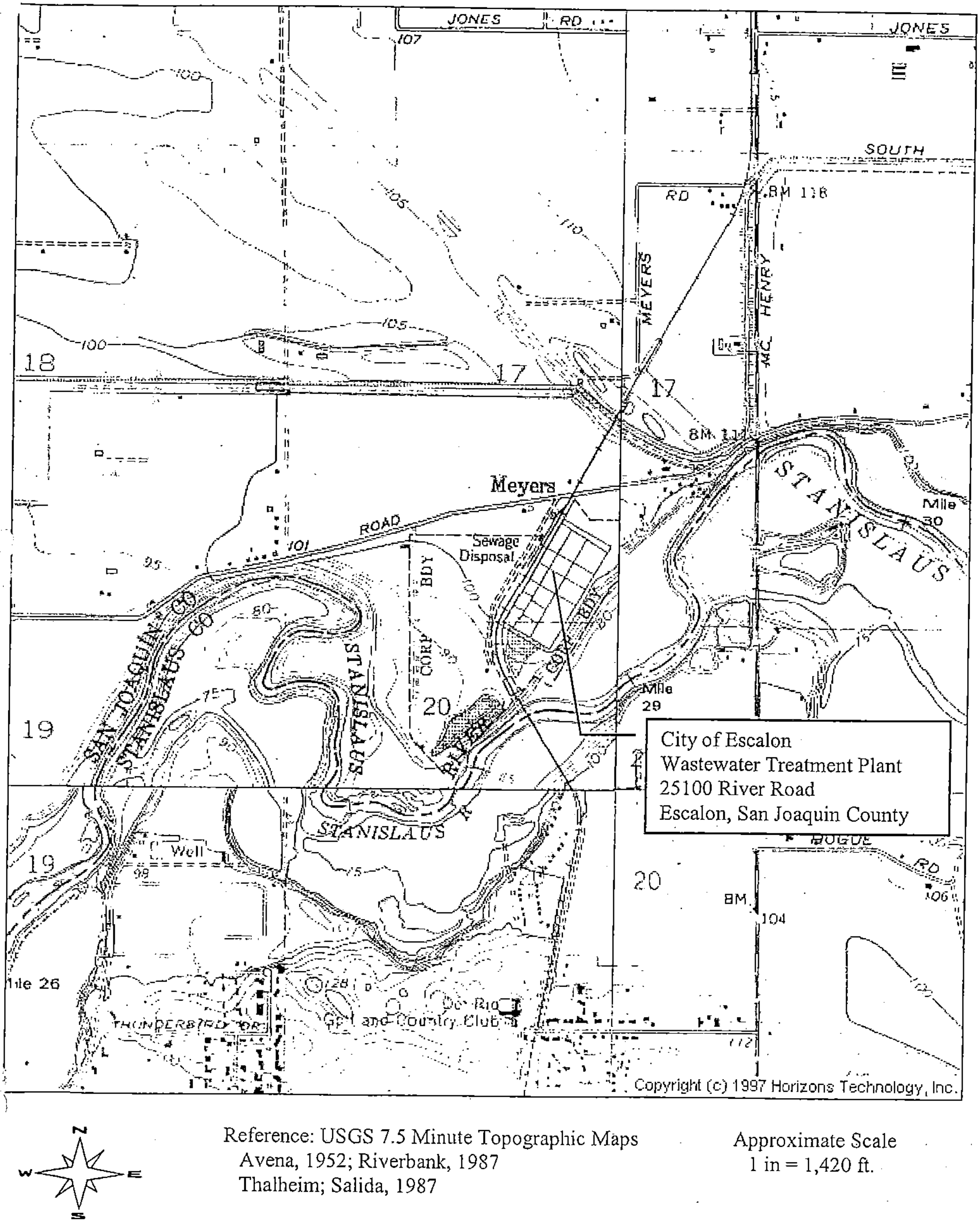
14. The Board will review this Order periodically and may revise requirements when necessary.

I, GARY M. CARLTON, Executive Officer, do hereby certify the foregoing is a full, true, and correct copy of an Order adopted by the California Regional Water Quality Control Board, Central Valley Region, on 16 June 2000.

GARY M. CARLTON, Executive Officer

Attachments
TRO: 6/16/00

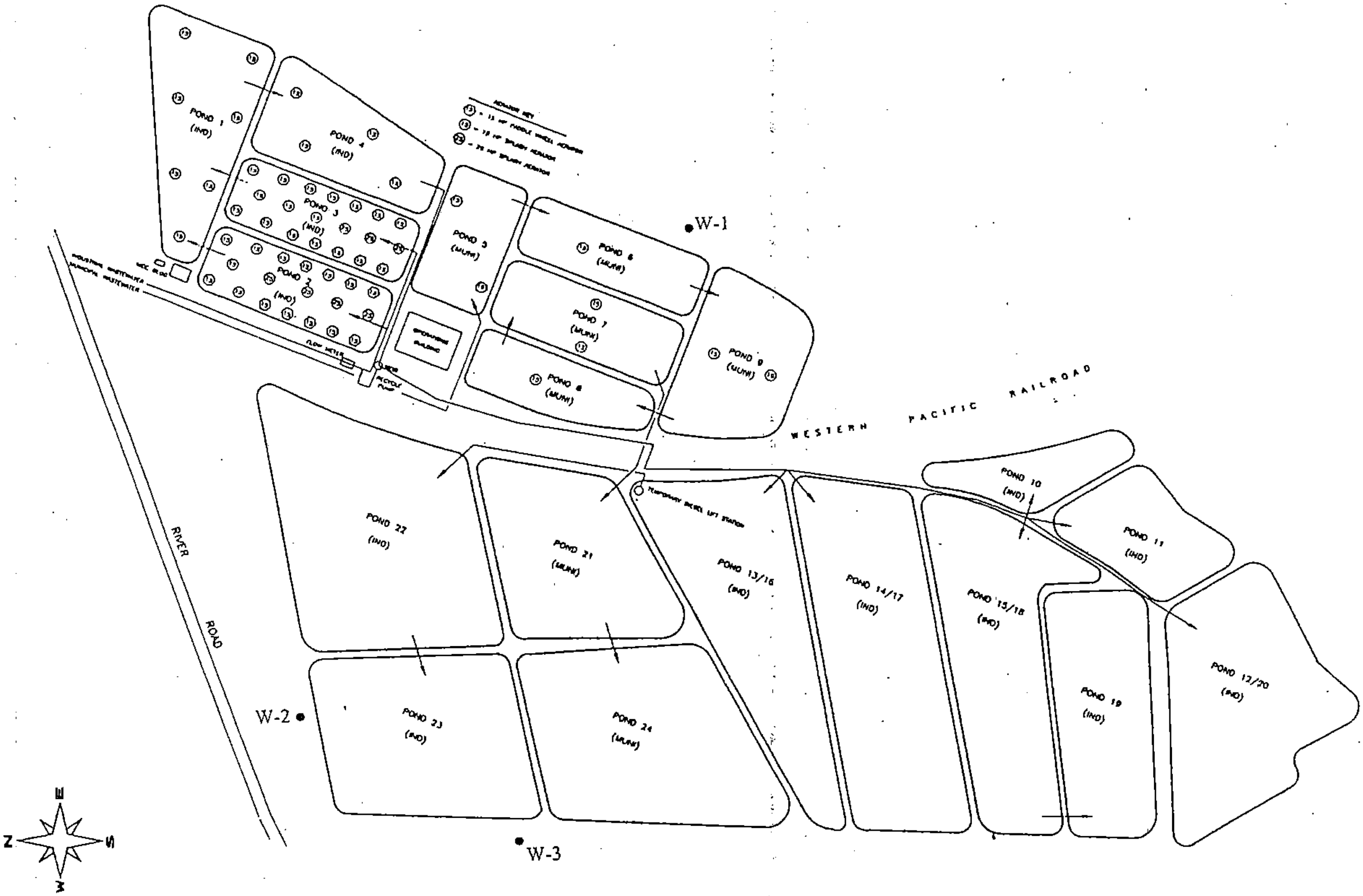
ATTACHMENT A



Reference: USGS 7.5 Minute Topographic Maps
Avena, 1952; Riverbank, 1987
Thalheim; Salida, 1987

Approximate Scale
1 in = 1,420 ft.

ATTACHMENT B



Scale: Not to Scale	Drawing Reference: Nolte Associates, Figure 4 Wastewater Treatment Facilities Improvements Preliminary Design Study, March 2000	**SITE PLAN** PROPOSED CONSTRUCTION IMPROVMENTS WASTEWATER TREATMENT PLANT ESCALON, SAN JOAQUIN COUNTY

INFORMATION SHEET

ORDER NO. 5-00-142
CITY OF ESCALON WASTEWATER TREATMENT PLANT
SAN JOAQUIN COUNTY

The City of Escalon operates a domestic and industrial wastewater treatment and disposal facility at 25100 W. River Road, in Escalon, San Joaquin County. The facility receives domestic wastewater from the City of Escalon. Industrial wastewater, generated by Escalon Premier Brands (tomato canner) and Eckert Cold Storage (bell pepper canner) is discharged to the WWTP between the months of July and November. The peak discharge of industrial wastewater occurs in September and October, when approximately 90 to 95 million gallons per month of industrial wastewater is discharged.

The industrial wastewater is discharged to the WWTP via a 30-inch gravity pipeline which also conveys stormwater from the City of Escalon during the wet season. The industrial dischargers presently screen the wastewater prior to discharge to the WWTP. Escalon Premier Brands is in the process of evaluating pretreatment alternatives. Odor complaints were documented related to the industrial wastewater discharge in 1997, 1998, and 1999.

The domestic wastewater and the industrial wastewater are conveyed, treated, and disposed of in separate ponds. Domestic wastewater is discharged to five treatment ponds, all of which are mechanically aerated. Approximately 120 horsepower of mechanical aeration devices are located in the domestic wastewater ponds. Industrial wastewater is discharged to four treatment ponds, all of which are mechanically aerated. Approximately 610 horsepower of mechanical aeration devices are located in the industrial wastewater ponds.

The Discharger plans to construct improvements in the WWTP which will provide better treatment of industrial wastewater and increased flow capacity. The improvements consist of deepening the existing treatment ponds, construction of two new ponds, reconfiguration of existing ponds, piping changes, and addition of mechanical aeration devices. Enlargement of the treatment ponds is recommended; the enlargement will result in an additional 3.1 million gallons of storage capacity. The treatment ponds will be enlarged by deepening, the surface area of the ponds will not change significantly. The evaporation/percolation pond configuration improvements will result in an additional 13.7 acres of surface area and 18.3 million gallons of storage capacity. The improvements will allow an additional 1.41 million gallons per day of wastewater flow. An additional 210 horsepower of mechanical aerators are also planned for the treatment ponds.

Depth to the first water bearing zone is approximately 30-35 feet. These WDRs require characterization of the existing groundwater quality and quarterly groundwater monitoring. Three groundwater monitoring wells exist at the facility; however because of the groundwater flow direction, they are not located in areas which provide useful data on groundwater conditions.

Because the WWTP improvements have not yet been constructed, these WDRs contain interim discharge limits. The interim limits will be replaced with higher flow limits upon Executive Officer approval of a technical report describing the implemented WWTP improvements.

The Monitoring and Reporting Program requires monitoring of the domestic influent and effluent, industrial influent and effluent, and groundwater and surface water. Monitoring data will be used to determine whether wastewater treatment and/or pond mitigation efforts are necessary.

Appendix B

Water Balances

SITE NAME:	Escalon	WATER BALANCE - Existing Facilities. Perc rate assumes all ww in disposal ponds percs each month and the ponds are at disposal capacity based on limited usage of disposal ponds.	COLOR LEGEND
BASE FLOW (MGD):	0.60		DESC. TEXT
DESIGN PRECIPITATION:	2005		CLIMATIC DATA
PERC./ INFIL. & INFLOW	Interim		DESIGN INPUT

CLIMATOLOGICAL FACTORS

PRECIP/AVG. PRECIP RATIO	1.14
OCT.-APR EVAP/AVG EVAP RATIO[1]	0.88
MAY-SEP EVAP/AVG EVAP RATIO[1]	0.95
ETo COEFFICIENT	1.00
LAND PRECIP COLLECTED	0.90
I/I ADJUSTMENT RATIO[2]	0.42
I/I REDUCTION FACTOR FOR NEW CONSTRUCT	0.00
ESTIMATED EDU	
TOTAL[3]	2380
EXISTING	2380
NEW	0

TREATMENT POND CHARACTERISTICS

TREATMENT POND GROSS AREA (ac)	8.1
EVAP./PERC. AREA (ac)	7.0
DESIGN PERC. RATE (in/day)[5]	0.62
STORAGE AVAILABLE (MG)	5
AERATOR INDUCED EVAP AREA (ac)[4]	11

DISPOSAL POND CHARACTERISTICS

EXISTING		Disposal Pond STORAGE PROVIDED	
GROSS AREA (ac)	8.3	TOTAL STORAGE AVAILABLE (MG)	19
EVAP./PERC. AREA (ac)	7.2	TOTAL STORAGE AVAILABLE (af)	75
EXISTING STORAGE (MG)	19	GROSS AREA (ac)	8.3
NEW		EVAP./PERC. AREA (ac)	7.2
GROSS AREA (ac)	0.00	DESIGN PERC. RATE (in/day)[5]	2.160
EVAP./PERC. AREA (ac)	0.00		
NEW STORAGE (MG)	0.00		

BASIC INPUT	PARAMETER\ MONTHS IN WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	WATER YEAR
	DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
	AVG ETo (in)	0.87	1.71	3.43	5.24	6.70	7.40	7.85	6.75	4.93	3.37	1.66	0.87	50.78
	AVG PRECIP (in)	0.63	1.34	2.12	2.37	2.14	1.94	1.08	0.47	0.09	0.03	0.04	0.20	12.45
	I/I FLOW ESTIMATED BASED ON RATIO (MG)	0.26	0.56	0.89	1.00	0.90	0.81	0.45	0.20	0.04	0.01	0.02	0.08	5.23
	FRACTION OF DESIGN PERC RATE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DESIGN CONDITIONS	PRECIPITATION (in)	0.03	0.34	3.06	2.77	2.78	2.01	1.55	1.11	0.21	0.00	0.00	0.28	14.14
	EVAPORATION (in)	3.31	1.67	0.75	0.58	1.51	3.09	4.23	5.65	6.93	7.56	6.63	4.63	46.54
	I/I FLOW (MGD)[6]	0.10	0.00	0.00	0.08	0.03	0.00	0.01	0.00	0.00	0.00	0.01	0.03	0.26
	WWTP INFLUENT FLOW (MGD)	0.697	0.592	0.555	0.676	0.621	0.600	0.608	0.593	0.571	0.587	0.607	0.623	
	WWTP INFLUENT VOLUME (MG)	21.59	17.76	17.20	20.96	17.39	18.59	18.23	18.40	17.13	18.20	18.83	18.68	223.0
TREATMENT PONDS	PERCOLATION (in)	19.2	18.6	19.2	19.2	17.3	19.2	18.6	19.2	18.6	19.2	19.2	18.6	225.8
	PERC. VOLUME (MG)	3.7	3.5	3.7	3.7	3.3	3.7	3.5	3.7	3.5	3.7	3.7	3.5	43.0
	EVAP. VOLUME (MG)	1.6	0.8	0.4	0.3	0.7	1.5	2.1	2.8	3.4	3.7	3.2	2.3	22.8
	PRECIP. VOLUME (MG)	0.0	0.1	0.7	0.6	0.6	0.4	0.3	0.2	0.0	0.0	0.0	0.1	3.1
	POND DISPOSAL (MG)	5.3	4.3	3.4	3.3	3.4	4.7	5.3	6.2	6.9	7.4	6.9	5.7	62.7
	TO DISPOSAL/STORAGE (MG)	16.3	13.5	13.8	17.6	13.9	13.9	13.0	12.2	10.2	10.8	11.9	12.9	160.2
PERCOLATION PONDS	PERCOLATION (in)	67.0	64.8	67.0	67.0	60.5	67.0	64.8	67.0	64.8	67.0	67.0	64.8	788.4
	PERC. VOLUME (MG)	13.1	12.7	13.1	13.1	11.8	13.1	12.7	13.1	12.7	13.1	13.1	12.7	154.3
	EVAP. VOLUME (MG)	0.6	0.3	0.1	0.1	0.3	0.6	0.8	1.1	1.4	1.5	1.3	0.9	9.1
	PRECIP VOLUME (MG)	0.0	0.1	0.7	0.6	0.6	0.4	0.3	0.2	0.0	0.0	0.0	0.1	3.1
	STORAGE POND DISPOSAL POTENTIAL (MG)	13.7	12.9	12.6	12.6	11.5	13.3	13.2	14.0	14.0	14.6	14.4	13.5	160.2
EFFLUENT IN STORAGE	BEGINNING VOLUME IN STORAGE POND (MG)	0	3	4	5	10	12	13	13	11	7	3	1	
	STORAGE GAIN (MG)	3	1	1	5	2	1	0	-2	-4	-4	-2	-1	
	FINAL STORAGE (MG)	3	4	5	10	12	13	13	11	7	3	1	0	

