APPENDIX C

WATER AND WASTEWATER FEASIBILITY ANALYSIS

WATER AND WASTEWATER FEASIBILITY ANALYSIS

FOR CHUMASH CAMP 4 PROPERTY FEE-TO-TRUST APPLICATION ENVIRONMENTAL ASSESSMENT

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CHAPTER 1

INTRODUCTION

The Santa Ynez Band of Chumash Indians (Tribe) proposes a project to develop approximately 1,433 acres of land as part of a trust land acquisition. This water and wastewater technical feasibility study is in support of the Environmental Assessment (EA) prepared in support of the Tribe's application for the Bureau of Indian Affairs (BIA) to take the 1,433-acre Project into Trust. by the Tribe to reduce potential adverse impacts to environmental resources. The EA is being prepared by Analytical Environmental Services (AES), Sacramento, California. The project alternatives evaluated in this EA consist of:

> **Alternative A** (Proposed Project) – 1,433± acre trust land acquisition and development of 143 five-acre residential lots for Tribal members. The remaining land uses would entail 300 acres of vineyards (256 acres existing), 206 acres of open space/recreational, 131 acres of riparian corridor and oak woodland conservation, and 3 acres of Special Purpose Zone- Utilities;
>
> **Alternative B** (Reduced Development Intensity Alternative) – Identical trust land acquisition and development of 143 one-acre residential lots for Tribal members. The remaining land uses would entail 775 acres of open space/recreational, 30 acres of Tribal Government/Development(including 80,000 square feet of Tribal facilities), and the same acreages of vineyard, riparian corridor and oak woodland conservation, and utilities land uses as proposed under Alternative A; and
>
> **Alternative C** (No Action Alternative) – No federal action or proposed development. The "No Action" alternative is not discussed further in this report, as no technical evaluation is warranted for this alternative.

A summary of project components under the two development alternatives (A and B) is provided in Table 1-1. Full details of the Project Descriptions and alternatives can be found in the EA prepared by AES for this Project.

ALTERNATIVE A – PROPOSED PROJECT

Alternative A consists of two main components: (1) the placement of 5 parcels totaling approximately 1,433± acres into Federal trust status for the Tribe; and (2) the development of 143 five-acre residential plots with the remaining acreage dedicated to agriculture, open space/recreational, conservation of riparian corridors and oak woodland, and development of utilities. Development of the site would include domestic water connections, a wastewater treatment plant (WWTP), and supporting roads and infrastructure. Alternative A is described in more detail in the following sections.

TABLE 1-1

SUMMARY OF PROJECT DEVELOPMENT ALTERNATIVES[a]

Project Components	Alternative	
	A	B
Land Taken into Trust	1,433± acres	1,433± acres
Residential Development	143 five-acre lots	143 one-acre lots
Designated Tribal Land Uses	300 acres of Agriculture (existing), 206 acres of Open Space/Recreational – General/Trails, 98 acres of Resource Management Zone – Riparian Corridors, 33 acres of Resource Management Zone – Oak Woodland, and 3 acres of Special Purpose Zone- Utilities	300 acres of Agriculture (existing), 755 acres of Open Space/Recreational – General/Trails, and 30 acres of Special Purpose Zone -Tribal Government/Development 98 acres of Resource Management Zone – Riparian Corridors, 33 acres of Resource Management Zone – Oak Woodland, and 3 acres of Special Purpose Zone- Utilities
Water Source	Groundwater	Groundwater
Wastewater Treatment	Onsite WWTP	Onsite WWTP

[a]*Source: AES, 2012*

Proposed Residential Development.

Under Alternative A, the Tribe would develop residential plots on Parcels 2, 3 and 4 of the project site. The proposed housing would consist of up to 143 five-acre residential plots with construction of single-family detached houses of varying sizes ranging from 3,000 to 5,000 square feet. Development on each five-acre plot would include approximately 0.35 acres of disturbance for building pad development, driveway construction, utility installations, and landscaping. Additionally, new domestic water connections, improved access roads, driveways, a new wastewater treatment plant, and utilities would also be constructed to support the residences. A site plan identifying the proposed residential plots is shown in Figure 1-1.

Designated Tribal Land Uses

In addition to the proposed residential development, the Tribe would designate the following land uses on the subject property:

Agricultural

The Tribe would continue operating an existing 256-acre vineyard located on Parcel 1 and a portion of Parcel 2 (refer to Figure 1-1). An additional 44 acres would be designated for agricultural use on Parcel 2 to allow for expansion of the existing vineyard operation. The vineyard is currently in operation and includes a storage reservoir, existing access roadways, and a processing/shipping area. No winemaking facilities are currently located on the project site, and there are no plans to develop a winery on the project site. Various structures are located within the agricultural lands including an old abandoned house and operational horse stables.

Open Space/Recreational – General/Trails

Approximately 206 acres of the project site would be designated as open space and recreation. Passive trails would be designated for pedestrian use and equestrian trails would be developed to provide recreation for residents and guests in coordination with the horse stables located on the existing agricultural lands. The open space/recreational area adjacent to State Route (SR) 154 would be utilized as a viewshed protection zone. No residential development is planned within the zone adjacent to SR-154 to protect the viewshed of the scenic highway.

Special Purpose Zone- Utilities (WWTP)

To support the development of residential plots, a central tertiary WWTP would be developed on three acres of the agricultural lands. The tertiary WWTP is described in more detail below.

