CSA-7A WASTEWATER SYSTEM CAPACITY STUDY
INTERIM REPORT

OAK SHORES
CALIFORNIA

WALLACE GROUP PROJECT
#0298-0009 TRACT 2162

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Chapter 1 - Executive Summary

1.1 Introduction and Purpose

The County of San Luis Obispo (County) provides wastewater collection, treatment, and disposal services to the community of Oak Shores through County Service Area No. 7A (CSA-7A). The existing service area includes 583 existing homes and 329 vacant parcels that are authorized to connect to the wastewater system. According to a recent study commissioned by the County in 2005, the existing treatment and disposal system requires improvement in order to provide adequate service to permitted lots. The existing treatment plant is permitted for 100,000 gpd. The average annual flow to the plant averaged over the past 12 years is 43,645 gpd. A significant amount of additional development is pending in the Oak Shores area. The County is currently working with land developers to enter into a Memorandum of Understanding (MOU) with respect to system capacity, processing requirements, and needed improvements to provide for orderly development.

Wallace Group has been retained by the owner of Tract 2162 (see Figure 2-1 for location of Tract 2162) to prepare this Wastewater System Capacity Study Interim Report to address the specific needs of Phases 2 through 6 of Tract 2162 and the general needs of the other potential developments. This report addresses the available permitted capacity of the existing WWTP, the collection system, the wastewater treatment plant, and the disposal system. A detailed hydrogeologic and mounding analysis of the Kavanaugh effluent disposal site is also provided (Appendix A). The purpose of this Interim Report is to (1) present major treatment and collection alternatives for consideration by the County (2) to identify the available permitted capacity within the existing Waste Discharge Requirements, Order No. 01-130 in order to provide the County with a basis with which to record the final maps for Tract 2162. Once an alternative is selected, a Final Report will then be prepared detailing the improvements required to implement the selected alternative.

1.2 Wastewater Flows

Wastewater flow characteristics in the recreational community of Oak Shores have been the subject of several past technical reports. The existing community contains primarily vacation homes, and the current permanent full-time occupancy level is estimated at 26%. Peak and seasonal wastewater flows were discussed in detail by the County's consultant, Garing Taylor & Associates in a report entitled, "Capacity and Demand Report, CSA-7A Wastewater Treatment Plant and Disposal Facilities (October, 2005)." Many of the methods employed in the County's study were used, and flow estimates were prepared for future conditions as a function of development. Key flow factors are listed as follows:

- Actual average annual flow per unit at the historical occupancy rate of 26% ranges between 75 gpd/conn and 108 gpd/conn depending on which years of recorded flows are used to compute the average.

- Design average annual flow per unit varies with full-time occupancy levels. The flow per unit is estimated at 120 gallons per day (gpd/unit), 128 gpd/unit, and 162 gpd/unit for occupancies of 26% (existing), 40% (future), and 100% (worst case), respectively.

- The average flow during the maximum month (ADMMF) is estimated at 162 gpd/unit and is the basis for the design average annual flow of units that are occupied on a full time basis.

- Peak Day Dry Weather Flow is estimated at 352 gpd/unit.

- The Peak Hour Flow is estimated at 0.45 gallons per minute per unit.

Additional flow summaries are provided in Tables 1-1 and 1-2 as follows:

Table 1-1 Average Daily Flow During The Maximum Month

Development Entity	Number of Units	Cumulative Units	ADMMF (gpd) (162 gpd/conn)
Existing CSA-7A Dwellings*	583	583	93,210
Future CSA-7A Dwellings	269	852	138,024
Davis/King Tract 2162 Phase 1	60	912	147,744
Davis/King Tract 2162 Phase 2-6	285	1197	193,914
Bean/Lynch Canyon Properties (Hughes) Tract 2520	307	1504	243,648
Munari	134	1638	265,356
Vaughn/Taylor/ Crawford	25	1663	269,406
Lynch Canyon Properties (Hughes)	50	1713	277,506

* This value is based off of the actual highest recorded ADMMF in gpd and not by multiplying the design ADMMF of 162 gpd/conn by 583 as was done for the other values in the table

Table 1-2 Design Average Annual Flow as a Function of Full-Time Occupancy				
Development Entity	**Cum. Units**	**Design Average Annual Flow by Full-Time Occupancy Rate in ac-ft/yr and gpd**		
		26% (0.134 AF/u)	40% (0.143 AF/u)	100% (0.182 AF/u)
Existing CSA-7A Dwellings	583	78 ac-ft/yr (69,700 gpd)	83 ac-ft/yr (74,400 gpd)	106 ac-ft/yr (94,700 gpd)
All CSA-7A permitted units	912	122 ac-ft/yr (109,100 gpd)	130 ac-ft/yr (116,400 gpd)	166 ac-ft/yr (148,200 gpd)
Davis/King Tract 2162 Phase 2-6	1197	160 ac-ft/yr (143,200 gpd)	171 ac-ft/yr (152,800 gpd)	218 ac-ft/yr (194,500 gpd)
Completion of pending developments	1713	230 ac-ft/yr (204,900 gpd)	245 ac-ft/yr (218,700 gpd)	311 ac-ft/yr (278,300 gpd)

Peak Day Dry Weather Flows were estimated as follows:

- Existing CSA-7A units – 205,200 gpd
- All currently permitted units with Tract 2162 (1,197 units) – 421,300 gpd
- All pending development (1,713 units) – 603,000 gpd

1.3 Collection System

A collection system analysis was prepared to assess potential points of connection and impacts of future development. The existing collection system for Oak Shores discharges to a trunk main, known as the Interceptor, that is located beneath the Nacimiento Lake high water level. The Interceptor is a source of concern for the County due in part to the potential for inflow and spills. Future connections to the Interceptor were avoided. A detailed collection system analysis was performed for Phases 2 through 6 of Tract 2162. Two alternatives were developed for review as follows:

Alternative A

- Convey flows from Phases 2 through 6 of Tract 2162 directly to the existing wastewater treatment plant through a 6” force main with a length of 4,200 ft.
- A portion of the force main would be constructed on two pipe bridges that were previously contemplated as County capital improvements.
- The cost of Alternative A is estimated at $1,020,000.

Alternative B

- Convey flows from Phases 2 through 6 of Tract 2162 to a new wastewater treatment plant (WWTP) site adjacent to the existing spray field. This alternative would require a 6" force main with a total length of 11,000 ft.
- This alternative corresponds to treatment plant Alternative B described below.
- The cost of Alternative B is estimated at $2,100,000.

With respect to other future developments, general criteria and recommendations were made for connections to the existing collection system. In general, developers should anticipate connecting to portions of the collection system that are only tributary to final reaches of the Interceptor. Downstream improvements would be required to route flows directly to the existing influent lift station near the WWTP.

1.4 Effluent Disposal

The effluent disposal system was analyzed in terms of existing and proposed capacity. A detailed study of the Kavanaugh percolation pond site was conducted including field testing, development of a hydrogeologic model, and water quality analysis. The Regional Water Quality Control Board was consulted to determine system constraints and future regulatory issues. A potential 10-acre spray field site on the Lynch Canyon Properties (Hughes) property to the west of Kavanaugh Creek was also analyzed for future use. The results of the analysis are as follows:

- The amount of available permitted capacity (see Table 5-1) in WDR 01-130, assuming a future full-time occupancy rate of 26%, is sufficient to allow the County to final the maps for:
    - Tract 2162 phases 2-5 (1,137 units) with 10 acre-ft of effluent storage if the 12-year AAF is used to predict actual future flows
    - Tract 2162 phases 2-6 (1,197 units) with 7.9 acre-ft of effluent storage if the last 5 years of AAF are used to predict future flows,
    - 923 units with 13.3 acre-ft of effluent storage if the maximum AAF recorded is used to predict future flows.
- The Kavanaugh percolation pond site has the capacity to dispose of an annual average flow of 145,000 gpd or 162 ac-ft/yr without excessive mounding or daylighting.
- Disposal capacity at Kavanaugh is not constrained by salt loading, but will require effluent nitrogen removal to a level below 10 mg/l as the community grows. Nitrogen removal will be required for the existing approved 913 lots when the number of developed lots reaches 896 if full-time occupancy is 40%.
- The existing spray field and Kavanaugh site are adequate to dispose of effluent generated from existing CSA-7A lots and Phases 2 through 6 of Tract 2162 plus an additional 310 units at the current full-time occupancy rate of 26%. If the full-time occupancy rate increases to 40% in the future, then there is adequate capacity for Tract 2162 phases 2-6 (1,197 units) plus an additional 215 units from other proposed developments.
- The existing spray field, Kavanaugh, and proposed Lynch Canyon Properties (Hughes) site provide adequate disposal capacity for all anticipated development (1,713 lots), even if the full-time occupancy rate increases to 40% in the future as summarized in Table 1-3.

Table 1-3 Development Potential as a Function of Disposal Capacity				
Disposal Areas	**Capacity (ac-ft)**	**Allowable Number of Housing Units by Average Full-Time Occupancy Rate**		
		26% (0.134 AF/u)	40% (0.143 AF/u)	100% (0.182 AF/u)
Existing spray field	40	298	279	219
Existing spray field + Kavanaugh	202	1,507	1,412	1,109
Existing spray field + Kavanaugh + Lynch Canyon Properties (Hughes)	259	1,932	1,811	1,423

1.5 Wastewater Treatment

The existing CSA-7A wastewater treatment plant includes two 400,000 gallon aerated lagoons and one 1.6 million gallon settling/maturation pond. The system is designed to treat an Average Daily Maximum Month Flow of 100,000 gpd to a secondary level. Due to the disposal limitations referenced above, a change in treatment technology will be required in order to limit nitrogen discharges from the WWTP to less than 10 mg/l. The County has expressed concerns that a more advanced treatment technology would result in higher fees to current users. In order to address this concern and provide options for consideration, two alternatives were developed. In Alternative A, the existing treatment plant would be upgraded to provide nitrogen removal, and flows from future development would be routed to the plant. In Alternative B, a new WWTP would be constructed adjacent to the existing spray field site. This new WWTP would be constructed solely for future development. A pond technology known as Biolac® was selected as the preferred approach due to its simplicity of use and track record. The alternatives are described in additional detail in Tables 1-4 and 1-5 below:

Table 1-4 Preliminary Configuration for Treatment Alternative A (Upgrade Existing)	
Process	**Preliminary Sizing and Description**
Headworks (new)	Automated 3 mm screening / washing system
Biological treatment using **Biolac®** process for nitrogen removal	Two parallel units. Phase 1: **425,000 gpd** (existing 912 lots + Tract 2162) and Phase 2: additional **185,000 gpd** (1,713 lots). At build out, approximately 0.61 mgd.
Energy requirements	Comparable to existing aerated ponds on a unit basis
Solids production	At 40% full-time occupancy, approx. **31 dry tons** per year. Existing two aerated ponds to be converted to aerated sludge storage. Drying beds to be provided to minimize O&M cost.
Effluent Pump Station (new)	New station sized for PDDWF of 425,000 gpd in Phase 1 and 605,000 gpd in Phase 2. Pressure increase on existing effluent force main is less than 2 psi for Phase 2.

Table 1-5 Preliminary Configuration for Treatment Alternative B (New WWTP)	
Process	**Preliminary Sizing and Description**
Headworks	Automated 3 mm screening / washing system
Biological treatment using **Biolac®** process for nitrogen removal	Two parallel units. Phase 1: **100,000 gpd** (Tract 2162, Phases 2 through 6) and Phase 2: additional **185,000 gpd**. At build out, approximately 0.29 mgd.
Energy requirements	Comparable to existing aerated ponds on a unit basis
Solids production	At 40% full-time occupancy, approximately **15 dry tons** per year. Drying beds and aerated sludge holding to be provided.
Effluent Pump Station	Not required – gravity flow to existing spray field ponds.

A preliminary review of operations and maintenance costs was conducted. If Alternative A is implemented (Upgrade Existing WWTP), the recommended Biolac® system is not expected to significantly increase the cost of operations and maintenance.

1.6 Recommended Improvements

The implementation of Alternative A (Upgrade Existing WWTP) is recommended. The estimated cost of the recommended project to provide service to Phases 2 through 6 of Tract 2162 is summarized in Table 1-6 as follows:

Table 1-6 Preliminary Cost Estimate for Recommended Project (Alternative A)	
Component	**Estimated Total Project Cost**
Force Main to convey Tract 2162 flow to the Existing WWTP	$1,021,000
Upgrade of existing WWTP to Biolac system (Phase 1)	$6,006,000
New percolations ponds at the Kavanaugh Site	$1,825,000
Total Cost	**$8,852,000**

Cost sharing arrangements and final phasing will be worked out with the County prior to issuance of a Final Report. The actual cost of project implementation for a private developer is expected to be significantly less than the above estimates, which include a 50% contingency and an 82% project multiplier.

2.1 Introduction

The County of San Luis Obispo provides wastewater collection, treatment, and disposal services to the community of Oak Shores through County Service Area No. 7A (CSA-7A). The existing service area is shown in Figure 2-1, which includes Tracts 378, 379, 380, 381, and Phase 1 of Tract 2162. The system is regulated by the Regional Water Quality Control Board under Waste Discharge Requirements, Order No. 01-130 (WDR 01-130). A copy of the order is provided in Appendix D.6. The existing community is primarily residential in nature, and only 26% of the existing homes are occupied on a full time basis. The capacity of the existing treatment and disposal system was analyzed by the County's consultant, Garing Taylor & Associates (GTA) in a report entitled, "Capacity and Demand Report, CSA-7A Wastewater Treatment Plant and Disposal Facilities (October, 2005)." A copy of the report is provided electronically in Appendix D.1. For the purpose of final map processing (not design), the available permitted capacity of WDR 01-130 provides the County with a basis to allow recordation of the Final Map for Tract 2162 without an amended discharge permit, assuming that effluent equalization facilities are provided. The GTA report concludes that from a design standpoint, existing facilities will be inadequate in the future if full-time occupancy patterns change significantly or if all presently permitted vacant lot owners develop their properties. There are approximately 583 existing homes in the community, and 912 residential lots are permitted and available for development.

Future development potential in the Oak Shores area is described in the County's Nacimiento Area Plan (2003). The Area Plan indicates that the community can be expected to grow from 913 recorded lots to a maximum of 1,786 lots. Additional background is provided in Appendix D.3. Tract 2162 is an approved phased tentative map with a remaining entitlement of 285 lots contained in five phases (Phases 2 through 6). Prior to the recordation of additional phases of Tract 2162, a final will-serve commitment from CSA-7A must be obtained. As described in Chapter 3, a number of other proposed developments are ready to begin the Tentative Map process, which requires the issuance of a preliminary will-serve commitment. The County is currently working with representatives from various development interests to create a Memorandum of Understanding (MOU). The purpose of the MOU is to provide an organized and mutually-beneficial process for the annexation of developments into CSA-7A (see Appendix D.9).

2.2 Purpose and Scope

Wallace Group has been retained by the developer of Tract 2162 to prepare an analysis of the existing collection, treatment, and disposal system for CSA-7A. The existing collection system was previously analyzed by the County in 2004 in an unofficial report entitled "Interceptor Bypass Study" (see Appendix D.3). The existing treatment and disposal system has also been recently analyzed as indicated above. Given the work already completed to date, the purpose of this study is to analyze the capacity of the

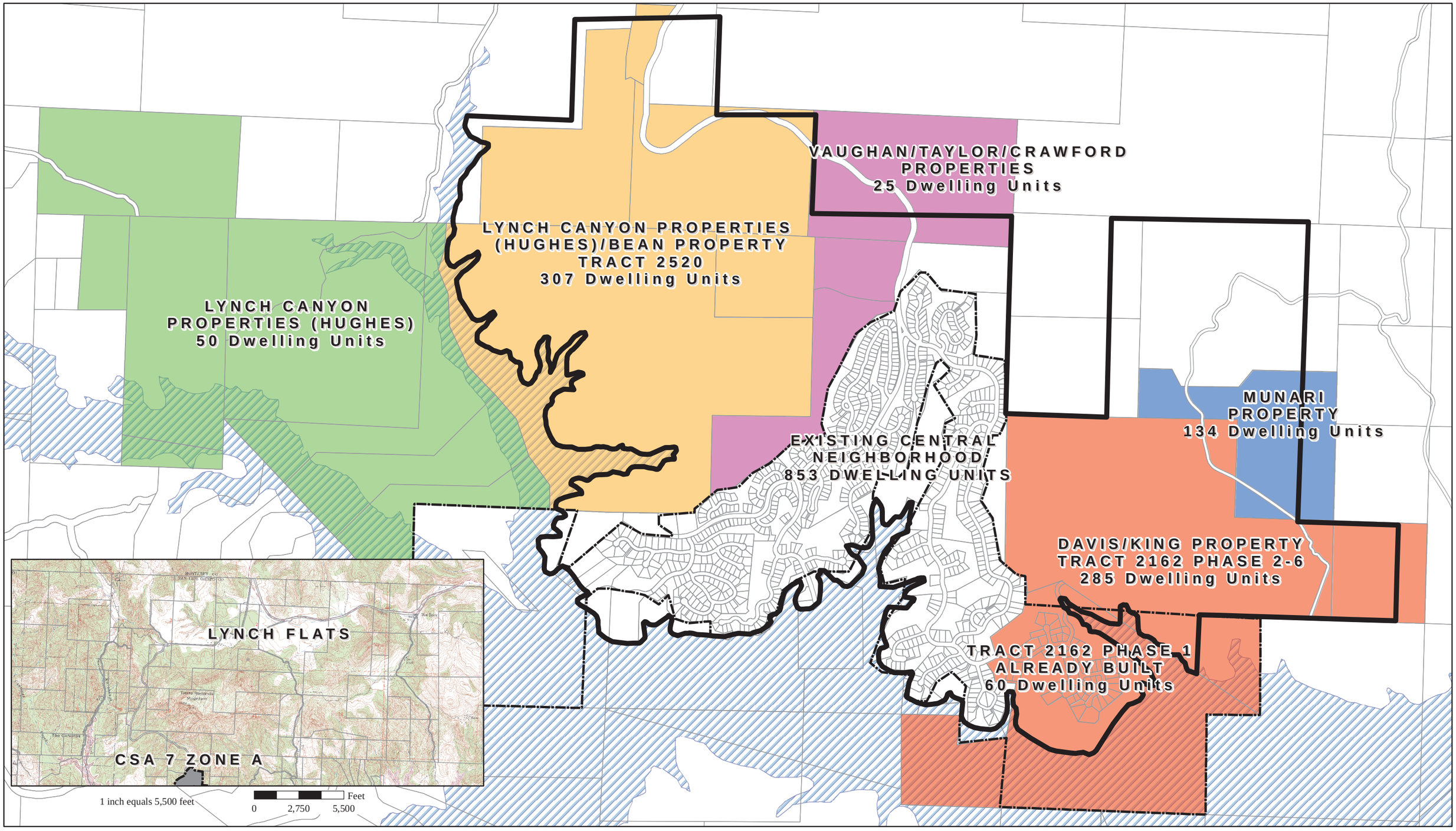
wastewater system to provide for future development. This study has been titled an "interim" report to provide the County with an opportunity to provide input on system alternatives prior to the completion of detailed facilities planning. The scope of this interim report is as follows:

- Provide an estimate of wastewater flows for various levels of full-time occupancy using methodologies developed in the 2005 Capacity and Demand Report. Future projections are limited to known developments as defined by the participants in the MOU.
- Estimate the available permitted capacity within the framework of WDR 01-130 using actual average annual flows to predict future flows from a permitting standpoint.
- Determining the effluent storage needed to "flatten" out effluent flows in order to realize the maximum available permitted capacity under WDR 01-130.
- Analyze the impact of Tract 2162 on the existing collection system, and develop recommendations and cost estimates for the purpose of bonding.
- Provide general collection system recommendations and criteria for the connection of other planned developments.
- Prepare a detailed hydrogeologic evaluation of the Kavanaugh Effluent Disposal Site that identifies both hydraulic and water quality limitations.
- Prepare an integrated feasibility and alternatives analysis for treatment and disposal facilities necessary to provide for future development. This analysis includes consideration of existing and future regulatory constraints.
- Develop treatment and disposal alternatives for the County's consideration. The selected alternative will provide a basis for detailed facilities planning.
- Recommend a collection, treatment, and disposal alternative for providing service to Phases 2 through 6 of Tract 2162. General feasibility will be established for other developments to facilitate issuance of a preliminary will-serve commitment.

2.3 Next Steps and Approval Process

Based on the information contained in this report, the developer of Tract 2162 is seeking Final Map approval for Phases 2 through 6.

1. Final alternatives for collection, treatment, and disposal will be selected.
2. The basis for cost sharing will be established in accordance with the benefit provided to existing and proposed development.
3. A Final Report will be provided with a higher level of facilities planning. The report will include bonding estimates based on the negotiated cost sharing arrangement.



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Legend

- BEAN PROPERTY
- DAVIS/KING PROPERTY
- LYNCH CANYON PROPERTIES
- MUNARI PROPERTY
- VAUGHAN/TAYLOR/CRAWFORD
- Oak Shores Village Reserve Area
- Nacimiento Lake
- CSA 7 Zone A

CSA-7A Wastewater System Capacity Study Interim Report

Figure 2-1 CSA-7A and Proposed Development

N W E S

1 inch equals 1,200 feet

0 600 1,200 2,400 Feet

3.1 Land Use and Proposed Development

The current County Service Area 7A (CSA-7A) includes the community of Oak Shores which is located on the north shore of Lake Nacimiento within the Nacimiento Planning Area. The County Land Use Element, the Nacimiento Area Plan, and the standards in Chapter 22.102 serve as the specific plan for the development of Oak Shores. The 1974 Oak Shores Specific Plan originally provided for 4,000 units for the entire community. Since then, anticipated land uses have been significantly reduced by the Land Use Element. The reductions in development potential are primarily adjustments in the village boundary to exclude northern portions of the Lynch Flat area. The resultant maximum allowable number of dwelling units within the Oak Shores village reserve line is 1,786 including RV sites. The village is 1,576 acres in area. Just to the north of the village reserve line is the Tierra Redonda Mountain sensitive resource area (SRA) which encompasses approximately 1,300 acres with 320 acres under Bureau of Land Management ownership. Oak Shores has been envisioned as a resort community of vacation and retirement homes with various recreational uses.

The village is divided into three neighborhoods: west, central, and east. The maximum of 1,786 total units within the village reserve line is allocated between the three neighborhoods as follows: 405 dwelling units in the west neighborhood, 853 dwelling units and a marina activity center in the central neighborhood, and 528 dwelling units in the east neighborhood. The 853 lots in the central neighborhood are divided into four tracts: 378, 379, 380, and 381. As of 2005, 583 of the 853 lots in the central neighborhood had been developed leaving 271 for future development. Recently Phase 1 of Tract 2162 which includes 60 lots was recorded in the east neighborhood.

Figure 2-1 shows the Oak shores community and the relationship between the three neighborhoods as well as the existing and proposed developments in each. The proposed level of development that is being considered for the purposes of this report is detailed in Table 3-1. Table 3-1 is not intended to illustrate the order of development but rather, the number of dwelling units which are associated with each owner. This distribution of dwelling units will be the basis for the wastewater flow demand analysis in this chapter. The purpose of this report is to provide a technical tool for the implementation of the MOU referenced in Chapter 2. For this reason, the total number of contemplated units (1,713) is 4% less than the above-referenced build-out number of 1,786.

Table 3-1 Proposed Oak Shores Development		
Development Entity	**Number of Units**	**Cumulative Units**
Existing CSA-7A Dwellings*	583	583
Future CSA-7A Dwellings	269	852
Davis/King Tract 2162 Phase 1	60	912
Davis/King Tract 2162 Phase 2-6	285	**1197**
Bean/Lynch Canyon Properties (Hughes) Tract 2520	307	1504
Munari	134	1638
Vaughn/Taylor/Crawford	25	1663
Lynch Canyon Properties (Hughes)	50	**1713**

* *taken from Garing Taylor & Assoc. as of 2005*

3.2 Existing Actual Flows

Information on past and current wastewater flow patterns was obtained from the San Luis Obispo County Public Works Department. The data includes daily wastewater influent totals from the totalizing influent flow meter at the Oak Shores WWTP from 1994 through 2005. The data was used as a basis for predicting future flows. During the analysis, missing daily flows and several multiple occurrences of daily flows were discovered, which invalidates monthly total flows. In order to correct for this, the actual number of flow recordings per month were used to determine average daily flow for any given month. Table 3-3 shows the monthly average flow data in gallons per day. Table 3-4 is a similar summary with monthly average flow in gallons per day per connection to be used in the establishment of design flows for future developments. The number of sewer connections for each year of flow data was taken from the 2005 Capacity and Demand Report.

Table 3-2 shows the actual average annual flow for each of the past twelve years on record. From the County flow data, it is seen that the AAF varies between a minimum of 31,686 gpd in 1994 to a maximum of 52,596 in 1998. When the AAF for each year is divided by the number of connections during that year, the AAF per connection ranges between 62 gpd/conn and 108 gpd/conn. Various periods of flow data were used to establish overall averages for annual flow. The results are shown in Table 3-2.

Table 3-2 Actual Average Annual Flow for Various Periods			
Month	**12-Year Ave AAF** (gpd/conn, '94-'05)	**Last 5 Years Ave AAF** (gpd/conn, '01-'05)	**Max AAF** (gpd/conn, occurred in 1998)
January	67	63	70
February	71	59	128
March	79	65	95
April	94	85	112
May	106	101	128
June	104	95	133
July	121	107	159
August	110	98	141
September	83	72	96
October	67	60	77
November	66	57	83
December	62	52	74
Ave AAF	**86**	**76**	**108**

Table 3-3 Summary Table of CSA-7A Wastewater Influent Flows averaged over each calendar month

	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	Maximums	Minimums	Mean
January	23,913	32,746	28,310	43,166	34,103	39,667	35,184	39,019	34,834	34,310	26,755	40,065	43,166	23,913	34,339
February	26,243	30,135	35,682	29,589	62,452	44,554	40,790	34,545	34,623	35,738	29,229	30,548	62,452	26,243	36,177
March	26,716	53,800	38,590	35,243	46,516	46,232	47,681	42,783	34,313	33,681	27,161	44,194	53,800	26,716	39,743
April	33,613	60,313	41,523	33,370	54,624	58,500	54,335	48,490	40,277	36,241	37,900	76,493	76,493	33,370	47,973
May	37,361	56,968	51,955	42,617	62,534	60,973	52,407	50,519	51,587	45,904	43,943	93,210	93,210	37,361	54,165
June	37,603	61,800	45,337	41,790	64,937	62,275	58,311	49,740	62,527	46,540	45,700	61,306	64,937	37,603	53,155
July	43,106	74,197	56,116	52,838	77,690	69,459	65,190	58,990	65,741	59,897	52,261	62,832	77,690	43,106	61,527
August	35,887	59,345	51,487	57,778	68,784	66,600	57,075	56,750	55,035	54,503	46,359	61,794	68,784	35,887	55,950
September	29,833	53,383	36,545	40,897	47,070	48,480	45,703	38,431	43,280	37,410	35,213	46,431	53,383	29,833	41,890
October	25,555	39,700	31,088	30,506	37,784	43,016	35,687	29,639	44,210	31,665	28,629	33,503	44,210	25,555	34,248
November	28,480	34,163	29,496	36,440	40,457	39,932	34,020	33,260	36,317	30,793	27,807	29,413	40,457	27,807	33,381
December	31,413	29,994	36,319	31,642	36,303	33,121	31,930	30,717	34,407	26,247	25,770	26,476	36,319	25,770	31,195
Average Annual Flow (gpd)	31,686	49,416	40,427	39,732	52,596	50,699	45,996	42,317	44,612	37,860	35,510	49,368			43,352
Max Day (gpd)	96,200	164,400	128,100	114,600	127,100	153,300	129,900	120,100	138,100	139,300	130,000	194,800			
Min Day (gpd)	2,800	0	13,400	14,400	12,800	17,700	10,100	13,700	16,500	12,800	2,680	3,390			
Aprox # of Connections	449	458	464	474	488	494	512	524	550	568	575	583			

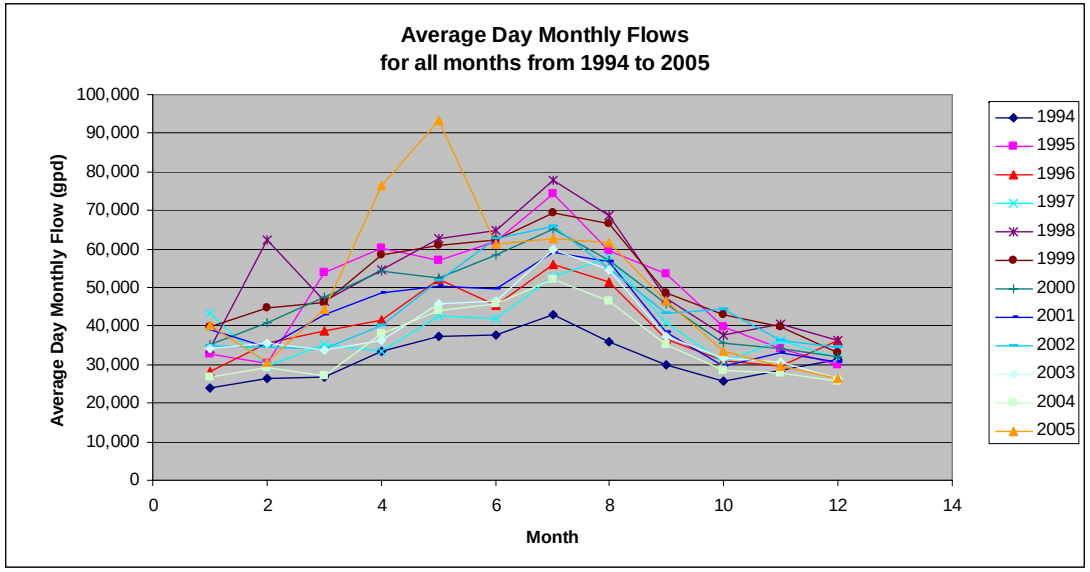


Figure 3-1 Summary Chart of CSA-7A Wastewater Influent Flows averaged over each calendar month

3.3 Design Flows for Future Development

The average and peak wastewater flow conditions of interest for sewer design are defined below.

Average Day Maximum Month Flow (ADMMF) – the maximum flow occurring in one calendar month divided by the number of days in that month. As explained previously, the number of actual records in the month was used to compute this flow rate instead of the number of days. For many months the two were not equal. According to the County flow data, the ADMMF in gallons per day occurred in May, 2005 with an average of 93,210 gpd but the per connection ADMMF used for design occurred in July, 1995 with 162 gpd/conn as shown in Table 3-4 and Figures 3-3 and 3-4.

Design Average Annual Flow (AAF) – obtained by dividing the total projected design flow discharged in one calendar year by 362.25 days. The per connection design flow of 162 gpd/conn, as discussed above, was used to establish the total projected design flow at different levels of occupancy. Using 162 gpd/conn for the determination of design average annual flow is conservative and is the same methodology employed by the 2005 Capacity and Demand Report. For most communities, other peak flow conditions are derived by multiplying the AAF by a peaking factor. As explained below, due to the seasonal variation in wastewater flow in Oak Shores, AAF will not be used to compute peak flow conditions; rather, actual historical peak flow data will be employed to determine appropriate design flows.

Peak Day Dry Weather Flow (PDDWF) – the maximum flow occurring in one day during the dry season. Due to the fact that Oak Shores is a resort community with high summertime use, especially on holiday weekends, a corresponding Peak Day Wet Weather Flow was not determined. PDDWF is used in the sizing of treatment processes. According to the County flow records, the PDDWF in gallons per day is 194,800 gpd. This flow occurred on Sunday, May 29, 2005 during Memorial Day Weekend. The per connection PDDWF used for design occurred on Sunday, July 2, 1995 with 352 gpd/conn as shown in Table 3-4 and Figures 3-3 and 3-4.

Peak Hour Dry Weather Flow (PHDWF) – the maximum flow rate that occurs in a single hour during dry weather. For the same reasons as stated above for PDDWF, Peak Hour Wet Weather Flow was not determined. Hourly flow rates are not currently recorded, and therefore existing data was unavailable. In the GTA Capacity and Demand Report (2005), a peaking factor of 4.0 was applied to the ADMMF to obtain the Peak Hourly Flow. The same factor will be utilized for the purpose of this report.