SUMMARY

NEEDED STORAGE VOLUME (MG) 13 40 ac-ft

ANNUAL INFLOW (MG)		ANNUAL OUTFLOW POTENTIAL (MG)		OVERALL BALANCE	
WASTEWATER	215	PERCOLATION	197	UNUSED POTENTIAL DISPOSAL CAPACITY (MG)	0
INFLOW AND INFILTRATION	8	EVAPORATION	32	(MUST NOT BE NEGATIVE)	
DIRECT PRECIPITATION	6			UNUSED STORAGE CAPACITY (MG)	11
TOTAL	229	TOTAL	229	(MUST NOT BE NEGATIVE)	

SITE NAME:	Escalon	WATER BALANCE - Existing Facilities	COLOR LEGEND
BASE FLOW (MGD):	1.16		DESC. TEXT
DESIGN PRECIPITATION:	100 year		CLIMATIC DATA
PERC./ INFIL. & INFLOW	100 year		DESIGN INPUT

CLIMATOLOGICAL FACTORS

PRECIP/AVG. PRECIP RATIO	1.81
OCT.-APR EVAP/AVG EVAP RATIO[1]	0.88
MAY-SEP EVAP/AVG EVAP RATIO[1]	0.95
ETo COEFFICIENT	1.00
LAND PRECIP COLLECTED	0.90
I/I ADJUSTMENT RATIO[2]	0.42
I/I REDUCTION FACTOR FOR NEW CONSTRUCT	0.00
ESTIMATED EDU	
TOTAL[3]	4654
EXISTING	2380
NEW	2274

TREATMENT POND CHARACTERISTICS

TREATMENT POND GROSS AREA (ac)	8.1
EVAP./PERC. AREA (ac)	7.0
DESIGN PERC. RATE (in/day)[5]	0.62
STORAGE AVAILABLE (MG)	5
AERATOR INDUCED EVAP AREA (ac)[4]	11

DISPOSAL POND CHARACTERISTICS

EXISTING	
GROSS AREA (ac)	20.1
EVAP./PERC. AREA (ac)	17.4
EXISTING STORAGE (MG)	22
NEW	
GROSS AREA (ac)	0.00
EVAP./PERC. AREA (ac)	0.00
NEW STORAGE (MG)	0.00

TOTAL STORAGE PROVIDED	
TOTAL STORAGE AVAILABLE (MG)	22
TOTAL STORAGE AVAILABLE (af)	85
GROSS AREA (ac)	20.1
EVAP./PERC. AREA (ac)	17.4
DESIGN PERC. RATE (in/day)[5]	2.160

	PARAMETER\ MONTHS IN WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	WATER YEAR
BASIC INPUT	DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
	AVG ETo (in)	0.87	1.71	3.43	5.24	6.70	7.40	7.85	6.75	4.93	3.37	1.66	0.87	50.78
	AVG PRECIP (in)	0.63	1.34	2.12	2.37	2.14	1.94	1.08	0.47	0.09	0.03	0.04	0.20	12.45
	I/I FLOW ESTIMATED BASED ON RATIO (MG)	0.51	1.09	1.73	1.93	1.74	1.58	0.88	0.38	0.07	0.02	0.03	0.16	10.14
	FRACTION OF DESIGN PERC RATE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DESIGN CONDITIONS	PRECIPITATION (in)	1.14	2.43	3.84	4.29	3.87	3.51	1.95	0.85	0.16	0.05	0.07	0.36	22.53
	EVAPORATION (in)	0.77	1.50	3.02	4.61	5.90	6.51	6.91	6.41	4.68	3.20	1.58	0.83	45.92
	I/I FLOW (MGD)[6]	0.03	0.07	0.10	0.11	0.11	0.09	0.05	0.02	0.00	0.00	0.00	0.01	0.61
	WWTP INFLUENT FLOW (MGD)	1.19	1.23	1.26	1.28	1.28	1.26	1.22	1.19	1.17	1.16	1.17	1.17	
	WWTP INFLUENT VOLUME (MG)	37.0	36.9	39.2	39.6	35.7	38.9	36.5	36.8	35.0	36.1	36.1	35.2	443.1
TREATMENT PONDS	PERCOLATION (in)	19.2	18.6	19.2	19.2	17.4	19.2	18.6	19.2	18.6	19.2	19.2	18.6	226.3
	PERC. VOLUME (MG)	3.7	3.5	3.7	3.7	3.3	3.7	3.5	3.7	3.5	3.7	3.7	3.5	43.1
	EVAP. VOLUME (MG)	0.4	0.7	1.5	2.3	2.9	3.2	3.4	3.1	2.3	1.6	0.8	0.4	22.5
	PRECIP. VOLUME (MG)	0.2	0.5	0.8	0.9	0.8	0.8	0.4	0.2	0.0	0.0	0.0	0.1	4.9
	POND DISPOSAL (MG)	3.8	3.8	4.3	5.0	5.4	6.1	6.5	6.6	5.8	5.2	4.4	3.9	60.7
	TO DISPOSAL/STORAGE (MG)	33.2	33.1	34.9	34.6	30.4	32.8	30.0	30.1	29.2	30.9	31.7	31.3	382.3
PERCOLATION PONDS	PERCOLATION (in)	67.0	64.8	67.0	67.0	60.5	67.0	64.8	67.0	64.8	67.0	67.0	64.8	788.4
	PERC. VOLUME (MG)	31.7	30.6	31.7	31.7	28.6	31.7	30.6	31.7	30.6	31.7	31.7	30.6	372.8
	EVAP. VOLUME (MG)	0.4	0.7	1.4	2.2	2.8	3.1	3.3	3.0	2.2	1.5	0.7	0.4	21.7
	PRECIP VOLUME (MG)	0.6	1.3	2.1	2.3	2.1	1.9	1.1	0.5	0.1	0.0	0.0	0.2	12.1
	STORAGE POND DISPOSAL POTENTIAL (MG)	31.4	30.0	31.0	31.5	29.3	32.8	32.9	34.2	32.8	33.1	32.4	30.8	382.3
EFFLUENT IN STORAGE	BEGINNING VOLUME IN STORAGE POND (MG)	0	2	5	9	12	13	13	10	6	2	0	0	
	STORAGE GAIN (MG)	2	3	4	3	1	0	-3	-4	-4	-2	-1	0	
	FINAL STORAGE (MG)	2	5	9	12	13	13	10	6	2	0	0	0	

SUMMARY

NEEDED STORAGE VOLUME (MG) 13 40 ac-ft

ANNUAL INFLOW (MG)		ANNUAL OUTFLOW POTENTIAL (MG)		OVERALL BALANCE	
WASTEWATER	425	PERCOLATION	416	UNUSED POTENTIAL DISPOSAL CAPACITY (MG)	0
INFLOW AND INFILTRATION	18	EVAPORATION	44	(MUST NOT BE NEGATIVE)	
DIRECT PRECIPITATION	17			UNUSED STORAGE CAPACITY (MG)	14
TOTAL	460	TOTAL	460	(MUST NOT BE NEGATIVE)	

SITE NAME:	Escalon	WATER BALANCE - Phase I conversion of P21 to lined aerated pond @ 1.23 MGD preceded by construction of 10 ac of disposal ponds	
BASE FLOW (MGD):	1.50		
DESIGN PRECIPITATION:	100 year		
PERC./ INFIL. & INFLOW	100 year		

COLOR LEGEND	
DESC. TEXT	
CLIMATIC DATA	
DESIGN INPUT	

CLIMATOLOGICAL FACTORS

PRECIP/AVG. PRECIP RATIO	1.81
OCT.-APR EVAP/AVG EVAP RATIO[1]	0.88
MAY-SEP EVAP/AVG EVAP RATIO[1]	0.95
ETo COEFFICIENT	1.00
LAND PRECIP COLLECTED	0.90
I/I ADJUSTMENT RATIO[2]	0.42
I/I REDUCTION FACTOR FOR NEW CONSTRUCT	0.00
ESTIMATED EDU	
TOTAL[3]	5000
EXISTING	2380
NEW	2620

TREATMENT POND CHARACTERISTICS

TREATMENT POND GROSS AREA (ac)	8.1
EVAP./PERC. AREA (ac)	7.0
DESIGN PERC. RATE (in/day)[5]	0.62
STORAGE AVAILABLE (MG)	5
AERATOR INDUCED EVAP AREA (ac)[4]	11
NEW	
TREATMENT POND GROSS AREA (ac)	4.3
EVAP./PERC. AREA (ac)	3.8
DESIGN PERC. RATE (in/day)[5]	0.00
STORAGE AVAILABLE (MG)	2.5
AERATOR INDUCED EVAP AREA (ac)[4]	2

DISPOSAL POND CHARACTERISTICS

EXISTING		TOTAL STORAGE PROVIDED	
GROSS AREA (ac)	15.7	TOTAL STORAGE AVAILABLE (MG)	24
EVAP./PERC. AREA (ac)	13.6	TOTAL STORAGE AVAILABLE (af)	94
EXISTING STORAGE (MG)	15	GROSS AREA (ac)	26.3
NEW		EVAP./PERC. AREA (ac)	22.9
GROSS AREA (ac)	10.67	DESIGN PERC. RATE (in/day)[5]	2.160
EVAP./PERC. AREA (ac)	9.28		
NEW STORAGE (MG)	9.00		

	PARAMETER\ MONTHS IN WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	WATER YEAR
BASIC INPUT	DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
	AVG ETo (in)	0.87	1.71	3.43	5.24	6.70	7.40	7.85	6.75	4.93	3.37	1.66	0.87	50.78
	AVG PRECIP (in)	0.63	1.34	2.12	2.37	2.14	1.94	1.08	0.47	0.09	0.03	0.04	0.20	12.45
	I/I FLOW ESTIMATED BASED ON RATIO (MG)	0.66	1.41	2.23	2.49	2.25	2.04	1.13	0.49	0.09	0.03	0.04	0.21	13.07
	FRACTION OF DESIGN PERC RATE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DESIGN CONDITIONS	PRECIPITATION (in)	1.14	2.43	3.84	4.29	3.87	3.51	1.95	0.85	0.16	0.05	0.07	0.36	22.53
	EVAPORATION (in)	0.77	1.50	3.02	4.61	5.90	6.51	6.91	6.41	4.68	3.20	1.58	0.83	45.92
	I/I FLOW (MGD)[6]	0.04	0.08	0.13	0.15	0.15	0.12	0.07	0.03	0.01	0.00	0.00	0.01	0.78
	WWTP INFLUENT FLOW (MGD)	1.54	1.58	1.63	1.65	1.65	1.62	1.57	1.53	1.51	1.50	1.50	1.51	
	WWTP INFLUENT VOLUME (MG)	47.7	47.5	50.5	51.0	46.1	50.2	47.1	47.4	45.2	46.6	46.6	45.4	571.2
TREATMENT PONDS	PERCOLATION (in)	19.2	18.6	19.2	19.2	17.4	19.2	18.6	19.2	18.6	19.2	19.2	18.6	226.3
	PERC. VOLUME (MG)	3.7	3.5	3.7	3.7	3.3	3.7	3.5	3.7	3.5	3.7	3.7	3.5	43.1
	EVAP. VOLUME (MG)	0.5	1.0	2.0	3.0	3.8	4.2	4.5	4.1	3.0	2.1	1.0	0.5	29.7
	PRECIP. VOLUME (MG)	0.2	0.5	0.8	0.9	0.8	0.8	0.4	0.2	0.0	0.0	0.0	0.1	4.9
	POND DISPOSAL (MG)	3.9	4.0	4.8	5.7	6.3	7.1	7.6	7.6	6.5	5.7	4.7	4.0	67.9
	TO DISPOSAL/STORAGE (MG)	43.8	43.6	45.7	45.3	39.8	43.1	39.5	39.8	38.6	40.8	41.9	41.4	503.2
PERCOLATION PONDS	PERCOLATION (in)	67.0	64.8	67.0	67.0	60.5	67.0	64.8	67.0	64.8	67.0	67.0	64.8	788.4
	PERC. VOLUME (MG)	41.7	40.3	41.7	41.7	37.6	41.7	40.3	41.7	40.3	41.7	41.7	40.3	490.6
	EVAP. VOLUME (MG)	0.5	0.9	1.9	2.9	3.7	4.1	4.3	4.0	2.9	2.0	1.0	0.5	28.6
	PRECIP VOLUME (MG)	0.8	1.7	2.7	3.0	2.7	2.5	1.4	0.6	0.1	0.0	0.1	0.3	15.9
	STORAGE POND DISPOSAL POTENTIAL (MG)	41.3	39.5	40.8	41.5	38.6	43.2	43.2	45.1	43.1	43.6	42.6	40.6	503.2
EFFLUENT IN STORAGE	BEGINNING VOLUME IN STORAGE POND (MG)	0	2	6	11	15	16	16	12	7	3	0	0	
	STORAGE GAIN (MG)	2	4	5	4	1	0	-4	-5	-4	-3	-1	1	
	FINAL STORAGE (MG)	2	6	11	15	16	16	12	7	3	0	0	1	

SUMMARY

NEEDED STORAGE VOLUME (MG) 16 49 ac-ft

ANNUAL INFLOW (MG)		ANNUAL OUTFLOW POTENTIAL (MG)		OVERALL BALANCE	
WASTEWATER	548	PERCOLATION	534	UNUSED POTENTIAL DISPOSAL CAPACITY (MG)	0
INFLOW AND INFILTRATION	24	EVAPORATION	58	(MUST NOT BE NEGATIVE)	
DIRECT PRECIPITATION	21			UNUSED STORAGE CAPACITY (MG)	13
TOTAL	592	TOTAL	592	(MUST NOT BE NEGATIVE)	

SITE NAME:	Escalon	WATER BALANCE - Phase II conversion of P 22 to lined aerated pond @ 1.82 MGD preceded by construction of 10 ac of Disposal ponds @ 1.5 MGD	COLOR LEGEND
BASE FLOW (MGD):	2.43		DESC. TEXT
DESIGN PRECIPITATION:	100 year		CLIMATIC DATA
PERC./ INFIL. & INFLOW	100 year		DESIGN INPUT

CLIMATOLOGICAL FACTORS	
PRECIP/AVG. PRECIP RATIO	1.81
OCT.-APR EVAP/AVG EVAP RATIO[1]	0.88
MAY-SEP EVAP/AVG EVAP RATIO[1]	0.95
ETo COEFFICIENT	1.00
LAND PRECIP COLLECTED	0.90
I/I ADJUSTMENT RATIO[2]	0.42
I/I REDUCTION FACTOR FOR NEW CONSTRUCT	0.00
ESTIMATED EDU	
TOTAL[3]	9723
EXISTING	2380
NEW	7343