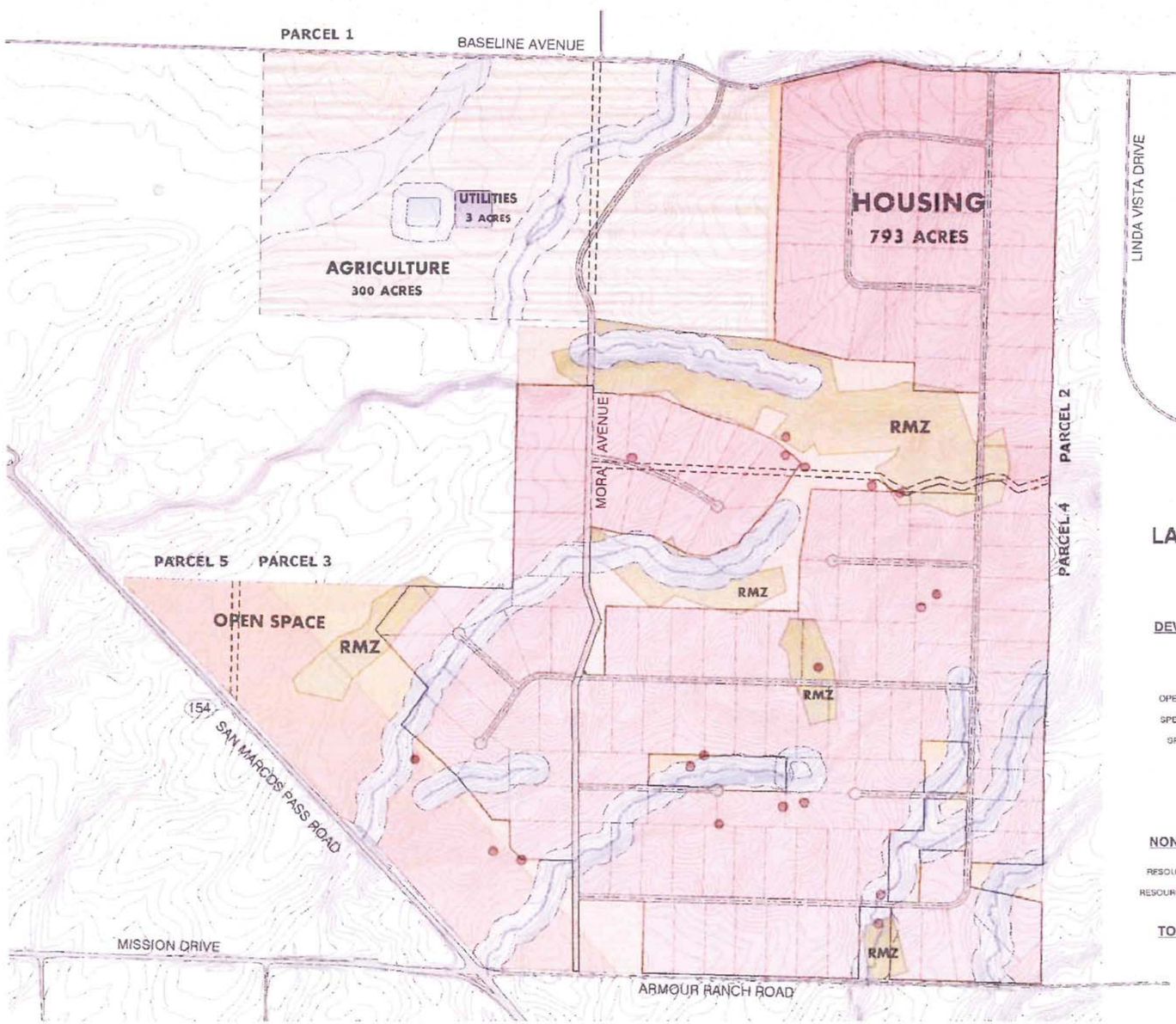
Water Supply

The Tribe would develop an on-site water supply system using groundwater to meet potable water demands. Groundwater wells would be located in reasonable proximity to the proposed residential developments. The Tribe would install an onsite domestic water storage tank as well as the appropriate water distribution pipelines to the proposed Tribal residences. Water quality would be no less stringent than Federal water quality and Federal Safe Drinking Water Act standards. Inspections of the water supply system and water quality by the U.S. Environmental Protection Agency (USEPA) would ensure compliance with applicable safe drinking water standards. Tertiary treated wastewater would be utilized to meet the irrigation water demands of the vineyard operation, common area landscaping, and other irrigated uses as feasible. The existing agriculture storage reservoir would be used to meet the recycled water storage requirements. Proposed water facilities are discussed in more detail in Chapter 2 of this Report.

Wastewater Treatment and Disposal

A new tertiary WWTP would be constructed on Parcel 1 (Figure 1-1) adjacent to the existing reservoir within the vineyards. The WWTP would be sized to accommodate the proposed wastewater generation rates of the Proposed Project. The tertiary treated wastewater would be recycled for use as agricultural irrigation for the existing agricultural operations, common area landscaping, and other irrigated uses as feasible on the project site. Drainage control would be installed along the perimeter of recycled water irrigation areas to prevent comingling with stormwater runoff. Recycled water runoff would be collected and disposed of via discharge to the WWTP.

The proposed WWTP and related facilities are discussed in more detail in Chapter 3 of this Report. In general terms, wastewater facilities would include a tertiary WWTP, sewer lift stations, conveyance systems, emergency storage, runoff/spill control, and a recycled water reservoir. The sewer lift stations would be developed within the residential areas as needed. The existing water reservoir located on Parcel 1 would be re-purposed to store recycled water from the WWTP, and enlarged if necessary. The reservoir would be equipped with provisions for potable water "make-up" water (with air-gap separation to protect the potable water supply) to supplement recycled water during high demand times. The existing water reservoir is currently lined and prior to use as a recycled water reservoir, the lining would be inspected for tears or other imperfections that may result in leakage. The proposed wastewater treatment system would be operated pursuant to U.S. Environmental Protection Agency (EPA) regulations.



LEGEND

- RESIDENTIAL ZONE
  LOW DENSITY PLANNED RESIDENTIAL DEVELOPMENT
- AGRICULTURAL ZONE
  AGRICULTURAL 1
- OPEN SPACE / RECREATION ZONE
  PASSIVE TRAILS, EQUESTRIAN TRAILS
- RESOURCE MANAGEMENT ZONE (RMZ)
  OLD WOODLAND
- RESOURCE MANAGEMENT ZONE (RMZ)
  RIPARIAN CORRIDORS
- SPECIAL PURPOSE ZONE
  GOVERNMENT CENTER
- SPECIAL PURPOSE ZONE
  PUBLIC UTILITIES - WWTP
- ROADS
- VIEW CORRIDOR
- CULTURAL RESOURCES

LAND USE SUMMARY

143 - 5 ACRE LOTS

DEVELOPABLE LAND	1,302 ACRES
RESIDENTIAL PRD	793 ACRES
AGRICULTURAL	300 ACRES
OPEN SPACE/RECREATION - GENERAL/TRAILS	206 ACRES
SPECIAL PURPOSE ZONE - GOVERNMENT	60 ACRES
SPECIAL PURPOSE ZONE - UTILITIES	3 ACRES

NON-DEVELOPABLE LAND	131 ACRES
RESOURCE MANAGEMENT ZONE - OAK WOODLAND	33 ACRES
RESOURCE MANAGEMENT ZONE - RIPARIAN CORRIDORS	98 ACRES

TOTAL SITE ACREAGE 1,433 ACRES

NORTH

0 1000' 2000'

SOURCE: SUMMIT PROJECT MANAGEMENT, 2011; AES 2011

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CHUMASH CAMP 4 FEE-TO-TRUST

Figure 1-1
Fee-To-Trust Land Use Summary - Alternative A

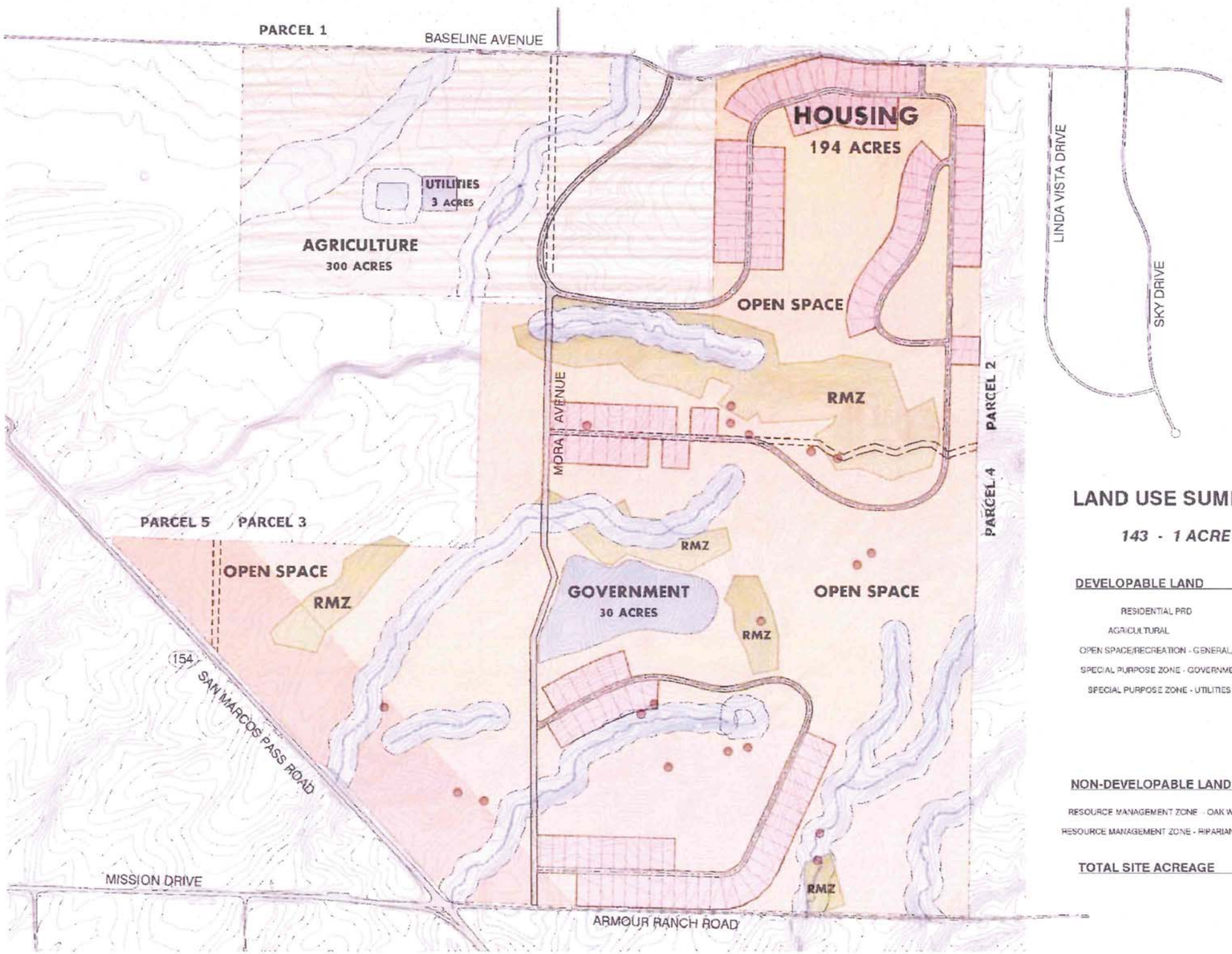
ALTERNATIVE B – REDUCED DEVELOPMENT INTENSITY

Alternative B would involve placing the 1,433-acre Camp 4 site into federal trust status for the benefit of the Tribe; however, under Alternative B, the residential parcel lot sizes would be reduced from 5 acres to 1 acre, decreasing the residential acreage from approximately 793± acres to approximately 194± acres. Development on each one-acre plot would include approximately 0.25 acres of disturbance for building pad development, driveway construction, utility installations, and landscaping. Additionally, new domestic water connections, improved access roads, driveways, a new WWTP, and utilities would also be constructed to support the residences. A site plan identifying the proposed residential plots is shown in Figure 1-2. In addition, approximately 30 acres of the project site would be reserved for approximately 80,000 square feet of Tribal government/ development space. The Tribal facilities would include development of a banquet/exhibition hall designed with an agriculture/equestrian theme, associated administrative spaces, a tribal office complex, and a tribal retreat including ceremony room and gymnasium. A breakdown of the components of the proposed Tribal facilities is displayed in Table 1-2. It is anticipated that the Tribal development would include office space for up to 75 Tribal employees and result in up to 100 events per year being held at the facilities. Approximately 400 parking spaces would be provided for the facilities.