Table 3-4 Summary Table of CSA-7A Wastewater Influent Flows averaged over each calendar month divided by the number of connections during that year

	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	Maximums	Minimums	Mean
January	53	71	61	91	70	80	69	74	63	60	47	69	91	47	67
February	58	66	77	62	128	90	80	66	63	63	51	52	128	51	71
March	60	117	83	74	95	94	93	82	62	59	47	76	117	47	79
April	75	132	89	70	112	118	106	93	73	64	66	131	132	64	94
May	83	124	112	90	128	123	102	96	94	81	76	160	160	76	106
June	84	135	98	88	133	126	114	95	114	82	79	105	135	79	104
July	96	162	121	111	159	141	127	113	120	105	91	108	162	91	121
August	80	130	111	122	141	135	111	108	100	96	81	106	141	80	110
September	66	117	79	86	96	98	89	73	79	66	61	80	117	61	83
October	57	87	67	64	77	87	70	57	80	56	50	57	87	50	67
November	63	75	64	77	83	81	66	63	66	54	48	50	83	48	66
December	70	65	78	67	74	67	62	59	63	46	45	45	78	45	62
Max Day (gpd)	214	359	276	242	260	310	254	229	251	245	226	334			
Min Day (gpd)	6	0	29	30	26	36	20	26	30	23	5	6			
Aprox # of Connections	449	458	464	474	488	494	512	524	550	568	575	583			

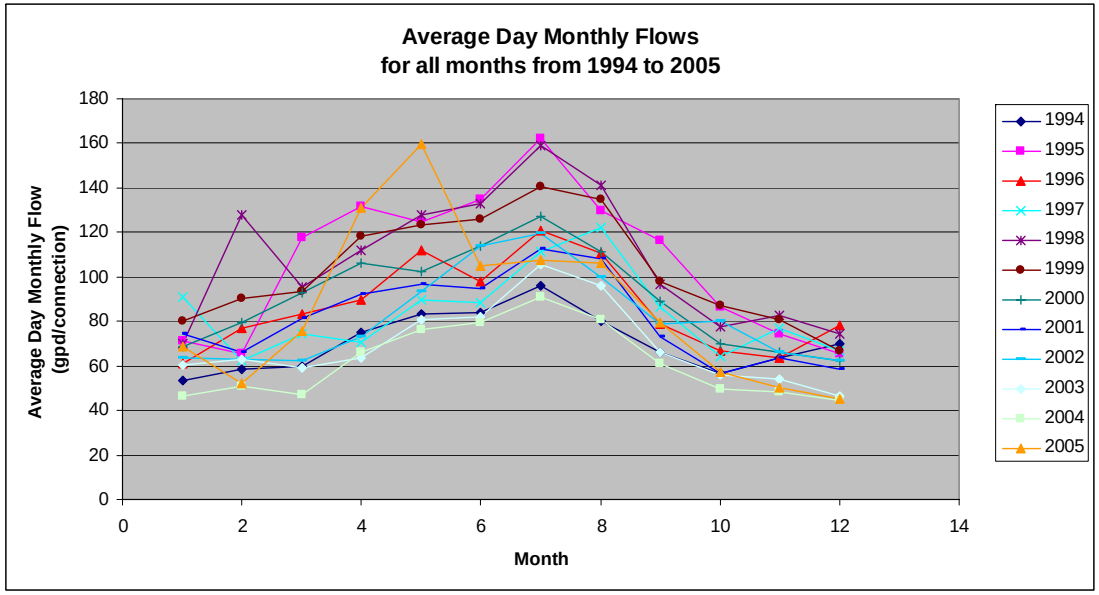
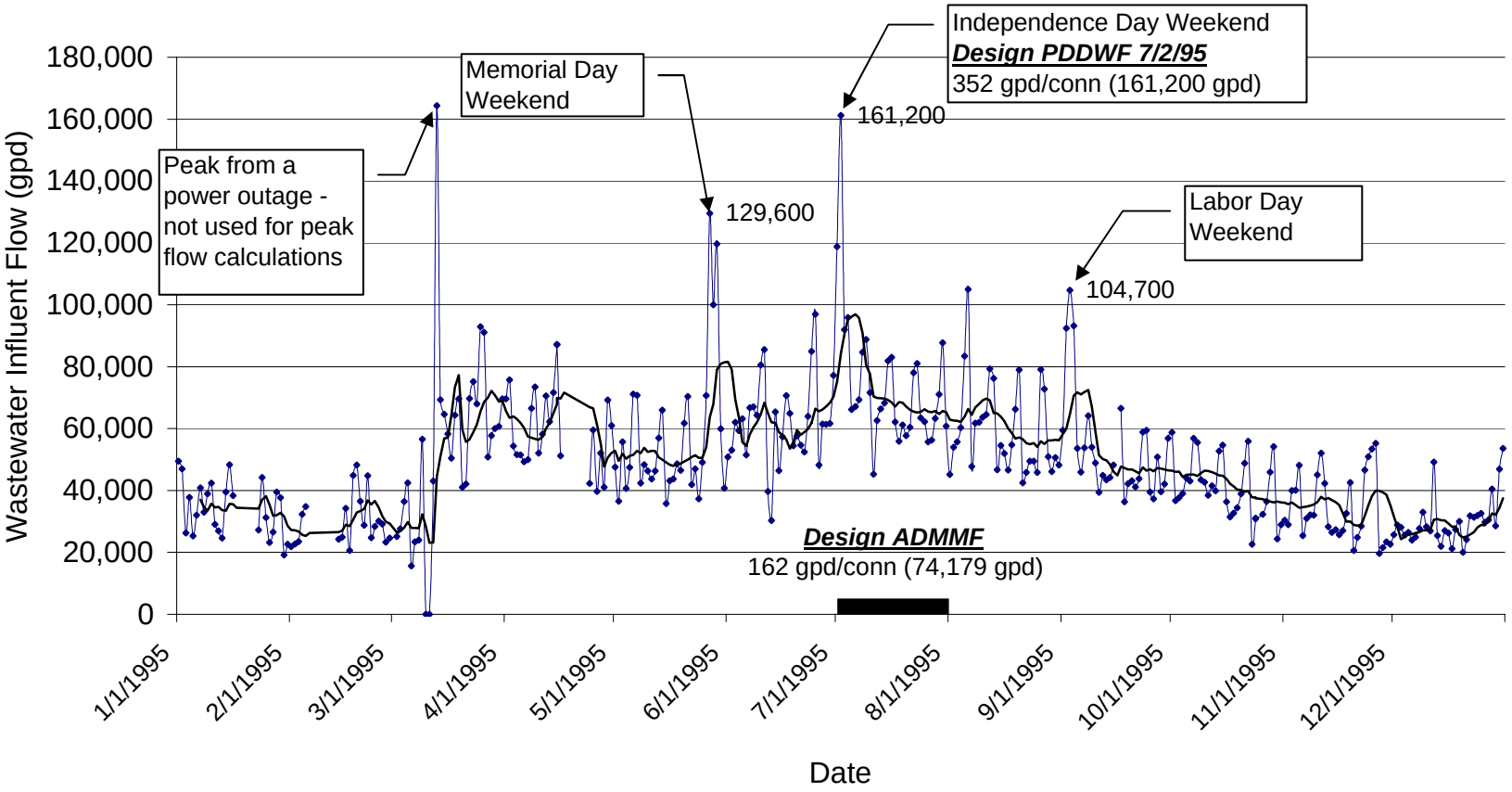


Figure 3-2 Summary Chart of CSA-7A Wastewater Influent Flows averaged over each calendar month divided by the number of connections during that year

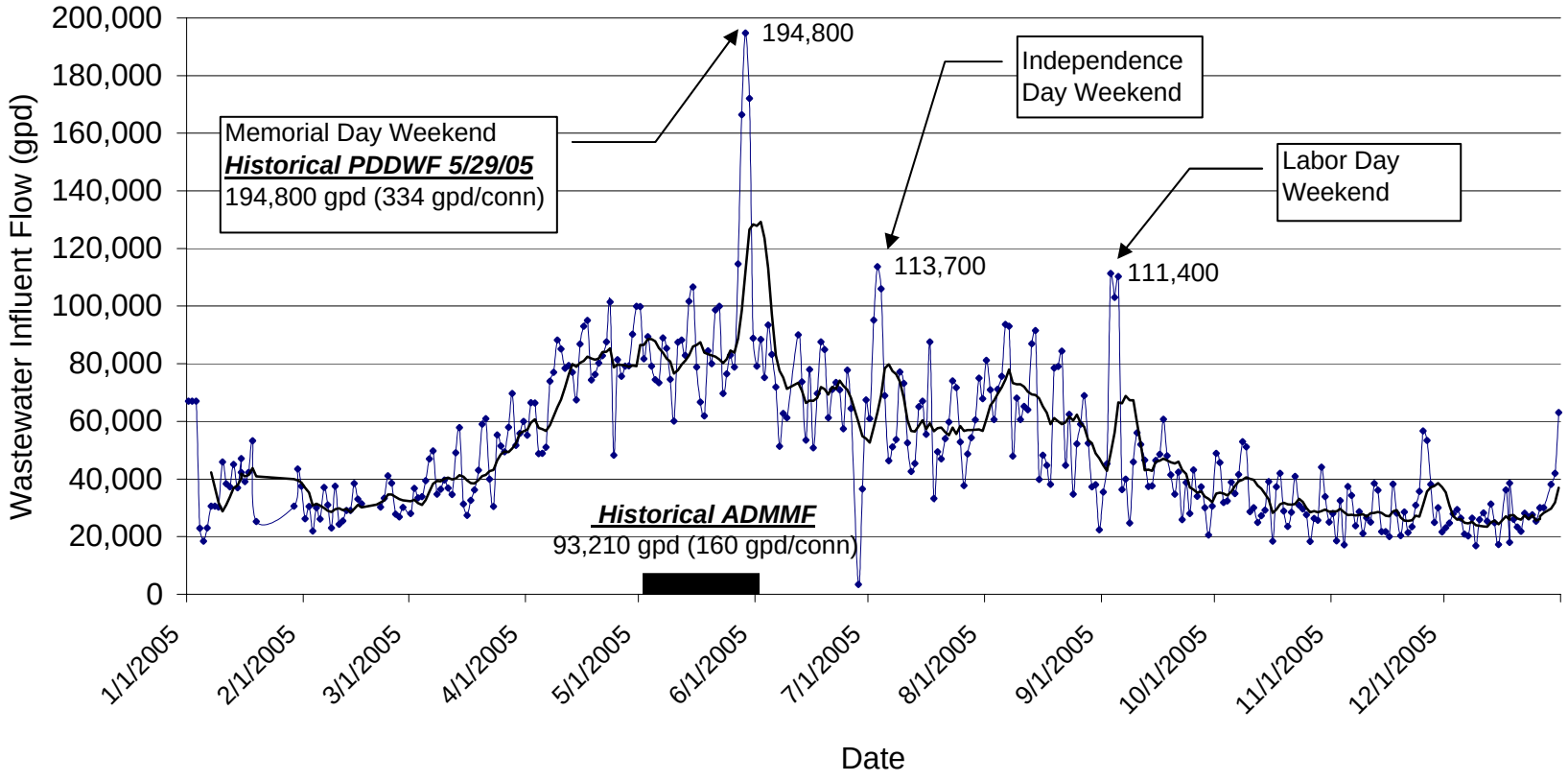
Figure 3-3
CSA-7A Wastewater Influent Flows for 1995[1]
with a 7 day moving average trendline



1 - from daily flow data provided by County of SLO

Figure 3-4
CSA-7A Wastewater Influent Flows for 2005[1]

with a 7 day moving average trendline



1 - from daily flow data provided by County of SLO

3.4 Maximum Monthly Flows

The Average Day Maximum Month Flow (ADMMF) is the maximum flow occurring in one calendar month divided by the number of days in that month. This report has adopted the methodology employed in the 2005 Capacity and Demand Report, which is summarized as follows:

- ADMMF is calculated based on actual monthly data as shown in Figures 3-3 and 3-4.
- ADMMF divided by the number of connections yields a flow per connection. A peaking factor was not used to obtain ADMMF in order to facilitate flow calculations as a function of full-time occupancy.
- The highest peak month on record occurred in 2005. However, the highest per unit flow for a single month occurred in July, 1995, with a flow per connection of 162 gpd. This ADMMF per unit is then used to predict future flows at full-time occupancy. The underlying assumption made by GTA and employed here is that the historical ADMMF occurred during a period of essentially 100% full-time occupancy.

Table 3-5 summarizes the anticipated ADMMF as a function of development.

Table 3-5 ADMMF for Proposed Oak Shores Developments

Development Entity	Number of Units	Cumulative Units	ADMMF (gpd)
Existing CSA-7A Dwellings*	583	583	93,210
Future CSA-7A Dwellings	269	852	138,024
Davis/King Tract 2162 Phase 1	60	912	147,744
Davis/King Tract 2162 Phase 2-6	285	1197	193,914
Bean/Lynch Canyon Properties (Hughes) Tract 2520	307	1504	243,648
Munari	134	1638	265,356
Vaughn/Taylor/ Crawford	25	1663	269,406
Lynch Canyon Properties (Hughes)	50	1713	277,506

* This value is based off of the actual highest recorded ADMMF in gpd and not by multiplying the design ADMMF of 162 gpd/conn by 583 as was done for the other values in the table

3.5 Seasonal Flow Variations

The calculation of seasonal flow variations is critical for the disposal analysis. Variations in full-time occupancy impact the total annual discharge of effluent, which impact both hydraulic and nitrogen loading calculations. Past studies have addressed the seasonal nature of Oak Shores using various methods. The methods employed by GTA allow for the calculation of flows as a function of full-time occupancy, and therefore they have been adopted for this analysis. The methodology is summarized as follows:

- Rational methods, including a review of Census and County assessor data, were used by GTA to estimate the number of seasonal and full-time units. The conclusion was that 26% of the units were permanently occupied at the time of the study. This estimate is consistent with full-time occupancy estimates given in the Oak Shores Phasing Plan which states "the present (1985) winter occupancy rate is between 20 and 25% and it is very doubtful that it will ever climb to 50%." (Oak Shores Village Phasing Plan, 1985).

- During the highest recorded average monthly flow, all units are assumed to be fully occupied (100% full-time occupancy), which yields a flow per connection of 162 gpd.

- It was assumed that wastewater flows from the full-time dwellings are constant throughout the year, and therefore any variation in the monthly flows is caused by the seasonal dwellings.

With the above assumptions, the following approach was used to estimate average monthly flows from seasonal units and the results are shown in Table 3-6. With the monthly flows from seasonal units estimated, it is possible to estimate the total flow for each calendar month at different rates of full time occupancy as follows:

$$MaximumFlowforMonth\left(\frac{\text{gal/day}}{\text{connection}}\right)=$$

$$FullTimeOccupancyRate\times\ FlowfromFulltimeUnits\ \left(\frac{\text{gal/day}}{\text{connection}}\right)+$$

$$\left(1-FullTimeOccupancyRate\ \right)\times FlowfromSeasonalUnits\left(\frac{\text{gal/day}}{\text{connection}}\right)$$

Example calculation for December

$$78\left(\frac{\text{gal/day}}{\text{connection}}\right)=0.26\times\ 162\ \left(\frac{\text{gal/day}}{\text{connection}}\right)+$$

$$\left(1-0.26\ \right)\times FlowfromSeasonalUnits\left(\frac{\text{gal/day}}{\text{connection}}\right)$$

Solving for $FlowfromSeasonalUnits$ gives:

$$FlowfromSeasonalUnits=48.9\left(\frac{\text{gal/day}}{\text{connection}}\right)$$

Table 3-6 Estimated Design Flow per connection from Seasonal Units		
Month	**Flow from Full-Time Units (gpd/conn)**	**Flow from Seasonal Units (gpd/conn)**
January	162	66.1
February	162	116.0
March	162	101.8
April	162	121.0
May	162	159.1
June	162	125.4
July	**162**	**162.0**
August	162	133.6
September	162	100.6
October	162	60.8
November	162	55.1
December	162	48.9

3.6 Full-Time Occupancy Rates and Associated Design Average Annual Flows

In order to provide for future flexibility, potential changes in full-time occupancy patterns have been analyzed. The GTA report cited research that indicated that the full-time occupancy is likely to increase from 26 % to 40% in the future. The water balance calculations described in Chapter 5 are conducted for occupancies of 26% and 40%, and general conclusions are indicated for occupancies of up to 100%. Full-time occupancy calculations do not impact the sizing of collection system and treatment plant components. Table 3-7 summarizes design average annual flows as a function of both development and full-time occupancy.

Table 3-7 Design Average Annual Flows as a Function of Full-Time Occupancy				
Development Entity	**Cum. Units**	**Design Average Annual Flow by Full-time occupancy Rate in ac-ft/yr and gpd**		
		26% (0.134 AF/u)	40% (0.143 AF/u)	100% (0.182 AF/u)
Existing CSA-7A Dwellings	583	78 ac-ft/yr (69,700 gpd)	83 ac-ft/yr (74,400 gpd)	106 ac-ft/yr (94,700 gpd)
Future CSA-7A Dwellings	852	114 ac-ft/yr (101,900 gpd)	122 ac-ft/yr (108,800 gpd)	155 ac-ft/yr (138,400 gpd)
Davis/King Tract 2162 Phase 1	912	122 ac-ft/yr (109,100 gpd)	130 ac-ft/yr (116,400 gpd)	166 ac-ft/yr (148,200 gpd)
Davis/King Tract 2162 Phase 2-6	1197	160 ac-ft/yr (143,200 gpd)	171 ac-ft/yr (152,800 gpd)	218 ac-ft/yr (194,500 gpd)
Bean/Lynch Canyon Properties (Hughes) Tract 2520	1504	202 ac-ft/yr (179,900 gpd)	215 ac-ft/yr (192,000 gpd)	274 ac-ft/yr (244,400 gpd)
Munari	1638	219 ac-ft/yr (195,900 gpd)	234 ac-ft/yr (209,100 gpd)	298 ac-ft/yr (266,100 gpd)
Vaughn/Taylor/ Crawford	1663	223 ac-ft/yr (198,900 gpd)	238 ac-ft/yr (212,300 gpd)	303 ac-ft/yr (270,200 gpd)
Lynch Canyon Properties (Hughes)	1713	230 ac-ft/yr (204,900 gpd)	245 ac-ft/yr (218,700 gpd)	311 ac-ft/yr (278,300 gpd)

3.7 Peak Day and Peak Hour Flow Rates

Previous capacity studies have used multiple methods for calculating Peak Day and Peak Hour Flows. The GTA method is utilized here due to its applicability to seasonal conditions. The GTA report determined that the PDDWF occurred on May 28, 1997 when 204,000 gallons was discharged from 474 connections giving a per connection rate of 430 gpd. A detailed review of the WWTP operator log revealed that the actual flow for May 28, 1997 was 20,400 gpd and it was simply entered into the spreadsheet incorrectly as 204,000 gpd. After correcting this value in the data, the new PDDWF occurred on July 2, 1995 during the Independence Day weekend when 161,200 gallons was discharged from 458 connections giving a per connection rate of 352 gpd as shown in Figures 3-3 and 3-4. This PDDWF of 352 gpd/unit was used to estimate the PDDWF for the proposed developments. The Peak Day that occurred on May 29, 2005 (194,800 gpd) does not govern due to the higher number of connections.

GTA utilized a peaking factor of 4.0 applied to ADMMF to determine the Peak Hour Flow (PHDWF). This report will use the same basis. This approach assumes a minimal amount of infiltration and inflow. The PHDWF is obtained by multiplying the ADMMF of 162 gpd/unit by 4.0 which gives 648 gpd/unit (0.45 gpm/unit). This calculation was made for each of the proposed developments for collection system design purposes and the results are shown in Table 3-8. ADMMF, PDDWF, and PHDWF are not impacted by changes in full-time occupancy given their derivation.

Table 3-8 PDDWF and PHDWF as a Function of Development

Development Entity	Number of Units	PDDWF (gpd) (352 gpd/conn)	PHDWF (gpm) (0.45 gpm/conn)
Existing CAS-7A Dwellings	583	205,216	262
Future CSA-7A Dwellings	269	94,688	121
Davis/King Tract 2162 Phase 1	60	21,120	27
Davis/King Tract 2162 Phase 2-6	285	100,320	128
Subtotal for Above Developments:	**1,197**	**421,344**	**538**
Bean/Lynch Canyon Properties (Hughes) Tract 2520	307	108,064	138
Munari	134	47,168	60
Vaughn/Taylor/Crawford	25	8,800	11
Lynch Canyon Properties (Hughes)	50	17,600	23
Total for All Developments	**1,713**	**602,976**	**771**

3.8 Wastewater Characteristics

The influent characteristics of the Wastewater Treatment Plant in terms of organic loading were analyzed and defined in the GTA Report. Influent values for 5-day BOD were generally less than 200 mg/l. Total influent nitrogen has not been historically tested. However, assuming a ratio of total nitrogen (as N) to BOD in the range of 6:1, the total influent nitrogen should be on the order of 30 to 40 mg/l. A design value of 40 mg/l (as N) would allow flexibility for future fluctuations in concentration. A more detailed influent characterization will be provided in the Final Report after a treatment alternative is selected.

4.1 Previous Studies

As described in Chapter 2 and Appendix D.3, the County previously studied the capacity of the existing collection system in its 2004 Interceptor Bypass Study. The study describes the risks and operational constraints associated with an existing trunk sewer that is located beneath the Nacimiento Lake high water level, known as the Interceptor. The study also identifies existing deficiencies at Lift Station No. 3, which is approximately 60 feet deep. All existing wastewater flows are conveyed by this lift station to the WWTP. If additional flows are routed to these facilities, the risk of operational problems and spills would generally increase.

4.2 Analysis Criteria

The capacity of the existing collection system to accommodate additional development is analyzed in this chapter. The improvements necessary to convey flows from Phases 2 through 6 of Tract 2162 are analyzed in detail, and general criteria for the connection of other developments are also provided. The following design criteria were used in the analysis:

- Additional connections to the existing Interceptor should be minimized. If connections are necessary, flows should be introduced as close to Lift Station No. 3 as practical.

- Peak hour flows should be conveyed with a ratio of flow depth to pipe diameter (d/D) of 0.50 or less. Given that peak hour flows are used, the collection system analysis is independent of the full-time occupancy rate.

- If flow is conveyed from a long force main or a series of pump stations, sulfide control measures should be required to prevent odor and corrosion. Such measures could include the installation chemical feed facilities to inhibit sulfide release (ferric chloride or equivalent), coating of exposed concrete, or where practical, alternative force main alignments.

- Force mains should be designed with velocities of more than 2 feet per second, but not more than 5 feet per second.

4.3 Collection System Alternatives for Tract 2162 - Phases 2 through 6

As shown in Figure 4-1, two alternatives have been identified for the connection of Phases 2 through 6 of Tract 2162. Both alternatives involve routing force mains directly to the headworks of a wastewater treatment plant. A connection to the existing gravity system was also contemplated. However, this alternative was rejected based on the following disadvantages:

- Significant improvements to the downstream gravity system would be required at a cost comparable with the recommended alternatives.

- Lift Station No. 3 would be impacted, which would require additional improvements and the associated developer contribution.

- Odor and corrosion control measures would be required.

The following alternatives were selected for additional analysis. These alternatives correspond to treatment plant options discussed in Chapter 6

Alternative A: Convey Flows Directly to the Existing WWTP

As shown in Figure 4-1, in this alternative a new force main would be constructed from the tract boundary at Pine Ridge Road, to Shoreline Road, and then to the treatment plant utilizing two pipe bridges as discussed in the 2004 Interceptor Bypass Study. The physical and hydraulic characteristics of the proposed force main are as follows:

- Peak Hour Flow: 128 gpm as indicated in Chapter 3. Upstream lift stations should be designed to convey the Peak Hour Flow or a self-cleaning velocity of 2 ft/sec. with the largest pump out of service.

- Diameter and material: 6” Class 200, PVC (C900) when buried and 6” Ductile Iron Pipe when exposed. A larger force main size was selected to provide flexibility for future flow increases. The minimum flow required to achieve a velocity of 2 ft/sec is 175 gpm.

- Approximately 4,000 linear feet will be buried, and approximately 200 linear feet will be installed on two new pipe bridges. The pipe bridges will be constructed with adequate space to add a second gravity pipe in the event the County implements West Side Improvement Project No. 11 in the future (see Appendix D.3).

- The force main will terminate at the existing (or new) headworks at the existing Wastewater Treatment Plant.

- The total project cost for Alternative A is estimated at $1,021,000 (see Appendix C).

Alternative B: Convey Flows to a new WWTP adjacent to the Existing Spray Field

In Alternative B, a new WWTP would be developed adjacent to the existing spray field site as described in Chapter 6. If this alternative is selected, a temporary connection to the existing gravity system at Pine Ridge Road may be appropriate while the new WWTP is under construction. Once the new WWTP is completed, all flows from Phases 2 through 6 of Tract 2162 would be conveyed in a new force main to the facility. The recommended physical and hydraulic characteristics of the force main are as follows:

- Peak Hour Flow as described in Alternative A

- Diameter and material: 6” Class 200, PVC (C900)

- The force main velocity is expected to be approximately 2 feet per second.

- The total length is estimated at 11,000 linear feet. Depending on the age of the pipeline, friction losses are expected to be between 30 and 40 feet of hydraulic head.

- The force main will terminate at the new headworks at a new Wastewater Treatment Plant.

- The total project cost for Alternative B is estimated at $2,102,000 (see Appendix C).

The construction of Alternative B would provide an opportunity for the County to install a parallel pipeline for the future conveyance of raw wastewater or secondary effluent. The recommended project alternative is discussed in Chapter 7.

4.4 Collection System Recommendations for West Side Connections

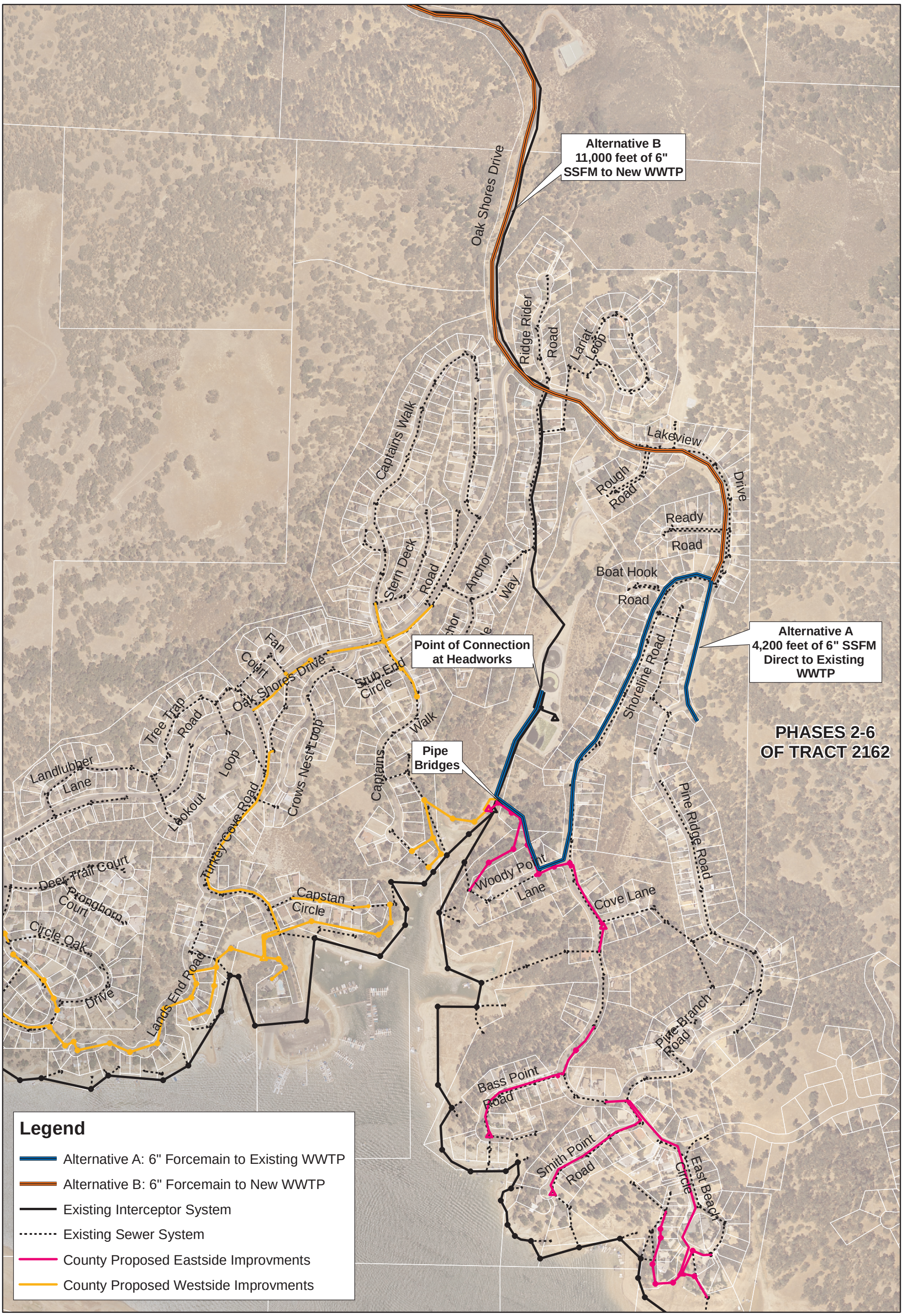
Assuming that future west side connections are routed to the existing WWTP (Alternative A), the primary requirement should be to avoid additional loading on the existing Interceptor. Figure 4-2 shows the recommended connection areas, and the area of downstream improvements that may be required. Recommended connection requirements are summarized as follows:

- If practical, connections should be made along Oak Shores Drive north of Fan Court, or along Captains Walk.

- Depending on the size of the development, downstream gravity improvements from the connection point to the existing WWTP are likely. The developer should contribute a pro-rata share to the completion of improvement at the intersection of Oak Shores Drive and Captains Walk, if such improvements are necessary to provide adequate capacity (see Interceptor Bypass Study, Westside Improvement No. 13).

- Depending on the size of the development and the proposed connection point, implementation of Westside Improvement No. 15 should be considered. If implemented, the new gravity sewer should be routed to Lift Station No. 3 in a manner consistent with the potential future replacement of Lift Station No. 3.

- Sulfide control measures should be implemented for developments that include long force mains or multiple lift stations in series.



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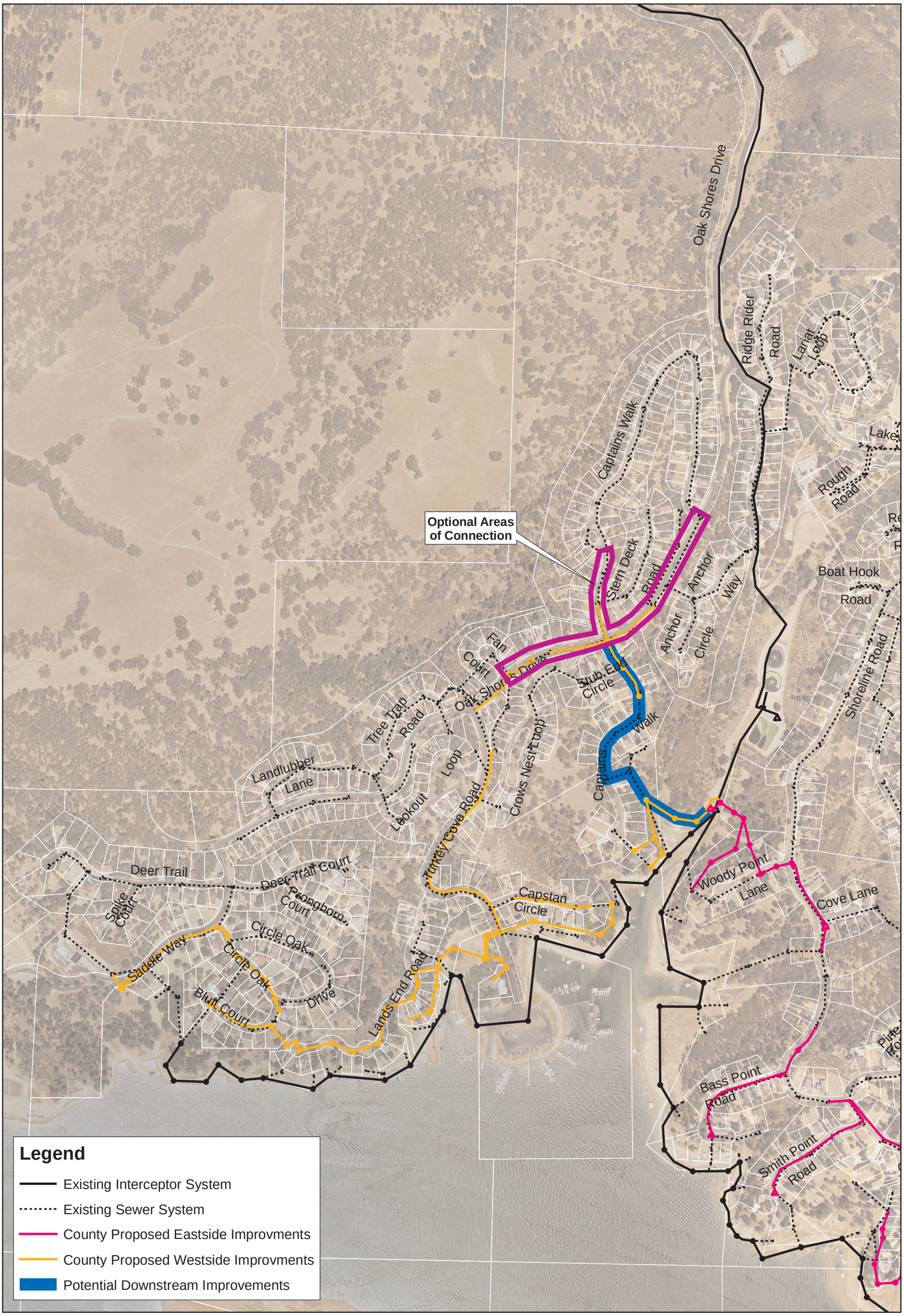
CSA-7A Wastewater System Capacity Study Interim Report

Figure 4-1 Tract 2162 Proposed Connection Alternatives

1 inch equals 500 feet

0 250 500 1,000 Feet

N W E S



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CSA-7A Wastewater System Capacity Study Interim Report

Figure 4-2 West Side Collection System Connections

1 inch equals 500 feet

0 250 500 1,000 Feet

N W E S

Figure 4-1 Tract 2162 Proposed Connection Alternatives

T

Figure 4-2 West Side Collection System Connections

5.1 Previous Studies

Effluent disposal options have been the subject of several studies by consultants and County staff. An electronic version of each major study is included in Appendix D. Recent studies that are pertinent to the current analysis are as follows:

- Various reports by Law and Crandall (1996, 1997, and 1999) and the Public Works Department (2001) recommended further development of the Kavanaugh effluent disposal site. However, these studies were generally limited to the surface characteristics of the site, and did not sufficiently address groundwater mounding and downstream water quality impacts.

- The Capacity and Demand Report prepared by Garing Taylor & Associates in 2005 highlighted the current need for disposal capacity at Kavanaugh. The report concludes that the capacity of the existing spray field and storage pond is limited to approximately 5 ac-ft of evapotranspiration, 10 ac-ft of evaporation from pond surfaces, and 41 ac-ft of deep percolation (pg. 23). In a very wet year, a portion of this capacity is needed for the disposal of 16 ac-ft of storm water runoff, leaving approximately 40 ac-ft available annually for disposal.

This current effort to assess disposal capacity included a more detailed analysis of downstream water quality and mounding impacts as described below. In addition, future regulatory constraints have a significant bearing on disposal and as a consequence, treatment. For this reason, the disposal portion of the study is covered prior to treatment (Chapter 6).

5.2 Regulatory Impacts on Wastewater System Alternatives

Revisions to WDR 01-130 will be required to provide for full development of the projects contemplated in the MOU. Several meetings were conducted with Regional Board staff to receive feedback on future requirements and permit issues. The development of additional spray field sites, if available, is expected to be straightforward if adequate wet weather storage is provided. However, Regional Board staff indicated that downstream groundwater quality impacts relating to the Kavanaugh site should be addressed in the initial application through appropriate testing and modeling. This analysis was completed as described in Section 5.4.

5.3 Permitted Capacity for Future Development

WDR 01-130 states in the **discharge** specifications that “Daily flow averaged over each month shall not exceed 100,000 gallons” which suggests that the current permitted capacity of the CSA-7A WWTP is dictated by the amount of effluent discharged in any one month. If summer-period equalization facilities are provided for the treated effluent, the amount of flow discharged to the disposal system during the summer months can be

equalized with the off-peak months. For the purposes of compliance with WDR 01-130, measuring the equalized effluent flow, which represents the actual discharge volume, should be a viable reporting method – a method that provides the County with increased flexibility. In fact, due to the seasonality of the flows from the Oak Shores community, the influent flows during the summer months are, even now, approaching the permitted capacity of 100,000 gpd. Table 3-3 shows that in May 2005 the average monthly flow was 93,210 gpd - only 6,790 gpd less than the permitted capacity of 100,000 gpd. The proposed "flattening" of the effluent discharge allows for a substantial increase in available permitted capacity. If summer period equalization is employed, the amount of additional permitted capacity is sufficient to allow the development of:

- Tract 2162 phases 2-5 (1,137 units) with 10 acre-ft of effluent storage if the 12-year AAF is used to predict actual future flows
- Tract 2162 phases 2-6 (1,197 units) with 7.9 acre-ft of effluent storage if the last 5 years of AAF are used to predict future flows,
- 923 units with 13.3 acre-ft of effluent storage if the maximum AAF recorded is used to predict future flows.