TREATMENT POND CHARACTERISTICS	
EXISTING	
TREATMENT POND GROSS AREA (ac)	8.1
EVAP./PERC. AREA (ac)	7.0
DESIGN PERC. RATE (in/day)[5]	0.62
STORAGE AVAILABLE (MG)	5
AERATOR INDUCED EVAP AREA (ac)[4]	22
NEW	
TREATMENT POND GROSS AREA (ac)	11.5
EVAP./PERC. AREA (ac)	10.0
DESIGN PERC. RATE (in/day)[5]	0.00
STORAGE AVAILABLE (MG)	7
AERATOR INDUCED EVAP AREA (ac)[4]	7

DISPOSAL POND CHARACTERISTICS			
EXISTING		TOTAL STORAGE PROVIDED	
GROSS AREA (ac)	8.4	TOTAL STORAGE AVAILABLE (MG)	29
EVAP./PERC. AREA (ac)	7.3	TOTAL STORAGE AVAILABLE (af)	113
EXISTING STORAGE (MG)	10	GROSS AREA (ac)	31.4
NEW		EVAP./PERC. AREA (ac)	27.3
GROSS AREA (ac)	23.00	DESIGN PERC. RATE (in/day)[5]	3.090
EVAP./PERC. AREA (ac)	20.00		
NEW STORAGE (MG)	19.55		

BASIC INPUT	PARAMETER\ MONTHS IN WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	WATER YEAR
	DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
	AVG ETo (in)	0.87	1.71	3.43	5.24	6.70	7.40	7.85	6.75	4.93	3.37	1.66	0.87	50.78
	AVG PRECIP (in)	0.63	1.34	2.12	2.37	2.14	1.94	1.08	0.47	0.09	0.03	0.04	0.20	12.45
	I/I FLOW ESTIMATED BASED ON RATIO (MG)	1.07	2.28	3.61	4.03	3.64	3.30	1.84	0.80	0.15	0.05	0.07	0.34	21.18
	FRACTION OF DESIGN PERC RATE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DESIGN CONDITIONS	PRECIPITATION (in)	1.14	2.43	3.84	4.29	3.87	3.51	1.95	0.85	0.16	0.05	0.07	0.36	22.53
	EVAPORATION (in)	0.77	1.50	3.02	4.61	5.90	6.51	6.91	6.41	4.68	3.20	1.58	0.83	45.92
	I/I FLOW (MGD)[6]	0.06	0.14	0.21	0.24	0.24	0.19	0.11	0.05	0.01	0.00	0.00	0.02	1.27
	WWTP INFLUENT FLOW (MGD)	2.49	2.57	2.64	2.67	2.67	2.62	2.54	2.48	2.44	2.43	2.43	2.45	
	WWTP INFLUENT VOLUME (MG)	77.3	77.1	81.9	82.7	74.7	81.3	76.3	76.8	73.2	75.4	75.5	73.5	925.6
TREATMENT PONDS	PERCOLATION (in)	19.2	18.6	19.2	19.2	17.4	19.2	18.6	19.2	18.6	19.2	19.2	18.6	226.3
	PERC. VOLUME (MG)	3.7	3.5	3.7	3.7	3.3	3.7	3.5	3.7	3.5	3.7	3.7	3.5	43.1
	EVAP. VOLUME (MG)	0.6	1.2	2.4	3.6	4.6	5.1	5.4	5.1	3.7	2.5	1.2	0.7	36.2
	PRECIP. VOLUME (MG)	0.2	0.5	0.8	0.9	0.8	0.8	0.4	0.2	0.0	0.0	0.0	0.1	4.9
	POND DISPOSAL (MG)	4.0	4.2	5.2	6.4	7.1	8.0	8.6	8.5	7.2	6.2	4.9	4.1	74.4
	TO DISPOSAL/STORAGE (MG)	73.3	72.8	76.7	76.3	67.5	73.3	67.7	68.3	66.0	69.3	70.6	69.4	851.2
PERCOLATION PONDS	PERCOLATION (in)	95.8	92.7	95.8	95.8	86.5	95.8	92.7	95.8	92.7	95.8	95.8	92.7	1127.9
	PERC. VOLUME (MG)	71.0	68.7	71.0	71.0	64.1	71.0	68.7	71.0	68.7	71.0	71.0	68.7	836.1
	EVAP. VOLUME (MG)	0.6	1.1	2.2	3.4	4.4	4.8	5.1	4.8	3.5	2.4	1.2	0.6	34.0
	PRECIP VOLUME (MG)	1.0	2.0	3.2	3.6	3.3	3.0	1.6	0.7	0.1	0.0	0.1	0.3	19.0
	STORAGE POND DISPOSAL POTENTIAL (MG)	70.6	67.8	70.0	70.8	65.2	72.9	72.2	75.0	72.1	73.3	72.1	69.0	851.2
EFFLUENT IN STORAGE	BEGINNING VOLUME IN STORAGE POND (MG)	0	3	8	15	20	22	22	17	10	4	0	0	
	STORAGE GAIN (MG)	3	5	7	5	2	0	-5	-7	-6	-4	-2	0	
	FINAL STORAGE (MG)	3	8	15	20	22	22	17	10	4	0	0	0	

SUMMARY

NEEDED STORAGE VOLUME (MG) 22 68 ac-ft

ANNUAL INFLOW (MG)		ANNUAL OUTFLOW POTENTIAL (MG)		OVERALL BALANCE	
WASTEWATER	887	PERCOLATION	879	UNUSED POTENTIAL DISPOSAL CAPACITY (MG)	0
INFLOW AND INFILTRATION	38	EVAPORATION	70	(MUST NOT BE NEGATIVE)	
DIRECT PRECIPITATION	24			UNUSED STORAGE CAPACITY (MG)	12
TOTAL	949	TOTAL	949	(MUST NOT BE NEGATIVE)	

SITE NAME:	Escalon	WATER BALANCE - 2035 Phase III Construction of 5 ac Disposal Pond	COLOR LEGEND	
BASE FLOW (MGD):	2.80		DESC. TEXT	
DESIGN PRECIPITATION:	100 year		CLIMATIC DATA	
PERC./ INFIL. & INFLOW	100 year		DESIGN INPUT	

CLIMATOLOGICAL FACTORS

PRECIP/AVG. PRECIP RATIO	1.81
OCT.-APR EVAP/AVG EVAP RATIO[1]	0.88
MAY-SEP EVAP/AVG EVAP RATIO[1]	0.95
ETo COEFFICIENT	1.00
LAND PRECIP COLLECTED	0.90
I/I ADJUSTMENT RATIO[2]	0.42
I/I REDUCTION FACTOR FOR NEW CONSTRUCT	0.00
ESTIMATED EDU	
TOTAL[3]	11200
EXISTING	2380
NEW	8820

TREATMENT POND CHARACTERISTICS

EXISTING	
TREATMENT POND GROSS AREA (ac)	8.1
EVAP./PERC. AREA (ac)	7.0
DESIGN PERC. RATE (in/day)[5]	0.62
STORAGE AVAILABLE (MG)	5
AERATOR INDUCED EVAP AREA (ac)[4]	22
NEW	
TREATMENT POND GROSS AREA (ac)	11.5
EVAP./PERC. AREA (ac)	10.0
DESIGN PERC. RATE (in/day)[5]	0.00
STORAGE AVAILABLE (MG)	7
AERATOR INDUCED EVAP AREA (ac)[4]	7

DISPOSAL POND CHARACTERISTICS

EXISTING		TOTAL STORAGE PROVIDED	
GROSS AREA (ac)	8.4	TOTAL STORAGE AVAILABLE (MG)	34
EVAP./PERC. AREA (ac)	7.3	TOTAL STORAGE AVAILABLE (af)	130
EXISTING STORAGE (MG)	10	GROSS AREA (ac)	36.6
NEW		EVAP./PERC. AREA (ac)	31.8
GROSS AREA (ac)	28.18	DESIGN PERC. RATE (in/day)[5]	3.090
EVAP./PERC. AREA (ac)	24.51		
NEW STORAGE (MG)	23.95		

BASIC INPUT	PARAMETER\ MONTHS IN WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	WATER YEAR
	DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
	AVG ETo (in)	0.87	1.71	3.43	5.24	6.70	7.40	7.85	6.75	4.93	3.37	1.66	0.87	50.78
	AVG PRECIP (in)	0.63	1.34	2.12	2.37	2.14	1.94	1.08	0.47	0.09	0.03	0.04	0.20	12.45
	I/I FLOW ESTIMATED BASED ON RATIO (MG)	1.23	2.63	4.16	4.65	4.19	3.80	2.12	0.92	0.18	0.06	0.08	0.39	24.40
	FRACTION OF DESIGN PERC RATE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DESIGN CONDITIONS	PRECIPITATION (in)	1.14	2.43	3.84	4.29	3.87	3.51	1.95	0.85	0.16	0.05	0.07	0.36	22.53
	EVAPORATION (in)	0.77	1.50	3.02	4.61	5.90	6.51	6.91	6.41	4.68	3.20	1.58	0.83	45.92
	I/I FLOW (MGD)[6]	0.07	0.16	0.24	0.27	0.27	0.22	0.13	0.05	0.01	0.00	0.00	0.02	1.46
	WWTP INFLUENT FLOW (MGD)	2.87	2.96	3.04	3.07	3.07	3.02	2.93	2.85	2.81	2.80	2.80	2.82	
	WWTP INFLUENT VOLUME (MG)	89.0	88.8	94.3	95.2	86.0	93.7	87.8	88.5	84.3	86.9	86.9	84.7	1066.2
TREATMENT PONDS	PERCOLATION (in)	19.2	18.6	19.2	19.2	17.4	19.2	18.6	19.2	18.6	19.2	19.2	18.6	226.3
	PERC. VOLUME (MG)	3.7	3.5	3.7	3.7	3.3	3.7	3.5	3.7	3.5	3.7	3.7	3.5	43.1
	EVAP. VOLUME (MG)	0.6	1.2	2.4	3.6	4.6	5.1	5.4	5.1	3.7	2.5	1.2	0.7	36.2
	PRECIP. VOLUME (MG)	0.2	0.5	0.8	0.9	0.8	0.8	0.4	0.2	0.0	0.0	0.0	0.1	4.9
	POND DISPOSAL (MG)	4.0	4.2	5.2	6.4	7.1	8.0	8.6	8.5	7.2	6.2	4.9	4.1	74.4
	TO DISPOSAL/STORAGE (MG)	85.0	84.5	89.1	88.8	78.9	85.6	79.3	79.9	77.1	80.7	82.1	80.6	991.7
PERCOLATION PONDS	PERCOLATION (in)	95.8	92.7	95.8	95.8	86.5	95.8	92.7	95.8	92.7	95.8	95.8	92.7	1127.9
	PERC. VOLUME (MG)	82.7	80.1	82.7	82.7	74.7	82.7	80.1	82.7	80.1	82.7	82.7	80.1	974.2
	EVAP. VOLUME (MG)	0.7	1.3	2.6	4.0	5.1	5.6	6.0	5.5	4.0	2.8	1.4	0.7	39.7
	PRECIP VOLUME (MG)	1.1	2.4	3.8	4.2	3.8	3.4	1.9	0.8	0.2	0.1	0.1	0.4	22.1
	STORAGE POND DISPOSAL POTENTIAL (MG)	82.3	79.0	81.6	82.5	76.0	84.9	84.1	87.4	84.0	85.4	84.0	80.4	991.7
EFFLUENT IN STORAGE	BEGINNING VOLUME IN STORAGE POND (MG)	0	3	9	17	23	26	27	22	14	7	2	0	
	STORAGE GAIN (MG)	3	6	8	6	3	1	-5	-8	-7	-5	-2	0	
	FINAL STORAGE (MG)	3	9	17	23	26	27	22	14	7	2	0	0	

SUMMARY

NEEDED STORAGE VOLUME (MG) 27 83 ac-ft

ANNUAL INFLOW (MG)		ANNUAL OUTFLOW POTENTIAL (MG)		OVERALL BALANCE	
WASTEWATER	1022	PERCOLATION	1017	UNUSED POTENTIAL DISPOSAL CAPACITY (MG)	0
INFLOW AND INFILTRATION	44	EVAPORATION	76	(MUST NOT BE NEGATIVE)	
DIRECT PRECIPITATION	27			UNUSED STORAGE CAPACITY (MG)	11
TOTAL	1093	TOTAL	1093	(MUST NOT BE NEGATIVE)	

SITE NAME:	Escalon	WATER BALANCE -Activated Sludge Treatment Phase I	COLOR LEGEND	
BASE FLOW (MGD):	1.36		DESC. TEXT	
DESIGN PRECIPITATION:	100 year		CLIMATIC DATA	
PERC./ INFIL. & INFLOW	100 year		DESIGN INPUT	

CLIMATOLOGICAL FACTORS	
PRECIP/AVG. PRECIP RATIO	1.81
OCT.-APR EVAP/AVG EVAP RATIO[1]	0.88
MAY-SEP EVAP/AVG EVAP RATIO[1]	0.95
ETo COEFFICIENT	1.00
LAND PRECIP COLLECTED	0.90
I/I ADJUSTMENT RATIO[2]	0.42
I/I REDUCTION FACTOR FOR NEW CONSTRUCT	0.00
ESTIMATED EDU	
TOTAL[3]	4544
EXISTING	0
NEW	4544

TREATMENT POND CHARACTERISTICS	
TREATMENT POND GROSS AREA (ac)	3.8
EVAP./PERC. AREA (ac)	1.9
DESIGN PERC. RATE (in/day)[5]	0.00
STORAGE AVAILABLE (MG)	0
AERATOR INDUCED EVAP AREA (ac)[4]	18

DISPOSAL POND CHARACTERISTICS			
EXISTING		TOTAL STORAGE PROVIDED	
GROSS AREA (ac)	20.1	TOTAL STORAGE AVAILABLE (MG)	39
EVAP./PERC. AREA (ac)	17.4	TOTAL STORAGE AVAILABLE (af)	151
EXISTING STORAGE (MG)	22	GROSS AREA (ac)	26.0
NEW		EVAP./PERC. AREA (ac)	22.6
GROSS AREA (ac)	5.9	DESIGN PERC. RATE (in/day)[5]	2.160
EVAP./PERC. AREA (ac)	5.2		
NEW STORAGE (MG)	17		