The remaining land uses and project components under Alternative B are identical to that proposed under Alternative A including: the construction of 143 residences ranging from 3,000 to 5,000 square feet, domestic water connections, and a WWTP. Public services, water supply, wastewater treatment and disposal, and roadway improvements would all be provided for Alternative B as described for Alternative A.

Table 1-2. Tribal Community Development – Onsite Facilities

Usage	Square Footage (sf)
Community Center	34,280
Community Center Administrative Support	3,110
Tribal Office Complex	12,025
Tribal Retreat	11,480
Circulation (Misc. at 30%)	18,269
Total Development	**79,164**



LAND USE SUMMARY

143 - 1 ACRE LOTS

DEVELOPABLE LAND	1,302 ACRES
RESIDENTIAL PRD	194 ACRES
AGRICULTURAL	300 ACRES
OPEN SPACE/RECREATION - GENERAL/TRAILS	775 ACRES
SPECIAL PURPOSE ZONE - GOVERNMENT	30 ACRES
SPECIAL PURPOSE ZONE - UTILITIES	3 ACRES

NON-DEVELOPABLE LAND	131 ACRES
RESOURCE MANAGEMENT ZONE - OAK WOODLAND	33 ACRES
RESOURCE MANAGEMENT ZONE - RIPARIAN CORRIDORS	98 ACRES

TOTAL SITE ACREAGE	1,433 ACRES

LEGEND

- RESIDENTIAL ZONE — LOW DENSITY PLANNED RESIDENTIAL DEVELOPMENT
- AGRICULTURAL ZONE — AGRICULTURAL 1
- OPEN SPACE / RECREATION ZONE — PASSIVE TRAILS, EQUESTRIAN TRAILS
- RESOURCE MANAGEMENT ZONE (RMZ) — OLD WOODLAND
- RESOURCE MANAGEMENT ZONE (RMZ) — RIPARIAN CORRIDORS
- SPECIAL PURPOSE ZONE — GOVERNMENT CENTER
- SPECIAL PURPOSE ZONE — PUBLIC UTILITIES - WWTP
- ROADS
- VIEW CORRIDOR
- CULTURAL RESOURCES

NORTH

0 1000' 2000'

SOURCE: SUMMIT PROJECT MANAGEMENT, 2011; AES 2011

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CHUMASH CAMP 4 FEE-TO-TRUST

Figure 1-2
Fee-To-Trust Land Use Summary - Alternative B

Protective Measures and Best Management Practices

Protective measures and best management practices (BMPs) pertinent to this water and wastewater feasibility study have been incorporated into the project design to eliminate or substantially reduce environmental impacts from the Proposed Project.

These measures and BMPs are discussed below.

Land Resources

All structures would meet the Tribe's building ordinance, which meets or exceeds Uniform Building Code (UBC) requirements.

Water Resources

- High water-demand plants would be minimized in landscaping plans. Native and drought-tolerant plant species (trees, shrubs, and ground cover) would be emphasized.
- Water-efficient fixtures and appliances would be installed in residences.

Public Services

- Structural fire protection would be provided through compliance with Tribal ordinances no less stringent than applicable Uniform Fire Code requirements. The Tribe would ensure that appropriate water supply and pressure is available for emergency fire flows.

CHAPTER 2

WATER SYSTEM

This chapter describes the projected water demands, water supply and distribution system requirements for the Chumash Camp 4 Project (Project) for Alternatives A and B. The water demand forecasts form the basis for assessing water supply requirements and identifying distribution system requirements. Recycled water demands are discussed in detail in Chapter 3.

PROJECTED POTABLE WATER DEMAND

Potable water demands projected for the Project will form the basis for recommendations for needed water supply, and for laying out conceptual water system components including potable water distribution and fire suppression system, water storage and pumping requirements. Detailed hydraulic analyses of the conceptual water system are beyond the scope of this study; however, general water system infrastructure is described in this Chapter. Water demands for Alternatives A and B are based on the program descriptions presented in Chapter 1 of this Report.

Key factors, assumptions and details used to formulate water demands for both Alternatives A and B include the following:

- Residential Units, 3.5 persons per dwelling unit, or ~500 permanent population
- Domestic water demand, 90 gallons per capita per day (gpcd)
- All homes furnished with low-flow fixtures

Hydraulic Demand Parameters

Water system demands are important characteristics of water systems, as these parameters are used to size pumping, storage, and distribution system facilities. Demands calculated for this Project will be used to evaluate water distribution system requirements. Since this is a planned future Project, existing operations data is not available to be used as part of the water demand analysis. Therefore, trends from other communities will be used to estimate demand factors herein.

Hydraulic demand parameters are defined as follows:

- Average Day Demand (ADD). The ADD is the average water demand calculated over the year. This demand is generally determined by production records, however, since the Project is a new development, the ADD must be estimated based on industry standards.
- Night Time Demand (NTD). The NTD is the production of water during low flow periods, typically seen in the middle of the night. These flows are critical for properly sizing pumps to meet these low demands. The NTD peaking factor for communities can vary considerably.
- Maximum Day Demand (MDD). The MDD is the maximum daily production of water needed to meet the peak day demand of the year. This is generally during the

summer as a result of increased residential irrigation demand. The MDD peaking factor for communities of similar size can range from 1.6 to 2.0. To be conservative, for this analysis, a peaking factor of 2.0 (2.0 times the ADD) will be used.

- Peak Hour Demand (PHD). The PHD of the system is critical in sizing water mains and pumping facilities. During peak hour demand, customers will generally experience low service pressures in areas with undersized mains and/or lack of looped distribution pipelines. The PHD is generally determined by calculating the specific demand within the day, by monitoring tank levels and pumping records. A PHD factor of 3.5 (3.5 times the ADD) was assigned to the entire system, based on engineering judgment and data from other similar municipalities. It is also noted, however, that the fire flow will be the highest water demand in the system.

It is noted that irrigation of the existing vineyards, open space/recreational land use designations, and Tribal Government Center irrigation demands (Alternative B), will be met using recycled water from the wastewater treatment plant, and make-up groundwater from on-site irrigation wells. Refer to Chapter 3 for more information on recycled water uses and demands.

Landscape irrigation demands were developed in part, by referring to local weather data available on the California Irrigation Management Information System (CIMIS) web site for local Santa Ynez weather stations, and consideration of the type of landscaping to be irrigated. For all residential lots, turf area was estimated, in part by comparing sample properties surrounding the Project area to determine landscaped areas, turf/lawn areas, and buffer areas with no landscaping.