Table 5-1 illustrates the amount of effluent storage that would be needed to accommodate the above three scenarios while still remaining under the 100,000 gpd discharge limit. Figure 5-1 below is a graphical display of the storage required to keep the monthly discharged under 100,000 gpd if the 5 year AAF is used to predict actual future flows.

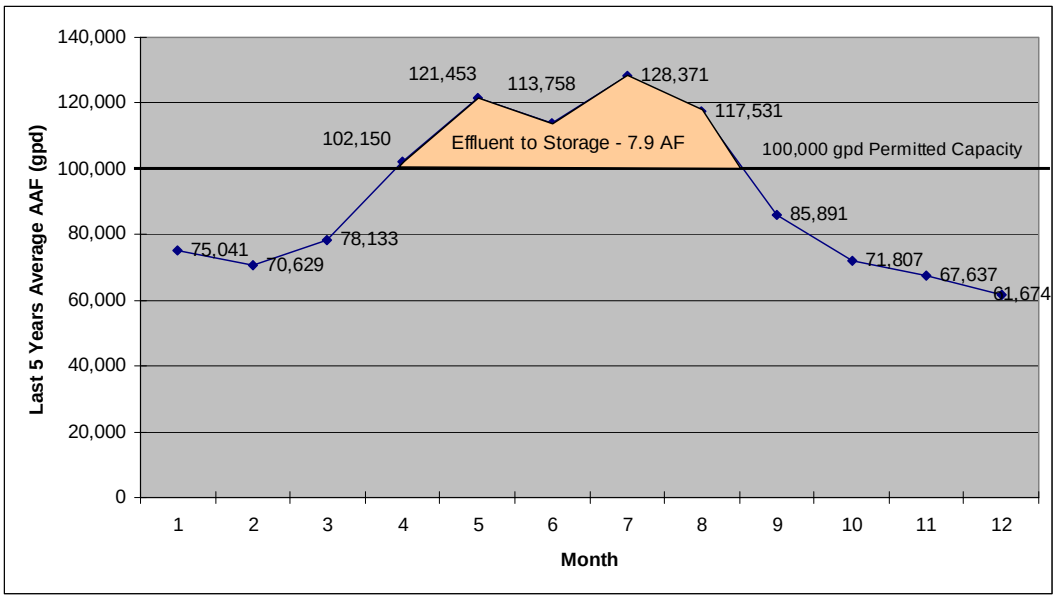

Figure 5-1 AAF for Build-Out of CSA-7A Through Tract 2162 using 5 Year AAF to predict Future Flows

Table 5-2 below shows the years in which the permitted capacity would be reached assuming that future development rates mirror the historical rate of 12.2 units per year. The permitted capacity would be reached in:

- 2031 if the 12 year average AAF was used to predict future flows
- Never if the 5 year average AAF was used to predict future flows
- 2019 if the maximum AAF was used to predict future flows

Table 5-1 Storage for Flow Equalization Required to Stay Within Current Permitted Capacity for Future Tract 2162 Flows Based on Historical Average Annual Flows

Table 5-2 Future Actual AAF from Proposed Developments Over Time

	Number of Units			Average Annual Flow (gpd)		
Year	Central Village	Tract 2162 Phase 1-6	Total	12-Year Ave (85 gpd/conn)	Last 5-Year Ave (75 gpd/conn)	Maximum (108 gpd/conn)
1994	449		449	38,165	33,675	48,492
1995	458		458	38,930	34,350	49,464
1996	464		464	39,440	34,800	50,112
1997	474		474	40,290	35,550	51,192
1998	488		488	41,480	36,600	52,704
1999	494		494	41,990	37,050	53,352
2000	512		512	43,520	38,400	55,296
2001	524		524	44,540	39,300	56,592
2002	550		550	46,750	41,250	59,400
2003	568		568	48,280	42,600	61,344
2004	575		575	48,875	43,125	62,100
2005	583		583	49,555	43,725	62,964
2006	595	12	607	51,612	45,540	65,578
2007	607	24	631	53,669	47,355	68,191
2008	620	36	656	55,726	49,170	70,805
2009	632	48	680	57,783	50,985	73,418
2010	644	60	704	59,840	52,800	76,032
2011	656	72	728	61,897	54,615	78,646
2012	668	84	752	63,954	56,430	81,259
2013	681	96	777	66,011	58,245	83,873
2014	693	108	801	68,068	60,060	86,486
2015	705	120	825	70,125	61,875	89,100
2016	717	132	849	72,182	63,690	91,714
2017	729	144	873	74,239	65,505	94,327
2018	742	156	898	76,296	67,320	96,941
2019	754	168	922	78,353	69,135	99,554
2020	766	180	946	80,410	70,950	102,168
2021	778	192	970	82,467	72,765	104,782
2022	790	204	994	84,524	74,580	107,395
2023	803	216	1,019	86,581	76,395	110,009
2024	815	228	1,043	88,638	78,210	112,622
2025	827	240	1,067	90,695	80,025	115,236
2026	839	252	1,091	92,752	81,840	117,850
2027	851	264	1,115	94,809	83,655	120,463
2028	853	276	1,129	95,965	84,675	121,932
2029	853	288	1,141	96,985	85,575	123,228
2030	853	300	1,153	98,005	86,475	124,524
2031	853	312	1,165	99,025	87,375	125,820
2032	853	324	1,177	100,045	88,275	127,116
2033	853	336	1,189	101,065	89,175	128,412
2034	853	345	1,198	101,830	89,850	129,384
2035	853	345	1,198	101,830	89,850	129,384

5.4 Kavanaugh Percolation Pond Hydraulic Analysis

Cleath and Associates was retained to prepare a hydrogeologic investigation of the Kavanaugh site. A copy of the draft report, which was issued in February 2006, is included as Appendix A. The Scope of the study included field and lab testing, model development, and water quality analysis. The following key conclusions were reached in the hydraulic portion of the report:

- The disposal capacity of the site is limited by downstream daylighting to an average flow of 145,000 gallons per day (gpd) or 162 ac-ft/yr. Additional flow can be discharged over shorter intervals. For example, at 165,000 gpd daylighting would not be expected for three years.

- The disposal capacity is not limited by effluent mounding or surface percolation. The recommended pond configuration is shown in Figure 5-2, which includes a five pond system. Three ponds would be in service, one pond would be out of service for drying, and one pond would be dry and ready for maintenance.

- The disposal capacity is available during wet years when Lake Nacimiento is at its maximum level.

- The estimated travel time from the disposal ponds to the subsurface flow of Kavanaugh Creek is estimated at 6 months. The effluent would not daylight in Kavanaugh Creek, but would remain in the dry wash underlying the creek bed.

The hydraulic flexibility of the Kavanaugh site is an important advantage from a water balance perspective. During wet weather months, effluent in excess of 145,000 gpd can be discharged to minimize storage requirements. Full-time occupancy patterns within CSA-7A impact the amount of flow discharged annually from each connection as described in Chapter 3. In keeping with the County's 2005 study, disposal capacity was analyzed for full-time occupancy rates of 26%, 40%, and 100%.

A cost estimate for developing a five-pond percolation system at the Kavanaugh site is summarized in Appendix C. The total project cost is estimated at $1,825,000.

5.5 Kavanaugh Percolation Pond Water Quality Analysis

As requested by the Regional Water Quality Control Board, the groundwater quality impacts associated with the Kavanaugh site were investigated by Cleath (Appendix A). The scope of the study included a review of beneficial uses and water quality thresholds, review of historical data, field/lab testing, and water quality modeling. The key results of the study are as follows:

- The mineral quality of the existing groundwater underlying the Kavanaugh Creek area is poorer than the existing wastewater effluent. Total dissolved solids (TDS) loading is therefore not expected to constrain the use of the disposal site.

- The nitrate levels underlying the site are generally low, and beneficial downstream uses will result in a down-gradient groundwater limit of 10 mg/l (as

N). Regional Board staff has advised the project team that an appropriate compliance sampling point would be approximately 100 ft downstream of the discharge.

- Multiple sources of naturally occurring groundwater will mix with the percolated effluent as it travels to the downstream point of monitoring. The flow of naturally occurring groundwater was estimated by Cleath at 236,900 gpd (265 ac-ft/year). This mixing water contains a negligible level of nitrates.

- Given the ratio of mixing water to percolated effluent (1.6 to 1), low levels of nitrates must be maintained in the effluent at the design flow to protect beneficial uses.

As indicated in Chapter 3, the influent BOD at Oak Shores varies considerably, but has averaged less than 200 mg/l. The average total nitrogen concentration is estimated at 40 mg/l (as N). The existing data indicate that some biological nitrogen reduction is occurring in the existing pond system. However, as flows increase, effluent nitrogen concentrations are expected to approach levels typical for a partially-mixed pond system. For the purposes of a conservative analysis, the future total nitrogen concentration in the effluent is estimated at 40 mg/l. Biological nitrogen reduction in the soil column is neglected given the saturated conditions and the lack of data. Assuming that the existing spray field does not have a nitrogen limitation, and that existing spray field capacity is utilized first, the number of units that can be supported utilizing Kavanaugh can be calculated with an approximate mass balance as follows:

- The available mixing water totals 236,900 gpd with negligible nitrogen. The total effluent applied over one year depends on the full-time occupancy level. The average annual flow per unit is estimated at 120, 128, and 162 gpd for full-time occupancy levels of 26%, 40%, and 100%, respectively (see Chapter 3).

- Approximately 40 ac-ft should be removed from the annual effluent volume to account for spray field disposal in a wet year. The remaining effluent would be applied to Kavanaugh at a conservative concentration of 40 mg/l.

- The effluent volume that yields a blended concentration of 10 mg/l would establish the allowable number of units without nitrogen reduction in the treatment system.

The results of the mass balance are summarized in Table 5-3 as follows:

Table 5-3 Approximate No. of Allowable Units Without Nitrogen Removal

Full-time occupancy Condition	Annual Average Flow per Unit (gpd)	Allowable Units Based on Nitrogen Limitation
26% (Existing)	120	955
40% (Future)	128	896
100% (Worst Case)	162	708

The following conclusions can be drawn from this analysis:

- The existing service area can be built to 896 units if occupancy increases to 40% or 706 if the occupancy increases to 100% (worst case) before Nitrogen removal would be required in the future.

- Additional development beyond 912 units will require nitrogen removal in the wastewater treatment system.

- Given that the existing 912 properties may require nitrogen removal at some point in the future, depending on future occupancy, capital improvements that provide such a reduction now may result in reduced capital expenditures in the future. This economic benefit may be partially offset by an increase in operational costs as discussed in Chapter 6.

5.6 Future Spray Field Site on Lynch Canyon Properties (Hughes) Property

As shown in Figure 5-3, one developing property that may participate in the MOU contains approximately 10 acres of land that is suitable for spray disposal. Vehicle access to the property is planned via a bridge across Kavanaugh Creek, which can be designed to accommodate an effluent disposal pipeline. Given that the percolation characteristics of the property are unknown, a conservative water balance has been prepared assuming all required storage would be provided on site in a 1 acre footprint. The required storage amount is estimated at 10 ac-ft. Additional soils testing may demonstrate that more deep percolation of effluent can be considered. Based on the current analysis, approximately 57 acre-ft can be discharged to the proposed new spray field. As indicated in Section 5.7, assuming that a full-time occupancy rate of 40% represents a reasonable future design parameter, the Lynch Canyon Properties (Hughes) property is not necessary for service to Phases 2 through 6 of Tract 2162.

5.7 Water Balance Scenarios

In order to confirm storage requirements and account for percolating rainfall, water balance calculations have been performed for the proposed disposal system at build-out. The methodology, which is similar to that employed by Garing and Taylor, is explained in additional detail in Appendix B. The proposed Lynch Canyon Properties (Hughes) spray field site was calculated separately and maximized in terms of the number of potential units at a future full-time occupancy of 40%. Table 5-4 summarizes the annual capacity of the various disposal sites. Note that the annual storm water volume captured during a wet year (16 ac-ft) is subtracted from the capacity of the existing spray field / storage ponds.

Table 5-4 Disposal Area Hydraulic Capacity		
Disposal Area	**Annual Capacity (ac-ft)**	**Aver. Annual Flow (gpd)**
Existing spray field (incl. deep percolation)	40	35,700
Kavanaugh perc. ponds	162	145,000
Lynch Canyon Properties (Hughes) spray field	57	50,900

Table 5-5 provides a summary of the number of units that can be accommodated by available disposal areas as a function of full-time occupancy. This table was generated using the design flow rate of 162 gpd/conn for a full-time unit. It should be noted that these tables assume the availability of adequate storage. This assumption is valid given that the Kavanaugh effluent disposal site can accommodate more than 145,000 gpd during short periods of time (less than a year) without daylighting. Detailed water balance calculations are provided in Appendix B for specific scenarios of interest.

Table 5-5 Design Development Potential as a Function of Disposal Capacity				
Disposal Areas	**Capacity (ac-ft/yr)**	**Allowable Number of Housing Units by Average Full-Time Occupancy Rate**		
		26% (0.134 AF/u)	40% (0.143 AF/u)	100% (0.182 AF/u)
Existing spray field	40	298	279	219
Existing spray field + Kavanaugh	202	1,507	1,412	1,109
Existing spray field + Kavanaugh + Lynch Canyon Properties (Hughes)	259	1,932	1,811	1,423

Table 5-6 is included for comparison purposes and provides a summary of the number of units that could be accommodated by available disposal areas as function of various historical AAF averages for a full-time occupancy rate of 26%. The difference between the number of units allowed for a full-time occupancy rate of 26% based on the design flow rate of 162 gpd/conn from Table 5-5 and the units allowed for the same occupancy rate based on actual average annual flow rates which vary from 75 gpd/conn to 108 gpd/conn in Table 5-6 is significant. For purposes of comparison, if actual average annual flow rates are assumed based on historical flow data, the existing spray field, Kavanaugh site, and Lynch Canyon Properties (Hughes) spray field would be adequate to provide service to 2,140 units through 3,083 units depending on which average annual flow value was used as shown in Table 5-6. It is important to note that in discussions regarding future development and existing capacity, design flow rates used as a basis of comparison have been segregated from actual flow rates. Actual flow rates are generally substantially lower.

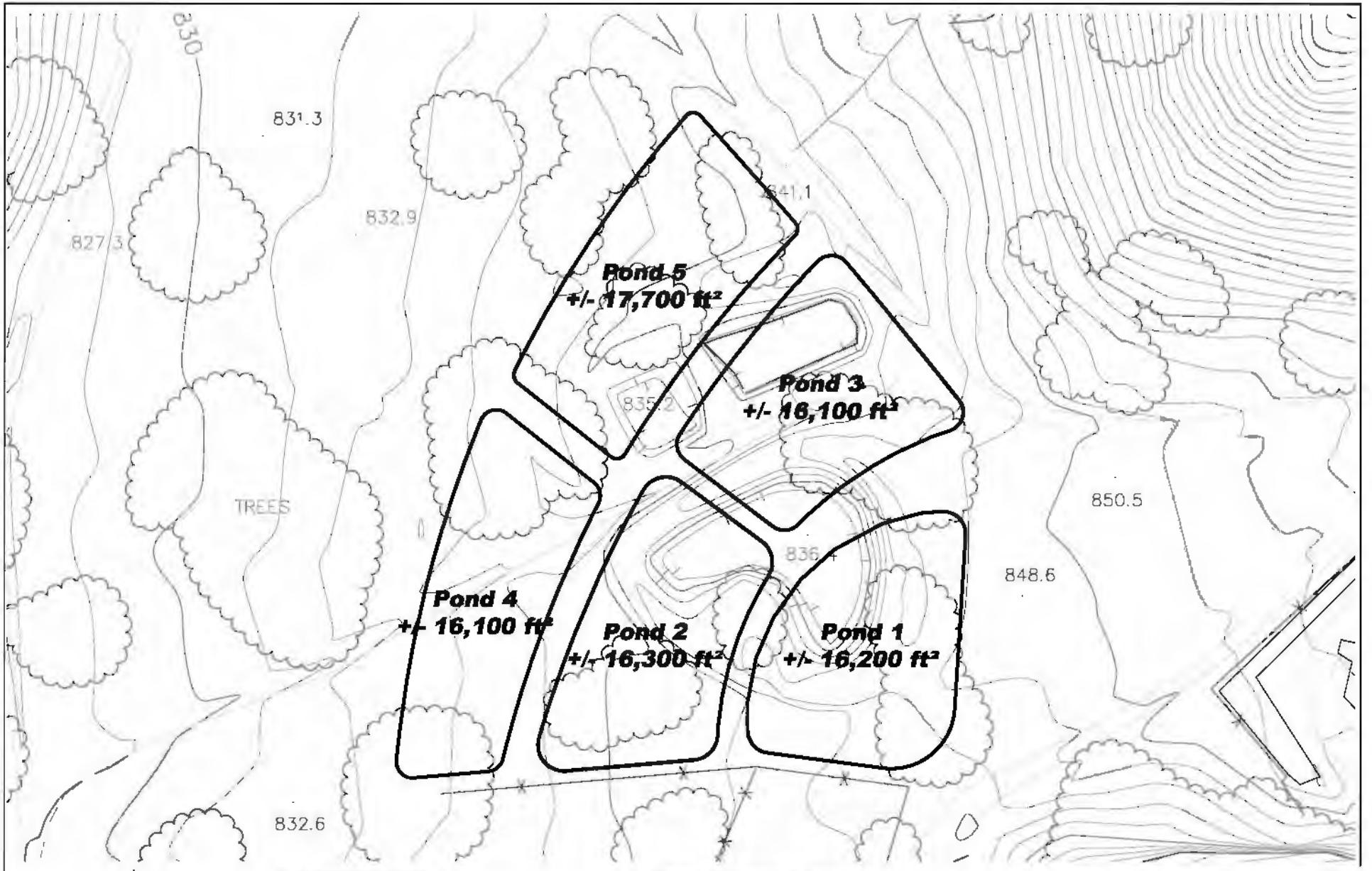

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Figure 5-2 Kavanaugh Effluent Disposal Site Preliminary Design

JOB No. :	298.009
DRAWING :	Oak Shores Ponds
DRAWN BY:	BGM
DATE :	03/14/06
SCALE :	NTS

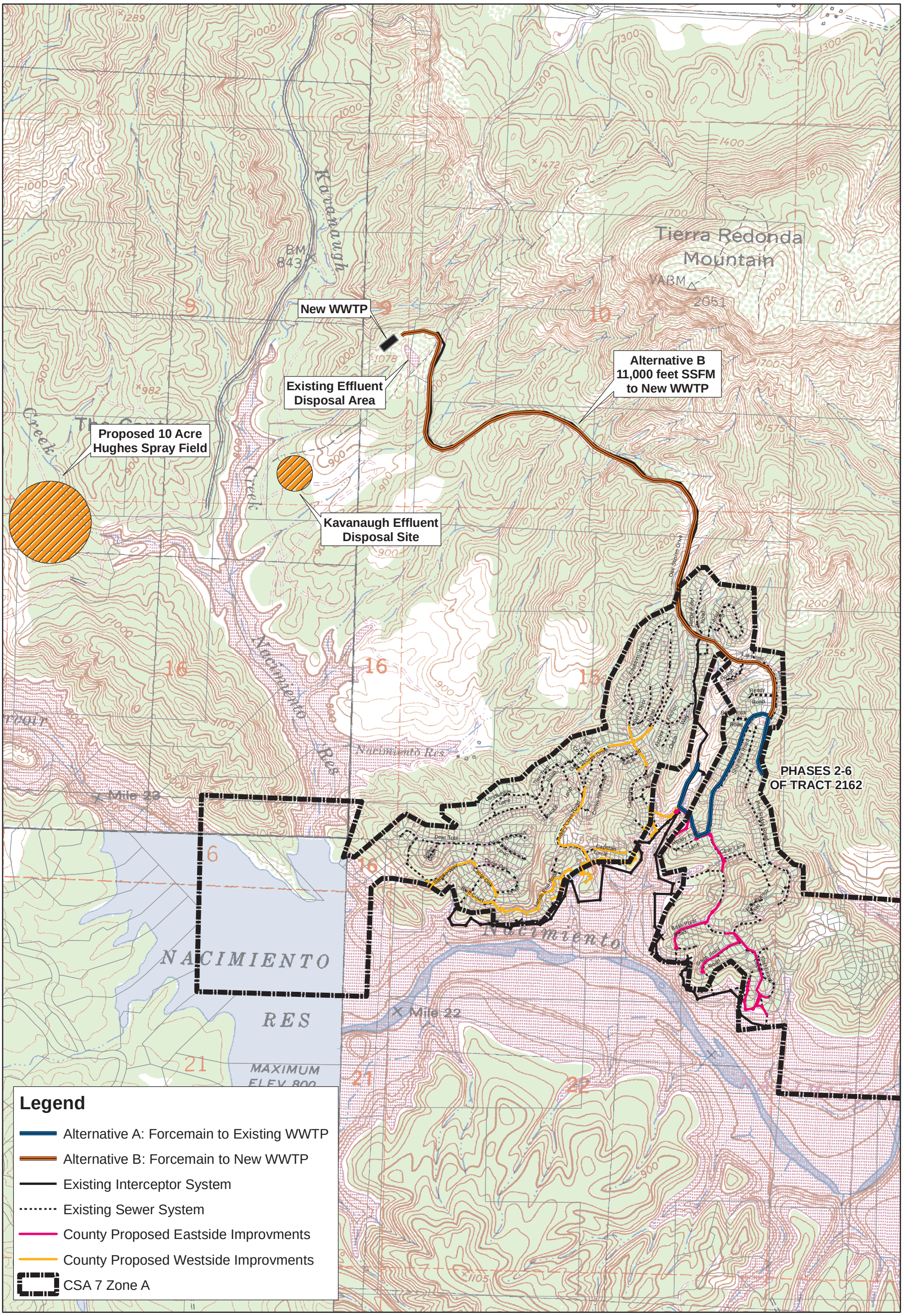

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Figure 5-3 Disposal Site Overview

1 inch equals 1,200 feet

0 600 1,200 2,400 Feet

N W E S

Table 5-6 Actual Development Potential as a Function of Disposal Capacity if Flows are Equalized Throughout the Year				
Disposal Areas	**Capacity (ac-ft/yr)**	**Allowable Number of Housing Units by Actual AAF**		
		12-Yr Ave AAF (0.095 AF/u)	Past 5 Yr Ave AAF (0.084 AF/u)	Max AAF (0.121 AF/u)
Existing spray field	40	421	476	330
Existing spray field + Kavanaugh	202	2,126	2,404	1,669
Existing spray field + Kavanaugh + Lynch Canyon Properties (Hughes)	259	2,726	3,083	2,140

The following conclusions can be reached from Table 5-5 and the water balance calculations for the existing spray field plus the Kavanaugh site (not including Lynch Canyon Properties):

- At the current full-time occupancy of 26%, the existing spray field and the Kavanaugh site are adequate to provide service to the proposed 1,197 units plus an additional 310 units. As stated above, nitrogen removal at the treatment plant will be required.

- Assuming a future full-time occupancy of 40%, the existing spray field and the Kavanaugh site are adequate to provide service to the existing 912 permitted lots and Phases 2 through 6 of Tract 2162, or a total of 1,197 lots. These sites can accommodate a full-time occupancy rate of just over 50%. As stated above, nitrogen removal at the treatment plant will be required.

- At 100% full-time occupancy, disposal capacity would be adequate for only 1109 units - 88 units less than the proposed 1197 lots. This number of units (88) equates to 16 ac-ft of effluent. This additional amount can be accommodated with minor additional percolation area or less than 2 acres of spray field. Alternative future disposal areas are discussed below. At the current rates of development (12 lots/year), it would be more than 20 years before build-out of the existing service area would be achieved. Given these factors, the proposed disposal arrangement is sufficiently conservative.

After the Lynch Canyon Properties (Hughes) spray field is added, additional conclusions can be reached as follows:

- Build out of CSA-7A can be achieved with a full-time occupancy rate of between 40% and 50%.

- At 100% full-time occupancy, approximately 10 acres of additional spray field area would be required. Assuming that the community grows at a maximum rate of 2.3%, it would take over 40 years to reach 1,423 units. As a result, the proposed approach is sufficiently conservative.

5.8 Future Disposal System Modifications

The Kavanaugh site provides suitable conditions for long term effluent disposal. However, the proposed site is not ideal from the perspective of future development interests due to its proximity to potential home sites. Alternative percolation pond areas that are more suitable to the existing land owner are available to the north of the Kavanaugh site. In addition, interest has been expressed on behalf of several developers to pursue alternative effluent water application methods such as subsurface drip irrigation and subsurface percolation at Kavanaugh. These methods can be developed, pilot-tested, and proved over time. As a result, the use of open percolation ponds at the Kavanaugh site may eventually be replaced by alternative but equivalent disposal methods. In order to preserve the agronomic value of treated effluent, the use of self regenerating water softeners should be prohibited in both the existing permitted areas and the proposed new subdivisions.

Figure 5-2 Kavanaugh Effluent Disposal Site Preliminary Design

Figure 5-3 Disposal Site Overview

6.1 Previous Studies

The existing wastewater treatment plant (WWTP) was analyzed by Wallace Group in 2004 and again by GTA in 2005. Both studies focused on the capabilities of the existing plant to stabilize organic material given varying influent conditions and water temperatures. In the County's study, GTA concluded that the existing WWTP was inadequate under certain conditions. The purpose of this chapter is to analyze treatment options that will work effectively within the disposal constraints set forth in Chapter 5. Specifically, the total nitrogen in the WWTP should not exceed 10 mg/l as an annual average. Given that nitrogen disposal capacity exists for the currently permitted 912 lots (at 40% full-time occupancy), alternatives have been developed that provide the County with flexibility to address potential rate payer concerns.

6.2 Analysis Criteria

Wastewater flow and loading are set forth in Chapter 3. General design criteria used for the development of alternatives include the following:

- Technologies that minimize operations and maintenance costs and requirements should be pursued. Effluent quality should meet requirements, but higher quality effluent should not be pursued at the expense of higher O&M costs.

- Technologies should have a proven track record for total nitrogen removal to a concentration of less than 10 mg/l (as N).

- Phasing constraints should be considered, including the need to maintain service at the existing WWTP.

- In general, treatment processes should be designed biologically for the Peak Daily Flow (PDDWF) and hydraulically for the Peak Hour Flow unless adequate equalization volume exists.

- Technologies that can provide the required nitrogen removal function at a minimum capital cost should be favored, but not at the expense of efficient operations.

6.3 Wastewater Treatment Plant Alternatives

Two alternatives were developed for analysis. In Alternative A, the existing WWTP would be upgraded to provide nitrogen removal. Some participation from the existing rate payers is anticipated with this option. When completed, all rate payers would pay the same O&M charges. In Alternative B, all new development would connect to a new

Existing ponds to remain in service during Biolac construction. Convert to aerated sludge storage.

Approximate location of new sludge drying beds.

Effluent pump station to be bypassed during construction.

New Biolac Process
Phase 1: 425,000 gpd
Phase 2: 185,000 gpd
Total Peak Day
Flow: 610,000 gpd

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Figure 6-1 Alternative A- Upgrade Existing WWTP

1 inch equals 100 feet

0 50 100 200 Feet

N W E S

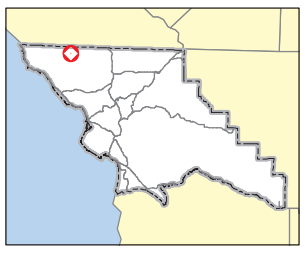

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Figure 6-2 Alternative B New WWTP for Development

1 inch equals 100 feet

0 50 100 200 Feet

N W E S

WWTP adjacent to the existing spray field. A separate rate structure would be developed, and the existing rate payers would fund future upgrades to the existing WWTP without further contributions from development.

Alternative A: Upgrade Existing WWTP

The existing WWTP is described in detail in Appendix D.1, but essentially consists of the following elements for biological treatment:

- Two aerated ponds, each with a volume of approximately 400,000 gallons and one 10 hp surface aerator in each pond.
- One maturation / final settling pond with a volume of approximately 1.6 million gallons.

The existing system is inadequate for the removal of nitrogen without a substantial change in the process design. Technologies that were considered include the following:

- Oxidation ditch
- Sequencing batch reactors (SBR)
- Biolac® (Parkson, Inc.) with Wave Oxidation for nitrogen removal

The Biolac® system was the only technology that met the project criteria of low O&M effort and reasonable capital cost. The system is described in additional detail in Appendix D.8, and includes the following elements:

- The Biolac® system utilizes the extended aeration process with an accompanying long sludge age and stable operations.
- Lined rectangular ponds are utilized for aeration chambers, with a hydraulic residence time of approximately 30 hours.
- Efficient aeration is provided with fine bubble diffusers that can be accessed from the surface of the pond for maintenance. A proprietary mixing approach is also employed which reduces the required total horsepower. Positive displacement blowers provide the required air volume.
- A final clarifier in constructed integrally with the aeration pond with a poured-in-place concrete hopper and vertical end wall. Sludge recycle is accomplished through an air lift pump using the same air source as the diffusers.
- The Wave Oxidation process automatically cycles air valves to create periodic anoxic conditions in alternating zones. The system has a successful track record for total nitrogen removal to levels below 10 mg/l in California. BOD and SS are expected to be less than 30 mg/l.
- In order to avoid the fouling of fine bubble diffusers, influent screening is recommended. As a result, the existing headworks would be replaced with an automated screening device that also provides for washing, compaction, and bagging of screenings.

After receiving further direction from the County on the desired treatment alternative, a Preliminary Design Report (PDR) will be prepared setting forth the sizing and configuration of various unit processes. For the purpose of preliminary cost estimating, the following configuration was used:

Table 6-1 Preliminary Configuration for Treatment Alternative A (Upgrade Existing)	
Process	**Preliminary Sizing and Description**
Headworks	Automated 3 mm screening / washing system such as Helisieve (see www.parkson.com) or approved equal. The headworks would be sized for the build out peak hour flow and would include a bar screen bypass.
Biolac® process	Two parallel units, the first would be sized for a PDDWF of **425,000 gpd** with a minimum operating volume of 530,000 gallons (**Phase 1**). The second would be added in the future with a capacity of **185,000 gpd** and a minimum volume of 230,000 gallons (**Phase 2**). Freeboard would be provided to accommodate daily equalization in the basins. The basins would be constructed at the location of the existing 1.6 MG pond (see Figure 6-1). The existing 400,000 gallon aeration basins would remain in service throughout construction.
Energy requirements	Blower size estimated at 20 to 30 hp for Phase 1, which compares favorably to existing lagoon system which utilizes 20 hp. Phase 2 total load estimated at 30 to 40 hp.
Clarifiers	Integral to basin with concrete hoppers and vertical end walls. Sludge return would be provided with an air lift pump
Solids production	Existing ponds to be converted to aerated sludge holding. Estimated annual sludge volume in Phase 1, assuming 40% full-time occupancy, yield of 0.7, and BOD removal of 190 mg/l is **31 dry tons** per year. Drying beds to be considered to minimize O&M cost.
Effluent Pump Station	New station sized for PDDWF of 425,000 gpd in Phase 1 and 605,000 gpd in Phase 2. Pressure increase on existing effluent force main is estimated at approximately 1 psi for Phase 1 and 2 psi for Phase 2. The new station would be constructed prior to taking the existing station out of service.

The proposed locations of the upgrades to the existing WWTP are shown in Figure 6-1. The operations and maintenance costs for the above-referenced upgraded facility are expected to be similar on a per-lot basis when compared with the existing WWTP. The following preliminary information is provided to support this conclusion:

- Other similar Biolac® facilities are being operated with part time staffing similar to the County operation. Daily sludge wasting is not required.
- The upgraded WWTP would require a licensed Grade II Wastewater Treatment Plant Operator. The County's current lead operator possesses such a license.

- The electrical requirements would be equivalent to the existing WWTP, if not more efficient.
- If drying beds are implemented in conjunction with long term aerobic sludge holding, sludge disposal can be accomplished at a reasonable cost. Assuming a solids content in the bed-dried material of 30%, approximately 103 wet tons will be hauled off each year, with an annual cost of approximately $8,000 / year.

An estimate for the capital cost of Alternative A is provided in Appendix C. The estimated total project cost for Phase 1 is $6,006,000.

Alternative B: Construct Separate WWTP for Development Only

The operations costs for an upgraded WWTP appear similar to the existing WWTP as indicated above. However, an alternative has been assembled that would allow the County to ensure a separate rate structure and approach for future development. In Alternative B, a separate treatment plant would be constructed adjacent to the existing spray field as shown in Figure 6-2. This approach would require a new force main from Phases 2 through 6 of Tract 2162 to the new location – a length of approximately 11,000 ft (see Chapter 4). The effluent from the new plant would flow by gravity to the existing spray field storage ponds. The disadvantage of this alternative is that the County would now have two plants to operated, maintain, adjust, and sample. New rate payers in the proposed developments would bear the cost of this inefficiency. The new WWTP is described as follows:

Table 6-2 Preliminary Configuration for Treatment Alternative B (New WWTP)

Process	Preliminary Sizing and Description
Headworks	Automated 3 mm screening / washing system such as Helisieve (see www.parkson.com) or approved equal. The headworks would be sized for the build out peak hour flow and would include a bar screen bypass.
Biolac® process	Two parallel units, the first would be sized to treat Phases 2 through 6 of Tract 2162 with a PDDWF of **100,000 gpd** and a minimum operating volume of 125,000 gallons (**Phase 1**). The second would be added in the future with a capacity of **185,000 gpd** and a minimum volume of 230,000 gallons (**Phase 2**).
Energy requirements	Blower size estimated at 10 to 15 hp for Phase 1, and 20 to 30 hp for Phase 2.
Clarifiers	Integral to basin with concrete hoppers and vertical end walls. Sludge return would be provided with an air lift pump
Solids production	A third pond for aerobic sludge holding would be provided. Estimated annual sludge volume after Phase 2, assuming 40% full-time occupancy, yield of 0.7, and BOD removal of 190 mg/l is **15 dry tons** per year. Drying beds will be considered to minimize O&M cost.
Effluent Pump Station	Not required – gravity flow to existing spray field ponds.

A cost estimate for Alternative B is provided in Appendix C. The total project for Phase 1 is estimated at $2,703,000.

6.4 Existing Effluent Pump Station and Force Main

The existing effluent pump station and force main have been the subject of analysis in previous studies by Wallace Group and the County. For the purpose of this report, only Alternative A would have a potential to impact the existing effluent force main. The effluent pump station requires no further analysis since it would be replaced in Alternative A. The County has indicated in previous reports that the existing effluent force main is “at capacity” in terms of its pressure rating. However, after accounting for the Peak Day Flow for Tract 2162, a pressure increase of only 1 psi is expected in the existing force main. Variable frequency drives or soft starts can also be installed on the new effluent pumps to minimize any potential for surge. Given the minimal pressure increase, no improvements to the effluent force main are recommended at this time. A rating curve for the effluent force main as a function of flow is provided in Appendix E.