BASIC INPUT	PARAMETER\ MONTHS IN WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	WATER YEAR
	DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
	AVG ETo (in)	0.87	1.71	3.43	5.24	6.70	7.40	7.85	6.75	4.93	3.37	1.66	0.87	50.78
	AVG PRECIP (in)	0.63	1.34	2.12	2.37	2.14	1.94	1.08	0.47	0.09	0.03	0.04	0.20	12.45
	I/I FLOW ESTIMATED BASED ON RATIO (MG)	0.60	1.28	2.02	2.26	2.04	1.85	1.03	0.45	0.09	0.03	0.04	0.19	11.88
	FRACTION OF DESIGN PERC RATE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DESIGN CONDITIONS	PRECIPITATION (in)	1.14	2.43	3.84	4.29	3.87	3.51	1.95	0.85	0.16	0.05	0.07	0.36	22.53
	EVAPORATION (in)	0.77	1.50	3.02	4.61	5.90	6.51	6.91	6.41	4.68	3.20	1.58	0.83	45.92
	I/I FLOW (MGD)[6]	0.04	0.08	0.12	0.13	0.13	0.11	0.06	0.03	0.01	0.00	0.00	0.01	0.71
	WWTP INFLUENT FLOW (MGD)	1.40	1.44	1.48	1.50	1.50	1.47	1.43	1.39	1.37	1.36	1.37	1.37	
	WWTP INFLUENT VOLUME (MG)	43.3	43.2	45.9	46.3	41.9	45.6	42.8	43.1	41.0	42.3	42.3	41.2	519.0
TREATMENT PONDS	PERCOLATION (in)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	PERC. VOLUME (MG)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	EVAP. VOLUME (MG)	0.4	0.8	1.6	2.5	3.2	3.5	3.7	3.5	2.5	1.7	0.9	0.4	24.8
	PRECIP. VOLUME (MG)	0.1	0.2	0.4	0.4	0.4	0.3	0.2	0.1	0.0	0.0	0.0	0.0	2.2
	POND DISPOSAL (MG)	0.3	0.6	1.3	2.1	2.8	3.2	3.5	3.4	2.5	1.7	0.8	0.4	22.6
	TO DISPOSAL/STORAGE (MG)	43.0	42.6	44.7	44.3	39.1	42.4	39.2	39.7	38.5	40.6	41.5	40.8	496.4
PERCOLATION PONDS	PERCOLATION (in)	67.0	64.8	67.0	67.0	60.5	67.0	64.8	67.0	64.8	67.0	67.0	64.8	788.4
	PERC. VOLUME (MG)	41.0	39.7	41.0	41.0	37.1	41.0	39.7	41.0	39.7	41.0	41.0	39.7	483.1
	EVAP. VOLUME (MG)	0.5	0.9	1.8	2.8	3.6	4.0	4.2	3.9	2.9	2.0	1.0	0.5	28.1
	PRECIP VOLUME (MG)	0.8	1.7	2.7	3.0	2.7	2.4	1.4	0.6	0.1	0.0	0.1	0.3	15.7
	STORAGE POND DISPOSAL POTENTIAL (MG)	40.7	38.9	40.2	40.9	38.0	42.6	42.6	44.4	42.5	43.0	41.9	40.0	495.5
EFFLUENT IN STORAGE	BEGINNING VOLUME IN STORAGE POND (MG)	0	2	6	10	13	14	14	11	6	2	0	0	
	STORAGE GAIN (MG)	2	4	4	3	1	0	-3	-5	-4	-2	0	1	
	FINAL STORAGE (MG)	2	6	10	13	14	14	11	6	2	0	0	1	

SUMMARY

NEEDED STORAGE VOLUME (MG) 14 43 ac-ft

ANNUAL INFLOW (MG)		ANNUAL OUTFLOW POTENTIAL (MG)		OVERALL BALANCE	
WASTEWATER	498	PERCOLATION	483	UNUSED POTENTIAL DISPOSAL CAPACITY (MG)	-1
INFLOW AND INFILTRATION	22	EVAPORATION	53	(MUST NOT BE NEGATIVE)	
DIRECT PRECIPITATION	18			UNUSED STORAGE CAPACITY (MG)	25
TOTAL	537	TOTAL	536	(MUST NOT BE NEGATIVE)	

SITE NAME:	Escalon	WATER BALANCE - Activated Sludge Treatment Phase II	COLOR LEGEND	
BASE FLOW (MGD):	2.23		DESC. TEXT	
DESIGN PRECIPITATION:	100 year		CLIMATIC DATA	
PERC./ INFIL. & INFLOW	100 year		DESIGN INPUT	

CLIMATOLOGICAL FACTORS

PRECIP/AVG. PRECIP RATIO	1.81
OCT.-APR EVAP/AVG EVAP RATIO[1]	0.88
MAY-SEP EVAP/AVG EVAP RATIO[1]	0.95
ETo COEFFICIENT	1.00
LAND PRECIP COLLECTED	0.90
I/I ADJUSTMENT RATIO[2]	0.42
I/I REDUCTION FACTOR FOR NEW CONSTRUCT	0.00
ESTIMATED EDU	
TOTAL[3]	7420
EXISTING	0
NEW	7420

TREATMENT POND CHARACTERISTICS

TREATMENT POND GROSS AREA (ac)	2.2
EVAP./PERC. AREA (ac)	1.9
DESIGN PERC. RATE (in/day)[5]	0.00
STORAGE AVAILABLE (MG)	0
AERATOR INDUCED EVAP AREA (ac)[4]	18

DISPOSAL POND CHARACTERISTICS

EXISTING	
GROSS AREA (ac)	25.9
EVAP./PERC. AREA (ac)	22.5
EXISTING STORAGE (MG)	39
NEW	
GROSS AREA (ac)	17.3
EVAP./PERC. AREA (ac)	15.0
NEW STORAGE (MG)	14.6

TOTAL STORAGE PROVIDED	
TOTAL STORAGE AVAILABLE (MG)	54
TOTAL STORAGE AVAILABLE (af)	208
GROSS AREA (ac)	43.1
EVAP./PERC. AREA (ac)	37.5
DESIGN PERC. RATE (in/day)[5]	2.160

BASIC INPUT	PARAMETER\ MONTHS IN WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	WATER YEAR
	DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
	AVG ETo (in)	0.87	1.71	3.43	5.24	6.70	7.40	7.85	6.75	4.93	3.37	1.66	0.87	50.78
	AVG PRECIP (in)	0.63	1.34	2.12	2.37	2.14	1.94	1.08	0.47	0.09	0.03	0.04	0.20	12.45
	I/I FLOW ESTIMATED BASED ON RATIO (MG)	0.98	2.09	3.30	3.69	3.33	3.02	1.68	0.73	0.14	0.05	0.06	0.31	19.40
	FRACTION OF DESIGN PERC RATE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DESIGN CONDITIONS	PRECIPITATION (in)	1.14	2.43	3.84	4.29	3.87	3.51	1.95	0.85	0.16	0.05	0.07	0.36	22.53
	EVAPORATION (in)	0.77	1.50	3.02	4.61	5.90	6.51	6.91	6.41	4.68	3.20	1.58	0.83	45.92
	I/I FLOW (MGD)[6]	0.06	0.13	0.19	0.22	0.22	0.18	0.10	0.04	0.01	0.00	0.00	0.02	1.16
	WWTP INFLUENT FLOW (MGD)	2.28	2.35	2.42	2.44	2.44	2.40	2.33	2.27	2.23	2.23	2.23	2.24	
	WWTP INFLUENT VOLUME (MG)	70.8	70.6	75.0	75.7	68.4	74.5	69.8	70.3	67.0	69.1	69.1	67.3	847.6
TREATMENT PONDS	PERCOLATION (in)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	PERC. VOLUME (MG)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	EVAP. VOLUME (MG)	0.4	0.8	1.6	2.5	3.2	3.5	3.7	3.5	2.5	1.7	0.9	0.4	24.8
	PRECIP. VOLUME (MG)	0.1	0.1	0.2	0.3	0.2	0.2	0.1	0.0	0.0	0.0	0.0	0.0	1.3
	POND DISPOSAL (MG)	0.3	0.7	1.4	2.2	3.0	3.3	3.6	3.4	2.5	1.7	0.8	0.4	23.5
	TO DISPOSAL/STORAGE (MG)	70.4	69.9	73.6	73.5	65.4	71.2	66.2	66.9	64.5	67.4	68.3	66.9	824.1
PERCOLATION PONDS	PERCOLATION (in)	67.0	64.8	67.0	67.0	60.5	67.0	64.8	67.0	64.8	67.0	67.0	64.8	788.4
	PERC. VOLUME (MG)	68.2	66.0	68.2	68.2	61.6	68.2	66.0	68.2	66.0	68.2	68.2	66.0	803.4
	EVAP. VOLUME (MG)	0.8	1.5	3.1	4.7	6.0	6.6	7.0	6.5	4.8	3.3	1.6	0.8	46.8
	PRECIP VOLUME (MG)	1.3	2.8	4.4	5.0	4.5	4.1	2.3	1.0	0.2	0.1	0.1	0.4	26.1
	STORAGE POND DISPOSAL POTENTIAL (MG)	67.7	64.8	66.9	68.0	63.2	70.8	70.8	73.8	70.6	71.4	69.8	66.5	824.1
EFFLUENT IN STORAGE	BEGINNING VOLUME IN STORAGE POND (MG)	0	3	8	15	20	22	22	17	10	4	0	0	
	STORAGE GAIN (MG)	3	5	7	5	2	0	-5	-7	-6	-4	-1	0	
	FINAL STORAGE (MG)	3	8	15	20	22	22	17	10	4	0	0	0	

SUMMARY

NEEDED STORAGE VOLUME (MG) 22 68 ac-ft

ANNUAL INFLOW (MG)		ANNUAL OUTFLOW POTENTIAL (MG)		OVERALL BALANCE	
WASTEWATER	813	PERCOLATION	803	UNUSED POTENTIAL DISPOSAL CAPACITY (MG)	0
INFLOW AND INFILTRATION	35	EVAPORATION	72	(MUST NOT BE NEGATIVE)	
DIRECT PRECIPITATION	27			UNUSED STORAGE CAPACITY (MG)	32
TOTAL	875	TOTAL	875	(MUST NOT BE NEGATIVE)	

SITE NAME:	Escalon	WATER BALANCE - Activated Sludge Treatment Phase III	COLOR LEGEND	
BASE FLOW (MGD):	2.80		DESC. TEXT	
DESIGN PRECIPITATION:	100 year		CLIMATIC DATA	
PERC./ INFIL. & INFLOW	100 year		DESIGN INPUT	

CLIMATOLOGICAL FACTORS

PRECIP/AVG. PRECIP RATIO	1.81
OCT.-APR EVAP/AVG EVAP RATIO[1]	0.88
MAY-SEP EVAP/AVG EVAP RATIO[1]	0.95
ETo COEFFICIENT	1.00
LAND PRECIP COLLECTED	0.90
I/I ADJUSTMENT RATIO[2]	0.42
I/I REDUCTION FACTOR FOR NEW CONSTRUCT	0.00
ESTIMATED EDU	
TOTAL[3]	9333
EXISTING	0
NEW	9333

TREATMENT POND CHARACTERISTICS

TREATMENT POND GROSS AREA (ac)	2.2
EVAP./PERC. AREA (ac)	1.9
DESIGN PERC. RATE (in/day)[5]	0.00
STORAGE AVAILABLE (MG)	0
AERATOR INDUCED EVAP AREA (ac)[4]	18

DISPOSAL POND CHARACTERISTICS

EXISTING		TOTAL STORAGE PROVIDED	
GROSS AREA (ac)	25.9	TOTAL STORAGE AVAILABLE (MG)	63
EVAP./PERC. AREA (ac)	22.5	TOTAL STORAGE AVAILABLE (af)	245
EXISTING STORAGE (MG)	39	GROSS AREA (ac)	54.6
NEW		EVAP./PERC. AREA (ac)	47.4
GROSS AREA (ac)	28.7	DESIGN PERC. RATE (in/day)[5]	2.160
EVAP./PERC. AREA (ac)	24.9		
NEW STORAGE (MG)	24.2		

	PARAMETER\ MONTHS IN WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	WATER YEAR
BASIC INPUT	DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
	AVG ETo (in)	0.87	1.71	3.43	5.24	6.70	7.40	7.85	6.75	4.93	3.37	1.66	0.87	50.78
	AVG PRECIP (in)	0.63	1.34	2.12	2.37	2.14	1.94	1.08	0.47	0.09	0.03	0.04	0.20	12.45
	I/I FLOW ESTIMATED BASED ON RATIO (MG)	1.23	2.63	4.16	4.65	4.19	3.80	2.12	0.92	0.18	0.06	0.08	0.39	24.40
	FRACTION OF DESIGN PERC RATE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DESIGN CONDITIONS	PRECIPITATION (in)	1.14	2.43	3.84	4.29	3.87	3.51	1.95	0.85	0.16	0.05	0.07	0.36	22.53
	EVAPORATION (in)	0.77	1.50	3.02	4.61	5.90	6.51	6.91	6.41	4.68	3.20	1.58	0.83	45.92
	I/I FLOW (MGD)[6]	0.07	0.16	0.24	0.27	0.27	0.22	0.13	0.05	0.01	0.00	0.00	0.02	1.46
	WWTP INFLUENT FLOW (MGD)	2.87	2.96	3.04	3.07	3.07	3.02	2.93	2.85	2.81	2.80	2.80	2.82	
	WWTP INFLUENT VOLUME (MG)	89.0	88.8	94.3	95.2	86.0	93.7	87.8	88.5	84.3	86.9	86.9	84.7	1066.2
TREATMENT PONDS	PERCOLATION (in)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	PERC. VOLUME (MG)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	EVAP. VOLUME (MG)	0.4	0.8	1.6	2.5	3.2	3.5	3.7	3.5	2.5	1.7	0.9	0.4	24.8
	PRECIP. VOLUME (MG)	0.1	0.1	0.2	0.3	0.2	0.2	0.1	0.0	0.0	0.0	0.0	0.0	1.3
	POND DISPOSAL (MG)	0.3	0.7	1.4	2.2	3.0	3.3	3.6	3.4	2.5	1.7	0.8	0.4	23.5
	TO DISPOSAL/STORAGE (MG)	88.7	88.1	92.9	93.0	83.0	90.4	84.2	85.0	81.8	85.2	86.1	84.3	1042.7
PERCOLATION PONDS	PERCOLATION (in)	67.0	64.8	67.0	67.0	60.5	67.0	64.8	67.0	64.8	67.0	67.0	64.8	788.4
	PERC. VOLUME (MG)	86.3	83.5	86.3	86.3	78.0	86.3	83.5	86.3	83.5	86.3	86.3	83.5	1016.4
	EVAP. VOLUME (MG)	1.0	1.9	3.9	5.9	7.6	8.4	8.9	8.3	6.0	4.1	2.0	1.1	59.2
	PRECIP VOLUME (MG)	1.7	3.5	5.6	6.3	5.7	5.1	2.9	1.2	0.2	0.1	0.1	0.5	33.0
	STORAGE POND DISPOSAL POTENTIAL (MG)	85.6	81.9	84.6	86.0	79.9	89.6	89.6	93.3	89.3	90.4	88.3	84.1	1042.7
EFFLUENT IN STORAGE	BEGINNING VOLUME IN STORAGE POND (MG)	0	3	9	17	24	27	28	23	15	7	2	0	
	STORAGE GAIN (MG)	3	6	8	7	3	1	-5	-8	-8	-5	-2	0	
	FINAL STORAGE (MG)	3	9	17	24	27	28	23	15	7	2	0	0	

SUMMARY

NEEDED STORAGE VOLUME (MG) 28 86 ac-ft

ANNUAL INFLOW (MG)		ANNUAL OUTFLOW POTENTIAL (MG)		OVERALL BALANCE	
WASTEWATER	1022	PERCOLATION	1016	UNUSED POTENTIAL DISPOSAL CAPACITY (MG)	0
INFLOW AND INFILTRATION	44	EVAPORATION	84	(MUST NOT BE NEGATIVE)	
DIRECT PRECIPITATION	34			UNUSED STORAGE CAPACITY (MG)	35
TOTAL	1100	TOTAL	1100	(MUST NOT BE NEGATIVE)	

Appendix C

Activated Sludge-Alternative Conceptual Layout

Z:\Projects\1 Drawings\ESCA05-001\escalon WWTF prelim.dwg
Aug 14, 2006 - 1:58pm

U:project_graphics/Escalon_City_of/ESCA05-001 - Wastewater/|Figure_08_01_Activated_Sludge_Alternative.ai 8/15/06 cms