Potable Water Demand – Alternative A

Potable water demands for Alternative A are summarized in Table 2-1. Key factors, assumptions and details used to formulate water demands for Alternative A include the following:

- 5-acre Lot size, 1.85 acres of low water demand landscaping per lot, water demand 1.0 acre feet per year (AFY)/acre
- Pad disturbance area, 0.35 acres
- Assumes 0.15 acres of irrigated turf/lawn area per lot
- Lawn/Turf irrigation demand, 3.0 AFY/acre

Potable Water Demand – Alternative B

Potable water demands for Alternative A are summarized in Table 2-2. Specific water demands for the Tribal Government Center are summarized in Table 2-3. Key factors, assumptions and details used to formulate water demands for Alternative A include the following:

- 1-acre Lot size, 0.40 acres of low water demand landscaping per lot, 1.0 AFY/acre
- Pad disturbance, 0.25 acres
- Assumes 0.10 acres of irrigated turf/lawn area per lot, 3 AFY/acre irrigation demand

Table 2-1. Summary of Potable Water Demands – Alternative A

User	Unit	Type of Unit	Peak Hour Demand per Unit, gpm[b]	Annual Demand, AFY
Residential - indoor	143	SFR[a]	0.30	50.5
Residential - landscape drought tolerant LS	1.85	acres	3.44	264.6
Residential - Lawn	0.15	acres	0.84	64.4
TOTAL	**---**	**---**	**655**	**379.4**

[a]*Single-family residence*

[b]*Gallons per minute*

Table 2-2. Summary of Potable Water Demands – Alternative B

User	Unit	Type of Unit	Peak Hour Demand per Unit, gpm	Annual Demand, AFY
Residential - indoor	143	SFR	0.30	50.5
Residential - landscape drought tolerant LS	0.4	acres	0.74	57.2
Residential - Lawn	0.1	acres	0.56	42.9
TOTAL	**---**	**---**	**229**	**150.6**

Table 2-3. Summary of Potable Water Demands – Tribal Government Center

Building Use	Unit	Quantity	Demand	Unit	Demand, gpm	Demand, AFY
Community Center	Event	100/yr	10	gpd/person@1,000 persons/event	69.4	3.1
Admin	Employee	75 ea	20	gpd/employee	9.4	1.2
Tribal Office			Included in employees above			
Tribal Retreat			Included in events above			
TOTAL					**78.8**	**4.3**

Note: Tribal Government Center irrigation demands met using recycled water and non-potable irrigation water.

WATER SUPPLY

This section discusses the existing site hydrogeology, existing water wells and expected water quality, the existing storage reservoir, and water supply needs.

Net Potable Water Demand – Alternative A

Net potable water demands for Alternative A are summarized in Table 2-4. The net project demand considers new potable water demands only, therefore does not include the existing water demand for the vineyard (which will be irrigated with recycled water and irrigation water from on-site irrigation wells). Key factors, assumptions and details used to formulate potable water demands for Alternative A were discussed earlier in this Chapter 2.

Table 2-4. Net Potable Water Demand for Alternative A (5-Acre Parcels)

User	Unit	Type of Unit	Annual Demand (AFY)
Residential - indoor use	143	SFR	51
Residential - landscape irrigation (drought tolerant)	1.85	acres	265
Residential – lawn irrigation	0.15	acres	64
Treated wastewater for irrigation (90% of indoor use)	--	--	<45>
NET PROJECT POTABLE WATER DEMAND	--	--	335

SFR – single family residence

Net Potable Water Demand – Alternative B

Net potable water demands for Alternative B are summarized in Table 2-5. As in Alternative A, the net potable water project demand considers new water demands only, therefore does not include the existing water demand for the vineyard (to be irrigated with recycled water and irrigation water from on-site irrigation wells). Specific potable water demands for the Tribal Government Center are summarized in Table 2-3. Key factors, assumptions and details used to formulate water demands for Alternative B were presented earlier in Chapter 2.

Table 2-5. Net Potable Water Demand for Alternative B (1-Acre Parcels)

User	Unit	Type of Unit	Annual Demand (AFY)
Residential – indoor use	143	SFR	51
Residential – landscape irrigation (drought tolerant)	0.4	acres	57
Residential – lawn irrigation	0.1	acres	43
Tribal Government Center (indoor)	--	--	4
Treated wastewater for irrigation (90% of indoor use)	--	--	<49>
NET PROJECT DEMAND	--	--	106

SFR – single family residence

Site Hydrogeology

Existing water supply at the site is entirely from groundwater resources within the Santa Ynez Uplands Groundwater Basin. The basin comprises the eastern portion of the groundwater basins of the Santa Ynez River watershed. These basins lie between the San Rafael Mountains to the north and east, the Purisima Hills to the northwest and the Santa Ynez Mountains to the south. The Santa Ynez Upland Groundwater Basin is located north of the Santa Ynez River between Buellton and the east end of Lake Cachuma. It underlies 130 square miles and is widest in the west and narrows to the east.

The shape of the basin is controlled by east-west trending folding and faulting of sedimentary beds and has also been influenced by historical stages and flow of the Santa Ynez River. It is bounded by a topographical groundwater divide from the San Antonio Basin to the northwest, faults and impermeable rocks of the San Rafael Mountains to the north and east, and by nonwater-bearing Tertiary age formations to the south that separate it from the Santa Ynez River alluvial basin. Average rainfall within the basin varies from a maximum of about 24 inches per year in the higher elevations to a minimum of about 15 inches per year in the southern and central areas. Rainfall and stream seepage are the primary sources of recharge to the basin.

(DWR, 1980, SB County Groundwater Report, 2008).

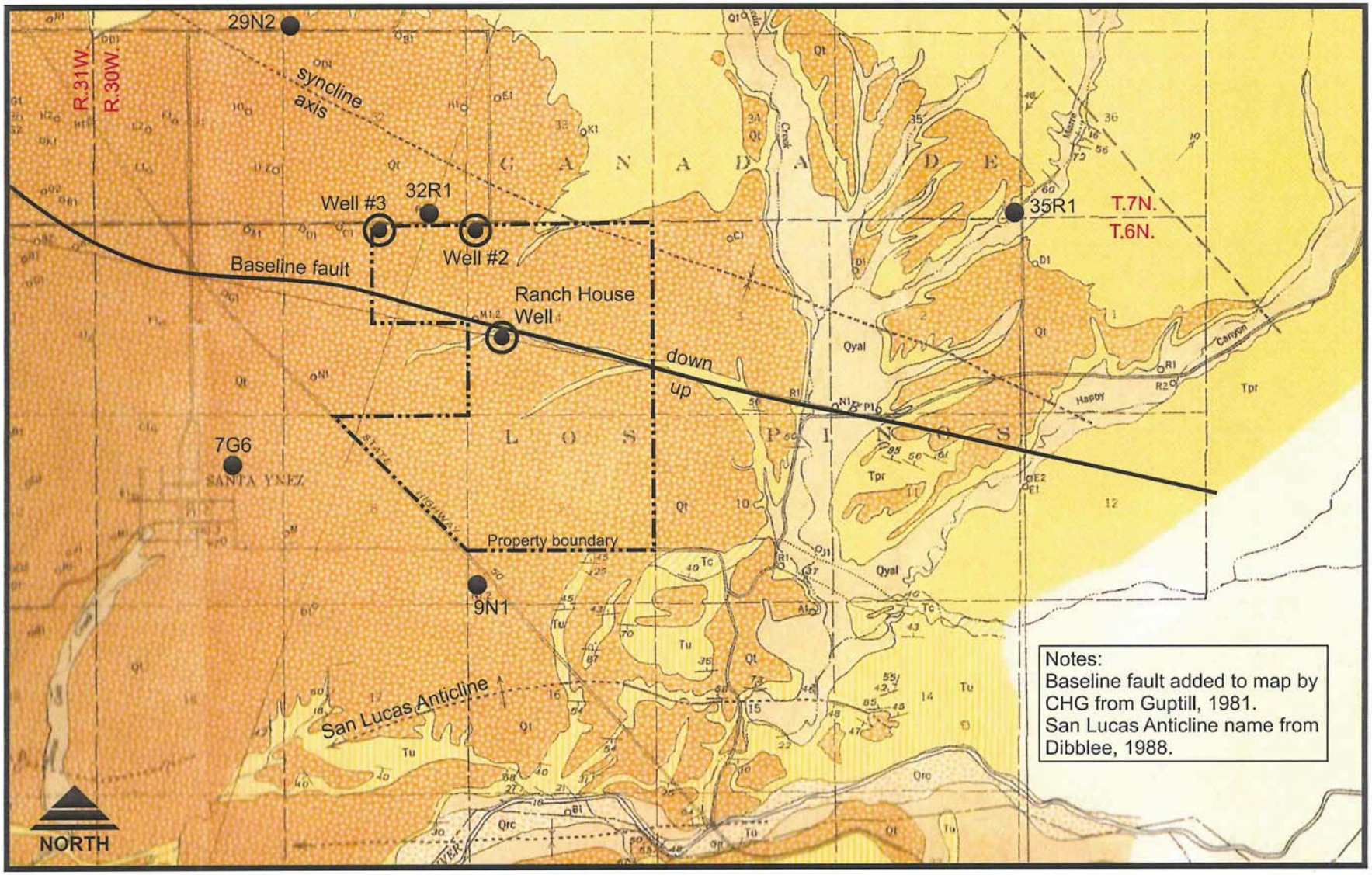
Loosely consolidated sand and gravel aquifers of the Plio-Pleistocene age Paso Robles Formation are the major source of groundwater in the basin. The formation consists of deposits of sand and gravel interbedded with clay and silt in discontinuous, lenticular beds. The Pliocene-age Careaga Formation lies underneath the Paso Robles Formation as unconsolidated fine to medium grained marine sand and lesser silt. Although it is water bearing within the basin, it is generally tapped by wells only in the southern margins of the basin where it has been uplifted to relatively shallow depths.