Figure 6-1 Alternative A - Upgrade Existing WWTP

Figure 6-2 Alternative B New WWTP for Development

7.1 Recommended Project Alternative

After the County reviews the proposed alternatives, a final project will be selected for implementation. Conservative estimates for the purpose of bonding have been provided to allow for short term approvals as described in Chapter 2. Cost estimates will be refined after an alternative is selected.

Alternative A is recommended for implementation for the following reasons:

- The County's primary objection to Alternative A is the potential for higher operations and maintenance costs, and the associated impact on existing rate payers. However, the proposed technology was selected to minimize this impact. Significant cost increases are not anticipated in terms of labor, electrical power, or sludge handling. In addition, the economy of scale associated with a single centralized facility is likely to result in the lowest overall cost to all CSA-7A rate payers.

- Given that the existing service area will not require nitrogen removal at a future full-time occupancy of 40%, a different rate structure for future development could still be justified. A detailed rate analysis can be performed after final design is completed, and additional costs associated with nitrogen removal can be born by the new lots.

- The existing WWTP structures will require replacement in the future. If Alternative A is implemented, much of the plant will be completely refurbished. This approach will benefit existing rate payers by deferring replacement-related expenses. The proposed collection system approach will result in two new pipe bridges that can be used by the County in the future to bypass the Interceptor.

The elements and cost of the recommended project are summarized as follows:

Table 7-1 Preliminary Cost Estimate for Recommended Project (Alternative A)

Component	Estimated Total Project Cost
Force Main to convey Tract 2162 flow to the Existing WWTP	$1,021,000
Upgrade of existing WWTP to Biolac® system (Phase 1)	$6,006,000
New percolations ponds at the Kavanaugh Site	$1,825,000
Total Cost	**$8,852,000**

Equitable cost sharing formulas will be developed in coordination with the County after review of this Interim Report is complete.

7.2 Project Schedule

The following schedule is anticipated for the recommended project through approval of bonding estimates for Tract 2162. A detailed implementation schedule through construction will be submitted with the Preliminary Design Report.

Table 7-2 Project Schedule and Milestones (Alternative A)

Project Milestone	Estimated Completion Dates
Selection of a final alternative by County	April 5, 2006
Approval of Phase 2 improvement plans for Tract 2162 based on this Interim Report and selection of an alternative.	April 14, 2006
Submission of Preliminary Design Report (PDR) for selected alternative with bonding estimates	May 31, 2006
Approval of PDR and County approval of bonding estimates for the purpose of map recordation	June 23, 2006

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Nolte and Associates, Oak Shores Treatment and Disposal Facilities Evaluation. January 1988

Nolte and Associates, Public Services and Facilities Element of the Nacimiento/San Antonio Planning Area General Plan. January 1973

Wallace Group, CSA 7A Wastewater Collection, Treatment, and Disposal System Capacity Assessment for Service to Phase I, Tract 2162. Revised October 2004

Appendix A: Cleath & Associates Analysis of Kavanaugh Percolation Pond Site

Appendix B: Water Balance Calculations

Appendix C: Construction Cost Estimates

Appendix D: Electronic Information (on compact disk)

Contents of compact disk:
D.1 – Capacity and Demand Report, CSA-7A, 2005, Garing Taylor & Associates
D.2 - Preliminary Plan and Cost Estimate, CSA-7A, 2005, Garing Taylor & Associates
D.3 - CSA-7A Interceptor Bypass Study, 2004, SLO County Public Works (unofficial)
D.4 – Oak Shores Disposal Area Expansion Project, 2001, SLO County Public Works
D.5 – Oak Shores Effluent Disposal Study…, 1989, Boyle Engineering Corp.
D.6 - Waste Discharge Requirements 01-130
D.7 – CSA-7A Budget for 05/06
D.8 - Manufacturer's data on Biolac®
D.9 - Draft Memorandum of Understanding
D.10 – SLO County CSA-7A Wastewater Flow Records

Appendix E: Hydraulic Calculations

Appendix A

Cleath & Associates Analysis of Kavanaugh Percolation Pond Site

HYDROGEOLOGIC INVESTIGATION
of the
KAVANAUGH EFFLUENT DISPOSAL SITE
OAK SHORES, LAKE NACIMIENTO

prepared for the

Wallace Group

DRAFT

February 2006

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Appendix A - Field Logs of Soil Borings
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INTRODUCTION

The Kavanaugh effluent disposal site is an emergency effluent disposal area for the Oak Shores community. There are three existing ponds and two ground water monitoring wells at the site. During wet weather, flows from the primary disposal area (spray field) pond are diverted to the Kavanaugh ponds. Wastewater facilities at Oak Shores are operated by County Service Area 7A, with effluent discharges to Kavanaugh subject to Central Coast Regional Water Quality Control Board revised waste discharge requirements order 01-130.

Expansion of the Kavanaugh disposal site has been discussed for many years, and several site evaluations have been performed (Boyle, 1989; Law/Crandall, 1996, 1997, 1999; SLO County Public Works, 2001; Wallace Group, 2004). Data collected during prior site investigations have included water quality, water levels, subsurface lithology, and infiltration tests. The location of the Kavanaugh effluent disposal area is shown in Figure 1.

Cleath & Associate was retained to characterize and quantify the hydrogeologic constraints on expanding effluent disposal operations at the Kavanaugh site. The investigation focused on ground water mounding, potential daylighting of effluent along Kavanaugh Creek, and impacts to water quality. This report summarizes the work performed and the results of the analyses.

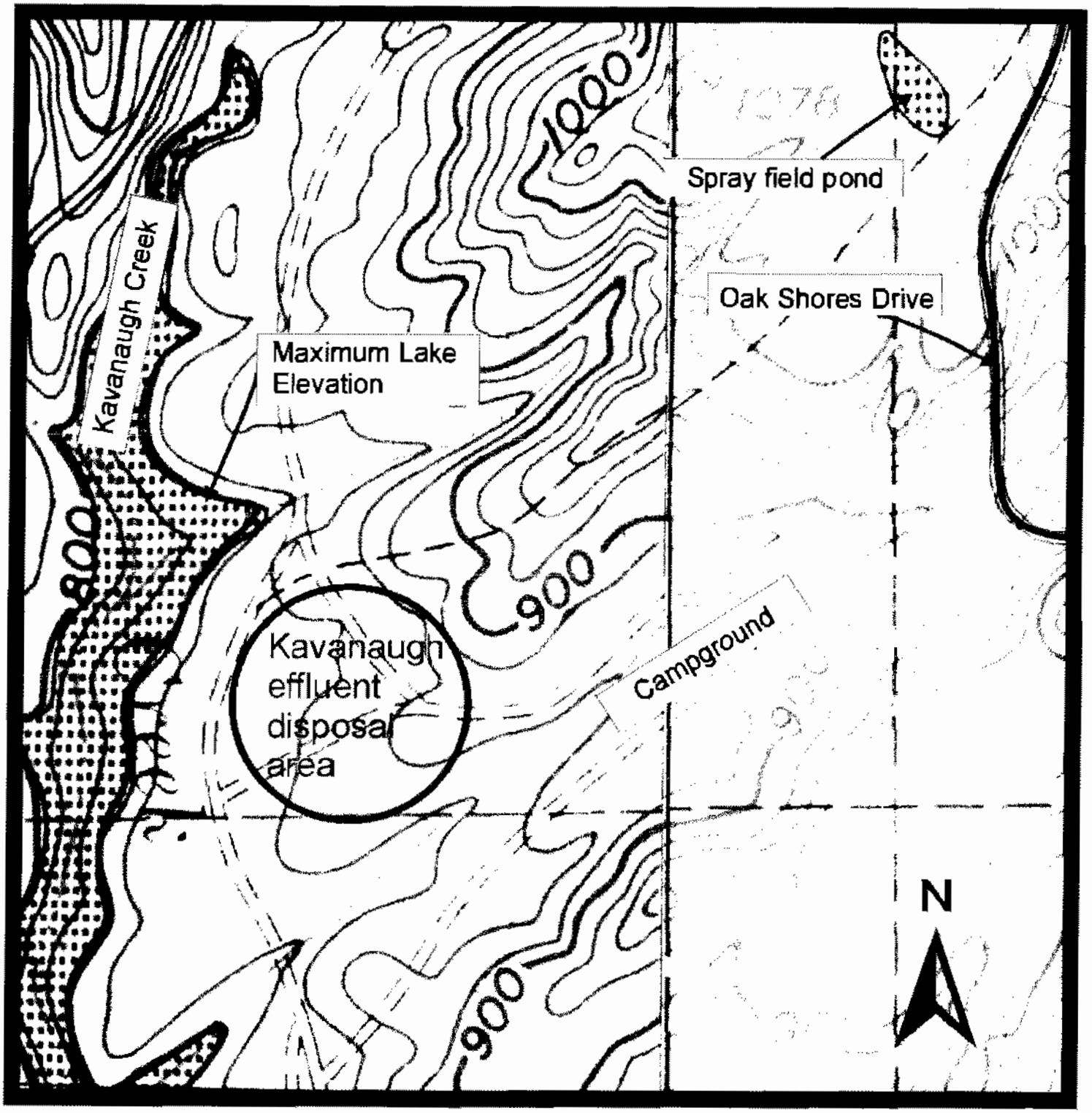
CONDUCT OF WORK

The investigation is divided into two studies, a disposal capacity study and a salt loading study. The following field activities were performed for the capacity study:

- Four test holes were drilled and logged to depths of up to 70 feet using hollow stem auger.
- Water levels were measured in test holes and on-site wells.
- Undisturbed soil samples were collected in brass sleeves for laboratory permeability (K) and sieve analyses.
- A bail down test and a constant discharge test were conducted at the two existing monitoring wells.

Results of the field investigation, along with information from prior work, were used for the following capacity study tasks:

- Ground water levels and pump test data interpretation.
- Geological interpretation of subsurface.



Base Map: USGS Topographic maps:
Tierra Redonda Mountain (photorevised 1979)
Bryson (photorevised 1979)

Map Scale: 1 inch = 400 feet

Figure 1
Location Map
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

- Permeability data interpretation.

- Construct steady-state and transient ground water flow models. The ground water flow models (MODFLOW) were used to evaluate site disposal capacity, including 100-year storm and maximum lake level conditions.

The following tasks were performed for the salt loading evaluation:

- The quality of local water resources was characterized, including ground water, surface water, and treated effluent using both new and existing data.

- Five water samples were collected for general mineral analyses. Two samples were taken from test holes. Two samples were taken from the existing monitoring wells on-site, and one sample was taken from a hand-auger hole in the dry wash of Kavanaugh Creek.

- Thresholds of significance for impacts to local water quality were determined using Regional Water Quality Control Board WDR Order No. 01-130.

- Mass loading of total dissolved solids and selected mineral salts, including nitrate, were evaluated for the project.

- Potential changes in water quality attributable to future disposal operations were compared to the thresholds of significance.

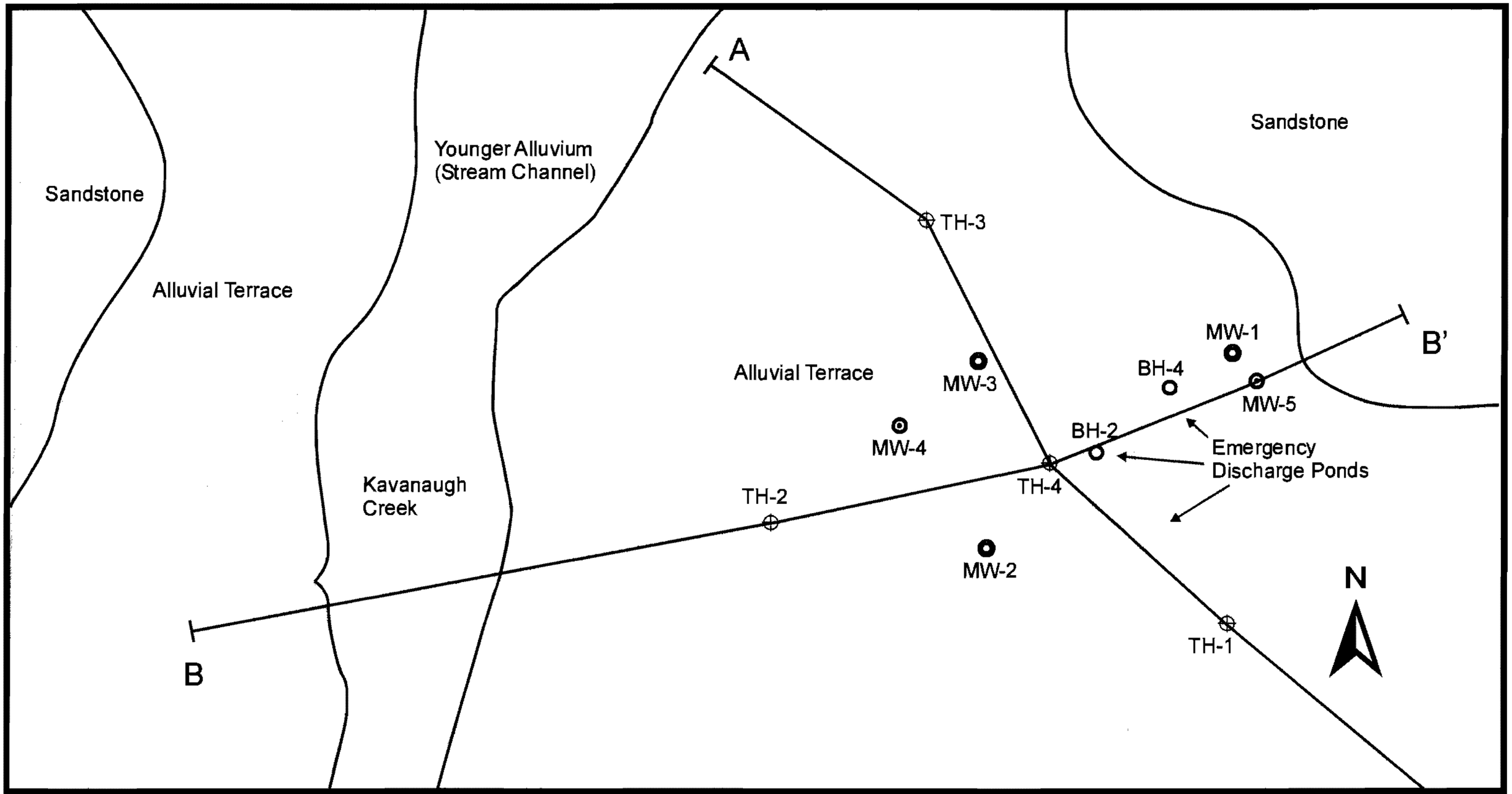
- Ground water travel times between the disposal ponds and Kavanaugh Creek were estimated.

HYDROGEOLOGIC SETTING

The Kavanaugh site encompasses approximately seven acres on an alluvial terrace above Kavanaugh Creek, with surface elevations generally between 830 and 840 feet above sea level. Dry wash elevations along Kavanaugh Creek range from 790 to 794 feet above sea level adjacent to the site. The sediments beneath the site are mostly unconsolidated silty sands and sands, with one prominent clay lense, which are underlain by Cretaceous-age shale and sandstone.

The unconsolidated deposits extend from ground surface to approximately 770 feet above sea level (up to 70 feet thick), where a nearly-horizontal bench has been eroded on the bedrock surface in the central and western portion of the site. Bedrock elevations rise abruptly to the northeast, and underlie the surrounding hills. Site geology and geological cross-sections are shown in Figures 2 and 3.

Ground water levels beneath the site were measured on November 22, 2005. Table 1 below summarizes the water level data.



Scale: 1 inch = 100 feet

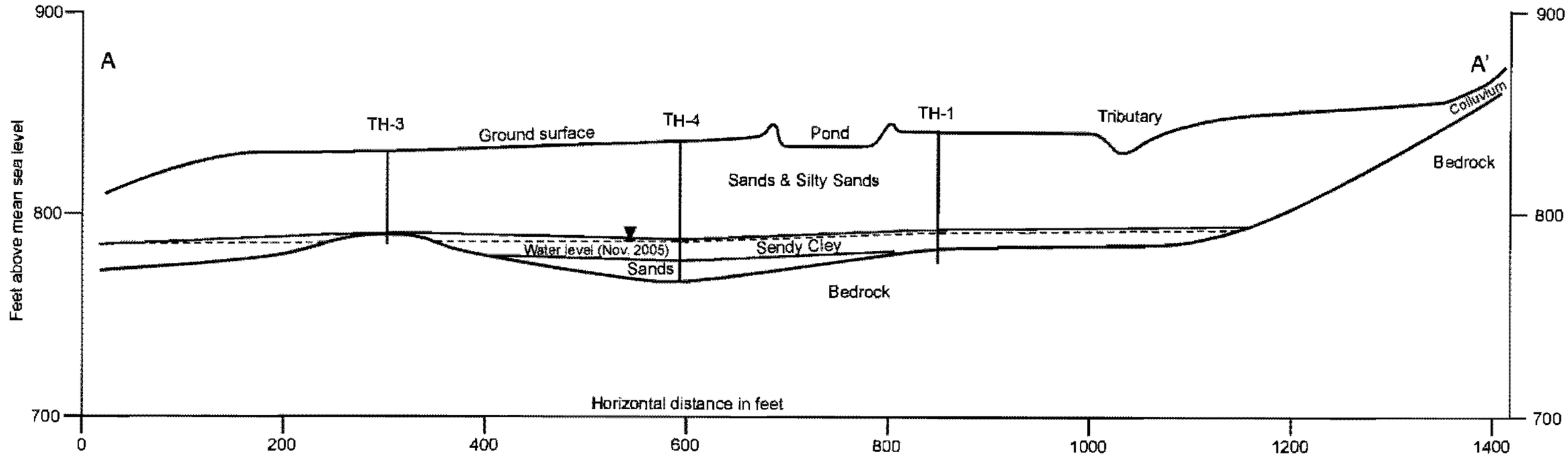
0 100

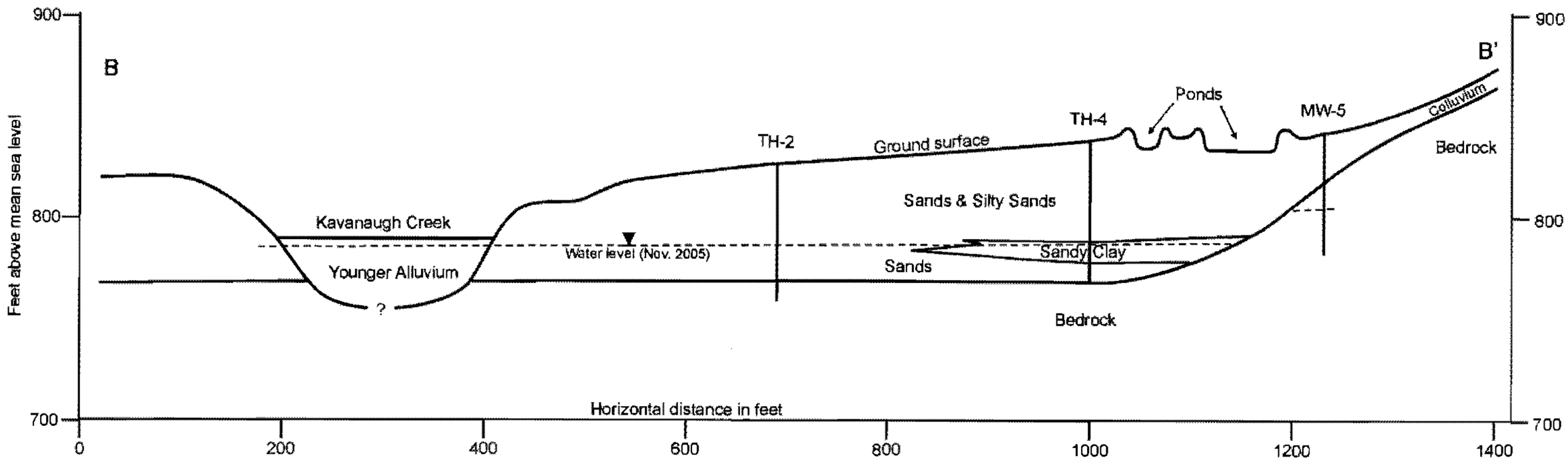
Legend:

- Law/Crandall Soil boring
- Law/Crandall Monitoring Well (destroyed)
- Law/Crandall Monitoring Well (existing)
- Cleath & Associates Soil Boring

Figure 2
Site Plan
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates





Cross-section locations shown on Figure 1

Figure 3
Geological Cross-sections
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

Table 1
Ground Water Elevations
November 22, 2005

Location	Depth to Water (ft)	Ground Surface Elevation (ft above sea level)	Ground Water Elevation (ft above sea level)
TH-1	54	843	789
TH-2	40	827	787
TH-3	(not present)	831	(not present)
TH-4	50	837	787
MW-4	44.3	830.26	785.96
MW-5	37.9	843.98	806.08
Kavanaugh Ck.	4	791	787

The piezometric surface is relatively flat beneath the central and western portions of the site (approximately 786 to 787 feet elevation). To the east, a rise in water elevation at TH-1 suggests subsurface inflow from the adjacent tributary drainage.

MW-5 is completed in the Cretaceous sandstone and has a much higher ground water elevation than the other locations. Water levels at MW-5 probably represent perched water flowing along the colluvium/bedrock interface.

Historical water level data at the site is available for December 1996, January 1997, and May 1999 through December 2000. The available historical data indicates water levels in the unconsolidated deposits ranged from a low of 780 above sea level (September 2000) to a high of 792 feet above sea level (February 2000). Water levels are likely higher than 792 feet elevation, however, when Lake Nacimiento rises into Kavanaugh Creek.

Kavanaugh Creek adjacent to the site becomes part of Lake Nacimiento when surface water elevations at the lake rise above the dry wash elevation. During the periods when historical ground water levels are available, lake levels reached a maximum of 791 feet (May 1999), which would be close to the invert of Kavanaugh Creek adjacent to the site. Lake levels, however, have reached the spill point of 800 feet elevation a few times in the last 10 years (1995, 1998, 2005), and evidence of encroachment of the lake into the Kavanaugh Creek valley is readily apparent from high water level marks along the banks adjacent to the site.

FIELD INVESTIGATION

On November 21 and 22, 2005, Cleath & Associates supervised the drilling of four test holes at the Kavanaugh site by S&G Drilling Company of Lompoc. The test holes were advanced using 8-inch diameter hollow-stem auger to depths of 65 feet (TH-1 and TH-2), 45 feet (TH-3), and 70 feet (TH-4), fully penetrating the unconsolidated terrace deposits. Undisturbed samples were collected at 5-foot intervals using a modified California split-spoon sampler, with standard penetration tests conducted during sampling. The boreholes were backfilled with a sand-cement slurry. Boring locations are shown in Figure 2. Lithologic logs of the boreholes are in Appendix A.

Water levels were measured in the test holes and existing on-site monitoring wells prior to backfilling on November 22, 2005. Water samples were also collected from two of the test holes (TH-1 and TH-4).

On November 29, bail-down tests were conducted at the two existing on-site monitoring wells (MW-4 and MW-5). Water samples were also collected from the wells at the conclusion of the bail down tests, along with a sample of Kavanaugh Creek underflow collected from a 5-foot hand-auger boring in the dry wash. The results of the bail-down tests indicated very low permeability at MW-5, which was bailed dry at an average discharge rate of less than 0.3 gallons per minute. By comparison, the sediments tapped by MW-4 were too permeable for analysis using the bail-down test. Therefore, a second test was conducted at MW-4 on January 16, 2006, using a submersible pump. Pumping test data are in Appendix B.

GROUND WATER FLOW MODEL

A numerical ground water flow model was developed to evaluate the hydraulic capacity of the site. The conceptual model, upon which the flow model was based, represents various lithologies beneath the site, the discharge pond geometry, Kavanaugh Creek, and elements of recharge and discharge.

Conceptual Model

Lithologies beneath the site were divided into five groups. Data from nine laboratory permeability tests, 19 laboratory sieve analyses, and two pumping test were used characterize the materials. The five groups are as follows:

1) Silty sands and sandy silts from ground surface to the top of the prominent clay lense. This interval includes all of the unsaturated zone beneath the ponds and is 40 to 50 feet thick.
2) The prominent clay lense, which is approximately 5 feet thick.

3) Loose, permeable sands immediately above bedrock (and beneath the prominent clay lense in the central portion of the site). This lithologic horizon is 5 to10 feet thick. A pumping test was conducted in this horizon at MW-4.
4) Bedrock. In most areas, bedrock is considered impermeable (no-flow). This group is modeled as a 15- to 20-foot thick low permeability layer immediately below the unconsolidated sediments.
5) Kavanaugh Creek alluvial deposits. These deposits are assumed to be entrenched into bedrock as a buried channel beneath Kavanuagh Creek, and are estimated to be approximately 35 feet thick.

Laboratory vertical permeability tests were plotted against silt content and a linear relationship was developed. Additional silt content analyses and the boring logs were used to estimate the vertical permeability profile values for the subsurface. The geometric mean of the permeability profile was then calculated at each borehole for lithologies above the prominent clay layer, and used to estimate the overall vertical permeability value. Laboratory data, a silt content versus permeability plot, and geometric mean calculations for the permeability analyses are in Appendix C.

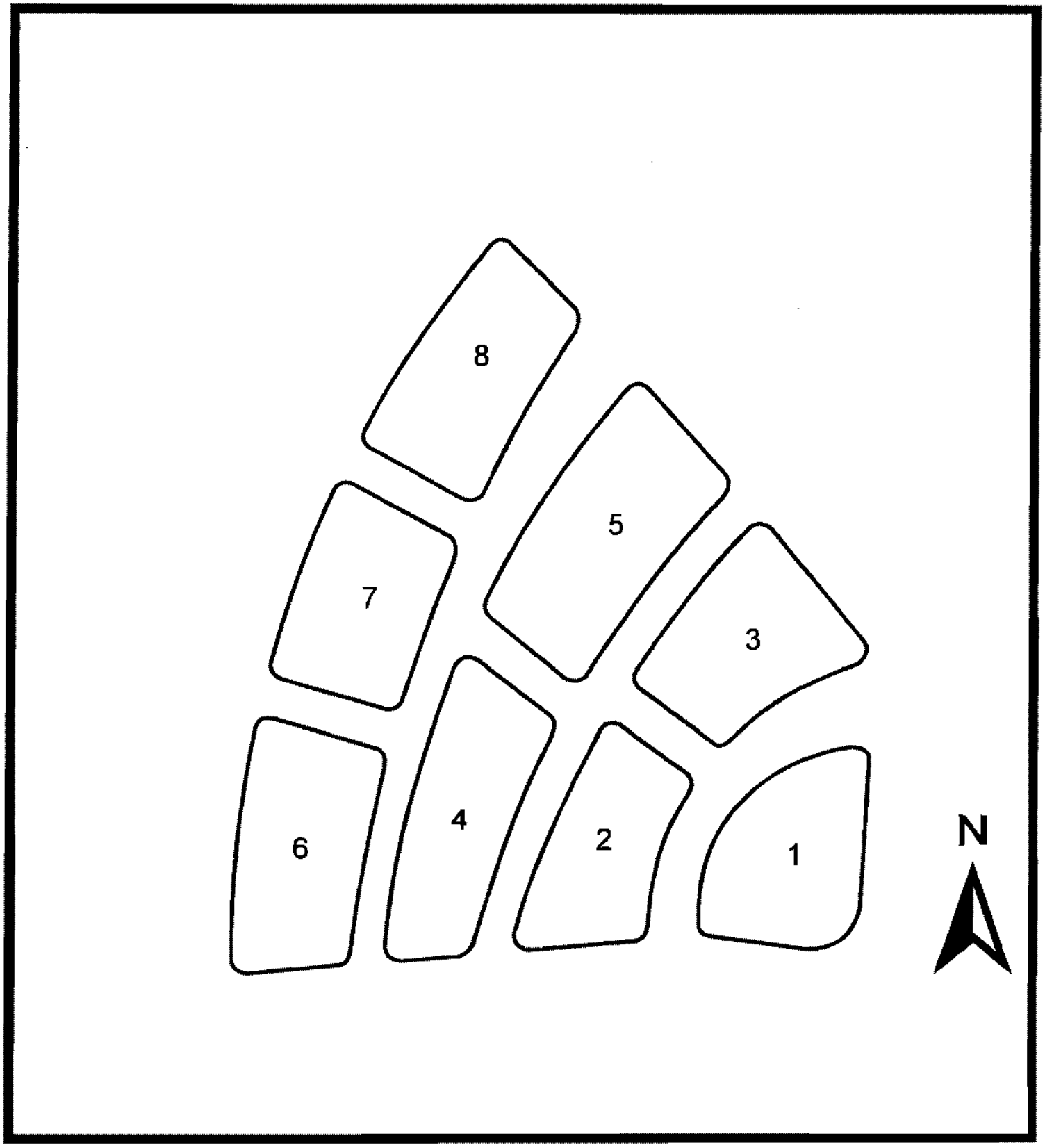
A constant discharge test was performed at MW-4 to determine horizontal permeability, which yielded a ratio of 8:1 horizontal to vertical for unconsolidated sediments. The permeability values for the subsurface lithologic groups are estimated as follows:

Lithologic Group	Horizontal permeability (ft/day)	Vertical permeability (ft/day)
Group 1 (silty sands)	12	1.5
Group 2 (clay)	0.08	0.01
Group 3 (basal sand)	100	12
Group 4 (bedrock)	0.0001	0.0001
Group 5 (creek alluvium)	200	24

Discharge pond geometry was digitized from a site plan provided by the Wallace Group (Figure 4). Two of the three existing ponds are used for model calibration, and up to 8 ponds are used for future treated effluent disposal scenarios. The future pond invert elevations are estimated to be between 828 and 835 feet above sea level.

Kavanaugh Creek is represented by a high permeability drain with a stage set within one-foot of the surface elevations of the dry wash. A general head boundary is used to represent Nacimiento Lake for maximum lake stage scenarios.

Recharge from percolation of precipitation is estimated at approximately 20 percent of the average 20 inches annual rainfall as measured at the Oak Shores gage. General head boundaries where Kavanaugh Creek enters and leaves the model domain maintain steady subsurface flow through the alluvial deposits during calibration runs. These elements of recharge and discharge were set to equilibrate at water levels approximating actual site conditions. During model scenarios involving steady-state discharge pond



Source: Wallace Group, 2005

Figure 4
Future Disposal Ponds
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

operation, the calibrated inflow from the upstream general head boundary was substituted with in-stream recharge to prevent mounded water from exiting the model in the upstream direction. The model domain is shown in Figure 5.

Flow Model Calibration

The model performance was checked in transient mode against a 12-day discharge test conducted by the County at the existing ponds sites. Between May 17 and May 28, 1999, approximately 408,000 gallons of effluent were percolated into Pond #1, and 209,000 gallons of effluent were percolated into Pond #2. During this period, water levels at five on-site wells were monitored. Two of the wells were completed in bedrock and three in the overlying unconsolidated sediments. The wells completed in unconsolidated sediments (MW-2, MW-3, and MW-4) are used for model calibration.

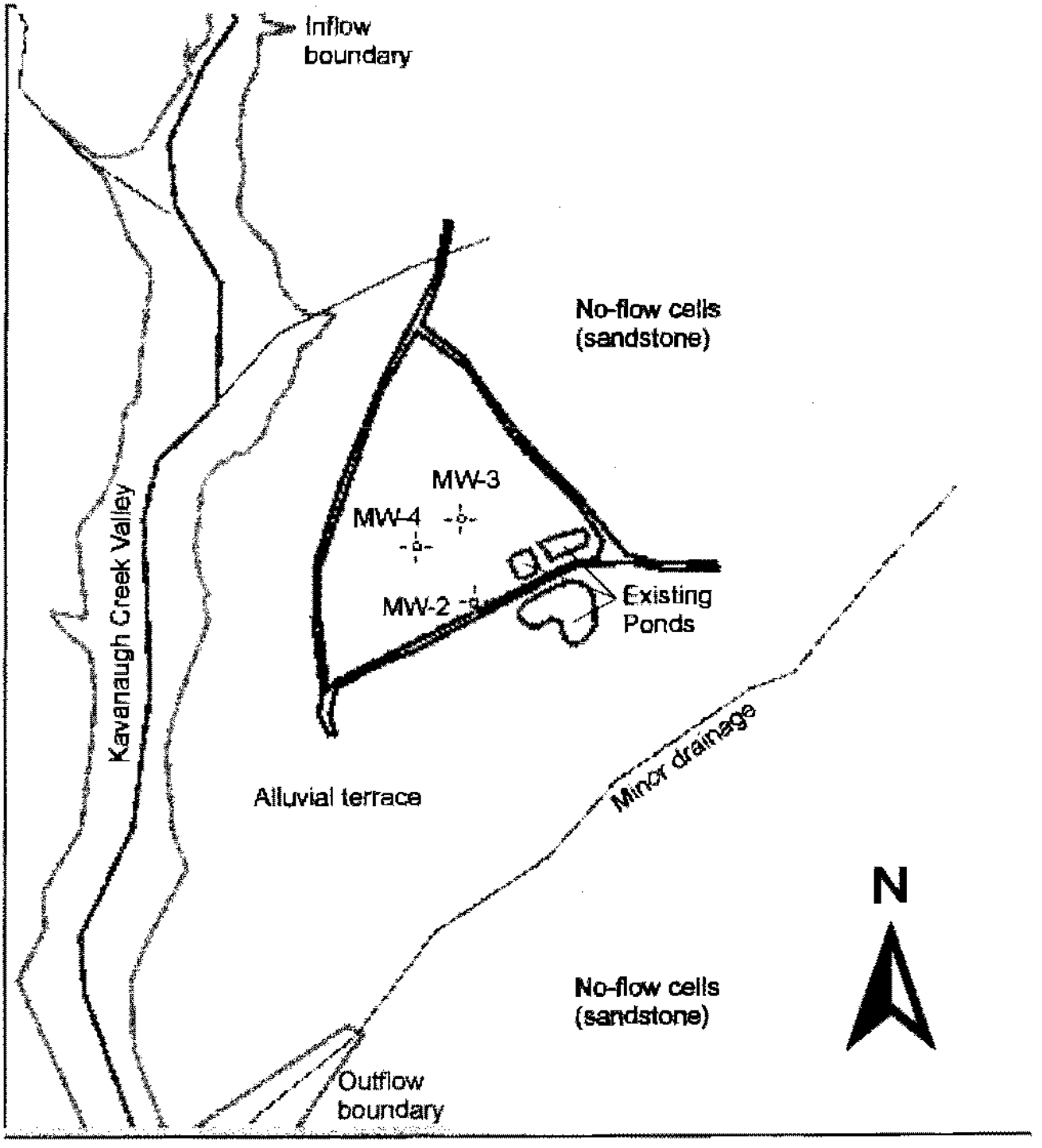
According to County records, water levels rose one foot at MW-2 and MW-3 during the test, while levels dropped one foot at MW-4. Three weeks after completion of the effluent infiltration test, water levels at MW-2 and MW-3 and MW-4 were all one foot higher than pre-test conditions. The water levels reported by the County are to the closest foot.