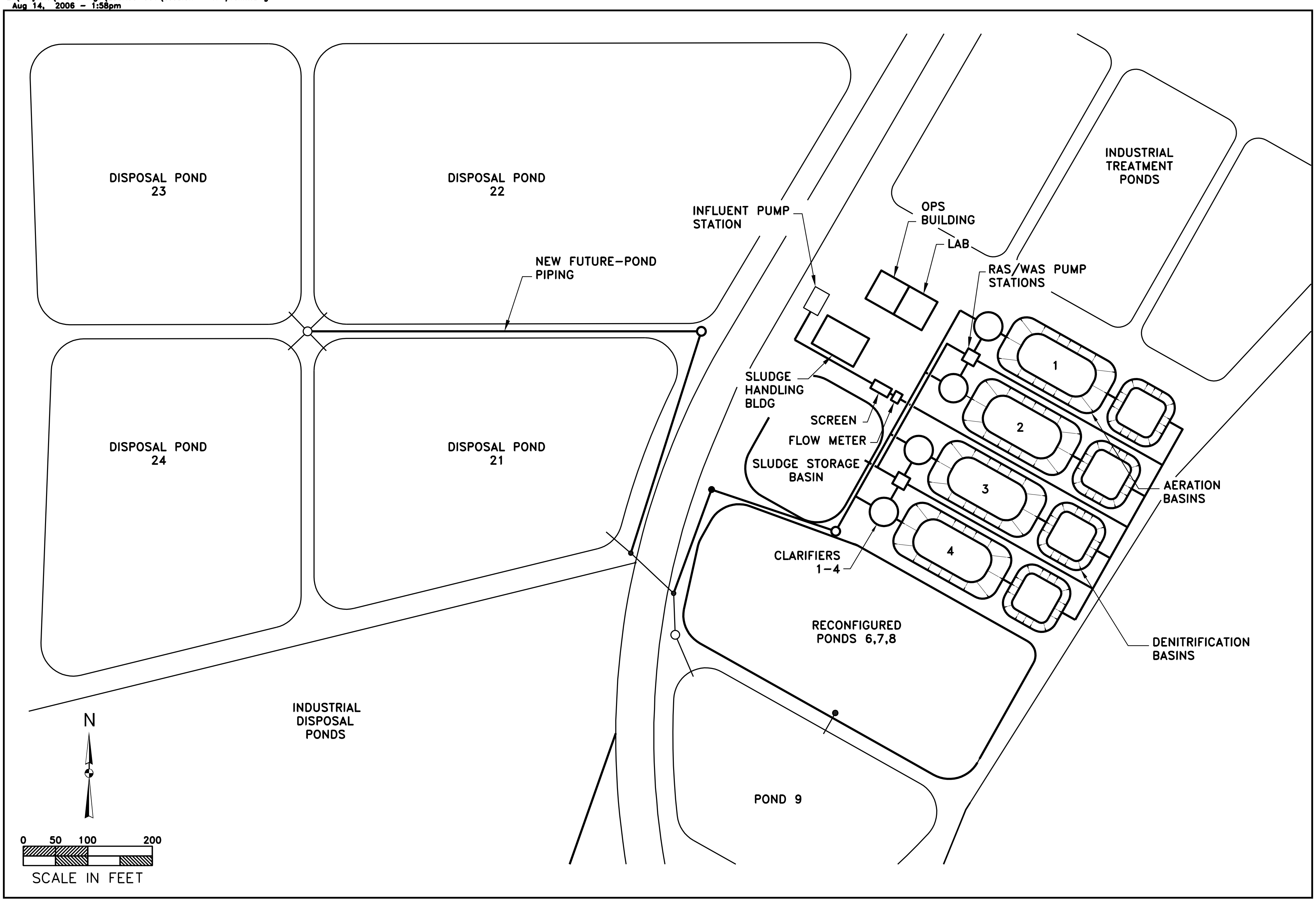


Figure C-1

Activated Sludge Alternative - Conceptual Layout

Appendix D

Historic Groundwater Monitoring Data

City of Escalon
Historic Groundwater Monitoring Data

Well ID	Date	Temp (^oC)	Field pH	Field EC (µS/cm)	DO (mg/L)	BOD_5 (mg/l)	NO_3 (mg/L)	NO_3-N (mg/L)	TDS (mg/L)	FDS (mg/L)	VDS (mg/L)	Total Coliform (MPN/100mL)	Ba (mg/L)	Ca (mg/L)	Mg (mg/L)	Na (mg/L)	K (mg/L)	Cl (mg/L)	SO_4 (mg/L)	Alk. as CaCO3 (mg/L)	Hardness (mg/L)	Volatile Organics (ug/L)
MW-1	9/5/2000	20.6	6.88	745	0.6	9.0		<0.5	568	356	212	<2	0.076	52	25	84	12.8	80	4	388	233	ND
MW-1	12/14/2000	19.7	6.96	780	1.01	<6	<2		475	390	85					94		44				ND
MW-1	3/7/2001	19.4	6.79	784	1.2	<2		<0.5	760	376	384	<2				88		48				1.9 chloroform
MW-1	5/30/2001	19.2	6.69	701	1.4	13.0		<0.5	532	436	96	4				83		73				ND
MW-1	8/28/2001	19.6	6.63	648	1.2	36.0		<0.5	520	356	164	<2	0.088	57	27	92	14.4	86	3	316	254	ND
MW-1	11/20/2001	18.8	6.87	706	1.1	3.0		<0.5	520	308	212	<2				87		53				ND
MW-1	3/13/2002	18.7	7.27	588	1.3	<2		<0.5	472	368	104	<1.1				84		62				ND
MW-1	6/12/2002	19.3	6.9	735	0.22	3.0		1.81	476	332	144	<1.1				87		92				ND
MW-1	9/20/2002	19.1	7.29	615	0.26	4.0	<1		525	325	200	absent	0.070	41	24	83	15.6	71	4	317	201	ND
MW-1	1/13/2003	18.8	7.32	717	0.27	7.0		<0.5	460	333	127	absent	<0.01	40	22	84	12.9	89	1	238	190	ND
MW-1	3/31/2003	19.2	6.8	694	0.49	<2		<0.5	456	334	122	absent				86		62				ND
MW-1	6/16/2003	18.9	6.88	665	0.67	5.0		<0.5	498	380	118	absent				86		46				ND
MW-1	9/30/2003	19.5	6.98	813	0.61	7	<2	<0.5	620	560	60	absent	0.08	42	23	82	16.3	41	11	321	200	ND
MW-1	12/11/2003	18.4	7.09	783	0.5	2		<0.5	630	440	190	absent				84		62				ND
MW-1	3/24/2004	18.6	7.04	669	0.73	5		<0.5	470	278	193	absent				84		55				ND
MW-1	6/23/2004	19.2	6.92	1183	0.6	8		<0.5	585	305	310	absent				86		66				ND
MW-1	8/30/2004	20.4	6.96	691	0.61	6		<0.5	650	295	355	absent				81		44				ND
MW-1	12/22/2004	18.3	7.16	840	3.58	4		<0.5	490	335	165	absent	0.04	34	18	80	9.2	62	7	306	159	ND
MW-1	2/22/2005	18.5	7.08	777	0.64	<5		<0.11	420	220	170	absent	0.13	33	17	76	11	67	1.6	290	170	ND
MW-1	6/8/2005	18.8	6.78	793	0.76	<5		<0.11	415	540	340	absent	0.06	37	18	81	15	67	5.1	242	140	ND
MW-1	8/23/2005	19.3	6.74	823	0.23	<5		<0.11	398	450	260	absent				82		77				ND
MW-1	11/15/2005	18.6	6.89	871	0.51	<5		<0.11	426	370	320	absent				89		86				ND
MW-1	3/8/2006	18.1	6.93	769	0.48	8.4		0.60	440	370	74	absent				87		65				ND
MW-2R	9/5/2000	21.9	7.06	228	3.1	11		6.0	284	184	100	130	0.057	25	10	15	1.5	12	13	110	104	ND
MW-2R	12/14/2000	19.0	7.09	272	4.05	<6	42		205	160	45					15		2				ND
MW-2R	3/7/2001	19.5	6.91	249	4.8	<2		7.0	340	228	112	<2				15		9				ND
MW-2R	5/30/2001	20.0	6.96	187	4.4	<2		4.7	212	152	60	<2				13		18				ND
MW-2R	8/28/2001	20.1	7.28	141	3.4	<2		3.6	164	104	60	<2	0.027	16	6	12	1.4	23	6	79	65	ND
MW-2R	11/20/2001	19.1	7.01	235	2.9	<2		7.4	172	108	64	<2				15		16				ND
MW-3	9/5/2000	22.9	7.03	669	2.6	<2		15.2	588	380	208	4	0.171	94	37	33	2.9	66	73	290	387	ND
MW-3	12/14/2000	20.3	7.04	598	2.4	<6	88		445	250	195					31		27				ND
MW-3	3/7/2001	19.8	6.91	587	3.0	<2		12.6	512	324	188	<2				30		34				ND
MW-3	5/30/2001	20.3	7.16	535	2.8	<2		13.1	496	316	180	<2				28		59				ND
MW-3	8/28/2001	20.7	6.98	484	1.9	<2		16.9	532	356	176	<2	0.146	83	33	31	3.4	50	47	189	343	ND
MW-3	11/20/2001	19.5	7.07	527	1.6	<2		18.7	420	220	200	<2				30		46				ND
MW-3	3/13/2002	19.3	8.5	506	1.4	<2		18.7	528	344	184	<1.1				30		53				ND
MW-3	6/12/2002	20.0	7.06	664	2.7	<2		40.4	484	392	92	<1.1				35		67				ND
MW-3	9/20/2002	20.6	7.30	439	2.6	<2	18.0		485	165	320	absent	0.120	53	27	27	4.4	74	29	179	244	ND
MW-3	1/13/2003	14.8	7.91	478	0.4	3		8.1	380	262	118	present	<0.01	40	19	61	1.7	57	33	138	178	ND
MW-3	3/31/2003	11.9	7.46	450	0.6	<2		3.6	388	284	104	absent				72		66				ND
MW-3	6/16/2003	14.3	7.41	511	0.5	3		2.0	520	360	160	absent				85		51				ND
MW-3	9/30/2003	21.0	7.40	668	0.13	<2	<2	<0.5	430	370	60	present	0.12	41	20	69	3.9	53	15	168	184	ND
MW-3	12/11/2003	21.6	7.45	680	0.67	3		<0.5	460	270	190	absent				75		76				ND
MW-3	3/24/2004	20.2	7.29	541	0.33	3		5.2	500	340	160	absent				70		53				ND
MW-3	6/23/2004	19.6	7.03	992	0.66	2		3.2	500	210	290	present				67		64				ND
MW-3	8/30/2004	19.6	7.47	512	0.54	4		3.4	540	300	240	absent				65		48				ND
MW-3	12/22/2004	18.4	7.07	1120	1.77	3		23.9	660	280	380	present	0.21	63	28	93	2.0	48	52	255	272	ND
MW-3	2/22/2005	19.6	7.04	1263	1.0	<5		41	800	620	640	absent	0.26	82	35	110	3.4	80	120	230	360	ND
MW-3	6/8/2005	17.6	6.96	1006	1.2	<5		35	651	400	240	absent	0.17	85	40	86	2.5	70	69	194	260	ND
MW-3	8/23/2005	18.0	7.09	811	0.54	<5		30	520	410	460	absent				75		95				ND
MW-3	11/15/2005	18.3	7.03	740	0.60	<5		13	460	390	210	present				76		117				ND
MW-3	3/8/2006	16.0	7.55	643	0.48	<3		9.8	460	380	80	present				72		68				ND
MW-4	9/5/2000	23.6	6.79	966	0.8	11		<0.5	612	544	68	<1	0.410	108	55	62	7.0	73	13	562	496	ND
MW-4	12/14/2000	22.1	6.91	1,189	0.75	<6	<2		690	555	135					80		39				ND
MW-4	3/7/2001	22.4	6.61	1,150	0.58	<2		<0.5	824	632	192	<2				78		50				ND
MW-4	5/30/2001	23.1	6.47	890	0.85	2		<0.5	612	528	84	<2				67		64				ND
MW-4	8/28/2001	23.0	6.63	767	0.5	<2		<0.5	712	464	248	<2	0.419	118	58	66	6.8	64	11	473	534	ND
MW-4	11/20/2001	21.8	6.77	863	0.3	5		<0.5	496	216	280	<2				67		62				3.2 Bromomethane, 1.0 Chloromethane
MW-4	3/13/2002	22.1	7.22	1,177	0.5	<2		<0.5	680	604	76	<1.1				67		62				ND
MW-4	6/12/2002	23.2	6.82	972	0.25	<2		0.5	532	368	164	<1.1				76		80				ND
MW-4	9/20/2002	22.3	7.26	761	0.21	3	<1		690	270	420	absent	0.160	32	53	65	11.8	62	8	360	298	ND
MW-4	1/13/2003	22.2	7.07	1,148	0.22	4		<0.5	747	467	280	present	0.460	116	58	74	5.8	69	9	560	528	ND
MW-4	3/31/2003	22.3	6.99	1,119	0.26	<2		<0.5	704	500	204	absent				75		59				ND
MW-4	6/16/2003	22.3	6.99	989	0.25	3		<0.5	612	532	80	absent				80		51				ND
MW-4	9/30/2003	23	6.87	1,118	0.52	6	<2	<0.5	510	380	130	present	0.44	95	52	62	5.6	37	37	808	451	ND
MW-4	12/11/2003	21	6.96	1,115	0.52	5		<0.5	660	480	180	present				64		55				ND
MW-4	3/24/2004	21.8	7.03	1,023	0.94	4		<0.5	610	460	150	absent				69		41				ND
MW-4	6/23/2004	22.1	6.89	1,569	0.64	3		<0.5	760	310	450	absent				73		59				ND

Source: Geological Technics Inc.