The Paso Robles and Careaga Formations have been folded into a north-dipping monocline north of the axis of the San Lucas Anticline which brings consolidated nonwater-bearing rocks of the Sisquoc and Monterey Formations to or near the ground surface south of the Chumash Camp 4 Project southern boundary. The water bearing zones of the Paso Robles Formation become increasingly thick and both the Paso Robles and the Careaga Formations become increasingly deep from south to north across the project area to the roughly east-west trending Baseline fault that crosses the northern half of the project property. The Baseline fault is a reverse fault, vertically offsetting fluvial terraces in the project area (Guptill, 1981), and effectively lowering the underlying Paso Robles and Careaga Formations north of the fault. The beds continue to dip to the north to the axis of a syncline crossing the northeast corner of the project area. North of the syncline, the beds become shallower.

Supply wells drilled between the Baseline fault and the syncline axis would encounter the greatest thickness of the Paso Robles Formation within and in the vicinity of the project boundaries. The fault may restrict groundwater flow across the fault plane, resulting in non-correlative groundwater levels in wells on opposite sides of the fault. A regional geologic map from the U.S. Geological Survey (1951) is included as Figure 2-1, showing the two fold axes with the Baseline fault added by CHG. The geologic maps by Dibblee (1988, 1993) were not used because of conflicting information on the two adjacent quadrangles.

Existing Water Well Production

Current water supply at the site is provided by two irrigation wells, serving irrigation requirements for the 256 acre vineyard, and by one ranch/domestic well that provides water for the ranch house and for stock watering. Well locations are shown on Figure 2-1. The two irrigation wells are located along Baseline Avenue situated within the down-dropped geologic structure between the Baseline fault and the synclinal axis north of the property, and the ranch/ domestic well is located near the trace of the fault.



Base map: Geologic Map & Sections of the Central Part of the Santa Ynez River Basin U.S.G.S. Water-Supply Paper 1107 Plate 2, Upson & Thomasson, 1951

0 4,000 8,000

Scale: 1 inch = 4,000 feet

Explanation

Qt	Terrace deposits
Tpr	Paso Robles Formation
Tc	Careaga Formation
Tu	Consolidated Tertiary rocks
◉	Existing onsite project well
●	Existing offsite well with water level data

Figure 2-1
Regional Geology Map
Chumash Camp 4
Cleath-Harris Geologists

The irrigation well #3 in the northwest corner of the property, was completed in 1984 to a total depth of 795 feet with perforations from 248 to 785 feet depth. The well was completed with 16-inch steel casing. According to the ranch manager, the well produces between 900 and 1,200 gallons per minute (gpm). During a four-hour pump test in November 1984, the static water level was 137 feet depth. Four pumping steps were performed beginning at 1,200 gpm and ending at 2,700 gpm with a final pumping level of 230 feet depth. A 60-minute pumping test was performed in August 1999 at rates of 1,960, 1,830 and 1,680 gpm with a maximum pumping level of 185 feet depth. The testing contractor recommended operational flow rates between 1,100 and 1,400 gpm for best operating efficiencies.

The irrigation well #2 is approximately one half mile east of well #3, and was completed in 1999 to a total depth of 740 feet. Perforation depth intervals are from 290 to 520 feet, 550 to 620 feet, and 660 to 730 feet. Casing is 16-inch diameter steel. According to the ranch manager, the well produces 1,700 gpm. During an eight-hour pumping test in December 1999, the static water level was measured at 178 feet depth. Pumping was performed in three steps at 1,500, 2,000 and 2,500 gpm with a maximum pumping level of 233 feet depth. The well is equipped with a 250 horse-power pump motor.

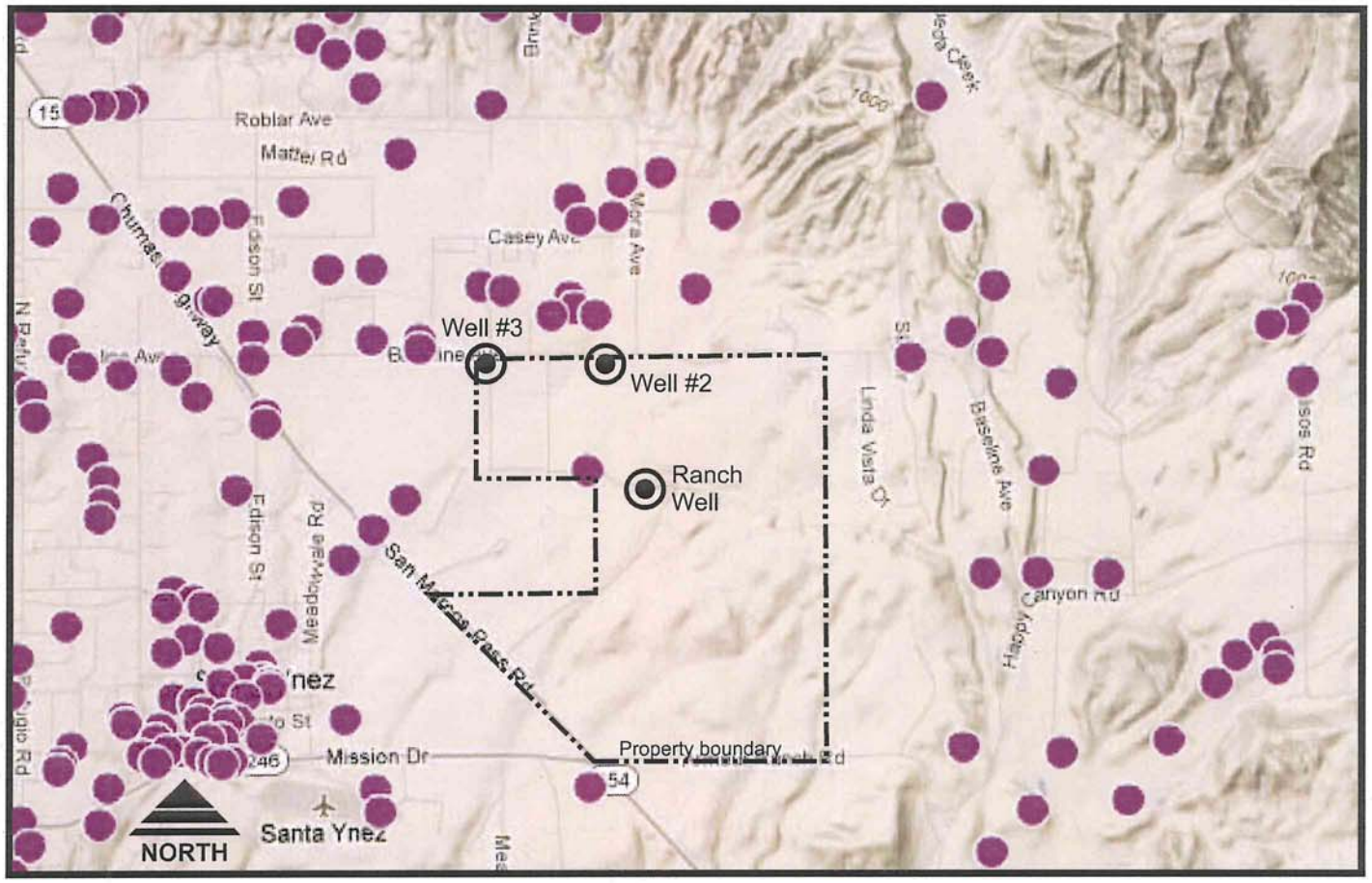
Both irrigation wells are equipped with an air line for measuring water levels and a flow meter. Water is pumped from the wells to a ½-acre lined reservoir for vineyard irrigation, that holds approximately 2-1/2 acre-feet.

The Ranch House Well was completed with eight-inch diameter steel casing and serves the cattle ranching area and the ranch house. The total depth of the well is 505 feet. A new, three horse-power pump was installed in 2005, and is capable of pumping 25 gpm. Static water level in July 2005 was 105 feet depth.