The transient model requires storage coefficient and specific yield estimates. At estimated values of 15 percent specific yield (equivalent to the storage coefficient for unconfined aquifers) the simulated change in water levels at the monitoring wells during the test resulted in a 0.8-foot rise and MW-2, a 0.5-foot rise at MW-3, and 0.1 feet of rise at MW-4, which after rounding is equivalent to the field observations. After approximately three weeks (one week after the test was terminated), simulated water levels at the monitoring wells showed a 1.4-foot rise at MW-2 and MW-3, and a 0.8-foot rise at MW-4, compared to pre-test water levels. This post-test calibration period shows that the delayed water level mounding at MW-4 (which is farther from the ponds) observed in the model is consistent with field observations.

SITE DISCHARGE CAPACITY

There are three hydrogeologic constraints on the surface application of treated effluent at the Kavanaugh site:

1) Shallow subsurface infiltration rates and plugging
2) Daylighting of effluent in Kavanaugh Creek
3) Maintaining separation between the ground water mound and the disposal ponds



Scale: 1 inch = 300 feet

Figure 5
Model Domain
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

Infiltration Rates

Prior testing at the site was focused on evaluating the first constraint. In January 1997, a five-day discharge test in a relatively small test pond was conducted. Approximately 10,500 gallons of effluent was discharged. Infiltration rates reportedly declined over time throughout the duration of the test, and averaged 0.74 ft/day. The decline in infiltration rate over time was attributed to a loss of permeability in the sediment beneath the test pond from clogging as suspended material in the effluent was filtered out and left behind.

In May 1999 a more extensive pilot test was conducted (used for model calibration, as discussed above). An average infiltration rate of 1.7 ft/day was reported for the test, with daily averages typically in excess of 1.5 ft/day, although the infiltration rate on the last day of testing at pond #1 was reported at 0.9 ft/day.

Based on the testing, the County's projected average infiltration rate for future disposal operations is 1.7 ft/day, and a minimum of 0.8 ft/day. For the purposes of this constraints analysis, the minimum application rate of 0.8 ft/day is assumed. The corresponding allowable application rate for an average disposal pond (16,500 square feet) would be up to 100,000 gallons per day.

Daylighting of effluent in Kavanaugh

Steady-state ground water flow modeling indicates that up to 145,000 gallons per day (gpd) of treated effluent may be discharged to the site without daylighting in the Kavanaugh Creek dry wash. The ponds closest to Kavanaugh (6, 7, and 8), however, cannot be used. If ponds 6, 7, and 8 are included in the disposal rotation, daylighting occurs at lower discharge rates.

A minimum of three ponds (ponds 1, 2, and 3) would be necessary for the 145,000 gpd disposal scenario. This would allow for one pond to be taken off-line for drying and cleaning, while the application rates at the two active ponds would be close to 72,500 gpd each (less than the assumed minimum infiltration rate of 100,000 gpd). The average annual hydraulic loading on a 3-pond system at 145,000 gpd would be equivalent to 0.4 ft day.

The most flexible system for the Kavanaugh site would involve five ponds, with three active, one drying, and one being cleaned. The average annual hydraulic loading on a 5-pond system would be 0.23 ft/day, equivalent to 14 percent of the County's projected average infiltration rate of 1.7 ft/day for future disposal operations based on the infiltration testing. The U.S. EPA Process Design Manual for Land Treatment of Municipal Waste (1981) recommends that annual hydraulic loading rates should be no greater than 10 to 15 percent of measured basin infiltration rates.

Development of the ground water mound is a gradual process, taking several years to reach a steady state condition. At 145,000 gallons per day, no daylighting of effluent would be expected in the Kavanaugh

Creek dry wash. Transient flow modeling indicates that at 165,000 gallons per day, daylighting would be expected within three years.

Kavanaugh Creek is one of the tributary streams flowing into Lake Nacimiento. The reservoir spillway elevation is 800 feet above sea level, and the creek bed is 790-794 feet above sea level opposite the disposal site. Lake levels have exceeded 790 feet during portions of 15 water years between 1958 and 2005 (averaging once every four years). These high lake levels back water up into the Kavanaugh Creek valley. The most recent periods of high lake levels were during the 1994-1995, 1997-1998, and 2004-2005 water years.

When Nacimiento Lake rises into Kavanaugh Creek, adjacent waters seep into the adjacent alluvial terrace deposits creating bank storage. This storage increases during the period of rising lake levels, and then returns to the creek when lake levels recede. The effect of transient bank storage on the ground water mound beneath the disposal site is an increase in overall mound height, with greater increases toward the creek.

To simulate the general effect of bank storage, an 145,000 gpd disposal rate was modeled using a 5-pond system over twelve months. After the 3rd month, an increase in lake levels to 800 feet elevation was simulated. The elevated lake levels were maintained over the next four months (through the 7th month), and then removed for the remaining 5 months. Development of the mound was monitored at three locations; beneath the disposal ponds, mid-way between the ponds and the creek, and adjacent to the creek.

The short-term rise in lake levels resulted in an increase of up to 2 feet in the mounding beneath the ponds (reached in the 8th month). An increased mound height of up to 8 feet was simulated mid-way to the creek, reaching a relatively sharp peak in the 7th month, and then falling off to 3 feet of net increase by the 12th month. Adjacent to the creek, the increase in water levels reached a maximum of 13 feet at the end of the 7th month, after which bank storage sharply dropped off, and the final difference in water levels with and without the lake rise was 2 feet after 12 months. During the first two months of elevated lake levels (the 4th and 5th month of the simulation), the normal hydraulic gradient between the ponds and the creek was reversed, with water near the creek moving back toward the ponds. Simulated hydrographs showing the increased mound heights during elevated lake conditions are shown in Figures 6a (beneath ponds), 6b (between ponds and creek), and 6c (adjacent to creek).

The above simulation demonstrates that elevated lake levels have the effect of increasing mound height beneath the ponds, but do not appear to increase the potential for daylighting of the mound. The effect of rising lake levels above 790 feet elevation is to temporarily reverse the flow of ground water back toward the ponds. As lake levels recede, bank storage would flow back into the creek, and the normal hydraulic gradient and water levels associated with the discharge mound would remain.

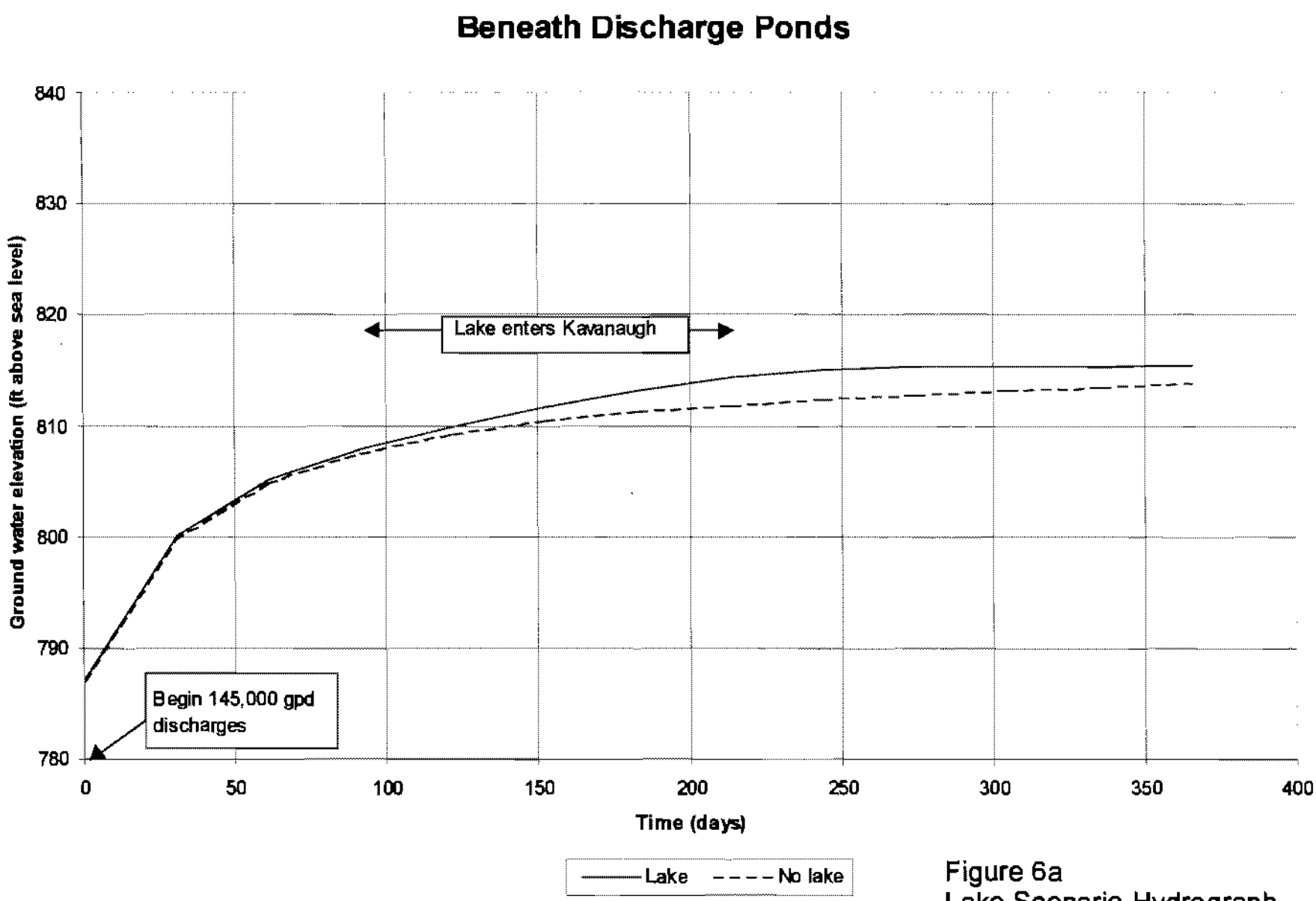


Figure 6a
Lake Scenario Hydrograph
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

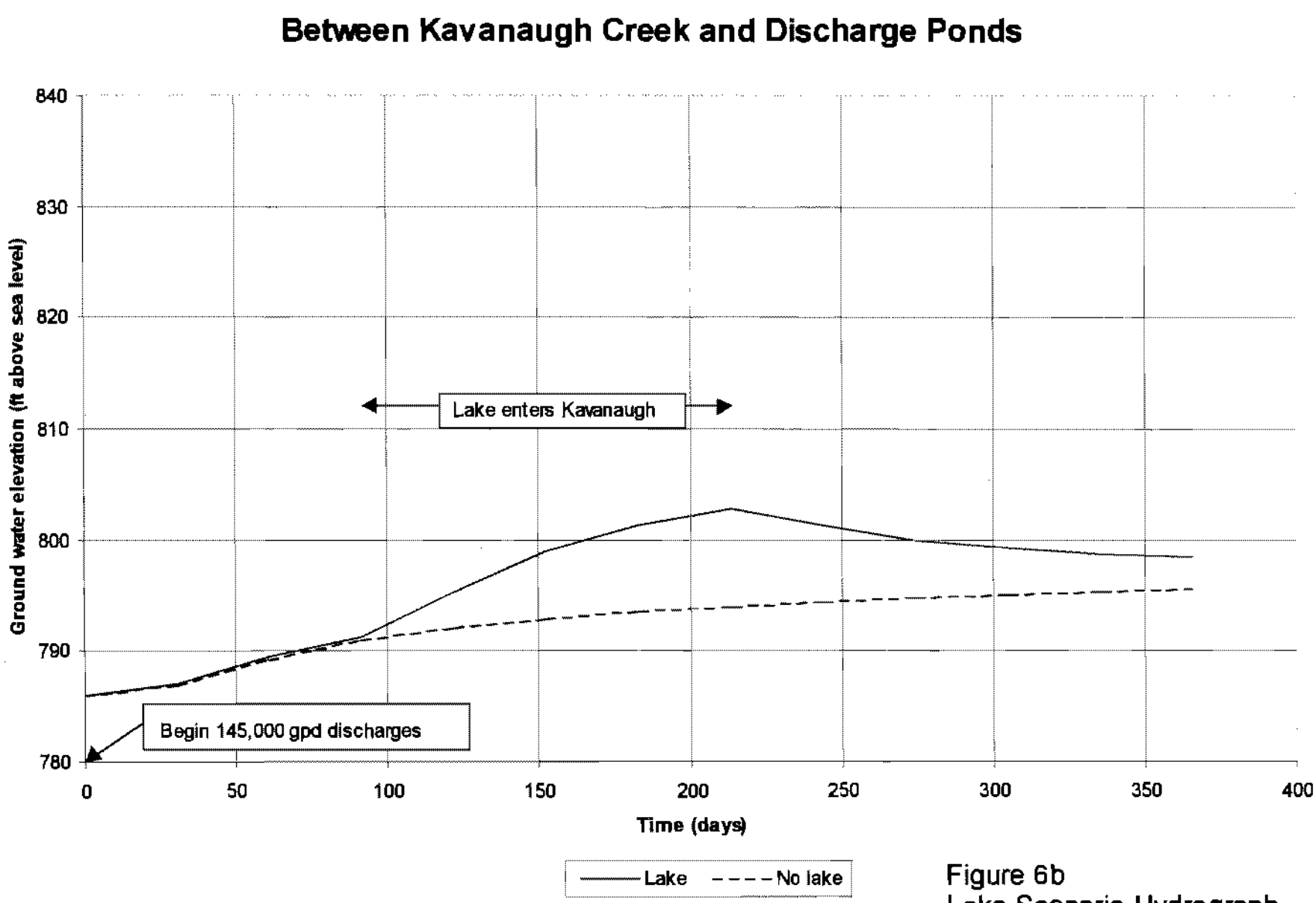


Figure 6b
Lake Scenario Hydrograph
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

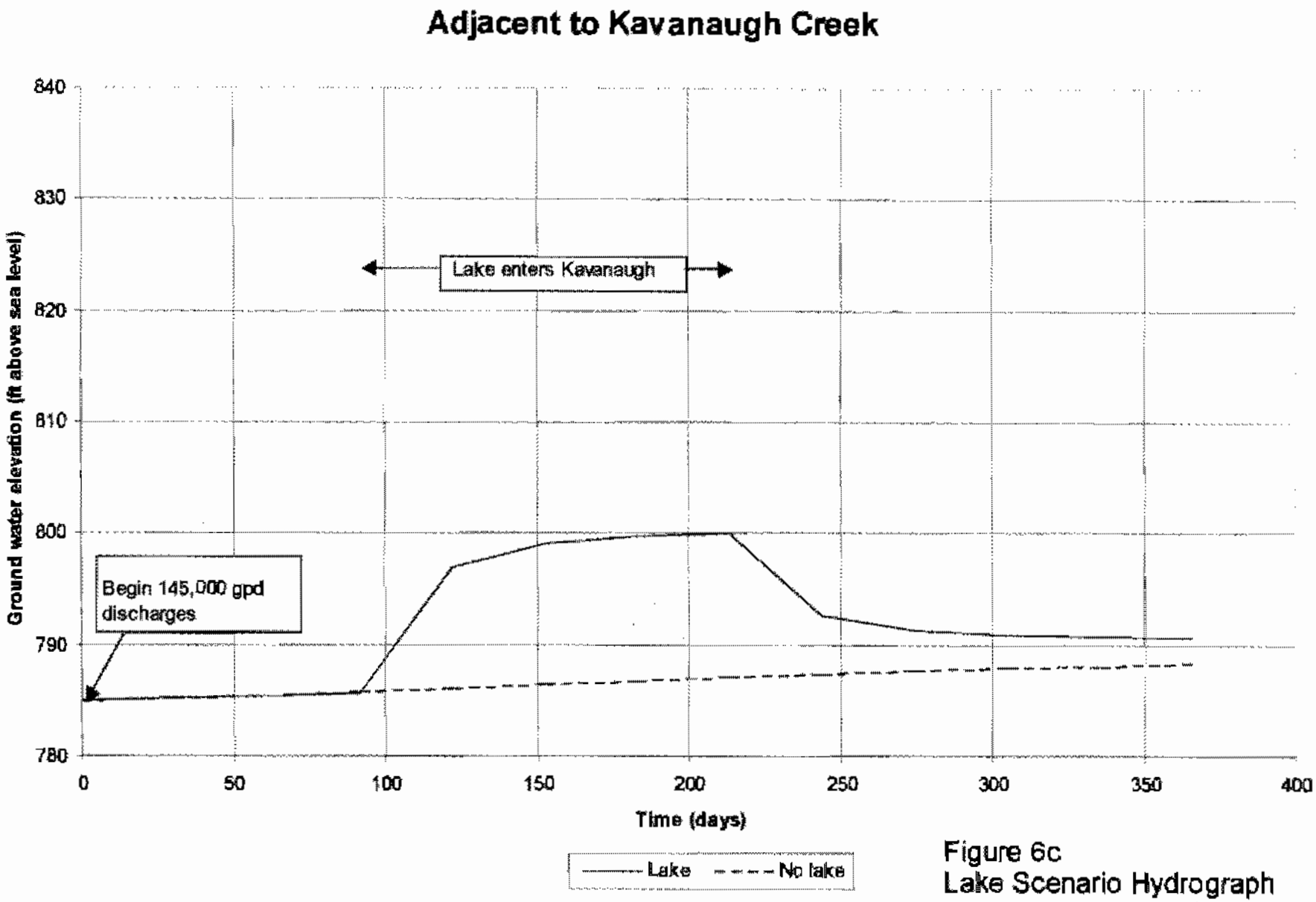


Figure 6c
Lake Scenario Hydrograph
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

Mound-to-pond separation

Separation between the ground water mound and the base of the ponds is desired to maintain the maximum disposal capacity of the pond. A minimum separation of 5 feet is assumed for this constraints analysis.

Pond invert elevations of up to 835 feet would require maximum mound heights of no more than 830 feet above sea level. Simulation of a 5-pond system indicates that the minimum mound-to-pond separation can be maintained with up to 240,000 gpd of effluent disposal. At this level of disposal, however, approximately 56,000 gpd of water is daylighting into the Kavanaugh Creek dry wash. If lake levels rise to 800 feet elevation, only 205,000 gpd of effluent can be disposed and still maintain the 5-foot minimum mound-to-pond separation.

The following table summarizes steady-state ground water modeling results for a 5-pond disposal system at Kavanaugh.

Kavanaugh Disposal Site
5-Pond System
Simulated Steady State Conditions

Creek Condition	Disposal Rate (gpd)	Daylighting (gpd)	Pond-to-mound (feet)
Dry Wash	145,000	none	16
Lake	145,000	none (creek flooded)	11
Dry Wash	240,000	56,000	5
Lake	240,000	none (creek flooded)	0 (ponds flooded)
Lake	205,000	none (creek flooded)	5

WATER QUALITY CHARACTERIZATION

Water resources in the vicinity of the Kavanaugh site include the following four groups:

1) Surface water in Lake Nacimiento

2) Oak Shores community water supply (Nacimiento Water Company)

3) Oak Shores community treated effluent

4) Ground water in bedrock

5) Ground water in alluvial deposits

Lake Nacimiento water quality has been studied for many years. To characterize the overall water quality, approximately 100 discrete samples from various locations and depths collected between 1997 and 2003 were averaged. Water type in the reservoir is calcium magnesium-bicarbonate with a total dissolved solids (TDS) averaging 186 milligrams per liter (mg/l). Nitrate concentrations averaged 0.1 mg/l.

Water service at Oak Shores is provided by the Nacimiento Water Company. The water company pumps water from a gallery well system completed in sediments directly underlying Lake Nacimiento. Water quality from the gallery well system closely matches reservoir water quality, and is a calcium magnesium-bicarbonate water with a TDS averaging 200 mg/l, based on 52 samples collected monthly between February 1997 and May 2001. One sample from the 1997-2000 series (collected April 6, 1999) includes a full general mineral analysis, and is used to represent the source water quality for the water company. TDS for the April 1999 sample is reported at 190 mg/l, and nitrate is reported at 0.5 mg/l (as nitrate).

Treated effluent at Oak Shores is currently disposed at spray fields, with seasonal discharges to emergency ponds at the Kavanaugh site. Water quality analyses for the effluent is also available in samples collected monthly between February 1997 and May 2001, including one sample (collected April 6, 1999) with a full general mineral analysis. Unlike the Nacimiento Water Company source of supply data, however, the April 1999 effluent sample cannot be used to characterize the effluent data set, because of a significant departure in TDS from the average. The average TDS of the effluent is 508 mg/l, while the April 1999 effluent sample reported 190 mg/l (very close to source water quality). Sodium and chloride data is available, however, and other major ions were estimated based on ratios in spray field data (maintaining electroneutrality). The resulting treated effluent water quality is a sodium calcium-chloride bicarbonate water.

Water quality data is also available at the spray field pond between March 1997 and April 2001. The spray pond effluent is a sodium calcium-chloride bicarbonate with an average TDS of 780 mg/l. Dissolved mineral pickup in the domestic use cycle generally contributes close to 300 mg/l of TDS to the source water, as reflected in the average TDS of the actual effluent samples. In the case of the spray pond, the TDS pickup is closer to 600 mg/l.

The reason for the excessive TDS pickup at the spray field pond is probably related to the disposal method and sampling location. The spray field system currently relies on atmospheric evaporation and on the evapotranspiration of water by grass covering the field. Most of the mineral salts in the effluent would concentrate over time in the soils underlying the field and leach back into the pond, which is at the

base of the field. Therefore, sampling at the ponds represents a unique water quality that does not necessarily represent effluent discharges.

The average nitrate concentration in the spray field pond was 5.6 mg/l. No analysis of other forms of nitrogen were reported in the spray field samples, although spray field operations are an oxidizing process and would be expected to convert ammonia (the dominant form of nitrogen in raw effluent) to nitrate. The relatively low concentration of nitrate in spray field pond effluent, as with the elevated TDS values, may also be related to the spray field process. Nitrogen uptake by field grasses is well-documented, and this process is likely contributing to nitrate removal from the effluent. Subsurface denitrification may also be occurring in leachate between the fields and the hydraulically downgradient pond.

Future discharges of treated effluent to the Kavanaugh site would not necessarily be limited to emergency overflow from a spray field process, however, and may come directly from aeration lagoons. Therefore, nitrogen as nitrate concentrations for discharges are assumed to average 30-40 mg/l, which would be closer to typical treatment plant values.

Ground water flows in the alluvial deposits and, in certain cases, within fractures in bedrock. The current quality of ground water in the vicinity of the Kavanaugh site has been characterized based on samples collected in November 2005 during test hole drilling and from existing monitoring wells.

The character and salinity of ground waters in the site vicinity are variable. Directly beneath the Kavanaugh site (November 2005 samples from TH1, TH2, and MW4), ground water in the alluvial sediments is calcium sodium-bicarbonate with an average TDS of 787 mg/l and a nitrate concentration of 4 mg/l. Water from well MW5, which taps bedrock and/or seepage from colluvial deposits overlying bedrock at the Kavanaugh site is magnesium calcium-sulfate bicarbonate with a TDS of 1,400 mg/l. The water quality of Kavanaugh Creek underflow, collected by hand-augering into the dry wash deposits, was calcium sodium-chloride bicarbonate with a TDS of 2,500 mg/l. The underflow water sample did not contain detectable concentrations of nitrate. Laboratory reports of recent water quality samples are included in Appendix D.

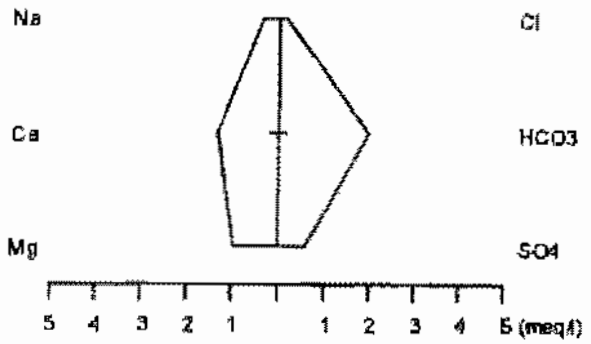
Stiff diagrams comparing the various water types in the site vicinity are shown in Figure 7a and 7b. Note difference in scale between the two groups of stiff diagrams.

WATER QUALITY THRESHOLDS OF SIGNIFICANCE

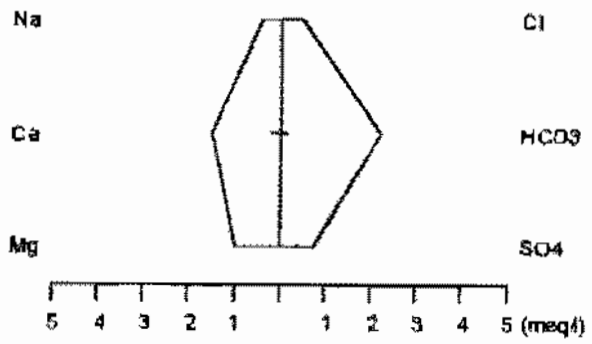
RWQCB WDR No. 01-130 addresses impacts to ground water quality with the following limitations:

- The discharges shall not cause a significant increase of mineral constituent concentrations in underlying ground waters.

Nacimiento Lake Water
(1997-2003 average)



Nacimiento Water Company
(Oak Shores Supply - 4/6/99)



Oak Shores Effluent
(1997-2001 average)

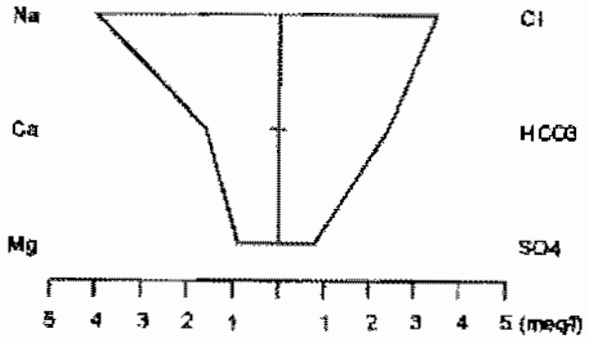
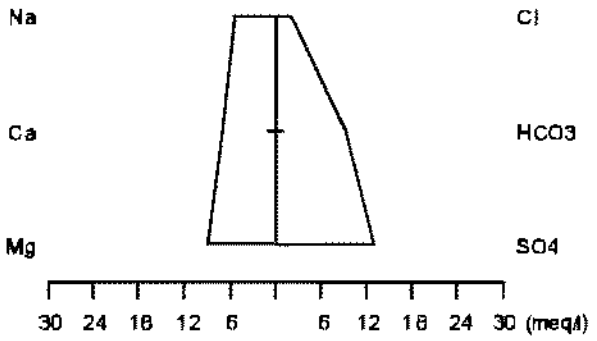


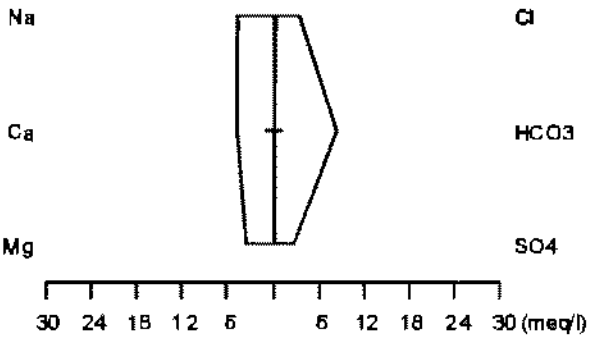
Figure 7a
Stiff Diagrams
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

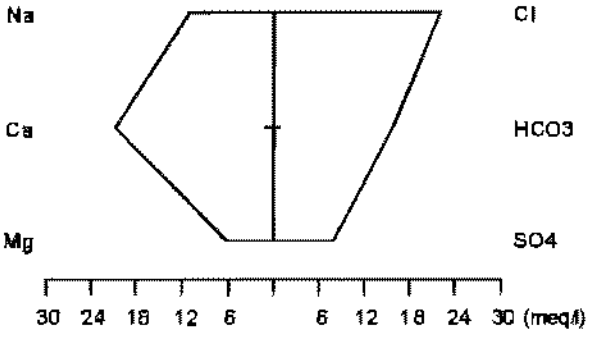
Bedrock
(MW-5 11/29/05)



Alluvium beneath ponds
(TH-1, TH-4, MW-4 average 11/22-29/05)



Kavanuagh Creek Underflow
(11/19/05)



Note increase in milliequivalents per liter (meq/l) scale between Figure 7a and 7b. Relative salinity is much greater in samples shown on Figure 7b, compared to 7a.

Figure 7b
Stiff Diagrams
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

- The discharge shall not cause concentrations of chemicals and radionuclides in ground water to exceed limits set forth in Title 22, Chapter 15, Articles 4, 4.5, 5, and 5.5 of the California Code of Regulations.

The interpretation of a significant increase in mineral constituents under the first limitation is site specific, and should consider the beneficial uses, background water quality, and assimilative capacity of the receiving water. The second limitation references primary drinking water standards contained in Title 22, Chapter 15, for inorganic and physical quality (Article 4), trihalomethanes (Article 4.5), radioactivity (Article 5) and organic chemicals (Article 6).

A significant increase in mineral constituents occurs when one of the present or anticipated beneficial uses of a local water resource is restricted due to changes in water quality attributable to the discharges. The RWQCB has identified the present and anticipated uses of ground water near the discharge site to include municipal and domestic water supply, agricultural water supply, industrial process supply, and industrial service supply. Ground water in the alluvial deposits ultimately drain to Lake Nacimiento, which has eleven present and anticipated beneficial uses listed in the Basin Plan, including municipal and domestic water supply and water contact recreation.

Given that there is a present and anticipated use for domestic and municipal supply of ground water in the site vicinity, the second limitation referencing Title 22 automatically applies to Kavanaugh discharges. Other types of limits on use could also be considered valid, such as Article 8 (secondary drinking water standards), and specific ion toxicity levels for sodium, chloride, and boron in regard to agricultural operations based on Ayers and Westcott (1985).

For the purposes of the Kavanaugh site impacts evaluation, the following mineral salt constituents are considered, with their associated threshold of significance:

- Total dissolved solids - 1,000 mg/l (potability restriction)
- Chloride - 350 mg/l (irrigation restriction)
- Sodium - 200 mg/l (irrigation restriction)
- Sulfate - 500 mg/l (potability restriction)
- Nitrate - 45 mg/l (potability restriction)
- Boron - 1 mg/l (irrigation restriction)

POTENTIAL GROUND WATER QUALITY IMPACTS

This impacts assessment compares changes in water quality due to future effluent discharges with respect to thresholds of significance for the mineral salt constituents listed above. The quality of the existing underlying ground water for the site is represented by recent samples from MW4, TH1, and TH4. Table 1 compares existing water quality with the thresholds.

Table 1
Water Quality Comparison

Constituent	Current ground water (mg/l)	Effluent* (mg/l)	Threshold of Significance (mg/l)
Total dissolved solids	787	500	1000
Chloride	119	123	350
Sodium	114	92	200
Sulfate	129	37	500
Nitrate	4	180	45
Boron	0.3	no data	1

*Notes: Effluent TDS, Cl, and Na average of 1997-2001 data. Effluent sulfate concentration derived from spray field pond samples. Effluent nitrogen as nitrate concentration assumed for future discharges.

Effluent water quality is lower in concentration than the existing underlying ground water for all constituents reviewed except nitrate (boron data not available). Provided the effluent quality does not change, discharges of treated effluent at the Kavanaugh discharge site will not cause concentrations of these constituents to exceed their threshold values. In fact, in some cases, discharges may cause constituent concentrations to decline.

Nitrate Loading

Nitrate loading will occur in ground water underlying the Kavanaugh Site. A review of historical data sets for existing and former monitoring wells show a direct relationship between the application of effluent and increases in nitrate in alluvial ground water beneath the ponds. Chemographs for nitrate are shown in Figures 8a, 8b, and 8c. The maximum historical values of nitrate in all three alluvial sediment monitoring wells (MW2, MW3, and MW4) were between 15 and 21 mg/l nitrate, which is comparable to the maximum levels of nitrate measured in the spray field pond effluent during the periods of effluent diversion to Kavanaugh.

Future discharges to the Kavanaugh site, however, would not necessarily be restricted to overflow from the spray field pond, and total nitrogen concentrations of up to 40 mg/l are anticipated, which could convert to nitrate concentrations of up to 180 mg/l.