City of Escalon
Historic Groundwater Monitoring Data

Well ID	Date	Temp (^oC)	Field pH	Field EC (µS/cm)	DO (mg/L)	BOD_5 (mg/l)	NO_3 (mg/L)	NO_3 -N (mg/L)	TDS (mg/L)	FDS (mg/L)	VDS (mg/L)	Total Coliform (MPN/100mL)	Ba (mg/L)	Ca (mg/L)	Mg (mg/L)	Na (mg/L)	K (mg/L)	Cl (mg/L)	SO_4 (mg/L)	Alk. as CaCO3 (mg/L)	Hardness (mg/L)	Volatile Organics (ug/L)
MW-4	8/30/2004	22.9	6.81	1,050	0.38	5		<0.5	950	370	580	absent				63		32				ND
MW-4	12/22/2004	21.3	7.03	1442	0.43	5		<0.5	760	510	250	absent	0.73	102	53	68	7.5	41	25	587	473	ND
MW-4	2/22/2005	22.1	6.78	1371	0.58	<5		<1.1	650	480	320	absent	0.64	99	49	66	9.8	52	34	560	480	ND
MW-4	6/8/2005	22.1	6.74	1185	0.56	<5		<0.11	573	320	290	absent	0.62	100	47	78	7.9	54	29	416	358	ND
MW-4	8/23/2005	22.7	6.54	1230	0.20	<5		<0.11	613	440	420	present				63		59				ND
MW-4	11/15/2005	22.4	6.76	1205	0.45	<5		<0.11	485	260	230	absent				96		65				ND
MW-4	3/8/2006	21.8	6.97	1120	0.36	<3		<0.5	600	510	90	absent				58		44				ND
MW-5	9/5/2000	19.9	6.89	741	0.7	10		21.4	684	544	140	500	0.159	106	52	46	1.8	92	5	379	479	ND
MW-5	12/14/2000	19.5	7.07	1,170	2	<6	112		815	640	175					75		41				ND
MW-5	3/7/2001	19.9	6.85	803	2.4	<2		5.9	620	416	204	<2				55		51				ND
MW-5	5/30/2001	20.4	6.82	789	2.6	4		6.8	592	412	180	<2				54		76				1.9 Dichlorodifluoromethane
MW-5	8/28/2001	20.2	7.1	534	2.8	6		2.0	508	384	124	<2	0.111	83	42	49	2.2	80	24	284	380	ND
MW-5	11/20/2001	19.3	7.0	786	2.0	3		21.7	736	524	212	<2				62		67				ND
MW-5	3/13/2002	19.4	7.22	1,125	1.5	<2		8.1	664	528	136	2.2				56		62				ND
MW-5	6/12/2002	19.9	6.83	1,041	0.6	<2		30.9	608	432	176	>23				63		83				ND
MW-5	9/20/2002	20.1	7.04	619	0.4	<2	51		685	325	360	present	0.140	49	52	52	5.2	64	33	279	336	ND
MW-5	1/13/2003	19.1	7.62	765	0.4	4		0.5	173	47	126	present	<0.01	86	50	47	1.6	41	7	403	421	ND
MW-5	3/31/2003	19.3	7.26	894	0.4	<2		0.7	640	416	224	absent				52		48				ND
MW-5	6/16/2003	19.8	7.28	839	0.8	3		5.8	752	620	132	absent				58		48				ND
MW-5	9/30/2003	19.5	7.04	1,099	0.35	<2	66	14.9	540	470	70	absent	0.23	104	67	59	2.4	44	22	485	536	ND
MW-5	12/11/2003	18.9	7.01	1,132	0.51	3		2.5	810	410	400	absent				61		67				ND
MW-5	3/24/2004	19.9	7.01	944	1.03	2		6.5	590	470	120	absent				61		50				ND
MW-5	6/23/2004	19.7	6.90	1,600	0.99	2		19.6	1,050	470	580	present				65		71				ND
MW-5	8/30/2004	20.4	7.05	1,030	0.94	4		16.5	850	330	520	absent				56		39				ND
MW-5	12/22/2004	21.5	6.89	1175	4.0	6		1.1	850	580	270	absent	0.29	83	50	57	1.8	48	11	472	413	ND
MW-5	2/22/2005	21.4	7.03	1028	1.15	<5		<1.1	480	220	570	absent	0.24	78	45	55	2.7	52	5.2	380	360	ND
MW-5	6/8/2005	21.6	6.84	896	0.60	<5		0.62	522	560	290	absent	0.23	77	43	59	3.4	70	25	302	286	ND
MW-5	8/23/2005	21.2	6.89	1024	0.59	<5		8.4	552	240	290	absent				55		46				ND
MW-5	11/15/2005	20.1	7.03	1140	0.41	<5		2.6	621	400	400	absent				60		100				ND
MW-5	3/8/2006	20.4	7.78	1078	0.26	<3		2.8	630	570	62	absent				62		63				ND
MW-6	9/5/2000	20.6	6.93	755	0.9	11		5.3	636	488	148	11	0.305	75	46	66	11.0	92	27	410	377	ND
MW-6	12/14/2000	20.7	7.19	822	1.35	<6	10		515	375	140					86		37				ND
MW-6	3/7/2001	18.5	7.07	671	1.5	<2		1.4	608	380	228	<2				64		27				ND
MW-6	5/30/2001	17.9	7.18	496	1.7	2		0.9	380	276	104	<2				50		23				ND
MW-6	8/28/2001	24.3	7.23	667	1.5	<2		<0.5	516	392	124	<2	0.163	102	41	58	5.2	76	9	379	424	ND
MW-6	11/20/2001	20.6	7.39	680	1.2	5		<0.5	532	348	184	<2				55		59				ND
MW-6	3/13/2002	17.8	7.45	953	1.1	<2		<0.5	536	356	180	<1.1				52		35				ND
MW-6	6/12/2002	17.2	7.22	745	0.62	<2		0.7	384	244	140	<1.1				53		50				ND
MW-6	9/20/2002	21.7	7.7	673	0.73	3	6		965	235	215	absent	0.490	31	44	48	9.3	71	23	267	259	ND
MW-6	1/13/2003	20.8	7.56	874	0.42	4		<0.05	584	340	244	present	0.780	104	46	53	3.8	89	6	410	449	ND
MW-6	3/31/2003	20.4	7.4	930	0.45	<2		<0.5	880	352	528	present				56		44				ND
MW-6	6/16/2003	19.7	7.45	544	0.39	4		<0.5	412	372	40	absent				44		32				ND
MW-6	9/30/2003	20.9	7.13	1044	0.14	8	2	0.5	560	370	190	absent	0.79	101	44	59	16.9	94	33	444	433	ND
MW-6	12/11/2003	21.6	7.33	947	0.66	5		<0.5	650	290	360	absent				55		62				ND
MW-6	3/24/2004	19.6	7.17	829	1.01	<2		<0.5	360	60	300	absent				61		48				ND
MW-6	6/23/2004	19.1	7.17	1302	0.89	<2		0.7	680	220	460	absent				60		41				ND
MW-6	8/30/2004	17.7	7.12	793	0.68	3		2.5	690	210	480	present				49		35				ND
MW-6	12/22/2004	21.9	7.27	1258	0.70	6		<0.5	600	390	210	absent	0.72	61	28	49	55.2	46	2	497	267	ND
MW-6	2/22/2005	21.6	7.29	1198	0.58	6.3		<1.1	550	400	150	absent	0.51	54	26	51	77	54	<5	460	220	ND
MW-6	6/8/2005	21.4	6.99	1145	0.46	<5		<0.11	551	319	370	absent	0.56	77	32	59	68	61	<0.5	364	228	ND
MW-6	8/23/2005	21.2	6.88	1209	0.46	<5		17	578	320	320	absent				58		77				ND
MW-6	11/15/2005	22.1	7.01	1168	0.33	<5		<0.11	572	490	330	absent				58		112				ND
MW-6	3/8/2006	19.1	7.81	1078	0.25	6.8-12		<0.5	570	470	110	absent				65		62				ND
MW-7	9/5/2000	25.5	6.76	1,075	0.3	27		<0.5	596	512	84	<2	0.219	84	42	71	25.5	80	1	543	383	ND
MW-7	12/14/2000	23.2	6.81	1,238	0.78	13	5		545	435	110					65		48				ND
MW-7	3/7/2001												No Water in Well									
MW-7	5/30/2001	23.5	6.51	984	1.1	103		<0.5	628	412	216	1,600				55		44				ND
MW-7	8/28/2001	20.1	6.62	730	0.4	3		<0.5	484	308	176	<2	0.248	59	30	48	32	44	1	385	271	ND
MW-7	11/20/2001	22.1	6.83	824	0.5	6		<0.5	440	284	156	<2				52		55				ND
MW-7	3/13/2002												No Water in Well									
MW-7	6/12/2002	24.4	6.67	1052	0.2	6		0.5	492	408	84	<1.1				64		66				ND
MW-7	9/20/2002	23.3	7.16	671	0.13	4	<1		385	225	160	absent	0.11	40	24	47	34.9	46	3	205	199	ND
MW-7	1/13/2003												No Water in Well									
MW-7	3/31/2003												No Water in Well									
MW-7	6/16/2003	20.3	7.89	856	2.03	5		<0.5	404	324	80	present				46		41				ND
MW-7	9/30/2003	22.6	6.86	883	0.38	9	<2	<0.5	350	230	120	absent	0.17	37	21	42	33.6	23	1	332	178	ND
MW-7	12/11/2003												No Water in Well									
MW-7	3/24/2004												No Water in Well									
MW-7	6/23/2004	22.7	6.80	930	0.44	7		3.2	760	270	490	absent				48		30				ND
MW-7	8/30/2004	23.5	6.82	992	1.1	9		<0.5	660	270	390	present				50		30				ND
MW-7	12/22/2004												No Water in Well									
MW-7	2/22/2005												No Water in Well									
MW-7	6/8/2005												No Water in Well									
MW-7	8/23/2005												No Water in Well									
MW-7	11/15/2005												No Water in Well									
MW-7	3/8/2006			no recharge		12		<0.5	430	370	66	absent				53		40				ND
MW-8	9/5/2000	19.5	7.22	560	0.3	10		0.8	452	408	44	130	0.144	38	20	82	2.9	87	6	268	177	ND

Source: Geological Technics Inc.

City of Escalon
Historic Groundwater Monitoring Data

Well ID	Date	Temp (°C)	Field pH	Field EC (µS/cm)	DO (mg/L)	BOD_5 (mg/l)	NO_3 (mg/L)	NO_3 -N (mg/L)	TDS (mg/L)	FDS (mg/L)	VDS (mg/L)	Total Coliform (MPN/100mL)	Ba (mg/L)	Ca (mg/L)	Mg (mg/L)	Na (mg/L)	K (mg/L)	Cl (mg/L)	SO_4 (mg/L)	Alk. as CaCO3 (mg/L)	Hardness (mg/L)	Volatile Organics (ug/L)
MW-8	12/14/2000	18.5	7.18	618	1.12	<6	4		350	335	15					92		55				ND
MW-8	3/7/2001	18.4	6.97	699	1.1	<2		4.3	740	352	388	<2				95		41				ND
MW-8	5/30/2001	19.5	6.9	609	1.6	4		1.6	428	280	148	<2				91		80				ND
MW-8	8/28/2001	19.6	7.28	511	0.9	<2		<0.5	508	272	236	<2	0.144	45	24	83	3.1	80	10	252	211	ND
MW-8	11/20/2001	18.5	7.11	559	0.7	4		<0.5	396	268	128	<2				84		85				ND
MW-8	3/13/2002	18.5	8.56	573	0.5	<2		2.3	508	384	124	<1.1				96		80				ND
MW-8	6/12/2002	19.8	7.03	616	0.52	<2		4.74	484	348	136	<1.1				92		98				ND
MW-8	9/20/2002	19.5	7.48	475	0.56	4	2		380	265	115	absent	0.130	27	17	74	11.2	71	7	221	137	ND
MW-8	1/13/2003	18.6	7.88	629	0.52	6		0.50	436	312	124	present	<0.01	37	21	91	5.1	83	7	255	179	ND
MW-8	3/31/2003	18.8	7.24	724	0.24	<2		3.6	536	356	180	absent				98		71				ND
MW-8	6/16/2003	18.9	7.3	630	0.48	3		0.7	508	344	164	absent				100		57				ND
MW-8	9/30/2003	19.4	7.22	691	0.52	<2	3	0.7	410	300	110	absent	0.14	34	21	91	6.1	41	26	250	171	ND
MW-8	12/11/2003	17.9	7.23	722	0.44	4		1.4	480	260	220	present				89		64				ND
MW-8	3/24/2004	18.6	7.11	640	0.92	3		16.9	330	90	240	present				71		41				ND
MW-8	6/23/2004	19.0	7.06	1163	0.81	<2		12.9	700	260	440	present				74		37				ND
MW-8	8/30/2004	20.1	7.11	740	0.69	5		3.8	610	290	320	present				82		55				ND
MW-8	12/22/2004	18.1	7.24	891	0.51	5		0.9	500	360	140	absent	0.22	45	25	94	5.2	55	11	316	215	ND
MW-8	2/22/2005	18.8	7.06	804	1.02	<5		1.3	490	530	450	absent	0.17	39	20	88	4.5	63	19	270	190	ND
MW-8	6/8/2005	18.6	6.90	937	0.84	<5		5.0	504	380	160	absent	0.27	73	36	96	5.6	80	17	288	246	ND
MW-8	8/23/2005	19.6	6.90	907	0.19	<5		6.8	472	330	300	absent				91		95				ND
MW-8	11/15/2005	18.6	7.02	932	0.53	<5		18	512	550	140	absent				78		88				ND
MW-8	3/8/2006	18.3	7.57	731	0.42	<3		5.5	440	370	71	present				70		66				ND
MW-9	3/13/2002	19.5	7.3	900	1.1	<2		10.8	584	444	140	<1.1				41		53				7.0 CHBrCl2, 1.6 CHBr3, 57 CHCl3, 2.5 CHBr2Cl
MW-9	6/12/2002	21.0	7.09	729	3.2	<2		24.6	504	400	104	3.6				28		48				2.7 CHCl3
MW-9	9/20/2002	21.0	7.46	488	3.24	<2	49		1,355	155	1,200	absent	0.110	42	39	18	6.9	27	58	265	174	ND
MW-9	1/13/2003	20.3	7.62	738	2.78	2		13.3	507	407	100	present	<0.01	100	44	24	3.7	39	58	290	431	ND
MW-9	3/31/2003	21.1	7.33	755	2.1	<2		13.5	556	400	156	present				25		27				ND
MW-9	6/16/2003	21.3	7.44	652	3.04	<2		11.3	540	376	164	absent				27		20				ND
MW-9	9/30/2003	21.1	7.22	742	2.90	<2	53	12.0	590	490	100	present	0.15	89	41	22	3.7	35	74	273	391	ND
MW-9	12/11/2003	20.1	7.24	769	3.70	<2		12.0	580	330	250	present				24		23				ND
MW-9	3/24/2004	20.4	7.15	646	2.85	<2		12.4	190	80	110	absent				24		25				ND
MW-9	6/23/2004	20.9	7.07	1207	2.5	<2		12.9	540	250	290	absent				25		28				ND
MW-9	8/30/2004	20.5	7.03	687	2.9	<2		18.5	670	180	690	absent				21		12				ND
MW-9	12/22/2004	19.6	6.98	1029	3.7	<2		22.3	580	370	250	>23	0.21	92	41	26	3.8	16	90	265	399	ND
MW-9	2/22/2005	20.4	7.02	901	3.73	<5		14	610	430	580	absent	0.20	89	37	23	3.3	21	73	270	350	ND
MW-9	6/8/2005	20.3	6.60	903	2.97	<5		20	517	390	190	absent	0.25	110	42	26	4.7	20	110	220	328	ND
MW-9	8/23/2005	20.4	7.38	882	0.22	<5		38	576	280	290	absent				22		20				ND
MW-9	11/15/2005	20.1	6.96	858	2.98	<5		31	645	320	410	present				21		19				ND
MW-9	3/8/2006	20.0	7.53	776	2.1	<3		13	500	390	110	absent				23		29				ND
MW-10	3/13/2002	20.7	8.39	597	0.8	<2		8.6	564	392	172	<1.1				48		44				7.0 CHBrCl2, 2.8 CHBr3, 76 CHCl3, 3.5 CHBr2Cl
MW-10	6/12/2002	21.3	7.18	759	3.02	<2		22.8	388	348	40	<1.1				44		55				0.66 CHBrCl2, 20 CHCl3
MW-10	9/20/2002	21.9	7.52	545	3.07	<2	60		530	305	225	absent	0.110	37	36	33	7.7	28	45	93	241	5.9 CHCl3
MW-10	1/13/2003	20.4	7.63	757	3.15	2		14.4	567	374	193	absent	<0.01	98	37	37	3.9	41	63	263	397	5.1 CHCl3
MW-10	3/31/2003	21.1	7.42	750	2.0	<2		10.6	556	408	148	present				39		18				3.4 CHCL3
MW-10	6/16/2003	21.4	7.53	650	2.93	<2		10.6	544	456	88	absent				37		25				0.75 CHCL3
MW-10	9/30/2003	21.1	7.33	804	3.14	<2	48	10.8	570	390	180	absent	0.15	98	39	32	5.0	35	56	319	406	0.71 CHCL3
MW-10	12/11/2003	20.2	7.41	828	3.5	<2		13.3	700	430	270	present				35		28				0.99 CHCL3
MW-10	3/24/2004	20.6	7.22	701	3.5	<2		12.0	550	300	220	absent				35		16				ND
MW-10	6/23/2004	21.3	7.19	1283	1.7	<2		12.9	570	250	320	absent				35		21				ND
MW-10	8/30/2004	20.9	7.25	705	2.1	<2		14.4	890	440	450	present				28		16				ND
MW-10	12/22/2004	19.4	7.08	1100	5.02	<2		14.7	580	370	210	absent	0.23	106	39	34	3.1	18	66	375	426	ND
MW-10	2/22/2005	19.3	7.21	915	3.17	9		10	650	340	290	absent	0.19	96	33	32	4.2	17	51	370	340	ND
MW-10	6/8/2005	20.2	6.87	990	2.81	<5		13	634	220	270	absent	0.28	130	44	36	6.0	18	88	292	358	ND
MW-10	8/23/2005	20.3	7.01	934	0.18	<5		19	565	450	230	present				31		18				ND
MW-10	11/15/2005	19.9	7.16	912	2.70	<5		20	578	360	340	absent				29		22				ND
MW-10	3/82006	19.4	7.43	875	1.6	<3		11	570	450	130	present				33		21				ND
MW-11	3/13/2002	21.2	8.4	768	0.6	<2		<0.5	644	492	152	>23.3				81		71				1.0 CHBrCl2, 8.3 CHCl3, 0.6 CHBr2Cl
MW-11	6/12/2002	21.8	6.84	1046	0.61	<2		1.35	500	420	80	<1.1				77		83				2.2 CHCl3
MW-11	9/20/2002	21.8	7.21	677	0.58	2	2		530	370	160	absent	0.130	40	39	92	17.3	64	30	291	260	0.56 CHCl3
MW-11	1/13/2003	21.4	7.27	1011	0.2	4		1.60	647	467	180	absent	0.120	110	45	75	6.1	44	33	470	460	ND
MW-11	3/31/2003	21.8	7.09	1017	0.22	<2		2.30	680	424	256	absent				78		62				ND
MW-11	6/16/2003	21.5	7.18	914	0.66	3		1.1	660	596	64	absent				77		53				ND
MW-11	9/30/2003	21.3	6.99	1078	0.53	4	53	10.8	580	470	110	absent	0.34	104	45	76	6.6	50	32	434	445	ND
MW-11	12/11/2003	20.3	7.04	1006	0.39	2		4.7	690	400	290	absent				74		83				ND
MW-11	3/24/2004	21	7.00	933	1.39	3		2.5	160	110	50	absent				77		53				ND
MW-11	6/23/2004	21.4	6.88	1517	0.76	5		1.8	730	370	360	present				77		59				ND
MW-11	8/30/2004	21.4	6.87	863	0.83	2		5.4	730	430	300	absent				73		44				ND
MW-11	12/22/2004	20.3	6.80	1345	1.04	4		2.9	640	500	140	absent	0.37	101	40	76	4.4	57	37	479	417	ND
MW-11	2/22/2005	21.0	7.08	1201	0.72	17		1.9	660	520	240	absent	0.40	100	39	77	6.4	59	31	410	400	ND
MW-11	6/8/2005	21.0	6.64	1234	0.58	<5		0.69	631	530	150	absent	0.55	126	47	81	11	70	20	416	372	ND
MW-11	8/23/2005	21.7	6.72	931	0.21	<5		13	451	350	470	present				72		65				ND
MW-11	11/15/2005	21.2	6.76	1018	0.44	<5		5.9	540	310	340	absent				73		78				ND
MW-11	3/8/2006	21.0	7.12	1015	0.41	<3		3.7	580	500	88	absent				68		52				ND