There is an eight-inch diameter steel-cased well with a windmill rod and column in the north-central portion of parcel 4. The well was dry to a total depth of 74 feet during a site visit by CHG in March 2012.

During the March 2012 CHG site visit, an active irrigation well was observed off the property on the adjacent parcel approximately three tenths of a mile west of the ranch house. The well is equipped with a submersible pump.

Several wells are present along the north side of Baseline Avenue that serve domestic supply and small irrigation demands including stock watering. Two wells serve the Santa Ynez Rancho Estates Mutual Water Company east of the project site. According to the ranch manager, an irrigation well also to the east of the project property serves a 50-acre vineyard. There are presumed to be several domestic wells serving the tract northeast of the project property. No wells were observed south of Armour Ranch Road during the March 2012 site visit by CHG. Offsite wells for which water level data are available through the California Department of Water Resources (DWR) internet site are shown in Figure 2-2.



Base map: California DWR Water data library online map.

0 4,000 8,000

Scale: 1 inch = 4,000 feet

Explanation

● Well locations with water level data available through California Dept. of Water Resources

Figure 2-2
Water-Level Location Map
Chumash Camp 4
Cleath-Harris Geologists

The irrigation demand for the existing vineyard located in the northern portion of the site is met by groundwater supply from the two Baseline Avenue wells by way of the ½-acre reservoir. Based on typical water use in Santa Ynez Valley vineyards, duty factors vary from 0.8 to 1.2 acre-feet per acre per year. For this assessment, one acre-foot per acre per year is estimated for the onsite vineyard water demand. Under the existing conditions at the site, the annual water demand for the 256 acre vineyard is estimated at 256 acre-feet per year (AFY).

Existing Water Well Quality

Water quality samples were obtained at the Baseline Avenue Well #2 on December 22, 1999 following the well completion and pump testing. No other water quality results were available for the onsite supply wells. The samples from Well #2 were analyzed for general minerals, general physical, and inorganic chemicals. No analytes were present in concentrations above the State of California maximum contaminant levels for drinking water. The total dissolved solids concentration was 480 milligrams per liter (mg/l) and total hardness concentration was 386 mg/l. Based on the laboratory results, water from Well #2 is suitable for irrigation water. Water quality testing to meet US EPA maximum contaminant levels established under the Safe Drinking Water Act is required from water supply wells prior to providing the domestic supply for the proposed project. Table 2-6 provides a summary of the analytical results.

Project Potable Water Supply Needs – Alternative A

Potable water demand calculations were prepared for this Alternative, and include residential demands for 143 five-acre lots including residential landscaping. It is estimated that 335 AFY of water demand is needed to serve the new project under Alternative A. The existing vineyard and open space/recreation demands will be served by blending groundwater from existing on-site irrigation wells and tertiary treated recycled water from the wastewater treatment facility. Peak hour demand for the potable water system is calculated as 655 gpm. Two new wells, rated at 750 gpm each, should be provided to supply the Project potable water, and for potable water supply redundancy.

Project Potable Water Supply Needs – Alternative B

Potable water demand calculations were prepared for this Alternative, and include residential demands for 143 one-acre lots including residential landscaping, and the Tribal Government Center. It is estimated that 106 AFY of water demand is needed to serve the new project under Alternative B. Although the Tribal Government Center increases water demand compared to Alternative A, this Alternative B also reduces residential irrigation demand considerably with the smaller lot size. As with Alternative A, the vineyard and open space/recreation demands, and Tribal Government Center landscape irrigation, will be served by groundwater supply wells and by tertiary treated recycled water from the wastewater treatment facility. Peak hour demand for the potable water system is calculated as 230 gpm. The estimated peak hour demand is considerably lower than Alternative A, due to significant reduction in residential irrigation demand. Two new wells, rated at 500 gpm each, should be provided to supply the Project potable water, and for potable water supply redundancy.

Table 2-6. Water Quality Results – Baseline Well #2

Analyte	Units	Results	MCL
Total Hardness (as $CaCO_3$)	mg/L	386	--
Calcium	mg/L	31	--
Magnesium	mg/L	75	--
Potassium	mg/L	2	--
Sodium	mg/L	26	--
Bicarbonate	mg/L	440	--
Sulfate	mg/L	22	250
Chloride	mg/L	35	250
Nitrate (NO_3)	mg/L	5.8	45
Fluoride	mg/L	0.2	2
pH	pH units	7.8	--
Electrical Conductivity	umhos/cm	827	1600
Total Dissolved Solids	mg/L	480	1000
Color	units	ND	15
Odor	TON	ND	3
Turbidity	NTU	ND	5
MBAS	mg/L	ND	0.5
Aluminum	ug/L	ND	1000
Antimony	ug/L	ND	6
Arsenic	ug/L	2	10
Barium	ug/L	269	1000
Beryllium	ug/L	ND	4
Cadmium	ug/L	ND	5
Chromium	ug/L	27	50
Copper	ug/L	ND	1000
Iron	ug/L	ND	300
Lead	ug/L	ND	15
Manganese	ug/L	ND	50
Mercury	ug/L	ND	2
Nickel	ug/L	ND	100
Selenium	ug/L	ND	50
Silver	ug/L	ND	100
Thallium	ug/L	ND	2
Zinc	ug/L	ND	5000

mg/L = milligrams per liter
ug/L = micrograms per liter
umhos/cm = micromhos per centimeter
TON = Threshold Odor Number
NTU = Nephelometric Turbidity Units
MCL = Maximum Contaminant Level
ND = Not detected above laboratory detection limit
NOTE: Samples obtained December 22, 1999

PROPOSED POTABLE WATER SUPPLY SYSTEM

The two existing wells on site have provided irrigation water supply for the vineyard and are considered reliable for future irrigation use based on their well design, their location within the deepest part of the groundwater basin, and the observed trend in rising water levels in the area. Although the basin is in a state of overdraft, changing pumping patterns and the importation of State Water in the 1980s and 1990s altered the amount of water extracted from the basin, resulting in more balanced groundwater conditions (SB County Groundwater Report, 2008). These changes in water use and the rising water level trends in the project area suggest that existing production rates of the two project irrigation wells can be relied upon to meet future irrigation demands.

To meet the proposed project potable water demands, however, two new potable water supply wells would be required to provide a groundwater supply redundancy, and also provide for flexibility in pumping schedules. While new wells located in the northeastern portion of the project area would likely provide the best onsite production, the nearby offsite wells northeast of the project area could experience significant water-level impacts from new project wells in that location (Figures 2-1 and 2-2). A new well located north of the Baseline fault but away from the northeast corner of the property could be expected to provide a reliable and adequate water supply and be less likely to cause significant offsite impacts than new wells located near the northeasterly project boundary. A new well placed south of, but near the Baseline fault would likely produce a reliable and adequate water supply for the project, and could be expected to cause minimal or no offsite water-level impacts. There are fewer offsite wells east of the southern project area than northeast of the site, and a well south of the fault could be located several thousand feet from the nearest offsite wells. Impacts across the fault from a new well would be minimized because of the expected restrictions to groundwater flow along the fault plane. Also, a well drilled south of the fault could tap permeable sands of the relatively unexploited Careaga Formation as it becomes shallower to the south away from the fault.

WASTEWATER

Wastewater treatment for both project Alternatives A and B is proposed to include onsite tertiary treatment for recycling and reuse in vineyard and landscape irrigation. The wastewater flow is assumed in this study to be equivalent to 90 percent of residential and Tribal Government Center indoor water use. The treated effluent will be pumped to the irrigation reservoir where it could be blended with groundwater produced from onsite wells and be available for vineyard and landscape irrigation. Because of probable increased storage requirements to accommodate effluent volumes, an additional treated effluent storage reservoir may be necessary (refer to Chapter 3 for further discussion).

The project may require up to two sites to be used for treated effluent storage ponds. The pond locations and design will be dependent on the treatment plant site selection and on site conditions. The options for percolation pond designs that could be used for wastewater disposal are very limited at the site because of the character of underlying soils. However, based on the water balance presented in Chapter 3, it is not anticipated that percolation disposal will be required. The Santa Ynez area has been identified as an area where existing septic system use is causing problems (Questa, 2003). According to the Septic System Sanitary Survey for Santa Barbara County report (Questa, 2003), one of the sources of the

wastewater related problems was the "highly restrictive soil-site conditions for a large portion of the area…" Because of the soil conditions, many disposal systems in the vicinity of the project area are based on drywell designs (Santa Barbara County, Office of Long Range Planning, September 2009).