The analysis of nitrate loading involves a mass balance calculation intended to represent the average, long-term nitrate concentration in the study area. The input sources to the mass balance calculation include the following:

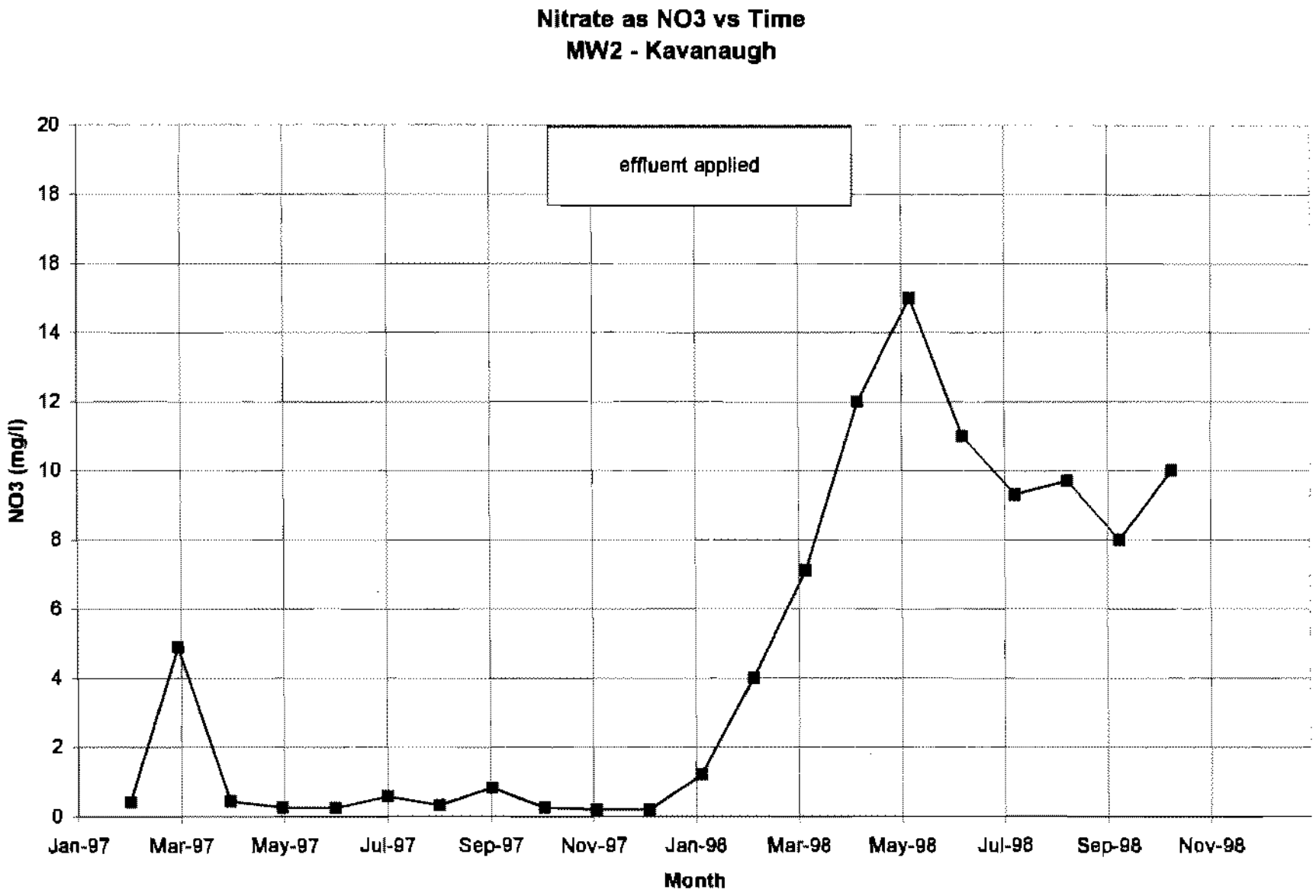


Figure 8a
Nitrate Chemograph
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

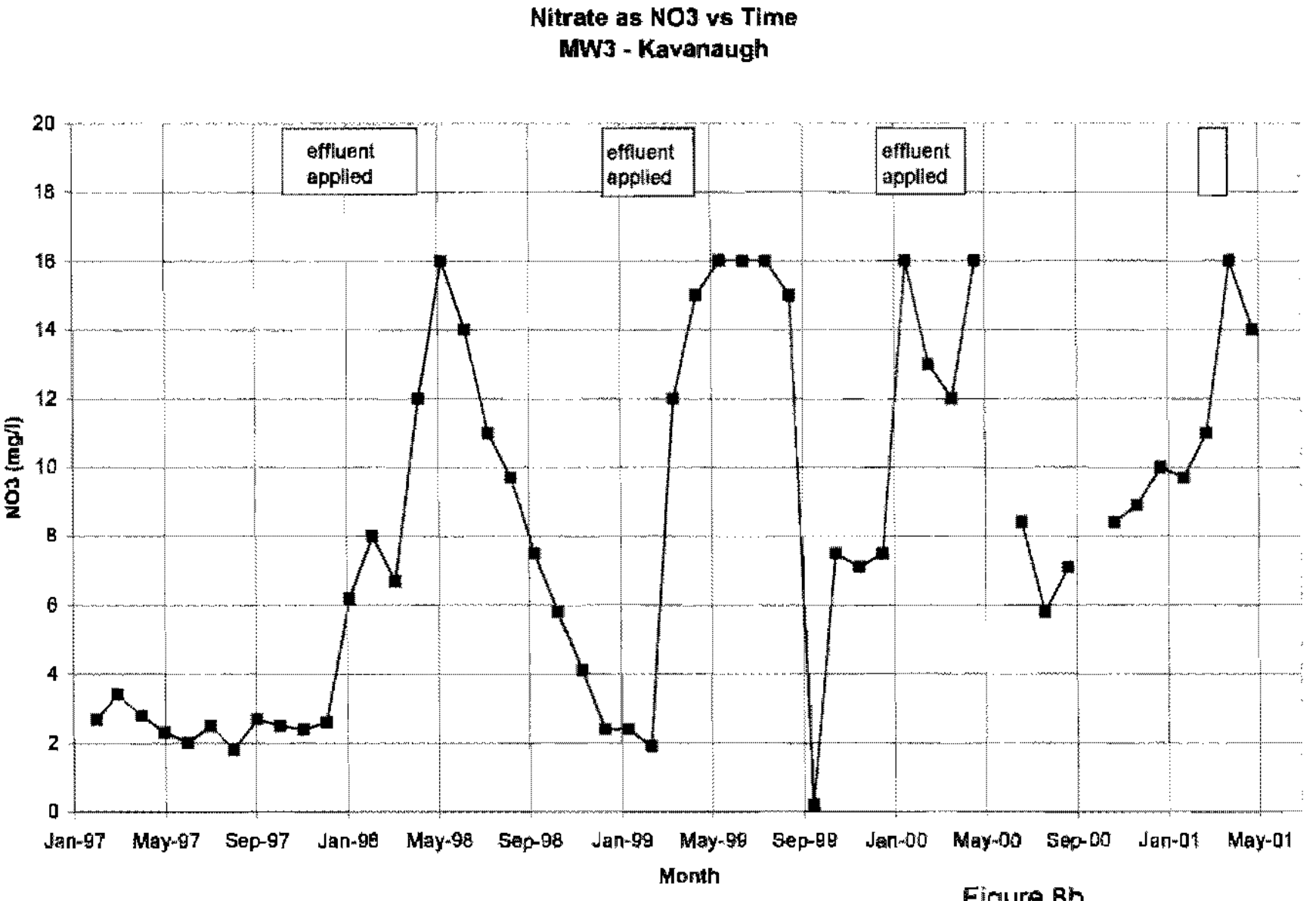


Figure 8b
Nitrate Chemograph
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

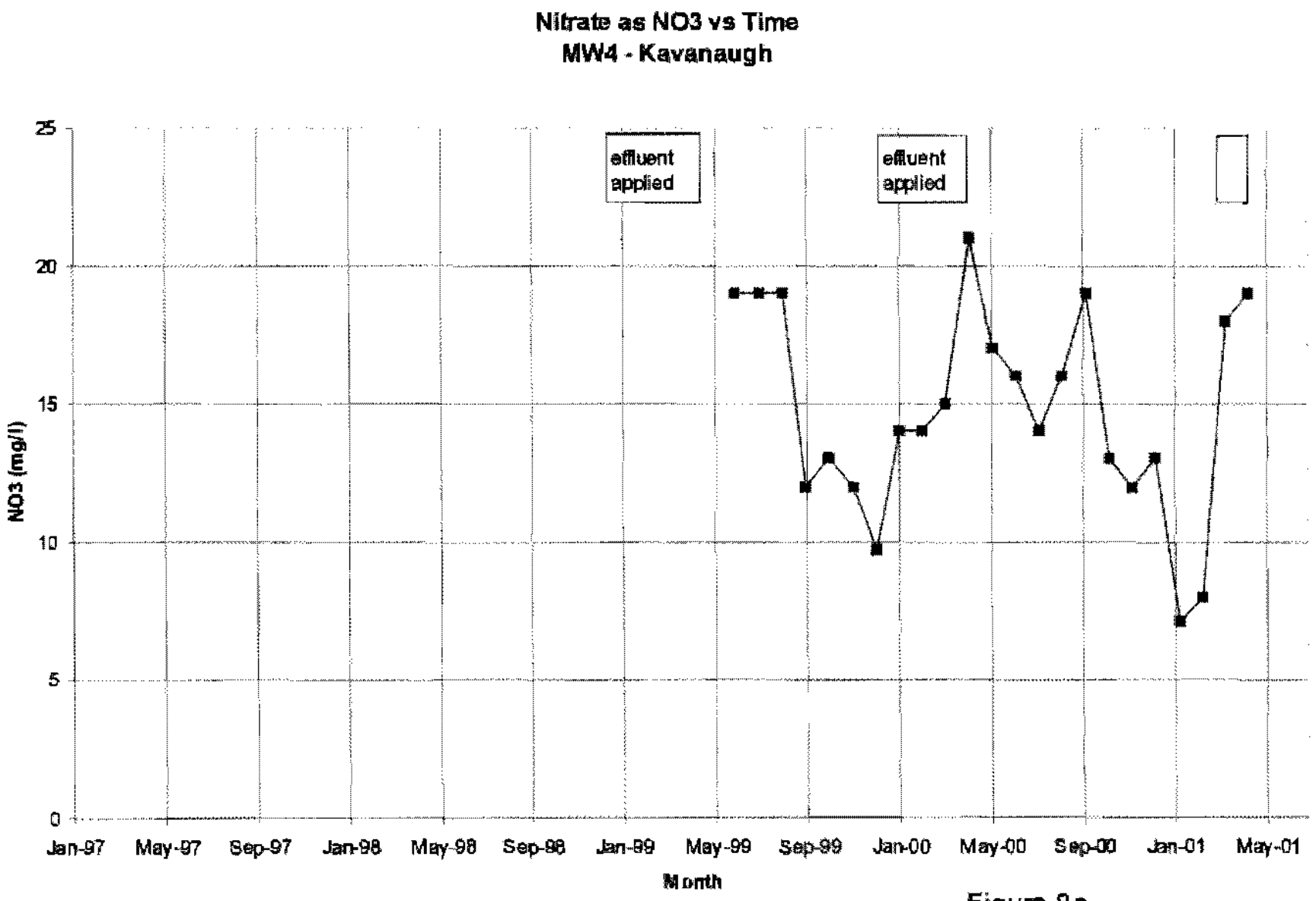


Figure 8c
Nitrate Chemograph
Kavanaugh Disposal Site
Oak Shores

Cleath & Associates

- Effluent discharges
- Subsurface inflow
- Percolation of precipitation
- Stream flow seepage
- Nacimiento Lake seepage

The effluent discharges are assumed to average 145,000 gallons per day, with a nitrate concentration of 180 mg/l as NO_3 (after conversion from ammonia and organic nitrogen). The resulting mass loading would be equivalent to 218 pounds of nitrogen per day. Denitrification may result in lower concentrations of nitrate in the leachate reaching ground water, even with a mound-to-pond separation between 11 and16 feet. Prior discharges of spray field effluent, however, resulted in nitrate concentrations in ground water collected at downstream monitoring wells that reached values similar to the effluent quality, suggesting limited denitrification. No subsurface denitrification has been assumed for the purpose of this nitrate loading analysis.

Subsurface inflow has been estimated using the calibrated steady state model, and is approximately 8,400 gallons per day. There was no detectable concentration of nitrates in the Kavanaugh Creek underflow sample collected for this investigation, therefore, no mass loading from this source.

Percolation of precipitation is estimated to be 20 percent of the annual average rainfall over the unconsolidated sediments of the study area. This value incorporates deep percolation of runoff from the sandstone units and minor drainages entering the study area. Percolation of precipitation can vary based on the soils, slopes, and vegetation (or hardscape). In addition, the actual amount of precipitation affects the percent of percolation.

For example, the percent percolation of precipitation over the 790 square-mile Paso Robles ground water basin is estimated at close to 8 percent (43,400 acre-feet per year for 13.5 inches average rainfall) using Blaney's curves developed from detailed studies in Ventura County (Fugro and Cleath & Associates, 2002). The 23 square-mile Atascadero subbasin, however, is estimated to deep percolate approximately 17 percent of rainfall (3,900 acre-feet per year for 18.8 inches of rainfall) using the same curves. The main reason for the difference in percent percolation of precipitation between the basin and its internal subbasin is average rainfall. The average annual rainfall at Oak Shores is approximately 20.3 inches per year, slightly greater than in the Atascadero subbasin. Therefore, considering the relatively flat alluvial terrace, well-drained soils, and relatively sparse vegetation in the study area, a value of 20 percent percolation would be appropriate.

The resulting estimate of percolation of precipitation for the study area averages 18,600 gallons per day. As with the subsurface inflow, the recharge from precipitation has no associated nitrate loading.

Stream flow seepage along Kavanaugh Creek also contributes inflow to ground water in storage within the study area. During future operation of the discharge ponds at 145,000 gallons per day, however, there would be little room left in the creek alluvial deposits for seepage, and most stream flow would run

through to Lake Nacimiento. Modeling estimates the amount of available room for stream seepage to be equivalent to 3,000 gallons per day. Since stream flow is seasonal, the opportunity for seepage would not be occurring year-round, and only a portion of the available room would be used. Assuming six months of stream flow, the contribution to ground water in storage from stream seepage is estimated at 1,500 gallons per day. No nitrate loading is associated with stream seepage.

The greatest natural source of recharge to ground water in storage beneath the Kavanaugh site is seepage from Lake Nacimiento during periods of high lake levels. As mentioned previously, Kavanaugh Creek is one of the tributary streams flowing into Lake Nacimiento. The reservoir spillway elevation is 800 feet above sea level, and the creek bed is 790-794 feet above sea level opposite the disposal site. High lake levels cause water to back up into the Kavanaugh Creek valley.

Transient ground water modeling was used to evaluate seepage from rising lake levels. Daily lake level data between 1958 and 2001 were used to determine the average number of days during a year that lake levels rise to each one-foot increment between 787 and 800 feet elevation. For example, the data shows that lake levels were at or above 795 feet elevation during 632 days out of the 15,727-day record. This equates to approximately 14.7 days per year, on average. A total of 120 days with rising (and falling) lake levels were loaded into the model. Inflow from the lake to ground water in storage within the study area was retrieved from the model during the rising lake period (60 days), and averaged 385,700 gallons per day (exceeding the effluent discharge rate). When distributed over the entire year, however, the average inflow from lake seepage would be 63,400 gallons per day. Nitrate concentrations in Lake Nacimiento are negligible, and no nitrate loading to the study area would be associated with lake seepage.

The cumulative inflow to the study area from the above sources is estimated to be 236,900 gallons per day with a total nitrogen mass of 218 pounds per day. During the initial years of expanded effluent discharge at the Kavanaugh site, the nitrate load would also be diluted by the existing ground water in storage within the study area. Over time, however, the nitrate concentration in ground water would rise until the amount of nitrogen mass flowing out of the study area was equal to the loading. Long-term outflow will be equivalent to the inflow, therefore, the long-term concentration can be calculated based on the information presented above.

Assuming the nitrogen mass was evenly distributed in the recharge water, the nitrate concentration in ground water in the study area is estimate to be 112 mg/l (as NO3). Not all the recharge water will mix evenly, and nitrate concentrations would likely be closer to the 180 mg/l concentration of the effluent beneath the discharge ponds, and less than the 112 mg/l average value farther away from the ponds and closer to Kavanaugh Creek.

Potential Surface Water Impacts

During normal conditions, ground water mounding beneath the discharge ponds will flow through the subsurface into the younger alluvium along Kavanagh Creek. Water quality of the underflow adjacent

to the site, with the exception of nitrate, has a greater salt content than the projected future inflow quality from the Kavanaugh disposal site. Nitrate loading, as discussed above, has the potential to drive nitrate concentration in ground water beneath the study area up to an estimated average value of 112 mg/l.

Operation of the disposal ponds should be managed such that there would be no daylighting of the ground water mound along the banks or dry wash of Kavanaugh Creek. There would be no direct contact between surface water and treated effluent. When lake levels are high, surface waters will seep into the alluvial terrace as bank storage, which will locally reverse the ground water flow direction away from the creek and toward the ponds. As lake levels recede, bank storage will drain back into the creek and hydraulic gradients will return to the normal condition. Mingling of bank storage water and the ground water mound would take place in the subsurface.

Treated effluent disposal at the Kavanaugh site would not increase the total dissolved solids concentration of ground water beneath the site, but it would increase the volume of subsurface outflow, thereby increasing salt loading to the reservoir. Assuming 145,000 gallons per day at 300 mg/l net TDS pickup (500 mg/l TDS added by effluent discharge minus 200 mg/l TDS removed by gallery well water supply), mass loading equivalent to approximately 363 pounds of mineral salts per day would be added to the watershed. On an annual basis, the mass loading would be approximately 66 tons.

Under steady-state conditions, mass loading from the Kavanaugh site would theoretically raise the salinity of the lake until an average of 66 additional tons of salt was being released through the dam on an annual basis. Average annual releases at the reservoir between 1951 and 1991 were 195,617 acre-feet, and in the recent wetter climate since the 1987-1991 drought, releases averaged closer to 230,000 acre-feet per year. At a nominal 200,000 acre-feet per year release through the dam, an increase in TDS of approximately 0.24 mg/l would result in 66 additional tons of mineral salts flowing downstream. Nitrate loading to the lake represents a fraction of the TDS load, and would be negligible from a standpoint of overall lake water quality.

Travel Time

Travel time between the ponds and Kavanaugh Creek are estimated to be in excess of six months. There was approximately two to three months delay between the onset of effluent discharges at the emergency ponds and observations of increasing nitrate concentrations at hydraulically downgradient monitoring wells MW2 and MW3. The horizontal distance between the emergency ponds and the monitoring wells measured 150 feet, resulting in an average horizontal subsurface travel time for nitrate solute of approximately 2 ft/day. Preliminary drawings of future pond configuration show the setback from Kavanaugh Creek for a 5-pond system would be close to 500 feet. Using the empirically measured solute velocity of approximately 2 ft/day, ground water could reach the creek underflow in just under 250 days. Flow model particle tracking for steady-state conditions at 145,000 gallons per day disposal indicates travel times as low as 180 days, however. The shorter travel times apply to water moving along the basal

sand, a flow path that would not have been measured historically due to the configuration of the prominent clay layer beneath the area of testing.

A subsurface residence time of at least six months would meet the criteria for a surface spreading project under the California Department of Health Services July 2003 draft guidelines for recharge reuse regulations. According to the guidelines, all the recharge water (wastewater) shall be retained underground for a minimum of six months prior to extraction for use as a drinking water supply, and shall not be extracted within 500 feet of a point of recharge. Note that during most years, the recharge water would remain underground as it flowed in Kavanaugh Creek underflow, extending the subsurface residence time underground. Only during high lake stages would mixing with bank storage water bring the underground residence time closer to six months.

CONCLUSIONS

The Kavanaugh site was evaluated for effluent disposal capacity and for potential water quality impacts at a 145,000 gallons per day disposal rate. Results of the investigation are summarized by the following conclusions:

1) The site can accept up to 145,000 gallons per day of effluent without the ground water mound daylighting into Kavanaugh Creek. A minimum of three ponds would be needed, and 5 ponds are recommended, averaging 16,500 square feet each. Mound-to-pond separation during normal operating conditions (lake levels below 790 feet) would be approximately 16 feet.

2) No significant impact on disposal capacity is expected from lake levels rising into Kavanaugh Creek, due to the barrier effect of bank storage. Lake rising will reduce mound-to-pond separation, although even with maximum lake levels, the site can maintain an 11-foot separation with 145,000 gpd of effluent disposal.

3) Disposal operations at the Kavanaugh site would not increase concentrations of total dissolved solids or key mineral salts, with the exception of nitrate, in underlying ground water or in Kavanaugh Creek underflow adjacent to the site.

4) Nitrate concentrations in effluent can be expected to reach 180 mg/l (after conversion from ammonia and organic nitrogen). The estimated impact of nitrate loading on underlying ground water would be to raise the average nitrate concentration to 112 mg/l, more than twice the drinking water standard of 45 mg/l.

5) Mass balance calculations show salt loading to Lake Nacimiento from 145,000 gallons per day treated effluent disposal would result in a long-term increase in surface water TDS of approximately 0.24 mg/l.

6) Travel time for percolated effluent between a 5-pond disposal system and Kavanaugh Creek would be at least six months.

REFERENCES

Ayers, R.S., and D. W. Westcott, 1985, Water Quality for Agriculture, FAO Irrigation and Drainage Paper 29 (rev. 1), Food and Agriculture Organization of the United Nations.

Boyle Engineering Corporation, September 1989, Oak Shores Effluent Disposal Study for the County of San Luis Obispo, California.

Fugro and Cleath & Associates, August 2002, Paso Robles Groundwater Basin Study, prepared for County of San Luis Obispo Public Works Department.

Law/Crandall, June 18, 1996, Report of Field Infiltration Testing, Oak Shores Effluent Disposal Site.

Law/Crandall, February 28, 1997, Report of Additional Field Infiltration Testing and Monitoring Well Installation, Oak Shores Effluent Disposal Site.

Law/Crandall, July 8, 1999, Report of Infiltration Assessment and Monitoring Well Installation, Oak Shores Effluent Disposal Site.

San Luis Obispo County Public Works Department, January 2001, Oak Shores Disposal Area Expansion Project.

Wallace Group, Revised October 2004, CSA-7A Wastewater Collection, Treatment, and Disposal System - Capacity Assessment for Services to Phase I, Tract 2162.

APPENDIX A

Soil Boring Logs

Oak Shores Kavanaugh Investigation

Date: November 21, 2005
Hole Number: TH-1
Elevation: 843
Geologist: S. Harris, Cleath & Associates
Drilling Company (Method): S/G Drilling (8 ¼-inch HSA)
Total Depth: 65 feet

Lithologic Log
Sample interval in feet below grade

Interval	Blow Counts	Description
4.5-6	4/4/6 (SPT)	**Sand with Silt (SP-SM);** trace gravel to 1", light yellowish brown (2.5Y 6/4), fine to medium grained sand, some white caliche veins, 10% silt, slightly moist.
9.5-11	6/9/9 (CAL)	**Sand with Silt (SW-SM);** trace fine gravel, light yellowish brown to light olive brown (2.5Y 6/4 – 5/4), fine to coarse grained sand, subangular to subrounded, 10% silt, slightly moist.
14.5-16	4/3/5 (SPT)	**Sand with Silt (SP-SM);** light yellowish brown (2.5Y 6/4) with white mottles and fine root holes, fine to medium grained sand, 10% silt, slightly moist.
19.5-21	15/13/16 (CAL)	**Silty Sand (SM);** pale yellow (2.5Y 7/4) with some white mottling, very fine to fine grained sand, medium dense, 30% silt, slightly moist.
24.5-26	8/6/8 (SPT)	**Sand (SW) and Sandy Silt (ML);** light yellowish brown (2.5Y 6/4), fine to coarse grained sand, fine sand with sandy silt in bottom 6", loose, 10% fines (60% fines in bottom).
29.5-31	20/15/19 (CAL)	**Sand (SW);** with gravel, light yellowish brown (2.5Y 6/4), fine to coarse grained sand, 15% gravel to ¾", mostly subrounded clasts, quartz and aphanitic fragments, slightly weathered resistant gray, red and yellow volcanic porphyries; fine sand with silt in bottom 6", slightly moist.
34.5-36	7/8/9 (SPT)	**Sand (SP);** light yellowish brown (2.5Y 6/4), fine grained with 1-2" sandy silt lenses.
39.5-41	10/13/14 (CAL)	**Sand (SW);** trace fine gravel, light yellowish brown (10YR 6/4), fine to coarse grained sand, wet, silty lenses in bottom ring.

TH-1 (continued)

Interval	Blow Counts	Description
44.5-46	6/5/6 (SPT)	**Silty Sand (SP-SM);** trace gravel, dark yelloish brown (10YR 4/4), gravel to ½", fine to medium sand, mostly fine, 20% silt, 10% gravel, blue gray volcanic porphyry, with weathered yellowish red plagioclase phenocrysts to 3mm, wet.
49.5-51	5/5/10 (CAL)	**Sandy Clay (CL);** yellowish brown (10YR 5/4), with grayish brown (10YR 5/2) mottling, soft, rolls to ¼" threads, 40% fine to medium sand, saturated. Water at 50 feet.
54.5-56	3/7/11 (SPT)	**Shale;** yellowish brown to dark yellowish brown (10YR 5/4 – 4/4), with strong brown (7.5YR 5/6) mottling, highly weathered as clay, trace fine sand, some fragments with oxidized surfaces, mostly soft, but some harder sections. Slow drilling at 58 feet.
59.5-61	42/50 for 3" (CAL)	**Sandy Shale;** yellowish brown (10YR 5/4) and strong brown.
64.5-65	50 for 3" (SPT)	Sandstone; greenish gray, fine grained, dense.

End of Hole

Oak Shores Kavanaugh Investigation

Date: November 21, 2005
Hole number: TH-2
Elevation: 827
Geologist: S. Harris, Cleath & Associates
Drilling Company (Method): S/G Drilling (8 ¼-inch HSA)
Total Depth: 65 feet

Lithologic Log
Sample interval in feet below grade

Interval	Blow Counts	Description
4.5-6	2/3/3 (SPT)	**Sand (SP);** trace silt, yellowish brown (10YR 5/4), fine roots, humid, fine grained.
9.5-11	6/6/9 (CAL)	**Silty Sand (SM) with Sandy Silt (ML);** light yellowish brown (2.5Y 6/4), fine sand, 15% silt, fine root holes with white caliche veins. Sandy silt (60% silt) in bottom 6", humid.
14.5-16	6/8/11 (CAL)	**Sand (SP);** trace silt, light yellowish brown (2.5Y 6/4); upper 6" inches fine to coarse grained sand, then sand with silt, fine to medium grained, mostly fine, humid.
19.5-21	4/6/7 (SPT)	**Silty Sand (SP-SM);** trace quartzite and sandstone gravel, light yellowish brown (2.5Y 6/4), mostly fine grained sand, 15% silt, damp.
24.5-26	11/10/13 (CAL)	**Sand (SP) and Silty Sand (SP-SM);** light yellowish brown (2.5Y 6/4), fine to medium sand to 25.5', then silty sand, 15% fines, yellowish brown (10YR 5/4), fine grained, damp.
29.5-31	4/4/8 (SPT)	**Sand (SP);** yellowish brown (10YR 5/4), fine to medium grained, trace gravel as light gray medium grained sandstone, trace silt, moist.
34.5-36	7/7/10 (CAL)	**Sand (SP) and Silty Sand (SM);** yellowish brown (10YR 5/4), fine to medium grained, little to no fines to 35.5 feet, then silty sand, fine grained, 15% silt, moist.
39.5-41	5/10/15 (SPT)	**Silty Sand (SW-SM);** trace gravel, yellowish brown (10YR 5/4), fine to coarse grained sand, water at 40', sandstone and chert gravel clasts to 1", 30% fines.
44.5-46	7/7/13 (CAL)	**Sand with Silt (SP-SM);** trace gravel, light yellowish brown (2.5Y 6/4), mostly medium to coarse grained sand with gravel to 1", subrounded sandstone/quartzite, volcanic porphyries, soft shale and chert clasts, saturated.

TH-2 (continued)

Interval	Blow Counts	Description
49.5-51	9/18/33 (SPT)	**Sand (SW);** light yellowish brown (2.5Y 6/4), little to no fines, fine to coarse grained quartz sand, subangular to subrounded, saturated.
54.5-56	50/3" (CAL)	**Sandstone;** light reddish brown (2.5YR 6/4) very fine to fine sand, with 10% silt, dense,
59.5-61	50/3" (CAL)	**Sandstone** in sboe, dense
64.5-65	100/2"	**Sandstone** in shoe, dense

End of Hole

Oak Shores Kavanaugh Investigation

Date: November 22, 2005
Hole Number: TH-3
Elevation: 831
Geologist: S. Harris, Cleath & Associates
Drilling Company (Method): S/G Drilling (8 ¼-inch HSA)
Total Depth: 45 feet

Lithologic Log
Sample interval in feet below grade

Interval	Blow Counts	Description
4.5-6	6/5/6 (SPT)	**Silty Sand (SP-SM);** yellowish brown (10YR 5/4), fine grained sand, 10% silt, fine root holes, humid.
9.5-11	5/8/9 (CAL)	**Silty Sand (SP-SM);** yellowish brown (10YR 5/4), fine grained sand, 30% silt, white caliche mottles/veins, humid.
14.5-16	3/3/3 (SPT)	**Silty Sand (SP-SM);** yellowish brown (10YR 5/4), fine grained sand, 10% silt, humid, white caliche mottles/veins to 15 feet; then silt increases to 30% with fine root holes.
19.5-21	6/7/13 (CAL)	**Sandy Silt (ML)**, yellowish brown (10YR 5/4) to light yellowish brown (2.5Y 6/4), 40% fine grained sand, abundant white caliche veins, humid.
24.5-26	4/5/6 (SPT)	**Silty Sand (SM)**, yellowish brown (10YR 5/4) to light yellowish brown (2.5Y 6/4), some fine root holes, with sand beginning at 25.5 feet, fine grained, trace silt, yellowish brown (10YR 5/4), trace black charcoal to 3mm, humid.
29.5-31	9/11/12 (CAL)	**Clayey Sand (SC);** dark yellowish brown (10YR 4/4), to 30', then silty sand, fine to coarse grained sand, yellowish brown (10YR 5/4), 40% fines, humid.
34.5-36	5/6/7 (SPT)	**Silty Sand (SM);** brown (7.5YR 5/4) to dark yellowish brown (10YR 4/4), mostly fine grained sand, 20% fines, humid.
39.5-41	12/46/50-3" (CAL)	**Shale;** dense, yellowish brown (10YR 5/4), 20% fine grained sand, dark green mottling along fracture planes, humid to damp.
44.5-45.5	20/150 (CAL)	**Silty Sandstone**, grayish brown (2.5Y 5/2), dense, humid to damp.

End of Hole

Oak Shores Kavanaugh Investigation

Date: November 22, 2005
Hole Number: TH-4
Elevation: 837
Geologist: S. Harris, Cleath & Associates
Drilling Company (Method): S/G Drilling (8 ¼-inch HSA)
Total Depth: 70 feet

Lithologic Log
Sample interval in feet below grade

Interval	Blow Counts	Description
4.5-6	2/3/5 (SPT)	**Sand with Silt (SP-SM);** yellowish brown (10YR 5/4), mostly fine grained sand, 10% silt, fine roots, humid.
9.5-11	9/11/12 (CAL)	**Sand (SP);** light yellowish brown (2.5Y 6/4), trace silt, mostly fine to medium grained sand, 1" dark brown sandy silt at 9.5 to 10', humid.
14.5-16	5/4/6 (SPT)	**Silty Sand (SM);** light yellowish brown (2.5Y 6/4), 20% silt, fine grained sand, fine roots, white caliche veins, humid.
19.5-21	11/12/17 (CAL)	**Silty Sand (SM);** light yellowish brown (2.5Y 6/4), 40% silt, fine grained quartz sand, trace fine gravel, subangular to subrounded, some fine roots, white caliche veins, humid.
24.5-26	7/8/11 (SPT)	**Silty Sand (SM);** light yellowish brown (2.5Y 6/4), 25% silt, mostly fine grained sand, humid.
29.5-31	12/17/25 (CAL)	**Silty Sand (SM);** pale yellow (2.5Y 7/4), very fine grained sand, 15% silt to 30.5', then mostly fine sandy silt in bottom 6", light yellowish brown (2.5Y 6/4), humid.
34.5-36	5/5/8 (SPT)	**Silty Sand (SM);** yellowish brown (10YR 5/4), fine grained sand, 25% silt, some white caliche veins, damp.
39.5-41	7/13/16 (CAL)	**Silty Sand (SM);** yellowish brown (10YR 5/4), fine grained sand, 25% silt, some white caliche veins, light yellowish brown (2.5Y 6/4), fine to medium grained, moist.
44.5-46	5/5/6 (SPT)	**Sand (SP);** trace silt, light yellowish brown (2.5Y 6/4) fine grained, moist.
49.5-51	4/4/6 (CAL)	**Sandy Clay (CL);** yellowish brown (10YR 5/4), 20% fine grained sand, soft clay, wet with free water at 50', clay rolls to ¼" threads.
54.5-56	2/4/5 (SPT)	**Sandy Clay (CL);** yellowish brown (10YR 5/4), 30% fine grained sand, soft.

TH-4 (continued)

Interval	Blow Counts	Description
59.5-61	3/3/7 (CAL)	**Sand with Silt (SP-SM);** yellowish brown (10YR 5/4), fine to medium sand, 10% fines.
64.5-66	No Sample	(former drilling at 66 feet)
69.5-70	25/50 for 5"	**Silty Sandstone,** olive (5Y 5/3) to greenish gray (Gley 5/5GY), fine grained.