Source: Geological Technics Inc.

City of Escalon

Historic Groundwater Elevation and Monitoring Well Construction Information.

Date	Well ID	Depth to Water (ft)	Top of Casing Elevation (ft.)	Groundwater Elevation (ft.)	Well Diameter (in)	Boreholde Diameter (in)	Borehole Bottom Elevation (ft.)	Approximate Top of Screen Elevation (ft.)	Screen Length (ft)
1/12/2000	MW-1	44.98	109.08	64.10	NA	NA	NA	NA	NA
9/5/2000	MW-1	42.74	109.08	66.34	NA	NA	NA	NA	NA
12/14/2000	MW-1	44.96	109.08	64.12	NA	NA	NA	NA	NA
3/7/2001	MW-1	45.13	109.08	63.95	NA	NA	NA	NA	NA
5/30/2001	MW-1	43.93	109.08	65.15	NA	NA	NA	NA	NA
8/28/2001	MW-1	44.55	109.08	64.53	NA	NA	NA	NA	NA
11/20/2001	MW-1	44.83	109.08	64.25	NA	NA	NA	NA	NA
3/13/2002	MW-1	45.55	109.08	63.53	NA	NA	NA	NA	NA
6/12/2002	MW-1	44.68	109.08	64.40	NA	NA	NA	NA	NA
9/20/2002	MW-1	43.57	109.08	65.51	NA	NA	NA	NA	NA
1/13/2003	MW-1	45.79	109.08	63.29	NA	NA	NA	NA	NA
3/31/2003	MW-1	46.32	109.08	62.76	NA	NA	NA	NA	NA
6/16/2003	MW-1	43.85	109.08	65.23	NA	NA	NA	NA	NA
9/30/2003	MW-1	43.92	109.08	65.16	NA	NA	NA	NA	NA
12/11/2003	MW-1	46.09	109.08	62.99	NA	NA	NA	NA	NA
3/24/2004	MW-1	46.56	109.08	62.52	NA	NA	NA	NA	NA
6/23/2004	MW-1	44.72	109.08	64.36	NA	NA	NA	NA	NA
8/30/2004	MW-1	43.85	109.08	65.23	NA	NA	NA	NA	NA
12/22/2004	MW-1	45.68	109.08	63.40	NA	NA	NA	NA	NA
2/22/2005	MW-1	46.65	109.08	62.43	NA	NA	NA	NA	NA
6/8/2005	MW-1	42.75	109.08	66.33	NA	NA	NA	NA	NA
8/23/2005	MW-1	44.27	109.08	64.81	NA	NA	NA	NA	NA
11/15/2005	MW-1	45.36	109.08	63.72	NA	NA	NA	NA	NA
3/8/2006	MW-1	40.33	109.08	68.75	NA	NA	NA	NA	NA
7/14/2000	MW-2R	37.87	103.56	65.69	2	8	53.56	73.56	20
9/5/2000	MW-2R	37.00	103.56	66.56	2	8	53.56	73.56	20
12/14/2000	MW-2R	39.68	103.56	63.88	2	8	53.56	73.56	20
3/7/2001	MW-2R	40.18	103.56	63.38	2	8	53.56	73.56	20
5/30/2001	MW-2R	38.05	103.56	65.51	2	8	53.56	73.56	20
8/28/2001	MW-2R	35.50	103.56	68.06	2	8	53.56	73.56	20
11/20/2001	MW-2R	38.87	103.56	64.69	2	8	53.56	73.56	20
4/30/1997	MW-3	30.60	97.74	67.14	NA	NA	NA	NA	NA
7/18/1997	MW-3	33.96	97.74	63.78	NA	NA	NA	NA	NA
1/12/2000	MW-3	34.74	97.74	63.00	NA	NA	NA	NA	NA
9/5/2000	MW-3	31.91	97.74	65.83	NA	NA	NA	NA	NA
12/14/2000	MW-3	34.32	97.74	63.42	NA	NA	NA	NA	NA
3/7/2001	MW-3	35.32	97.74	62.42	NA	NA	NA	NA	NA
5/30/2001	MW-3	32.98	97.74	64.76	NA	NA	NA	NA	NA
8/28/2001	MW-3	33.63	97.74	64.11	NA	NA	NA	NA	NA
11/20/2001	MW-3	34.23	97.74	63.51	NA	NA	NA	NA	NA
3/13/2002	MW-3	35.59	97.74	62.15	NA	NA	NA	NA	NA
6/12/2002	MW-3	32.83	97.74	64.91	NA	NA	NA	NA	NA
9/20/2002	MW-3	30.66	97.74	67.08	NA	NA	NA	NA	NA
1/13/2003	MW-3	33.44	97.74	64.30	NA	NA	NA	NA	NA
3/31/2003	MW-3	35.43	97.74	62.31	NA	NA	NA	NA	NA
6/16/2003	MW-3	31.37	97.74	66.37	NA	NA	NA	NA	NA
9/30/2003	MW-3	32.87	97.74	64.87	NA	NA	NA	NA	NA
12/11/2003	MW-3	34.12	97.74	63.62	NA	NA	NA	NA	NA
3/24/2004	MW-3	36.15	97.74	61.59	NA	NA	NA	NA	NA
6/23/2004	MW-3	33.35	97.74	64.39	NA	NA	NA	NA	NA
8/30/2004	MW-3	32.51	97.74	65.23	NA	NA	NA	NA	NA

City of Escalon

Historic Groundwater Elevation and Monitoring Well Construction Information.

Date	Well ID	Depth to Water (ft)	Top of Casing Elevation (ft.)	Groundwater Elevation (ft.)	Well Diameter (in)	Boreholde Diameter (in)	Borehole Bottom Elevation (ft.)	Approximate Top of Screen Elevation (ft.)	Screen Length (ft)
1/12/2000	MW-1	44.98	109.08	64.10	NA	NA	NA	NA	NA
9/5/2000	MW-1	42.74	109.08	66.34	NA	NA	NA	NA	NA
12/14/2000	MW-1	44.96	109.08	64.12	NA	NA	NA	NA	NA
3/7/2001	MW-1	45.13	109.08	63.95	NA	NA	NA	NA	NA
5/30/2001	MW-1	43.93	109.08	65.15	NA	NA	NA	NA	NA
8/28/2001	MW-1	44.55	109.08	64.53	NA	NA	NA	NA	NA
11/20/2001	MW-1	44.83	109.08	64.25	NA	NA	NA	NA	NA
3/13/2002	MW-1	45.55	109.08	63.53	NA	NA	NA	NA	NA
6/12/2002	MW-1	44.68	109.08	64.40	NA	NA	NA	NA	NA
9/20/2002	MW-1	43.57	109.08	65.51	NA	NA	NA	NA	NA
1/13/2003	MW-1	45.79	109.08	63.29	NA	NA	NA	NA	NA
3/31/2003	MW-1	46.32	109.08	62.76	NA	NA	NA	NA	NA
6/16/2003	MW-1	43.85	109.08	65.23	NA	NA	NA	NA	NA
9/30/2003	MW-1	43.92	109.08	65.16	NA	NA	NA	NA	NA
12/11/2003	MW-1	46.09	109.08	62.99	NA	NA	NA	NA	NA
3/24/2004	MW-1	46.56	109.08	62.52	NA	NA	NA	NA	NA
6/23/2004	MW-1	44.72	109.08	64.36	NA	NA	NA	NA	NA
8/30/2004	MW-1	43.85	109.08	65.23	NA	NA	NA	NA	NA
12/22/2004	MW-1	45.68	109.08	63.40	NA	NA	NA	NA	NA
2/22/2005	MW-1	46.65	109.08	62.43	NA	NA	NA	NA	NA
6/8/2005	MW-1	42.75	109.08	66.33	NA	NA	NA	NA	NA
8/23/2005	MW-1	44.27	109.08	64.81	NA	NA	NA	NA	NA
11/15/2005	MW-1	45.36	109.08	63.72	NA	NA	NA	NA	NA
3/8/2006	MW-1	40.33	109.08	68.75	NA	NA	NA	NA	NA
7/14/2000	MW-2R	37.87	103.56	65.69	2	8	53.56	73.56	20
9/5/2000	MW-2R	37.00	103.56	66.56	2	8	53.56	73.56	20
12/14/2000	MW-2R	39.68	103.56	63.88	2	8	53.56	73.56	20
3/7/2001	MW-2R	40.18	103.56	63.38	2	8	53.56	73.56	20
5/30/2001	MW-2R	38.05	103.56	65.51	2	8	53.56	73.56	20
8/28/2001	MW-2R	35.50	103.56	68.06	2	8	53.56	73.56	20
11/20/2001	MW-2R	38.87	103.56	64.69	2	8	53.56	73.56	20
4/30/1997	MW-3	30.60	97.74	67.14	NA	NA	NA	NA	NA
7/18/1997	MW-3	33.96	97.74	63.78	NA	NA	NA	NA	NA
1/12/2000	MW-3	34.74	97.74	63.00	NA	NA	NA	NA	NA
9/5/2000	MW-3	31.91	97.74	65.83	NA	NA	NA	NA	NA
12/14/2000	MW-3	34.32	97.74	63.42	NA	NA	NA	NA	NA
3/7/2001	MW-3	35.32	97.74	62.42	NA	NA	NA	NA	NA
5/30/2001	MW-3	32.98	97.74	64.76	NA	NA	NA	NA	NA
8/28/2001	MW-3	33.63	97.74	64.11	NA	NA	NA	NA	NA
11/20/2001	MW-3	34.23	97.74	63.51	NA	NA	NA	NA	NA
3/13/2002	MW-3	35.59	97.74	62.15	NA	NA	NA	NA	NA
6/12/2002	MW-3	32.83	97.74	64.91	NA	NA	NA	NA	NA
9/20/2002	MW-3	30.66	97.74	67.08	NA	NA	NA	NA	NA
1/13/2003	MW-3	33.44	97.74	64.30	NA	NA	NA	NA	NA
3/31/2003	MW-3	35.43	97.74	62.31	NA	NA	NA	NA	NA
6/16/2003	MW-3	31.37	97.74	66.37	NA	NA	NA	NA	NA
9/30/2003	MW-3	32.87	97.74	64.87	NA	NA	NA	NA	NA
12/11/2003	MW-3	34.12	97.74	63.62	NA	NA	NA	NA	NA
3/24/2004	MW-3	36.15	97.74	61.59	NA	NA	NA	NA	NA
6/23/2004	MW-3	33.35	97.74	64.39	NA	NA	NA	NA	NA
8/30/2004	MW-3	32.51	97.74	65.23	NA	NA	NA	NA	NA

City of Escalon

Historic Groundwater Elevation and Monitoring Well Construction Information.

Date	Well ID	Depth to Water (ft)	Top of Casing Elevation (ft.)	Groundwater Elevation (ft.)	Well Diameter (in)	Boreholde Diameter (in)	Borehole Bottom Elevation (ft.)	Approximate Top of Screen Elevation (ft.)	Screen Length (ft)
12/22/2004	MW-3	34.99	97.74	62.75	NA	NA	NA	NA	NA
2/22/2005	MW-3	35.18	97.74	62.56	NA	NA	NA	NA	NA
6/8/2005	MW-3	31.68	97.74	66.06	NA	NA	NA	NA	NA
8/23/2005	MW-3	32.71	97.74	65.03	NA	NA	NA	NA	NA
11/15/2005	MW-3	34.46	97.74	63.28	NA	NA	NA	NA	NA
3/8/2006	MW-3	29.86	97.74	67.88	NA	NA	NA	NA	NA
7/14/2000	MW-4	43.88	109.88	66.00	2	8	49.88	74.88	20
9/5/2000	MW-4	43.91	109.88	65.97	2	8	49.88	74.88	20
12/14/2000	MW-4	46.20	109.88	63.68	2	8	49.88	74.88	20
3/7/2001	MW-4	47.01	109.88	62.87	2	8	49.88	74.88	20
5/30/2001	MW-4	44.22	109.88	65.66	2	8	49.88	74.88	20
8/28/2001	MW-4	44.71	109.88	65.17	2	8	49.88	74.88	20
11/20/2001	MW-4	45.99	109.88	63.89	2	8	49.88	74.88	20
3/13/2002	MW-4	46.81	109.88	63.07	2	8	49.88	74.88	20
6/12/2002	MW-4	44.71	109.88	65.17	2	8	49.88	74.88	20
9/20/2002	MW-4	45.08	109.88	64.80	2	8	49.88	74.88	20
1/13/2003	MW-4	47.31	109.88	62.57	2	8	49.88	74.88	20
3/31/2003	MW-4	46.85	109.88	63.03	2	8	49.88	74.88	20
6/16/2003	MW-4	43.82	109.88	66.06	2	8	49.88	74.88	20
9/30/2003	MW-4	44.89	109.88	64.99	2	8	49.88	74.88	20
12/11/2003	MW-4	47.07	109.88	62.81	2	8	49.88	74.88	20
3/24/2004	MW-4	47.92	109.88	61.96	2	8	49.88	74.88	20
6/23/2004	MW-4	44.29	109.88	65.59	2	8	49.88	74.88	20
8/30/2004	MW-4	44.97	109.88	64.91	2	8	49.88	74.88	20
12/22/2004	MW-4	47.41	109.88	62.47	2	8	49.88	74.88	20
2/22/2005	MW-4	47.83	109.88	62.05	2	8	49.88	74.88	20
6/8/2005	MW-4	43.39	109.88	66.49	2	8	49.88	74.88	20
8/23/2005	MW-4	45.06	109.88	64.82	2	8	49.88	74.88	20
11/15/2005	MW-4	46.38	109.88	63.50	2	8	49.88	74.88	20
3/8/2006	MW-4	41.95	109.88	67.93	2	8	49.88	74.88	20
7/14/2000	MW-5	30.19	92.82	62.63	2	8	52.82	72.82	20
9/5/2000	MW-5	26.15	92.82	66.67	2	8	52.82	72.82	20
12/14/2000	MW-5	30.59	92.82	62.23	2	8	52.82	72.82	20
3/7/2001	MW-5	30.45	92.82	62.37	2	8	52.82	72.82	20
5/30/2001	MW-5	29.30	92.82	63.52	2	8	52.82	72.82	20
8/28/2001	MW-5	29.73	92.82	63.09	2	8	52.82	72.82	20
11/20/2001	MW-5	30.31	92.82	62.51	2	8	52.82	72.82	20
3/13/2002	MW-5	31.25	92.82	61.57	2	8	52.82	72.82	20
6/12/2002	MW-5	33.16	92.82	59.66	2	8	52.82	72.82	20
9/20/2002	MW-5	24.91	92.82	67.91	2	8	52.82	72.82	20
1/13/2003	MW-5	30.53	92.82	62.29	2	8	52.82	72.82	20
3/31/2003	MW-5	31.63	92.82	61.19	2	8	52.82	72.82	20
6/16/2003	MW-5	28.77	92.82	64.05	2	8	52.82	72.82	20
9/30/2003	MW-5	26.50	92.82	66.32	2	8	52.82	72.82	20
12/11/2003	MW-5	31.57	92.82	61.25	2	8	52.82	72.82	20
3/24/2004	MW-5	31.43	92.82	61.39	2	8	52.82	72.82	20
6/23/2004	MW-5	29.78	92.82	63.04	2	8	52.82	72.82	20
8/30/2004	MW-5	27.16	92.82	65.66	2	8	52.82	72.82	20
12/22/2004	MW-5	30.21	92.82	62.61	2	8	52.82	72.82	20
2/22/2005	MW-5	30.15	92.82	62.67	2	8	52.82	72.82	20
6/8/2005	MW-5	28.84	92.82	63.98	2	8	52.82	72.82	20