The soil types found at the site are shown on Figure 2-3 and are based on soil surveys by the Natural Resources Conservation Service. The capacity of the soils to transmit water is considered very low to moderately low in soils underlying 73 percent of the project site and moderately low to moderately high in soils underlying 21 percent of the site. The Botella loam (BoA) has the highest saturated hydraulic conductivity, but it is located within the vineyard in the northern portion of the site. The Chamise shaly loam (ChF) extends across a large area and may provide minimal percolation rates; however, it is a thin soil underlain by less permeable material and underlies moderately steep slopes. Based on the soil types and conductivities listed in the soil survey, percolation rates underlying the site are generally inferred to be very slow. Soil characteristics for each soil type are summarized in Table 2-7 below:

Table 2-7. Soil Properties on Project Site

Soil Type	Percent of Property	Percent Slopes	Depth to restrictive feature (inches)	Drainage Class	Capacity of the most limiting layer to transmit water
BoA	5.4	0-2	80	well drained	mod high (0.2 to 0.57 in/hr)
CeC	0	5-9	34-46	well drained	mod low to mod high (0.06 to 0.20 in/hr)
ChF	21.2	15-45	22-40	well drained	mod low to mod high (0.06 to 0.20 in/hr)
ChG2	0.1	30-75	10-20	well drained	mod low to mod high (0.06 to 0.20 in/hr)
PtC	30.7	2-9	20-26	well drained	very low to mod low (0.00 to 0.06 in/hr)
PtD	13.3	9-15	12-20	well drained	very low to mod low (0.00 to 0.06 in/hr)
PtE	15.7	15-30	6-26	well drained	very low to mod low (0.00 to 0.06 in/hr)
SnC	7.9	2-9	20-30	mod well drained	very low to mod low (0.00 to 0.06 in/hr)
SnD	5.7	9-15	20-29	mod well drained	very low to mod low (0.00 to 0.06 in/hr)

Base map: U.S. Department of Agriculture, NRCS, Custom Soil Resource Report for Northern Santa Barbara Area, California

0 2,000 4,000

Scale: 1 inch = 2,000 feet

Explanation

BoA	Botella loam, 0-2% slopes
CeC	Chamise sandy loam, 5-9% slopes
ChF	Chamise shaly loam, 15-45% slopes
ChG2	Chamise shaly loam, 30-75% slopes, eroded
PtC	Positas fine sandy loam, 2-9% slopes
PtD	Positas fine sandy loam, 9-15% slopes
PtE	Positas fine sandy loam, 15-30% slopes
SnC	Santa Ynez gravelly fine sandy loam, 2-9% slopes
SnD	Santa Ynez gravelly fine sandy loam, 9-15% slopes

Figure 2-3
Soil Survey Map
Chumash Camp 4
Cleath-Harris Geologists

IMPACTS

The Santa Barbara County Groundwater Report for 2008 states that the Santa Ynez Uplands Groundwater Basin is in overdraft by about 2,000 AFY based on 2001 estimates. Available storage within the basin is estimated to be about 900,000 AF (La Freniere and French, 1968). Safe yield of this basin is estimated to be 11,500 AFY (for gross pumpage) and estimated pumpage of the basin is 11,000 AFY (Ahlroth, 2001).

Implementation of the proposed project results in a net increase in water production of 335 AFY for Alternative A, and 106 AFY for Alternative B. Should the basin be in overdraft, any increase in groundwater production in the basin adds to that deficit; however, groundwater levels in U.S. Geological Survey monitored wells to the north, east and west of the site have risen since the mid 1990s. Because of the increased importation of water that offsets pumping in the basin, these stabilizing water levels indicate a lessened severity of overdraft conditions in the project area.

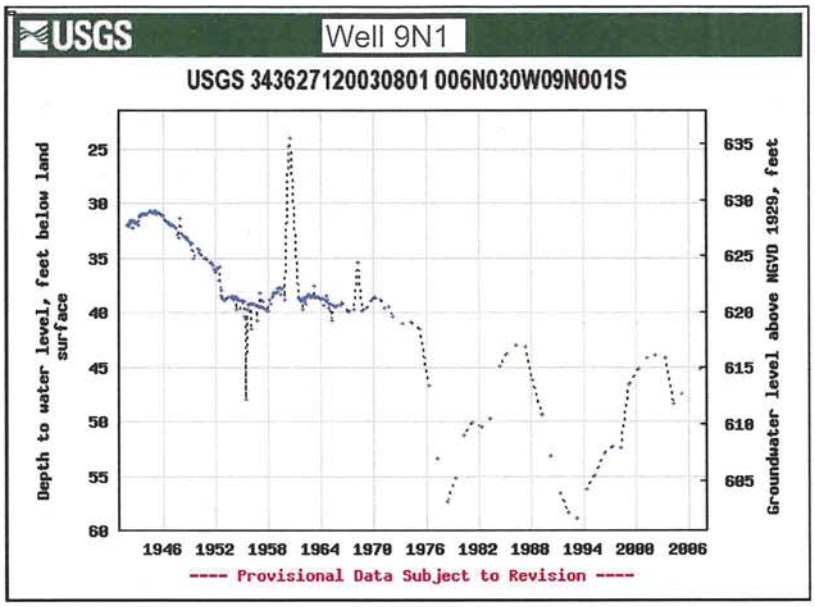
Hydrographs for the U.S. Geological Survey monitored wells are shown in Figures 2-4 and 2-5. A map showing locations for other offsite wells for which water level data are available with the DWR is shown on Figure 2-2. Increased well production above existing conditions at the site may adversely impact neighboring wells depending on where the onsite wells are located and the amount of pumping that occurs. The recommendations in the following paragraph are provided to reduce/prevent these potential impacts.

RECOMMENDATIONS TO REDUCE IMPACTS

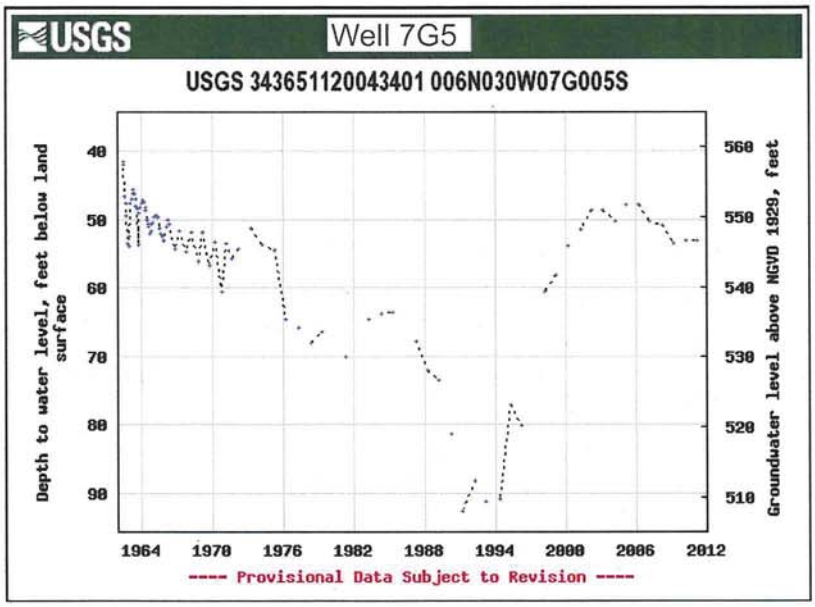
Potential impacts to offsite wells may be reduced through various options that would reduce groundwater production and/or use imported water to meet demand. Water conservation methods may be appropriate for residential indoor use, Tribal Government Center use, residential landscape use and in the existing vineyard irrigation. There are many resources and water conservation programs/techniques available. For example, the County of Santa Barbara Public Works Department has information on water conservation measures on their website (http://www.sbwater.org/), some of which have been incorporated into this report. These conservation measures are consistent with best management practices to reduce water demands. In addition to use of low flow fixtures and drought tolerant landscaping (see Chapter 1), the following recommendations can be considered:

- Drip irrigation, drought-tolerant planting, and dry-farming techniques are recommended where appropriate.
- A reduction in the amount of space set aside for residential landscape areas should be considered.
- Residential lawns have the highest water demand of the various land uses. Reduction of lawn size below the 0.15 acres per residence for Alternative A and the 0.1 acres for Alternative B would significantly reduce this demand.
- Irrigation controllers/timers should be used to control duration and timing of irrigation to minimize losses.
- Stormwater runoff capture for landscape irrigation, where possible.