End of Hole

APPENDIX B

Aquifer Testing Data

Pumping Test, MW4 - January 16 2006

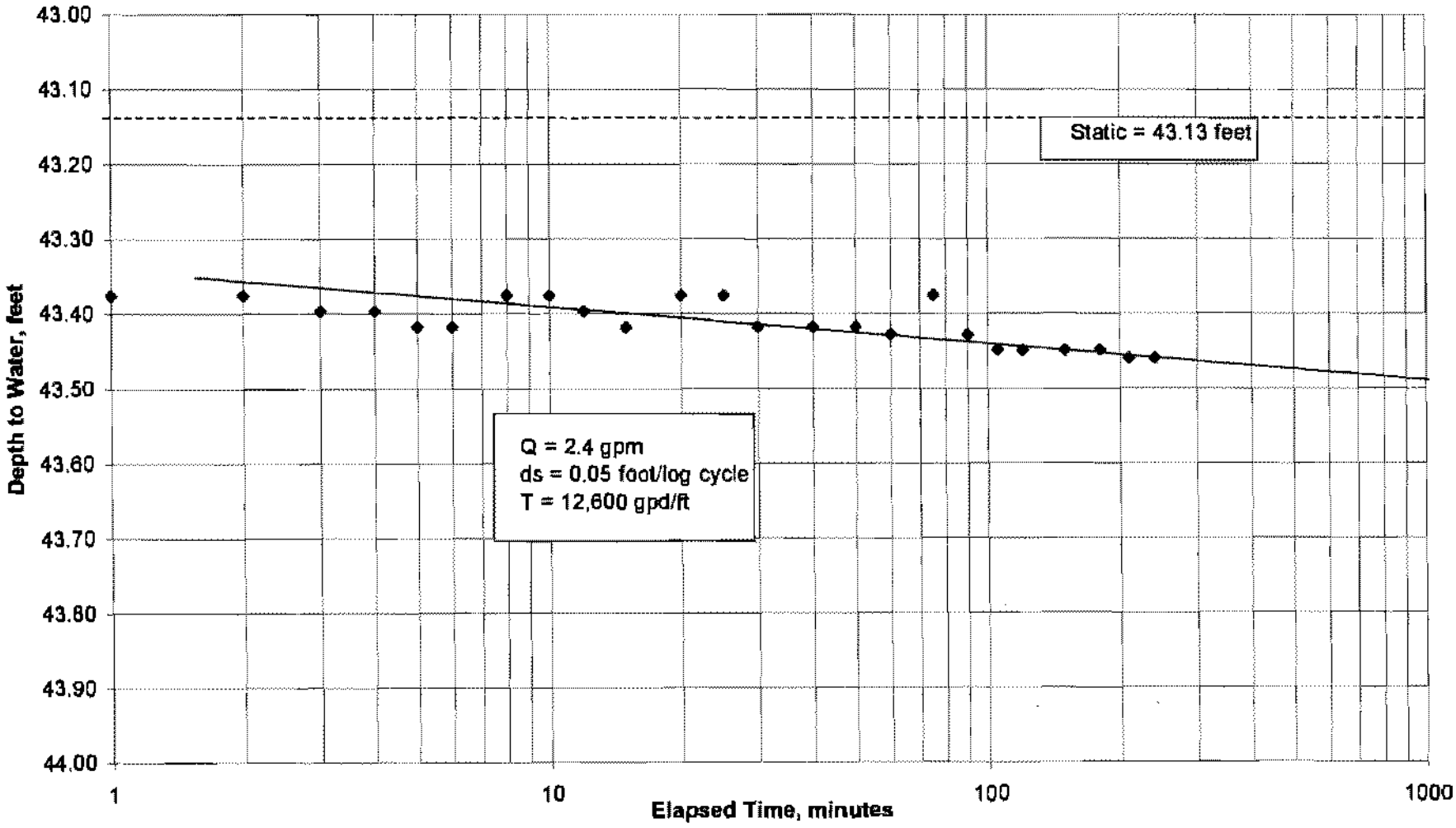
Day	Time	Elapsed Time		Depth to Water	Drawdown	Meter	Recorded Pumping Rate
Mo./Day/Yr	hr:min	hours:minutes	minutes	feet	feet	cubic feet	gallons per minute
1/16/2006	13:02	0	0	43.13	0		Start
	13:03	0:01	1	43.38	0.25		
	13:04	0:02	2	43.38	0.25		2.73
	13:05	0:03	3	43.40	0.27		
	13:06	0:04	4	43.40	0.27		
	13:07	0:05	5	43.42	0.29		
	13:08	0:06	6	43.42	0.29		2.5
	13:10	0:08	8	43.38	0.25		2.5
	13:12	0:10	10	43.38	0.25		
	13:14	0:12	12	43.40	0.27		2.44
	13:17	0:15	15	43.42	0.29		2.34
	13:22	0:20	20	43.38	0.25		2.38
	13:27	0:25	25	43.38	0.25		2.36
	13:32	0:30	30	43.42	0.29		2.38
	13:42	0:40	40	43.42	0.29		2.38
	13:52	0:50	50	43.42	0.29		2.36
	14:02	1:00	60	43.43	0.38		2.38
	14:17	1:15	75	43.38	0.25		2.34
	14:32	1:30	90	43.43	0.30		2.38
	14:47	1:45	105	43.45	0.32		2.36
	15:02	2:00	120	43.45	0.32		2.4
	15:32	2:30	150	43.45	0.32		2.4
	16:02	3:00	180	43.45	0.32		2.4
	16:32	3:30	210	43.46	0.33		2.42
	17:02	4:00	240	43.46	0.33		2.42
	STOP					Average	2.4

Recovery Well MW4

Day	Time	Elapsed Time		Depth to Water	Drawdown	Ratio
Mo./Day/Yr	hr:min	hours:minutes	minutes	feet	feet	t/t(o)
Recovery						
1/16/2006	17:02	4:00	0	43.46	0.33	
	17:03	4:01	1	43.19	0.05	76
	17:04	0:03	2	43.18	0.05	38.5
	17:05	4:03	3	43.17	0.05	26
	17:06	4:04	4	43.17	0.04	19.8
	17:07	4:05	5	43.17	0.04	16
	17:08	4:05	6	43.16	0.04	13.5
	17:10	4:08	8	43.15	0.03	10.4
	17:12	4:10	10	43.15	0.03	8.5
	17:14	4:12	12	43.15	0.02	7.3
	17:17	4:15	15	43.15	0.02	6
	17:22	4:20	20	43.15	0.02	4.8
	17:27	4:25	25	43.14	0.01	4
	17:32	4:30	30	43.14	0.01	3.5
	17:42	4:40	40	43.13	0.00	2.9
	17:52	4:50	50	43.13	0.00	2.5
	18:02	5:00	60	43.13	0.00	2.3
	STOP					

Pumping Test - Oak Shores Kavanugh MW4
January 16, 2006

Depth to Static Water Level: 43.13 feet **Pumping Rate: 2.4 gpm**



APPENDIX C

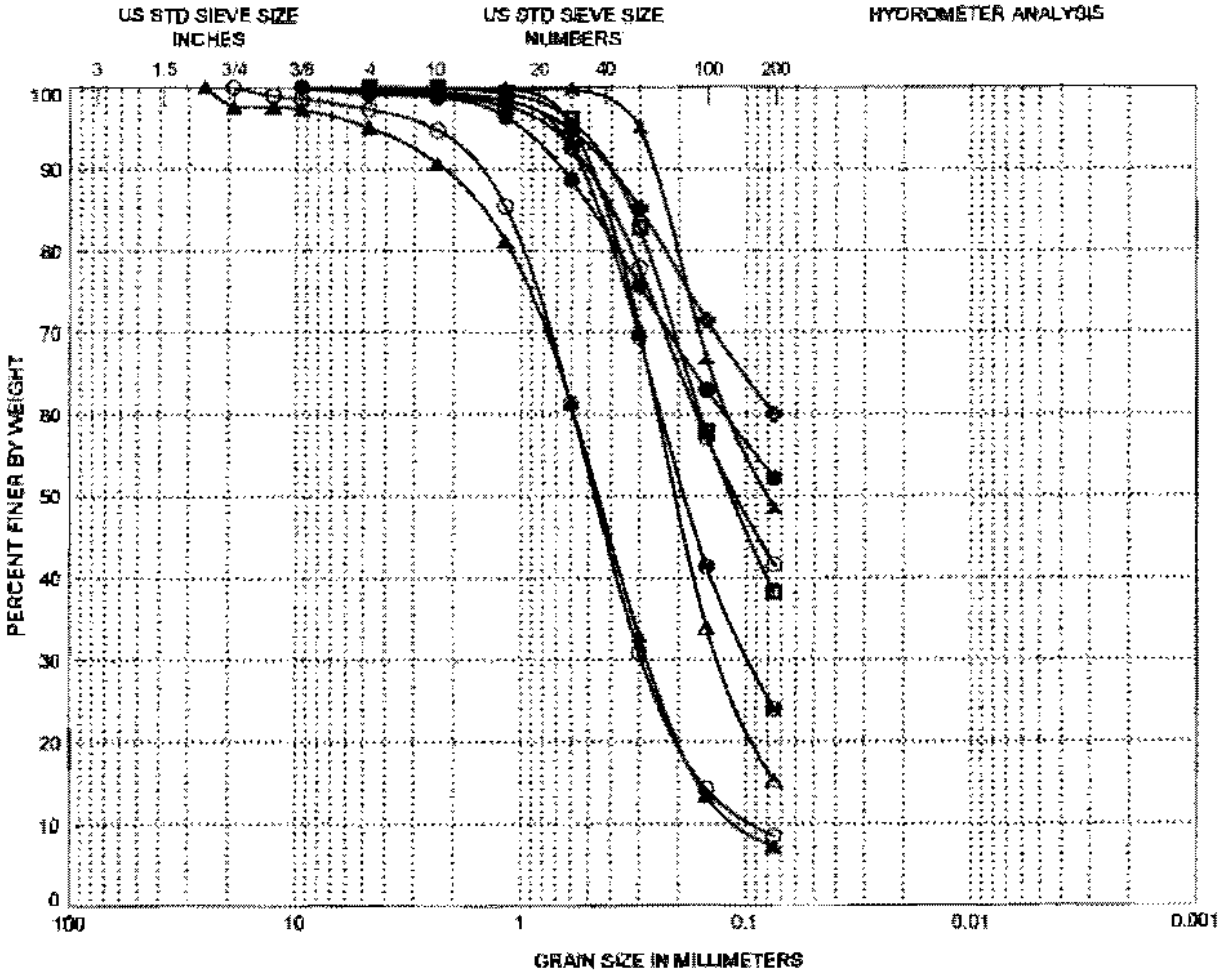
Laboratory Soil Testing

LABORATORY TEST SUMMARY

Project Name: Kavanaugh Expansion Site
Project Number: 3225.001
Date of Report: December 16, 2005
Attention: Spencer Harris

Test Hole No.	Sample Depth (ft)	Sample Classification (ASTM D2487)	Dry Density, pcf (ASTM D2937)	Moisture Content, % (ASTM D2937)	Permeability, cm/s (Constant or Falling Head)
TH-1	10.0	Well-graded SAND with silt (SW-SM)	100.5	4.0	3.9×10^{-3}
TH-1	50.0	Sandy CLAY (CL)	113.5	19.0	3.4×10^{-7}
TH-2	35.0	Silty SAND (SM)	97.5	5.3	4.3×10^{-3}
TH-2	45.0	SAND with silt (SP-SM)	118.8	11.3	2.4×10^{-3}
TH-3	10.0	Silty SAND (SM)	104.7	7.5	2.9×10^{-5}
TH-3	20.0	Sandy SILT (ML)	106.7	11.3	9.1×10^{-6}
TH-3	45.0	Silty SAND (SM)	125.1	13.6	1.6×10^{-5}
TH-4	20.0	Silty SAND (SM)	103.5	4.6	3.6×10^{-5}
TH-4	40.0	Silty SAND (SM)	105.4	4.9	8.5×10^{-4}

Cleath and Associates
Project No. 3225.001



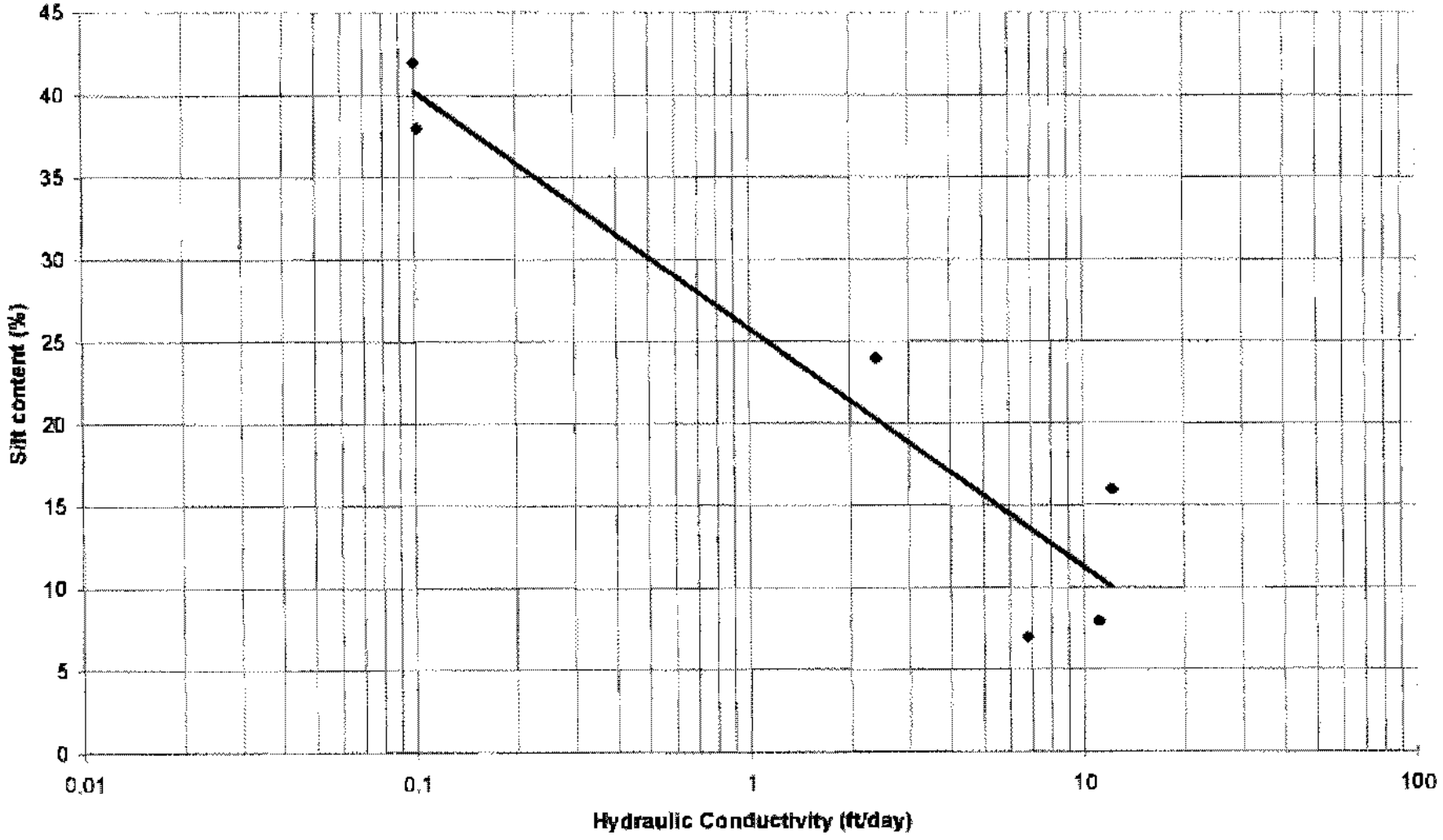
GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

	LEGEND (location)	(depth,ft)	CLASSIFICATION	Cc	Cu
○	TH-1	10.0	Well-graded SAND with silt (SW-SM)	1.5	6.5
●	TH-1	50.0	Sandy CLAY (CL)		
△	TH-2	35.0	Silty SAND (SM)		
▲	TH-2	45.0	SAND with silt (SP-SM)	1.2	5.7
⊙	TH-3	10.0	Silty SAND (SM)		
✿	TH-3	20.0	Sandy SILT (ML)		
⅄	TH-3	45.0	Silty SAND (SM)		
■	TH-4	20.0	Silty SAND (SM)		
⊛	TH-4	40.0	Silty SAND (SM)		

GRAIN SIZE CURVES
Kavanaugh Expansion Site
Oak Shores, California

PLATE 2

Permeability (K) vs Silt Content
Kavanaugh Disposal Site



-200 INPUT

Job No.:	3225.001	Client:	Cleath and Associates	Date	1/26/2006
Lab ID:		Job Name	Kavanough Expansion Site	Page 1	OF 1

SPECIMEN INFORMATION AND MEASUREMENTS

Boring	Sample No.	Depth (ft)	Soil Description	Tare No./ Weight (g)	Wet mass + Tare (g)	Dry Mass + Tare (g)	Initial Water content (%)	Initial Dry Mass of Soil (g)	Dry Mass of soil after washing and shaking over No. 200 Sieve (g)	Percent Passing No. 200 Sieve
TH-1		20	Silty SAND (SM)	74.85	285.52	275.49	5.0	200.64	132.81	33.8
TH-1		30	Sandy SILT (ML)	77.26	298.53	281.9	8.1	204.64	88.58	56.7
TH-1		40	Silty SAND (SM)	88.47	283.85	269.99	7.6	181.52	133.23	26.6
TH-2		10	Sandy SILT (ML)	94.83	291.36	272.06	10.9	177.23	63.02	64.4
TH-2		15	Silty SAND (SM)	89.02	398.64	371.82	9.5	282.8	167.06	40.9
TH-2		25	Silty SAND (SM)	86.74	364.87	351.35	5.1	264.61	220.9	16.5
TH-3		30	Silty SAND (SM)	113.53	337.4	316.64	10.2	203.11	113.61	44.1
TH-4		10	Silty SAND (SM)	138.95	464.72	450.81	4.5	311.86	244.43	21.6
TH-4		30	Sandy SILT (ML)	145.51	344.3	333.22	5.9	187.71	58.56	68.8
TH-4		60	Poorly-graded SAND with silt (SP-SM)	143.81	473.87	425.8	17.0	281.99	250.5	11.2
							#DIV/0!	0		#DIV/0!
							#DIV/0!	0		#DIV/0!
							#DIV/0!	0		#DIV/0!
							#DIV/0!	0		#DIV/0!
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							#DIV/0!	0		#DIV/0!
							#DIV/0!	0		#DIV/0!

Tested By:	NJD	Date:1/25/06	Computed By:	NJD	Checked By	CS	Date	1/26/2006

Average permebilties (K)
Kavanaugh Disposal Site
Layer 1

Sample Depth (feet)	Hydraulic conductivity TH1 (ft/day)	TH2 (ft/day)	TH3 (ft/day)	TH4 (ft/day)
5	12	12	12	12
10	12	0.4	0.1	0.1
15	12	0.1	2.5	2.5
20	0.4	5.5	0.026	0.1
25	0.4	5.5	2.5	1.1
30	0.4	12	0.1	0.1
35	0.1	5.5	2.5	1.1
40	0.4	0.4		1.1
45	2.5	12		12
50		12		
Geometric mean	1.31	2.98	0.65	0.92

Average vertical K 1.5 ft/day
horizontal K 12 ft/day

APPENDIX D

Water Quality Results

CREEK ENVIRONMENTAL LABORATORIES, INC.
A Minority-owned Business Enterprise
141 SUBURBAN ROAD, SUITE C-5 • SAN LUIS OBISPO, CA 93401 • (805) 545-9838 • FAX (805) 545-0107

Cleath & Associates
Dwain Davis
P.O. Box 184
Templeton, CA 93465

Log Number: 05-C14327
Order: M6345
Project: Kavanaugh
Received: 11/23/05

REPORT OF ANALYTICAL RESULTS

SAMPLE DESCRIPTION	SAMPLED BY	SAMPLED DATE @ TIME	MATRIX
TH1	S. Harris	11/22/05@14:15	Aqueous

ANALYTE	RESULT	DLR	UNITS	METHOD	ANALYZED
Total Alkalinity as CaCO3	440	2	mg/L	SM 2320B	12/05/05
Chloride	84	1	mg/L	EPA 300.0	11/23/05
Electrical Conductance	1,100	1	umhos/cm	SM 2510	11/23/05
Nitrate as N	0.8	0.1	mg/L	EPA 300.0	11/23/05
Nitrate as NO3	3.3	0.4	mg/L	EPA 300.0	11/23/05
pH	7.4	0.0	units	EPA 150.1	11/23/05
Sulfate	90	0.5	mg/L	EPA 300.0	11/23/05
Total Dissolved Solids	720	10	mg/L	EPA 160.1	11/29/05
Boron	0.28	0.05	mg/L	EPA 200.7	12/06/05
Calcium	80	0.03	mg/L	EPA 200.7	12/06/05
Hardness	410	1	mg/L CaCO3	EPA 200.7	12/06/05
Copper	Not Detected	0.05	mg/L	EPA 200.7	12/06/05
Iron	Not Detected	0.1	mg/L	EPA 200.7	12/06/05
Potassium	9.2	0.1	mg/L	EPA 200.7	12/06/05
Magnesium	51	0.03	mg/L	EPA 200.7	12/06/05
Manganese	0.44	0.02	mg/L	EPA 200.7	12/06/05
Sodium	92	0.05	mg/L	EPA 200.7	12/06/05
Zinc	Not Detected	0.05	mg/L	EPA 200.7	12/06/05

DLR = Detection Limit for Reporting. Results of "Not Detected" are below DLR.

CREEK ENVIRONMENTAL LABORATORIES

Lab Director, Michael Ng

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Cleath & Associates
Dwain Davis
P.O. Box 184
Templeton, CA 93465

Log Number: 05-C14328
Order: M6345
Project: Kavanaugh
Received: 11/23/05

REPORT OF ANALYTICAL RESULTS

SAMPLE DESCRIPTION	SAMPLED BY	SAMPLED DATE @ TIME	MATRIX
TH4	S. Harris	11/22/05@15:00	Aqueous

ANALYTE	RESULT	DLR	UNITS	METHOD	ANALYZED
Total Alkalinity as CaC03	360	2	mg/L	SM 2320B	12/05/05
Chloride	94	1	mg/L	EPA 300.0	11/23/05
Electrical Conductance	1,100	1	umhos/cm	SM 2510	11/23/05
Nitrate as N	1.5	0.1	mg/L	EPA 300.0	11/23/05
Nitrate as NO3	6.6	0.4	mg/L	EPA 300.0	11/23/05
pH	7.4	0.0	units	EPA 150.1	11/23/05
Sulfate	76	0.5	mg/L	EPA 300.0	11/23/05
Total Dissolved Solids	640	10	mg/L	EPA 160.1	11/29/05
Boron	0.26	0.05	mg/L	EPA 200.7	12/06/05
Calcium	71	0.03	mg/L	EPA 200.7	12/06/05
Hardness	300	1	mg/L CaCO3	EPA 200.7	12/06/05
Copper	Not Detected	0.05	mg/L	EPA 200.7	12/06/05
Iron	0.2	0.1	mg/L	EPA 200.7	12/06/05
Potassium	2.1	0.1	mg/L	EPA 200.7	12/06/05
Magnesium	30	0.03	mg/L	EPA 200.7	12/06/05
Manganese	Not Detected	0.02	mg/L	EPA 200.7	12/06/05
Sodium	100	0.05	mg/L	EPA 200.7	12/06/05
Zinc	Not Detected	0.05	mg/L	EPA 200.7	12/06/05

DLR = Detection Limit for Reporting. Results of "Not Detected" are below DLR.

CREEK ENVIRONMENTAL LABORATORIES

Lab Director, Michael Ng

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Dwain Davis
P.O. Box 184
Templeton, CA 93465

Log Number: 05-C14610
Order: M6439
Project: Kavanaugh
Received: 11/30/05

REPORT OF ANALYTICAL RESULTS

SAMPLE DESCRIPTION	SAMPLED BY	SAMPLED DATE @ TIME	MATRIX
MW4	S. Harris	11/29/05@14:30	Aqueous

ANALYTE	RESULT	DLR	UNITS	METHOD	ANALYZED
Total Alkalinity as CaCO3	420	2	mg/L	SM 2320B	12/05/05
Chloride	180	20	mg/L	EPA 300.0	12/01/05
Electrical Conductance	1,600	1	umhos/cm	SM 2510	11/30/05
Nitrate as N	0.5	0.1	mg/L	EPA 300.0	11/30/05
Nitrate as NO3	2.2	0.4	mg/L	EPA 300.0	11/30/05
pH	7.0	0.0	units	EPA 150.1	11/30/05
Sulfate	220	0.5	mg/L	EPA 300.0	11/30/05
Total Dissolved Solids	1,000	10	mg/L	EPA 160.1	12/05/05
Boron	0.29	0.05	mg/L	EPA 200.7	12/07/05
Calcium	140	0.03	mg/L	EPA 200.7	12/07/05
Hardness	550	1	mg/L CaCO3	EPA 200.7	12/07/05
Copper	Not Detected	0.05	mg/L	EPA 200.7	12/07/05
Iron	8.1	0.1	mg/L	EPA 200.7	12/07/05
Potassium	2.9	0.1	mg/L	EPA 200.7	12/07/05
Magnesium	51	0.03	mg/L	EPA 200.7	12/07/05
Manganese	0.20	0.02	mg/L	EPA 200.7	12/07/05
Sodium	150	0.05	mg/L	EPA 200.7	12/07/05
Zinc	0.09	0.05	mg/L	EPA 200.7	12/07/05

DLR = Detection Limit for Reporting. Results of "Not Detected" are below DLR.

CREEK ENVIRONMENTAL LABORATORIES

Lab Director, Michael Ng

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A Minority-owned Business Enterprise
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Dwain Davis
P.O. Box 184
Templeton, CA 93465

Log Number: 05-C14611
Order: M6439
Project: Kavanaugh
Received: 11/30/05

REPORT OF ANALYTICAL RESULTS

SAMPLE DESCRIPTION	SAMPLED BY	SAMPLED DATE @ TIME	MATRIX
MW5	S. Harris	11/29/05@17:45	Aqueous

ANALYTE	RESULT	DLR	UNITS	METHOD	ANALYZED
Total Alkalinity as CaCO3	460	2	mg/L	SM 2320B	12/05/05
Chloride	69	1	mg/L	EPA 300.0	11/30/05
Electrical Conductance	1,900	1	umhos/cm	SM 2510	11/30/05
Nitrate as N	Not Detected	0.1	mg/L	EPA 300.0	11/30/05
Nitrate as NO3	Not Detected	0.4	mg/L	EPA 300.0	11/30/05
pH	7.1	0.0	units	EPA 150.1	11/30/05
Sulfate	620	10	mg/L	EPA 300.0	12/01/05
Total Dissolved Solids	1,400	10	mg/L	EPA 160.1	12/05/05
Boron	0.50	0.05	mg/L	EPA 200.7	12/07/05
Calcium	140	0.03	mg/L	EPA 200.7	12/07/05
Hardness	810	1	mg/L CaCO3	EPA 200.7	12/07/05
Copper	Not Detected	0.05	mg/L	EPA 200.7	12/07/05
Iron	2.3	0.1	mg/L	EPA 200.7	12/07/05
Potassium	11	0.1	mg/L	EPA 200.7	12/07/05
Magnesium	110	0.03	mg/L	EPA 200.7	12/07/05
Manganese	0.19	0.02	mg/L	EPA 200.7	12/07/05
Sodium	130	0.05	mg/L	EPA 200.7	12/07/05
Zinc	0.14	0.05	mg/L	EPA 200.7	12/07/05

DLR = Detection Limit for Reporting. Results of "Not Detected" are below DLR.

CREEK ENVIRONMENTAL LABORATORIES

Lab Director, Michael Ng

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CREEK ENVIRONMENTAL LABORATORIES, INC.
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Dwain Davis
P.O. Box 184
Templeton, CA 93465

Log Number: 05-C14612
Order: M6439
Project: Kavanaugh
Received: 11/30/05

REPORT OF ANALYTICAL RESULTS

SAMPLE DESCRIPTION	SAMPLED BY	SAMPLED DATE & TIME	MATRIX
Kavanaugh underflow	S. Harris	11/29/05@17:57	Aqueous

ANALYTE	RESULT	DLR	UNITS	METHOD	ANALYZED
Total Alkalinity as CaCO3	780	2	mg/L	SM 2320B	12/05/05
Chloride	780	20	mg/L	EPA 300.0	12/01/05
Electrical Conductance	3,800	1	umhos/cm	SM 2510	11/30/05
Nitrate as N	Not Detected	0.1	mg/L	EPA 300.0	11/30/05
Nitrate as NO3	Not Detected	0.4	mg/L	EPA 300.0	11/30/05
pH	6.9	0.0	units	EPA 150.1	11/30/05
Sulfate	380	10	mg/L	EPA 300.0	12/01/05
Total Dissolved Solids	2,500	10	mg/L	EPA 160.1	12/05/05
Boron	0.20	0.05	mg/L	EPA 200.7	12/07/05
Calcium	420	0.03	mg/L	EPA 200.7	12/07/05
Hardness	1,400	1	mg/L CaCO3	EPA 200.7	12/07/05
Copper	Not Detected	0.05	mg/L	EPA 200.7	12/07/05
Iron	0.8	0.1	mg/L	EPA 200.7	12/07/05
Potassium	6.2	0.1	mg/L	EPA 200.7	12/07/05
Magnesium	74	0.03	mg/L	EPA 200.7	12/07/05
Manganese	6.7	0.02	mg/L	EPA 200.7	12/07/05
Sodium	260	0.05	mg/L	EPA 200.7	12/07/05
Zinc	Not Detected	0.05	mg/L	EPA 200.7	12/07/05

DLR = Detection Limit for Reporting. Results of "Not Detected" are below DLR.

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Appendix B

Water Balance Calculations

Kavanaugh Effluent Disposal Ponds Water Balance

Introduction

In order to determine the capacity of the proposed expansion of the Kavanaugh percolation ponds while including the capacity of the existing spray fields, the water balance methodology used by GTA in the 2005 Capacity and Demand Report was adopted by this report to determine the number of new connections that could be added with various full-time occupancy rates and the resulting affect on effluent storage requirements and nitrogen loading. The 2005 Capacity and Demand Report estimated the capacity of the existing spray fields by using historical wastewater influent flow data in combination with data on discharges to the existing emergency percolation ponds at the Kavanaugh site.

The 2005 Capacity and Demand Report determined the existing spray field capacity to be approximately 40 acre-ft per year. This spray field capacity was included as a given in the water balances generated for this report by using the exact same methodology and input variables such as precipitation and evaporation, as the 2005 Capacity and Demand Report. The assumptions, a description of the methodology, and the background calculations for the water balances used in this report can be found in the 2005 Capacity and Demand Report which is included in Appendix D.1. The only changes that were made to the 2005 Capacity and Demand Report water balance format to enable analysis of future development and occupancy scenarios, were (1) the use of slightly different design flow rate per connection due to the removal of various inaccuracies in the historical flow data as well as (2) the method of calculating the capacity of the proposed percolations ponds. These modifications are explained below.

Water balance analyses were conducted for two development scenarios each assuming a reasonable future full-time occupancy of 40%: one for build-out of phases 2 through 6 of Tract 2162 which results in 1,197 total dwelling units and another for 1,313 dwelling units. The water balance analysis for the proposed spray field on Lynch Canyon Properties (Hughes) property west of Kavanuagh Creek, which is discussed to follow, showed a capacity to dispose of effluent from 400 dwelling units. The scenario for 1,313 total dwelling units was analyzed since the difference of the total proposed developments (1,713 dwelling units) and the Lynch Canyon Properties (Hughes) spray field capacity (400 dwelling units) is 1,313 dwelling units. The required effluent storage for each of these scenarios was determined.

Input Data

<u>Design Flow</u> - For this analysis, the design flow used is the combined ADMMF from full-time units (162 gpd/conn) and the ADMMF from seasonal units which varies depending on the calendar month which can be found in Table 3-5 in the body of this report.

<u>Percolation Pond Bottom Area</u> – The area of the bottom of the percolation ponds. The pond bottom area is the area through which the effluent will percolate. Any percolation through the berm slopes is not considered.

Percolation Rate for Kavanaugh Ponds – The percolation rate that was measured at the Kavanaugh percolation pond site. The average infiltration rate for the Kavanaugh site was determined by Law & Crandall to be 1.7 ft/day.

Percolation Pond Loading Rate – The obtainable infiltration rate for the percolation ponds during normal operation over long periods of time. The *Process Design Manual: Land Treatment of Municipal Waste Water* report from the EPA suggests using a loading rate of 10-15 percent of the minimum measured basin infiltration rate to account for drying cycles, clogging, and soil variability. A loading rate of 15 percent will be used for this report which results in is 0.26 ft/day.