City of Escalon

Historic Groundwater Elevation and Monitoring Well Construction Information.

Date	Well ID	Depth to Water (ft)	Top of Casing Elevation (ft.)	Groundwater Elevation (ft.)	Well Diameter (in)	Boreholde Diameter (in)	Borehole Bottom Elevation (ft.)	Approximate Top of Screen Elevation (ft.)	Screen Length (ft)
8/23/2005	MW-5	29.25	92.82	63.57	2	8	52.82	72.82	20
11/15/2005	MW-5	30.22	92.82	62.60	2	8	52.82	72.82	20
3/8/2006	MW-5	25.25	92.82	67.57	2	8	52.82	72.82	20
7/14/2000	MW-6	30.18	90.49	60.31	2	8	50.49	71.49	20
9/5/2000	MW-6	29.05	90.49	61.44	2	8	50.49	71.49	20
12/14/2000	MW-6	29.22	90.49	61.27	2	8	50.49	71.49	20
3/7/2001	MW-6	28.16	90.49	62.33	2	8	50.49	71.49	20
5/30/2001	MW-6	28.96	90.49	61.53	2	8	50.49	71.49	20
8/28/2001	MW-6	26.77	90.49	63.72	2	8	50.49	71.49	20
11/20/2001	MW-6	28.91	90.49	61.58	2	8	50.49	71.49	20
3/13/2002	MW-6	29.78	90.49	60.71	2	8	50.49	71.49	20
6/12/2002	MW-6	29.78	90.49	60.71	2	8	50.49	71.49	20
9/20/2002	MW-6	27.99	90.49	62.50	2	8	50.49	71.49	20
1/13/2003	MW-6	30.28	90.49	60.21	2	8	50.49	71.49	20
3/31/2003	MW-6	29.82	90.49	60.67	2	8	50.49	71.49	20
6/16/2003	MW-6	27.22	90.49	63.27	2	8	50.49	71.49	20
9/30/2003	MW-6	28.50	90.49	61.99	2	8	50.49	71.49	20
12/11/2003	MW-6	30.83	90.49	59.66	2	8	50.49	71.49	20
3/24/2004	MW-6	30.76	90.49	59.73	2	8	50.49	71.49	20
6/23/2004	MW-6	28.21	90.49	62.28	2	8	50.49	71.49	20
8/30/2004	MW-6	28.47	90.49	62.02	2	8	50.49	71.49	20
12/22/2004	MW-6	30.64	90.49	59.85	2	8	50.49	71.49	20
2/22/2005	MW-6	30.35	90.49	60.14	2	8	50.49	71.49	20
6/8/2005	MW-6	29.12	90.49	61.37	2	8	50.49	71.49	20
8/23/2005	MW-6	28.98	90.49	61.51	2	8	50.49	71.49	20
11/15/2005	MW-6	29.25	90.49	61.24	2	8	50.49	71.49	20
3/8/2006	MW-6	21.37	90.49	69.12	2	8	50.49	71.49	20
7/14/2000	MW-7	29.67	106.42	76.75	2	8	61.42	86.42	20
9/5/2000	MW-7	36.56	106.42	69.86	2	8	61.42	86.42	20
12/14/2000	MW-7	40.71	106.42	65.71	2	8	61.42	86.42	20
3/7/2001	MW-7	Dry	106.42	Dry	2	8	61.42	86.42	20
5/30/2001	MW-7	33.21	106.42	73.21	2	8	61.42	86.42	20
8/28/2001	MW-7	38.51	106.42	67.91	2	8	61.42	86.42	20
11/20/2001	MW-7	39.94	106.42	66.48	2	8	61.42	86.42	20
3/13/2002	MW-7	Dry	106.42	Dry	2	8	61.42	86.42	20
6/12/2002	MW-7	37.21	106.42	69.21	2	8	61.42	86.42	20
9/20/2002	MW-7	38.45	106.42	67.97	2	8	61.42	86.42	20
1/13/2003	MW-7	Dry	106.42	Dry	2	8	61.42	86.42	20
3/31/2003	MW-7	Dry	106.42	Dry	2	8	61.42	86.42	20
6/16/2003	MW-7	40.72	106.42	65.70	2	8	61.42	86.42	20
9/30/2003	MW-7	38.42	106.42	68.00	2	8	61.42	86.42	20
12/11/2003	MW-7	41.45	106.42	64.97	2	8	61.42	86.42	20
3/24/2004	MW-7	42.51	106.42	63.91	2	8	61.42	86.42	20
6/23/2004	MW-7	40.60	106.42	65.82	2	8	61.42	86.42	20
8/30/2004	MW-7	38.37	106.42	68.05	2	8	61.42	86.42	20
12/22/2004	MW-7	Dry	106.42	Dry	2	8	61.42	86.42	20
2/22/2005	MW-7	Dry	106.42	Dry	2	8	61.42	86.42	20
6/8/2005	MW-7	Dry	106.42	Dry	2	8	61.42	86.42	20
8/23/2005	MW-7	Dry	106.42	Dry	2	8	61.42	86.42	20
11/15/2005	MW-7	42.55	106.42	63.87	2	8	61.42	86.42	20
3/8/2006	MW-7	42.27	106.42	64.15	2	8	61.42	86.42	20

City of Escalon

Historic Groundwater Elevation and Monitoring Well Construction Information.

Date	Well ID	Depth to Water (ft)	Top of Casing Elevation (ft.)	Groundwater Elevation (ft.)	Well Diameter (in)	Boreholde Diameter (in)	Borehole Bottom Elevation (ft.)	Approximate Top of Screen Elevation (ft.)	Screen Length (ft)
7/14/2000	MW-8	37.22	103.88	66.66	2	8	58.88	78.88	20
9/5/2000	MW-8	35.95	103.88	67.93	2	8	58.88	78.88	20
12/14/2000	MW-8	39.17	103.88	64.71	2	8	58.88	78.88	20
3/7/2001	MW-8	39.88	103.88	64.00	2	8	58.88	78.88	20
5/30/2001	MW-8	37.99	103.88	65.89	2	8	58.88	78.88	20
8/28/2001	MW-8	38.16	103.88	65.72	2	8	58.88	78.88	20
11/20/2001	MW-8	38.33	103.88	65.55	2	8	58.88	78.88	20
3/13/2002	MW-8	39.80	103.88	64.08	2	8	58.88	78.88	20
6/12/2002	MW-8	38.47	103.88	65.41	2	8	58.88	78.88	20
9/20/2002	MW-8	36.68	103.88	67.20	2	8	58.88	78.88	20
1/13/2003	MW-8	39.77	103.88	64.11	2	8	58.88	78.88	20
3/31/2003	MW-8	40.97	103.88	62.91	2	8	58.88	78.88	20
6/16/2003	MW-8	38.15	103.88	65.73	2	8	58.88	78.88	20
9/30/2003	MW-8	37.42	103.88	66.46	2	8	58.88	78.88	20
12/11/2003	MW-8	40.29	103.88	63.59	2	8	58.88	78.88	20
3/24/2004	MW-8	40.82	103.88	63.06	2	8	58.88	78.88	20
6/23/2004	MW-8	39.09	103.88	64.79	2	8	58.88	78.88	20
8/30/2004	MW-8	37.16	103.88	66.72	2	8	58.88	78.88	20
12/22/2004	MW-8	39.97	103.88	63.91	2	8	58.88	78.88	20
2/22/2005	MW-8	40.19	103.88	63.69	2	8	58.88	78.88	20
6/8/2005	MW-8	37.32	103.88	66.56	2	8	58.88	78.88	20
8/23/2005	MW-8	37.60	103.88	66.28	2	8	58.88	78.88	20
11/15/2005	MW-8	39.48	103.88	64.40	2	8	58.88	78.88	20
3/8/2006	MW-8	36.10	103.88	67.78	2	8	58.88	78.88	20
3/11/2002	MW-9	39.44	102.35	62.91	NA	NA	NA	67.35	NA
3/13/2002	MW-9	39.45	102.35	62.90	NA	NA	NA	67.35	NA
6/12/2002	MW-9	36.34	102.35	66.01	NA	NA	NA	67.35	NA
9/20/2002	MW-9	35.79	102.35	66.56	NA	NA	NA	67.35	NA
1/13/2003	MW-9	38.70	102.35	63.65	NA	NA	NA	67.35	NA
3/31/2003	MW-9	39.46	102.35	62.89	NA	NA	NA	67.35	NA
6/16/2003	MW-9	35.10	102.35	67.25	NA	NA	NA	67.35	NA
9/30/2003	MW-9	36.62	102.35	65.73	NA	NA	NA	67.35	NA
12/11/2003	MW-9	38.62	102.35	63.73	NA	NA	NA	67.35	NA
3/24/2004	MW-9	40.31	102.35	62.04	NA	NA	NA	67.35	NA
6/23/2004	MW-9	36.40	102.35	65.95	NA	NA	NA	67.35	NA
8/30/2004	MW-9	36.25	102.35	66.10	NA	NA	NA	67.35	NA
12/22/2004	MW-9	39.32	102.35	63.03	NA	NA	NA	67.35	NA
2/22/2005	MW-9	40.01	102.35	62.34	NA	NA	NA	67.35	NA
6/8/2005	MW-9	33.70	102.35	68.65	NA	NA	NA	67.35	NA
8/23/2005	MW-9	36.09	102.35	66.26	NA	NA	NA	67.35	NA
11/15/2005	MW-9	38.51	102.35	63.84	NA	NA	NA	67.35	NA
3/8/2006	MW-9	35.50	102.35	66.85	NA	NA	NA	67.35	NA
3/11/2002	MW-10	40.86	103.99	63.13	NA	NA	NA	69.99	NA
3/13/2002	MW-10	40.72	103.99	63.27	NA	NA	NA	69.99	NA
6/12/2002	MW-10	37.53	103.99	66.46	NA	NA	NA	69.99	NA
9/20/2002	MW-10	37.72	103.99	66.27	NA	NA	NA	69.99	NA
1/13/2003	MW-10	40.81	103.99	63.18	NA	NA	NA	69.99	NA
3/31/2003	MW-10	40.85	103.99	63.14	NA	NA	NA	69.99	NA
6/16/2003	MW-10	37.16	103.99	66.83	NA	NA	NA	69.99	NA
9/30/2003	MW-10	37.92	103.99	66.07	NA	NA	NA	69.99	NA
12/11/2003	MW-10	40.60	103.99	63.39	NA	NA	NA	69.99	NA

City of Escalon

Historic Groundwater Elevation and Monitoring Well Construction Information.

Date	Well ID	Depth to Water (ft)	Top of Casing Elevation (ft.)	Groundwater Elevation (ft.)	Well Diameter (in)	Boreholde Diameter (in)	Borehole Bottom Elevation (ft.)	Approximate Top of Screen Elevation (ft.)	Screen Length (ft)
3/24/2004	MW-10	41.80	103.99	62.19	NA	NA	NA	69.99	NA
6/23/2004	MW-10	37.51	103.99	66.48	NA	NA	NA	69.99	NA
8/30/2004	MW-10	37.73	103.99	66.26	NA	NA	NA	69.99	NA
12/22/2004	MW-10	41.01	103.99	62.98	NA	NA	NA	69.99	NA
2/22/2005	MW-10	41.82	103.99	62.17	NA	NA	NA	69.99	NA
6/8/2005	MW-10	35.21	103.99	68.78	NA	NA	NA	69.99	NA
8/23/2005	MW-10	38.31	103.99	65.68	NA	NA	NA	69.99	NA
11/15/2005	MW-10	39.94	103.99	64.05	NA	NA	NA	69.99	NA
3/8/2006	MW-10	37.51	103.99	66.48	NA	NA	NA	69.99	NA
3/11/2002	MW-11	42.17	106.26	64.09	NA	NA	NA	71.26	NA
3/13/2002	MW-11	43.05	106.26	63.21	NA	NA	NA	71.26	NA
6/12/2002	MW-11	39.85	106.26	66.41	NA	NA	NA	71.26	NA
9/20/2002	MW-11	40.24	106.26	66.02	NA	NA	NA	71.26	NA
1/13/2003	MW-11	43.25	106.26	63.01	NA	NA	NA	71.26	NA
3/31/2003	MW-11	43.11	106.26	63.15	NA	NA	NA	71.26	NA
6/16/2003	MW-11	39.79	106.26	66.47	NA	NA	NA	71.26	NA
9/30/2003	MW-11	40.14	106.26	66.12	NA	NA	NA	71.26	NA
12/11/2003	MW-11	42.95	106.26	63.31	NA	NA	NA	71.26	NA
3/24/2004	MW-11	44.15	106.26	62.11	NA	NA	NA	71.26	NA
6/23/2004	MW-11	39.84	106.26	66.42	NA	NA	NA	71.26	NA
8/30/2004	MW-11	40.11	106.26	66.15	NA	NA	NA	71.26	NA
12/22/2004	MW-11	43.37	106.26	62.89	NA	NA	NA	71.26	NA
2/22/2005	MW-11	44.15	106.26	62.11	NA	NA	NA	71.26	NA
6/8/2005	MW-11	37.83	106.26	68.43	NA	NA	NA	71.26	NA
8/23/2005	MW-11	40.76	106.26	65.50	NA	NA	NA	71.26	NA
11/15/2005	MW-11	42.17	106.26	64.09	NA	NA	NA	71.26	NA
3/8/2006	MW-11	39.54	106.26	66.72	NA	NA	NA	71.26	NA

NA Information Not available

Source: Geological Technics Inc.