Calculations

Combined Maximum Monthly Flow per Conn $\left(\frac{\text{gpd}}{\text{conn}}\right)$ =

$$MaxFlowPerFullTimeConn\left(\frac{\text{gpd}}{\text{conn}}\right) \times FullTimeOcc + MaxFlowPerSeasonalConn \times \left(1 - FullTimeOcc\right)$$

Perc Pond Max Perc Based on Perc Rate (acre-ft) =

$$MeasuredPercRate\left(\frac{\text{ft}}{\text{day}}\right) \times PercDesignFactor \times \#Days \times PondBottomArea(\text{acres})$$

WATER BALANCE FOR PROPOSED KAVANAUGH EFFLUENT DISPOSAL PONDS

CSA-7A WASTEWATER SYSTEM CAPACITY STUDY
INTERIM REPORT

Assumptions

Areas Contributing to Evapotranspiration

1.3 Sprayfield area (ac)

Areas Contributing to Pond Evaporation

1.4 Sprayfield ponds surface area (ac)

0.99 WWTP Pond Surface Area (ac)

Areas Contributing Precipitation Runoff

2 Percolation Pond Surface Catchment Area (ac)

3.8 Sprayfield Ponds Catchment Area (ac)

1.09 Treatment Plant Catchment (ac) (Ponds +10%)

Areas Contributing to Percolation

~~0.78~~ ~~Percolation Pond Surface Area (ac)~~

~~0.4~~ ~~Percolation Pond infiltration rate (ft/day)~~

0.08537 Spray Field infiltration rate (ft/day)

1313 Number of connections

40% Full-Time Occupancy

Kavanaugh Perc Ponds		
Perc Pond Bottom Area[1]	**1.90**	**acres**
Measured Percolation Rate[2]	1.7	ft/day
Percolation Design Factor[3]	0.15	- -
Design Percolation Rate	**0.26**	**ft/day**
Maximum Loading at Kav site before Daylighting[4]	145,000	gpd on an annual basis

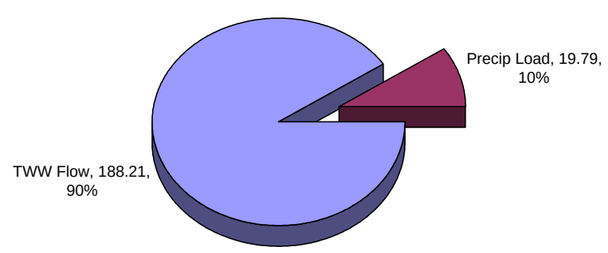
Water Balance

		Demand						Precipitation		Evapotranspiration				Evaporation		Percolation						Storage	
Month	Days per Month	Maximum Monthly Flow per Full-Time Connection (gpd/conn)	Maximum Monthly Flow per Seasonal Connection (gpd/conn)	**Combined Maximum Monthly Flow per Connection (gpd/conn)**	TWW Flow (gpd)	TWW Flow (gal)	**TWW Flow (acre-ft)**	Feb 100-year Monthly Precip (in)	**Precip Load (acre-ft)**	Pan Evap (in)	ETo/ETp Ratio	ETo (in)	**ETo (acre-ft)**	Reservoir Evap (in)	**Sprayfield and WWTP Reservoir Evap (acre-ft)**	Max Perc under Spray Fields (acre-ft)	Applied to Spray Fields (acre-ft)	**Perc Under Spray Fields (acre-ft)**	TWW Flow to Perc or Storage (acre-ft)	Perc Pond Max Perc based on perc rate (acre-ft)	**WWTP Direct Flow to Perc (acre-ft)**	Net Flow to or from Storage (acre-ft)	Cumulative Storage (acre-ft)
October	31	162	60.8	101.3	132,981	4,122,400	12.65	0.76	0.44	5.23	0.63	3.29	0.36	3.66	0.73	3.44	12.36	3.44	8.56	15.02	8.56	0.00	0.00
November	30	162	55.1	97.9	128,490	3,854,705	11.83	1.74	1.00	2.72	0.82	2.23	0.24	1.90	0.38	3.33	12.45	3.33	8.88	14.54	8.88	0.00	0.00
December	31	162	48.9	94.1	123,606	3,831,780	11.76	4.40	2.53	1.73	0.82	1.42	0.15	1.21	0.24	3.44	14.05	3.44	10.45	15.02	10.45	0.00	0.00
January	31	162	66.1	104.5	137,156	4,251,835	13.05	4.62	2.65	1.65	0.92	1.52	0.16	1.16	0.23	3.44	15.47	3.44	11.87	15.02	11.87	0.00	0.00
February	28.25	162	116.0	134.4	176,467	4,985,198	15.30	17.00	9.76	2.32	0.84	1.95	0.21	1.62	0.32	3.14	24.74	3.14	21.39	13.69	13.69	7.70	7.70
March	31	162	113.2	132.7	174,261	5,402,102	16.58	3.22	1.85	3.94	0.74	2.92	0.32	2.76	0.55	3.44	17.88	3.44	14.12	15.02	14.12	-0.90	6.81
April	30	162	121.0	137.4	180,406	5,412,186	16.61	1.83	1.05	5.63	0.59	3.32	0.36	3.94	0.78	3.33	16.88	3.33	13.19	14.54	13.19	-1.35	5.46
May	31	162	159.1	160.3	210,421	6,523,063	20.02	0.50	0.29	7.95	0.76	6.04	0.65	5.57	1.11	3.44	19.20	3.44	15.10	15.02	15.02	0.08	5.54
June	30	162	125.4	140.0	183,873	5,516,176	16.93	0.13	0.07	9.96	0.62	6.18	0.67	6.97	1.39	3.33	15.62	3.33	11.62	14.54	11.62	-2.92	2.63
July	31	162	162.0	162.0	212,706	6,593,886	20.24	0.02	0.01	11.46	0.72	8.25	0.89	8.02	1.60	3.44	18.65	3.44	14.32	15.02	14.32	-0.70	1.92
August	31	162	133.6	145.0	190,332	5,900,307	18.11	0.04	0.02	10.60	0.59	6.25	0.68	7.42	1.48	3.44	16.65	3.44	12.54	15.02	12.54	-1.92	0.00
September	30	162	100.6	125.2	164,335	4,930,052	15.13	0.20	0.11	7.83	0.71	5.56	0.60	5.48	1.09	3.33	14.15	3.33	10.22	14.54	10.22	0.00	0.00
Total	365.25					61,323,691	188.21	34.46	19.79	71.02		48.93	5.30	49.71	9.90	40.54	198.09	40.54	152.26	176.96	144.47		
Total (gpd)							167,895		17,650				4,729		8,833			36,161		157,864	**128,876**		
Average											0.73												**7.70**
Maximum																							6,872

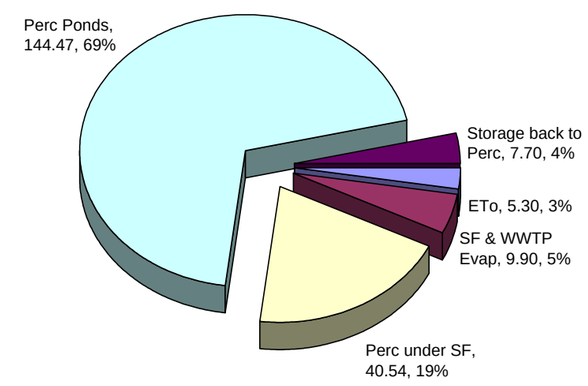
Notes:

1. Pond Bottom Area is the area of the bottom of the pond through which percolation occurs
2. Based on 2001 Oak Shores Disposal Area Expansion Project by SLO County and Law & Crandall Field Infiltration Testing Reports
3. U.S. EPA. Land Treatment of Municipal Wastewater. October 1981. pg 5-12 to 5-14 and
Water Environment Federation Manual of Practice No. 8. Design of Municipal Treatment Plants Volume II. 1992. pg 858-859 (function of soil variability, type of infiltration rate field test, and drying periods)
4. Cleath & Associates Hydrogeological Investigation of the Kavanaugh Effluent Disposal Site

What needs to be Disposed of (acre-ft)



Where the Effluent is Going (acre-ft)



WATER BALANCE FOR PROPOSED KAVANAUGH EFFLUENT DISPOSAL PONDS

CSA-7A WASTEWATER SYSTEM CAPACITY STUDY
INTERIM REPORT

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1.3 Sprayfield area (ac)

Areas Contributing to Pond Evaporation

1.4 Sprayfield ponds surface area (ac)

0.99 WWTP Pond Surface Area (ac)

Areas Contributing Precipitation Runoff

2 Percolation Pond Surface Catchment Area (ac)

3.8 Sprayfield Ponds Catchment Area (ac)

1.09 Treatment Plant Catchment (ac) (Ponds +10%)

Areas Contributing to Percolation

~~0.78~~ ~~Percolation Pond Surface Area (ac)~~

~~0.4~~ ~~Percolation Pond infiltration rate (ft/day)~~

0.08537 Spray Field infiltration rate (ft/day)

1197 Number of connections

40% Full-Time Occupancy

Kavanaugh Perc Ponds		
Perc Pond Bottom Area[1]	**1.90**	**acres**
Measured Percolation Rate[2]	1.7	ft/day
Percolation Design Factor[3]	0.15	- -
Design Percolation Rate	**0.26**	**ft/day**
Maximum Loading at Kav site before Daylighting[4]	145,000	gpd on an annual basis

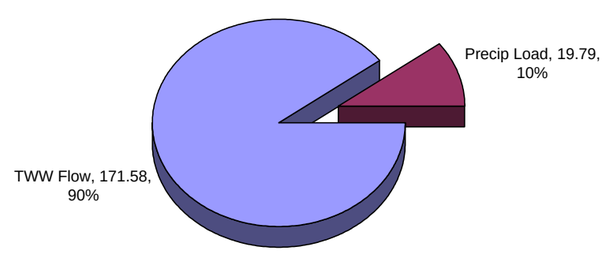
Water Balance

		Demand						Precipitation		Evapotranspiration				Evaporation		Percolation						Storage	
Month	Days per Month	Maximum Monthly Flow per Full-Time Connection (gpd/conn)	Maximum Monthly Flow per Seasonal Connection (gpd/conn)	**Combined Maximum Monthly Flow per Connection (gpd/conn)**	TWW Flow (gpd)	TWW Flow (gal)	**TWW Flow (acre-ft)**	Feb 100-year Monthly Precip (in)	**Precip Load (acre-ft)**	Pan Evap (in)	ETo/ETp Ratio	ETo (in)	**ETo (acre-ft)**	Reservoir Evap (in)	**Sprayfield and WWTP Reservoir Evap (acre-ft)**	Max Perc under Spray Fields (acre-ft)	Applied to Spray Fields (acre-ft)	**Perc Under Spray Fields (acre-ft)**	TWW Flow to Perc or Storage (acre-ft)	Perc Pond Max Perc based on perc rate (acre-ft)	**WWTP Direct Flow to Perc (acre-ft)**	Net Flow to or from Storage (acre-ft)	Cumulative Storage (acre-ft)
October	31	162	60.8	101.3	121,232	3,758,197	11.53	0.76	0.44	5.23	0.63	3.29	0.36	3.66	0.73	3.44	11.24	3.44	7.44	15.02	7.44	0.00	0.00
November	30	162	55.1	97.9	117,138	3,514,153	10.79	1.74	1.00	2.72	0.82	2.23	0.24	1.90	0.38	3.33	11.41	3.33	7.83	14.54	7.83	0.00	0.00
December	31	162	48.9	94.1	112,686	3,493,253	10.72	4.40	2.53	1.73	0.82	1.42	0.15	1.21	0.24	3.44	13.01	3.44	9.41	15.02	9.41	0.00	0.00
January	31	162	66.1	104.5	125,039	3,876,197	11.90	4.62	2.65	1.65	0.92	1.52	0.16	1.16	0.23	3.44	14.32	3.44	10.71	15.02	10.71	0.00	0.00
February	28.25	162	116.0	134.4	160,877	4,544,770	13.95	17.00	9.76	2.32	0.84	1.95	0.21	1.62	0.32	3.14	23.39	3.14	20.04	13.69	13.69	6.35	6.35
March	31	162	113.2	132.7	158,866	4,924,841	15.11	3.22	1.85	3.94	0.74	2.92	0.32	2.76	0.55	3.44	16.41	3.44	12.66	15.02	12.66	-2.36	3.99
April	30	162	121.0	137.4	164,468	4,934,034	15.14	1.83	1.05	5.63	0.59	3.32	0.36	3.94	0.78	3.33	15.41	3.33	11.72	14.54	11.72	-2.82	1.18
May	31	162	159.1	160.3	191,831	5,946,768	18.25	0.50	0.29	7.95	0.76	6.04	0.65	5.57	1.11	3.44	17.43	3.44	13.33	15.02	13.33	-1.18	0.00
June	30	162	125.4	140.0	167,628	5,028,836	15.43	0.13	0.07	9.96	0.62	6.18	0.67	6.97	1.39	3.33	14.12	3.33	10.12	14.54	10.12	0.00	0.00
July	31	162	162.0	162.0	193,914	6,011,334	18.45	0.02	0.01	11.46	0.72	8.25	0.89	8.02	1.60	3.44	16.86	3.44	12.53	15.02	12.53	0.00	0.00
August	31	162	133.6	145.0	173,517	5,379,031	16.51	0.04	0.02	10.60	0.59	6.25	0.68	7.42	1.48	3.44	15.05	3.44	10.94	15.02	10.94	0.00	0.00
September	30	162	100.6	125.2	149,817	4,494,496	13.79	0.20	0.11	7.83	0.71	5.56	0.60	5.48	1.09	3.33	12.82	3.33	8.89	14.54	8.89	0.00	0.00
Total	365.25				55,905,909	171.58	34.46	19.79	71.02		48.93	5.30	49.71	9.90		40.54	181.47	40.54	135.63	176.96	129.28		
Total (gpd)						153,062		17,650				4,729		8,833				36,161		157,864	**115,323**		
Average											0.73												
Maximum																						**6.35**	
																						5,667	

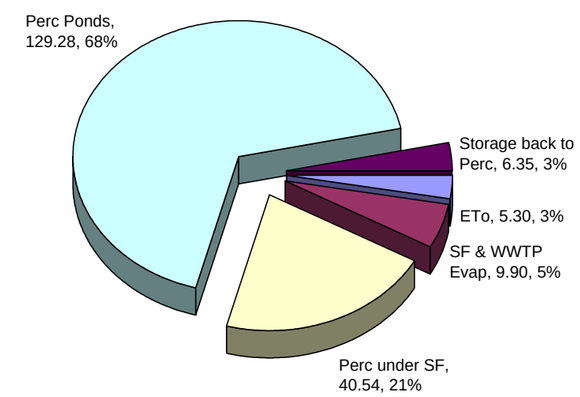
Notes:

1. Pond Bottom Area is the area of the bottom of the pond through which percolation occurs
2. Based on 2001 Oak Shores Disposal Area Expansion Project by SLO County and Law & Crandall Field Infiltration Testing Reports
3. U.S. EPA. Land Treatment of Municipal Wastewater. October 1981. pg 5-12 to 5-14 and
Water Environment Federation Manual of Practice No. 8. Design of Municipal Treatment Plants Volume II. 1992. pg 858-859 (function of soil variability, type of infiltration rate field test, and drying periods)
4. Cleath & Associates Hydrogeological Investigation of the Kavanaugh Effluent Disposal Site

What needs to be Disposed of (acre-ft)



Where the Effluent is Going (acre-ft)



Lynch Canyon Properties (Hughes)
Spray Field Water Balance

Introduction

The above water balance determined the capacity of the proposed disposal system at the Kavanaugh site and the resulting storage requirements for different development and occupancy scenarios. This second water balance analysis was conducted to determine the capacity of approximately 10 acres of supplemental spray fields proposed to be located on Lynch Canyon Properties (Hughes) property on the west side of the Kavanaugh drywash. This additional 10 acre spray field site would also include area for effluent storage to attenuate the peak flows from 100-yr storm events and high demand weekends. The results of this water balance analysis showed that with approximately 10 acres of spray fields along with approximately 10 AF of effluent storage could dispose of the effluent from 400 dwelling units at 40 percent full-time occupancy.

Methodology

The determination of capacity and proper sizing of the effluent storage facilities required during the design year depends on a number of input parameters. Certain values are fixed for a given project site, such as average infiltration rates for the soils as well as 100-year precipitation and evaporation. Other variables include average daily wastewater flows, spray disposal field size, storage pond size and geometry. A basic equation for analyzing the spray field capacity and the associated effluent storage requirements can be summarized as follows:

$$FLOW_{IN} = FLOW_{OUT} + STORAGE$$

$FLOW_{IN}$ - is composed of (1) ADMMF wastewater flows, and (2) precipitation falling into the storage ponds.

$FLOW_{OUT}$ - is composed of (1) evapotranspiration and percolation taking place at the spray disposal field, (2) evaporative losses from the surfaces of the storage ponds, as well as (3) spray losses from the spray field spray nozzles.

ADMMF wastewater flows were adjusted to account for the historical and estimated future seasonal occupancy of Oak Shores. Precipitation is highest during the fall and winter months when the evapotranspiration rates are typically at their lowest. It is during these months that storage facilities are required, since during these wet months, discharges to the spray disposal field are reduced due to saturated field conditions. The magnitude of the required effluent storage is lower for the Oak Shores community than it would be for most communities since the occupation is also lowest in the winter months which is reflected in the wastewater flows. A computer spreadsheet was developed to calculate each of the components of the “water balance” equation described above, evaluating the net storage required on a month to month basis beginning with October, which is the start of the rainy season.

Input Data

Design Flow - For this analysis, the design flow used is the combined ADMMF from full-time units (162 gpd/conn) and the ADMMF from seasonal units which varies depending on the calendar month which can be found in Table 3-5 in the body of this report.

Pond Mid Area - The area of the water surface when the water surface level is midway between the maximum allowable water height and the bottom of the pond. Pond mid-area is used for calculating the amount of water evaporating from the surfaces of the storage ponds. Evaporation amounts calculated from pond mid areas are conservative since the pond mid area is smaller than the area of the water surface at higher water levels during normal operation.

Pond Catch Area - The area of the pond defined by the top of the berm. All rain that falls within this area will enter into the pond. The area outside of this boundary is or will be graded to drain away from the pond.

Estimated Field Area - The area, in acres, of the spray disposal field. For this analysis, a field size of 10 acres was used. This size is conceptual in nature and no specific site feasibility study has been conducted.

Spray Disposal Field Design Percolation Rate - The percolation rate that is obtainable at the spray disposal field site. A value of 0.1 in/hr or 0.2 ft/day was chosen based on general SCS soil permeability data for the proposed spray field site. Actual in-situ infiltration test would need to be performed to determine the actual site specific infiltration rate.

Allowable Spray Field Percolation – When effluent is applied to the spray fields in excess of the crop requirements, deep percolation of the effluent results. There is a maximum amount of deep percolation that can be allowed and still provide a marketable crop or prevent the creation of nuisance conditions and management problems. The report: *Irrigation with Reclaimed Municipal Wastewater* from the California State Water Resources Control Board establishes this maximum daily percolation rate in the range of 4% to 6% of the minimum soil profile permeability. A value of 5% was chosen for this water balance analysis.

Runoff Coefficient – The ratio of runoff to precipitation. The total volume of stormwater runoff for each month was calculated using SCS methods. The detailed analysis was performed considering the following factors:

- SCS Soil characteristics
- Proposed land use at disposal site
- Projected soil moisture conditions
- Estimated irrigation and precipitation events

The average “runoff coefficients” are summarized in on the water balance spreadsheet.

100-Year Precipitation – The amount of annual rainfall that is equaled or exceeded once, on the average, every 100 years. The annual 100-year precipitation from data collected at the DWR, Nacimiento Dam weather station no. T 09 6056 is 33.58 inches. The DWR frequency analysis for the Nacimiento Dam station is included at the end of the water

balance analysis. The County of San Luis Obispo also provided precipitation data from the County's weather station #201 at the Oak Shores wastewater treatment plant which is provided at the end of this water balance analysis as well. The County precipitation data spans from 1975 to 2004, however there is an 8 year gap in the data making proper frequency analysis of the data to determine the annual 100-year precipitation impossible. The County station #201 shows an average annual precipitation of 20.34 inches compared to 14.22 inches at the DWR Nacimiento Dam station. Additionally, the maximum annual precipitation on record is 46.64 inches for the Oak Shores WWTP station and 30.12 inches for the Nacimiento Dam station. Therefore, without the ability to perform a proper frequency analysis with the County's precipitation data, an annual 100-year precipitation of 44 inches was assumed to account for the apparent difference in precipitation between the two stations. This estimated 100-year annual rainfall is distributed over twelve months by using Table 7 – "Monthly % Distribution of Mean Annual Precipitation", "*California-Vegetative Water Use in California*, 1974." The distribution factors are summarized in the water balance spreadsheet.

Evaporation - The annual average class A pan evaporation from 40 years of records from the DWR Nacimiento Dam weather station T 09 6056 was 70.16 inches. Pan evaporation data are higher than the actual evaporation from a large water body and must be adjusted to account for radiation and heat exchange effects. The adjustment factor is called the pan coefficient which ranges from 0.7 to 0.8 in the summer and is close to the actual pan evaporation rate in the winter.

Crop Evapotranspiration (ET_c) – The combined loss of water to the atmosphere by the combined processes of evaporation from plant and soil surfaces and transpiration from plant tissues. The ET/Epan ratios were taken from Table 5 of the *Vegetative Water Use in California* report from the California Department of Water Resources. The values for an improved pasture were used.

Spray Losses – All spray irrigation systems exhibit some degree of spray losses which depends on nozzle size, pressure, wind speed, and ET rate. *The Surface Irrigation Manual* by Dr. Charles M. Burt estimates that approximately 4% to 6% of the total applied water is lost before it reaches the crop. 6% was used for the purposes of this report due to the high temperatures and high evaporation rates for the Oak Shores area for much of the year.

Calculations

Column A

$$\text{Combined Maximum Monthly Flow per Conn}\left(\frac{\text{gpd}}{\text{conn}}\right) =$$

$$MaxFlowPerFullTimeConn\left(\frac{\text{gpd}}{\text{conn}}\right)\times FullTimeOcc + MaxFlowPerSeasonalConn\times\left(1-FullTimeOcc\right)$$

Column B

$$\text{Monthly WW Flow}\left(\frac{\text{gal}}{\text{day}}\right) = CombinedMaxMonthlyFlowPerConn\left(\frac{\text{gpd}}{\text{conn}}\right)\times NumberOfConnections$$

Column C

$$\text{Monthly Flow (acre-ft)} = \frac{MonthlyWWFlow\left(\frac{\text{gal}}{\text{day}}\right) \times \#Days}{7.48\frac{\text{gal}}{\text{ft}^3} \times 43{,}560\frac{\text{ft}^2}{\text{acre}}}$$

Column D

$$\text{Treatment \& Storage Pond Evaporation (acre-ft)} = E_{Pond}(\text{in}) \times \frac{PondMidArea(\text{acres})}{12\frac{\text{in}}{\text{ft}}}$$

Column E

$$\text{Water Deficit/Surplus in Soil } (\text{in}) = ET_{crop}(\text{in}) - Precipitation(\text{in}) \times (1 - RunoffCoef)$$

Column F

Allowable Percolation (W_p, in) =

$$SprayFieldPercRate\left(\frac{\text{in}}{\text{hr}}\right) \times AllowableSprayFieldPerc \times \left(\frac{\text{24 hours}}{\text{day}}\right) \times AllowableSprayDays$$

Column G

$$\text{Spray Field Loading Rate } (L_w, in) = WaterDeficitSurplusInSoil \text{ (in)}$$
$$+ AllowablePercolation \text{ (in)}, \text{ if } WaterDeficitSurplusInSoil > 0$$

or

$$\text{Spray Field Loading Rate } (L_w, in) = AllowablePercolation \text{ (in)}$$
$$\text{if } WaterDeficitSurplusInSoil \text{ (in)} < 0$$

Column H

$$\text{Spray Field Loading Rate } (V_w, acre-ft) = SprayFieldLoadingRate \text{ (in)} \times \frac{SprayFieldArea(\text{acres})}{12\frac{\text{in}}{\text{ft}}}$$

Column I

$$\text{Spray Losses (acre-ft)} = SprayLosses \times SprayFieldLoadRate \text{ (acre-ft)}$$

Column J

$$\text{Pond Catch Volume (acre-ft)} = Precipitation(\text{in}) \times \frac{PondCatchArea(\text{acres})}{12\frac{\text{in}}{\text{ft}}}$$

Column K

Available Change in Storage (acre-ft) =

$$MonthlyWWFlow(C) - TreatmentStoragePondEvaporation(D) - SprayFieldLoadRate(H)$$
$$-SprayLosses(I) + PondCatchVolume(G)$$

Column L

Cummulative Storage (acre-ft) $= PreviousMonthVolume + K$, if sum > 0

Or

Cummulative Storage (acre-ft) =0 , if sum < 0

Total Effluent Storage Volume Required = maximum value from column L

WATER BALANCE FOR PROPOSED LYNCH CANYON PROPERTIES (HUGHES) SPRAY FIELD

CSA-7A WASTEWATER SYSTEM CAPACITY STUDY
INTERIM REPORT

Design Data

Number of Connections	**400 Connections**
Full-Time Occupancy	**40%**
Storage Pond Mid Area[1]:	0.84 acres
Storage Pond Catch Area[2]:	1.00 acres

Spray Field Data

Estimated Field Area:	**10 acres**
Sprayfield Percolation Rate[5]:	0.1 in/hour
Allowable Sprayfield Percolation[6]:	5%
Spray Losses[3]	6%

		Demand					Evaporation			Spray Fields								Precipitation			Storage	
				(A)	(B)	(C)			(D)			(E)		(F)	(G)	(H)	(I)		(J)		(K)	(L)
Month	Days	Maximum Monthly flow per Full-Time Connection (gpd/conn)	Maximum Monthly flow per Seasonal Connection (gpd/conn)	Combined Maximum Monthly flow per Connection (gpd/conn)	Monthly WW Flow (gpd)	**Monthly WW Flow (acre-ft)**	Ave. Class A Pan Evaporation[7] (E_{pan}, in)	E_{Pond}/E_{pan} Ratio	**Treatment & Storage Pond Evaporation (E_{pond}, acre-ft)**	Pasture ET_{crop}/E_{pan} Ratio[8]	Evapotranspiration (ET_{crop}, in)	Net Evapotranspiration (ET_{crop}-P(1-C), in)	Allowable Spray Days[4]	Allowable Percolation (W_p, in)	Load Rate (L_w, in)	**Load Rate (V_w, acre-ft)**	**Spray Losses (acre-ft)**	Precipitation[9] (in)	**Pond Catch Volume (acre-ft)**	Net Flow to Storage (acre-ft)	Net Flow to or from Storage (acre-ft)	Cumulative Storage (acre-ft)
October	31	162	60.8	101.3	40,512	3.85	5.28	0.80	0.30	0.75	3.70	2.16	29	3.48	5.64	4.70	0.28	1.54	0.13	-1.30	0.00	0.00
November	30	162	55.1	97.9	39,144	3.60	2.67	0.90	0.17	0.73	1.82	-1.31	28	3.36	3.36	2.80	0.17	3.52	0.29	0.76	0.00	0.76
December	31	162	48.9	94.1	37,656	3.58	1.66	1.00	0.12	0.70	1.09	-4.69	29	3.48	3.48	2.90	0.17	8.89	0.74	1.13	1.13	1.89
January	31	162	66.1	104.5	41,784	3.98	1.56	1.00	0.11	0.71	1.04	-4.93	29	3.48	3.48	2.90	0.17	9.33	0.78	1.57	1.57	3.46
February	28.25	162	116.0	134.4	53,760	4.66	2.12	1.00	0.15	0.74	1.45	-4.30	26	3.12	3.12	2.60	0.16	8.71	0.73	2.48	2.48	5.95
March	31	162	113.2	132.7	53,088	5.05	3.76	0.90	0.24	0.76	2.67	-2.15	29	3.48	3.48	2.90	0.17	6.51	0.54	2.28	2.28	8.23
April	30	162	121.0	137.4	54,960	5.06	5.68	0.80	0.32	0.77	4.08	0.83	28	3.36	4.19	3.49	0.21	3.70	0.31	1.35	1.35	9.58
May	31	162	159.1	160.3	64,104	6.10	8.06	0.80	0.45	0.78	5.88	4.87	29	3.48	8.35	6.96	0.42	1.01	0.08	-1.64	-1.64	7.93
June	30	162	125.4	140.0	56,016	5.16	9.92	0.80	0.56	0.78	7.22	6.96	28	3.36	10.32	8.60	0.52	0.26	0.02	-4.49	-4.49	3.44
July	31	162	162.0	162.0	64,800	6.17	11.32	0.80	0.63	0.78	8.26	8.21	29	3.48	11.69	9.74	0.58	0.04	0.00	-4.80	-3.44	0.00
August	31	162	133.6	145.0	57,984	5.52	10.42	0.80	0.58	0.78	7.60	7.52	29	3.48	11.00	9.16	0.55	0.09	0.01	-4.77	0.00	0.00
September	30	162	100.6	125.2	50,064	4.61	7.71	0.80	0.43	0.77	5.54	5.15	28	3.36	8.51	7.09	0.43	0.40	0.03	-3.30	0.00	0.00
	365.25					57.34	70.16		4.05		50.35			40.92	76.61	63.85	3.83	44.00	3.67			

Required Storage Pond Volume: **9.6** **acre-ft**

Notes:
1. Pond Mid Area is used to compute evaporation. It is the water surface area when the water level is midway between the maximum allowable water height and the bottom of the pond.
2. Pond Catch Area is the area over which the pond will collect any incipient rain
3. Burt, Dr. Charles M. The Surface Irrigation Manual. 1995. Waterman Industries, Exeter, CA. pg 13.9
4. Assuming 2 days per month of spray field nonoperation due to harvesting, sprinkler maintenance, freezing temperatures, etc.
5. SCS Soil Map Data
6. California State Water Resources Control Board. Irrigation with Reclaimed Municipal Wastewater, A Guidance Manual. 1984. pg. 8-15
7. California Department of Water Resources. Nacimiento Dam, Station T09 6056 00 Total Monthly Evaporation (40 years). 2005
8. California Department of Water Resources. Vegetative Water Use in California. Bulletin No. 113-3. 1974. ET/Ep Ratios for Improved Pasture - Table 5
9. California Department of Water Resources. Nacimiento Dam, Station T09 6056 00 Rainfall Depth Duration Frequency (47 years). 2005. Rainfall for 100-yr Return Period Used. and San Luis Obispo County precipitation data from station #201, Oak Shores WWTP.
10. California Department of Water Resources. Vegetative Water Use in California. Bulletin No. 113-3. 1974. Monthly Precipitation Distribution - Table 7

Rainfall Distribution[10]

Month	% of Ave	inches
October	3.50%	1.54
November	8.00%	3.52
December	20.20%	8.89
January	21.20%	9.33
February	19.80%	8.71
March	14.80%	6.51
April	8.40%	3.70
May	2.30%	1.01
June	0.60%	0.26
July	0.10%	0.04
August	0.20%	0.09
September	0.90%	0.40
	100.00%	44

Evaporation Distribution[8]

Month	% of Ave	inches
October	7.52%	5.28
November	3.81%	2.67
December	2.37%	1.66
January	2.23%	1.56
February	3.03%	2.12
March	5.36%	3.76
April	8.10%	5.68
May	11.49%	8.06
June	14.14%	9.92
July	16.13%	11.32
August	14.85%	10.42
September	10.99%	7.71
	100.00%	70.16

SCS Runoff Data

Month	C
October	0%
November	11%
December	35%
January	36%
February	34%
March	26%
April	12%
May	0%
June	0%
July	0%
August	0%
September	0%

References

Bedient, Philip B., Huber, Wayne C., 2002, *Hydrology and Floodplain Analysis*, 3rd ed., Prentice Hall, Uppper Saddle River, NJ

California Department of Water Resources, 1975, *Vegetative Water Use in California*, Bulletin No. 113-3

California Department of Water Resources, 1976, *Rainfall Analysis for Drainage Design, Volume II, Long-Duration Precipitation Frequency Data*, Bulletin No. 195

California State Water Resources Control Board, 1984, *Irrigation with Reclaimed Municipal Wastewater*, Report No. 84-1 wr

California Department of Water Resources. Nacimiento Dam, Station T09 6056 00 Rainfall Depth Duration Frequency (47 years). 2005. Rainfall for 100-yr Return Period Used

California Department of Water Resources. Nacimiento Dam, Station T09 6056 00 Total Monthly Evaporation (40 years). 2005

County of San Luis Obispo, Precipitation Records from Oak Shores WWTP Weather Station #201

Irrigation Training and Research Center, 2003, *California Crop and Soil Evapotranspiration for Water Balances and Irrigation Scheduling/Design*, California Polytechnic State University, San Luis Obispo, CA, p31

U. S. Environmental Protection Agency, 1981, *Process Design Manual: Land Treatment of Municipal Waste Water*, EPA 625/1-81-013

Water Environment Federation, 1992, *Design of Municipal Waste Water Treatment Plants*, Vol. 2, Manual of Practice No. 8

Appendix C

Construction Cost Estimates

Project Cost Estimate

Force Main to Existing WWTP from Tract 2162 (Alternative A)

ENR Index estimated at mid-point of construction: **7920**

No.	Item Description	Quantity	Unit	Unit Cost	Cost	Remarks
1	Steel Pipe Bridge	80	LF	$570	$45,600	Estimates inflated from Eastside Project 11 - Interceptor Bypass Study ENR of 6957
2	Steel Pipe Bridge	50	LF	$570	$28,500	
3	6" Ductile Iron Pipe (esposed)	200	LF	$100	$20,000	
4	6" Cl 200, C900 PVC SSFM	4000	LF	$70	$280,000	
				Construction Subtotal:	**$374,100**	
				With Construction Contingency (50%)	**$561,150**	
				County Adjustment Factors (82%)	$460,143	
				TOTAL PROJECT COST	**$1,021,293**	

Force Main to new WWTP from Tract 2162 (Alternative B)

ENR Index estimated at mid-point of construction: **7920**

No.	Item Description	Quantity	Unit	Unit Cost	Cost	Remarks
1	6" Cl 200, C900 PVC SSFM	11000	LF	$70	$770,000	
				Construction Subtotal:	**$770,000**	
				With Construction Contingency (50%)	**$1,155,000**	
				County Adjustment Factors (82%)	$947,100	
				TOTAL PROJECT COST	**$2,102,100**	

Project Cost Estimate

Alternative A - Ph. 1 Upgrade of Existing WWTP to 425,000 gpd Biolac System

ENR Index estimated at mid-point of construction: **7920**

No.	Item Description	Quantity	Unit	Unit Cost	Cost	Remarks
1	Headwork incl. screening, bypass bar screen, new flow meter	1	LS	$250,000	$250,000	Cost of screening equip is approx. $50,000
2	New Biolac unit w/ intregral clarfiier	425,000	gpd	$4	$1,700,000	Based on information from Parkson, Inc.
3	Effluent pump station	1	LS	$150,000	$150,000	Assuming two vertical turbines w/ VFD's
4	Sludge drying beds	1	LS	$70,000	$70,000	Approx. area of 5,000 sq ft
5	Conversion of old ponds to aerated sludge holding	1	LS	$30,000	$30,000	Assuming new rotary / brush aerators
				Construction Subtotal:	**$2,200,000**	
				With Construction Contingency (50%)	**$3,300,000**	
				County Adjustment Factors (82%)	$2,706,000	
				TOTAL PROJECT COST	**$6,006,000**	

Alternative B - Ph 1 of new WWTP - 100,000 gpd Biolac System

ENR Index estimated at mid-point of construction: **7920**

No.	Item Description	Quantity	Unit	Unit Cost	Cost	Remarks
1	Headwork incl. screening, bypass bar screen, new flow meter	1	LS	$210,000	$210,000	Cost of screening equip is approx. $50,000
2	New Biolac unit w/ intregral clarfiier	100,000	gpd	$5	$500,000	Based on information from Parkson, Inc., unit cost adjusted for lower quantity
3	New electrical service to site	1	LS	$40,000	$40,000	Estimate for three phase service
4	Sludge drying beds	1	LS	$40,000	$40,000	Approx. area of 2,500 sq ft
5	New aerated sludge lagoon	1	LS	$200,000	$200,000	Lined pond w/ new brush aerator
				Construction Subtotal:	**$990,000**	
				With Construction Contingency (50%)	**$1,485,000**	
				County Adjustment Factors (82%)	$1,217,700	
				TOTAL PROJECT COST	**$2,702,700**	

Project Cost Estimate

Kavanaugh Percolation Disposal Facility

ENR Index estimated at mid-point of construction: **7920**

No.	Item Description	Quantity	Unit	Unit Cost	Cost	Remarks
1	New percolation ponds (five at 16,500 SF each)	2	acre	$300,000	$600,000	Based on recent perc pond bid costs in Templeton, Ca
2	Force main from spray field	50	LF	$570	$28,500	
3	Monitoring wells	1	LS	$20,000	$20,000	
4	Radio telemetry / alarm	1	LS	$20,000	$20,000	
				Construction Subtotal:	**$668,500**	
				With Construction Contingency (50%)	**$1,002,750**	
				County Adjustment Factors (82%)	$822,255	
				TOTAL PROJECT COST	**$1,825,005**	

Appendix D

Electronic Information (on compact disk)

Contents of compact disk:

D.1 – Capacity and Demand Report, CSA-7A, 2005, Garing Taylor & Associates
D.2 - Preliminary Plan and Cost Estimate, CSA-7A, 2005, Garing Taylor & Associates
D.3 - CSA-7A Interceptor Bypass Study, 2004, SLO County Public Works (unofficial)
D.4 – Oak Shores Disposal Area Expansion Project, 2001, SLO County Public Works
D.5 – Oak Shores Effluent Disposal Study…, 1989, Boyle Engineering Corp.
D.6 - Waste Discharge Requirements 01-130
D.7 – CSA-7A Budget for 05/06
D.8 - Manufacturer's data on Biolac®
D.9 - Draft Memorandum of Understanding
D.10 – SLO County CSA-7A Wastewater Flow Records

Appendix E

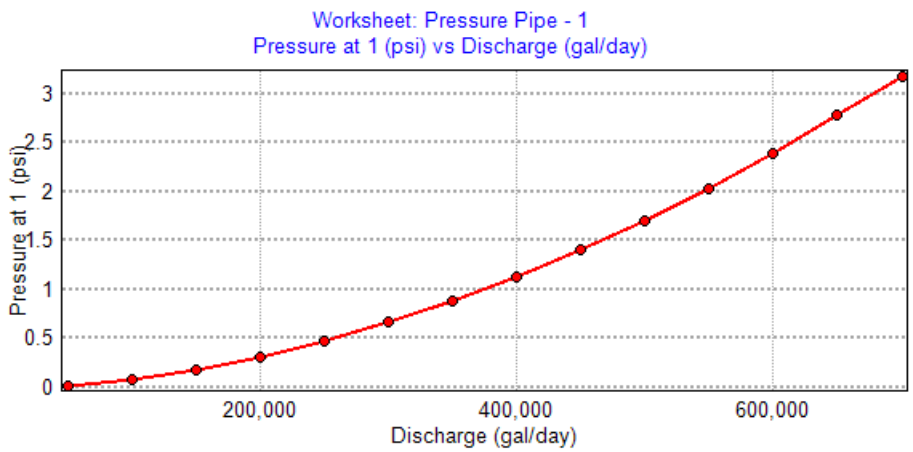
Hydraulic Calculations

Rating Curve for Pressure Pipe - 1

Project Description

Friction Method	Hazen-Williams Formula
Solve For	Pressure at 1

Input Data

Pressure 2	0.00	psi
Elevation 1	0.00	ft
Elevation 2	0.00	ft
Length	9500.00	ft
Roughness Coefficient	120.000	
Diameter	12.00	in
Discharge	200000.00	gal/day

Rating Curve Plot