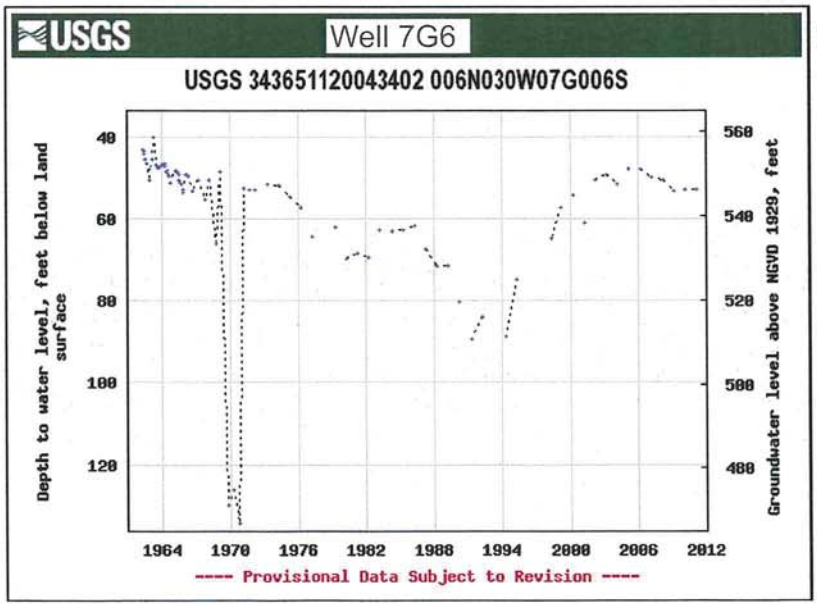
Siting proposed water wells as far as possible from existing offsite wells would result in lower water-level declines at neighboring wells. In addition, siting at least one of the new wells south of the Baseline fault (Figure 2-1) would reduce impacts to adjacent wells. The capacity of proposed wells to meet the project demand and water quality cannot be properly assessed without actually constructing and testing each well.



Latitude 34°36'27", Longitude 120°03'08" NAD27
Land-surface elevation 660.00 feet above NGVD29
The depth of the well is 160 feet below land surface.



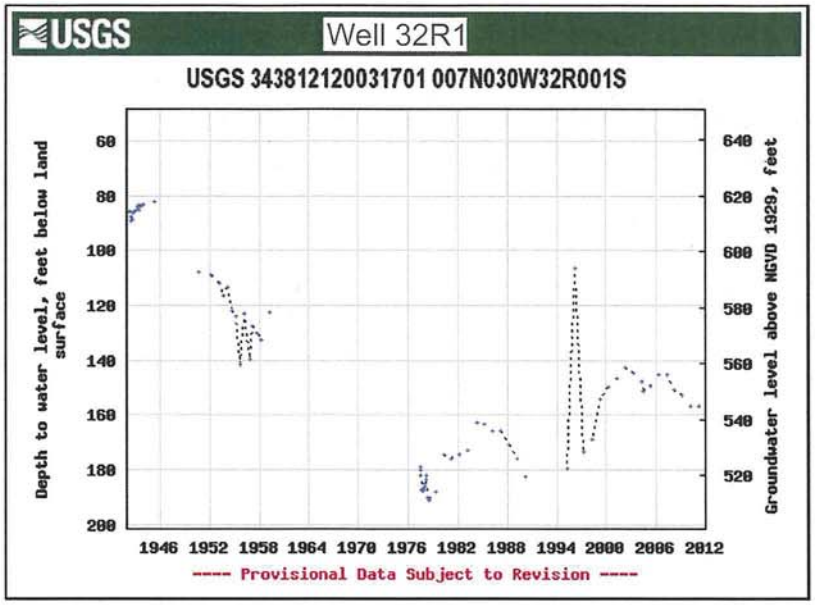
Latitude 34°36'51", Longitude 120°04'34" NAD27
Land-surface elevation 600.00 feet above NGVD29
The depth of the well is 158 feet below land surface.



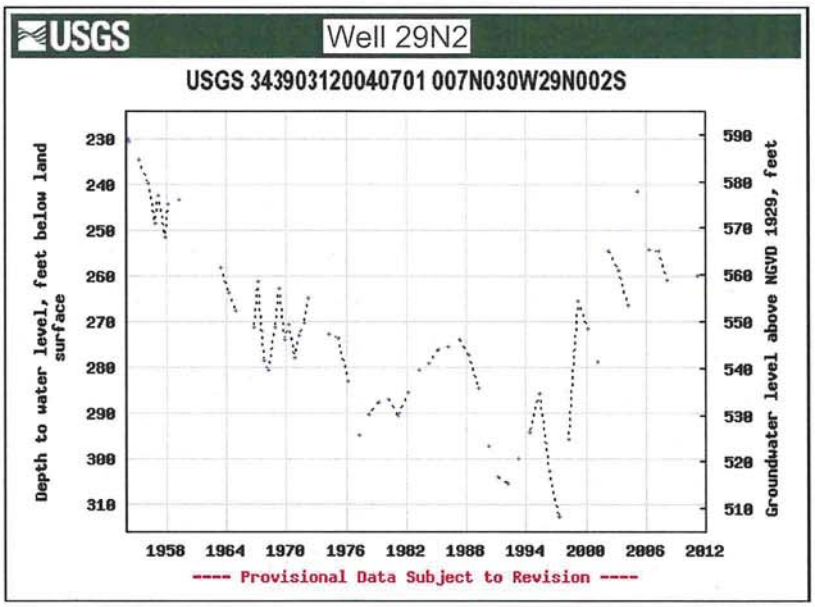
Latitude 34°36'51", Longitude 120°04'34" NAD27
Land-surface elevation 600.00 feet above NGVD29
The depth of the well is 410 feet below land surface.

Figure 2-4
Water Level Hydrographs
Wells 9N1, 7G5, 7G6
Chumash Camp 4

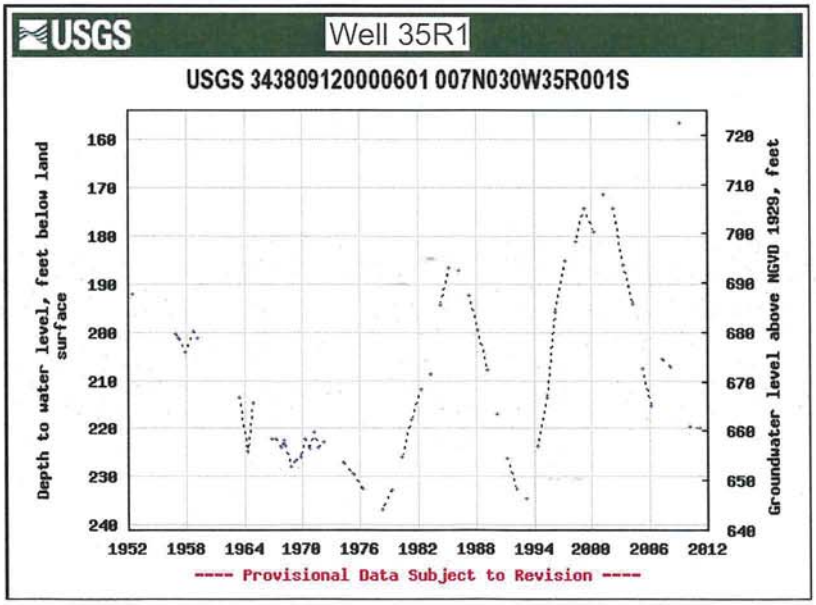
Cleath-Harris Geologists



Latitude 34°38'12", Longitude 120°03'17" NAD27
Land-surface elevation 701.00 feet above NGVD29
The depth of the well is 186 feet below land surface.



Latitude 34°39'03", Longitude 120°04'07" NAD27
Land-surface elevation 820.00 feet above NGVD29
The depth of the hole is 485 feet below land surface.



Latitude 34°38'09", Longitude 120°00'06" NAD27
Land-surface elevation 880.00 feet above NGVD29
The depth of the well is 288 feet below land surface.

Figure 2-5
Water Level Hydrographs
Wells 32R1, 29N2, 35R1
Chumash Camp 4

Cleath-Harris Geologists

PROPOSED WATER DISTRIBUTION SYSTEM

This section describes the proposed water distribution, storage and pumping system required to serve Project Alternatives A and B. The potable water and fire suppression demands are anticipated to be served by a single water distribution system. An overview of the water system, including storage reservoirs and pumping station, and water distribution system, are shown on Figures 2-6 and 2-7, respectively, for Alternatives A and B. It is noted that the locations of the two new domestic water supply wells are not shown on these figures, as the actual locations will need to be determined as part of detailed design.

Distribution System

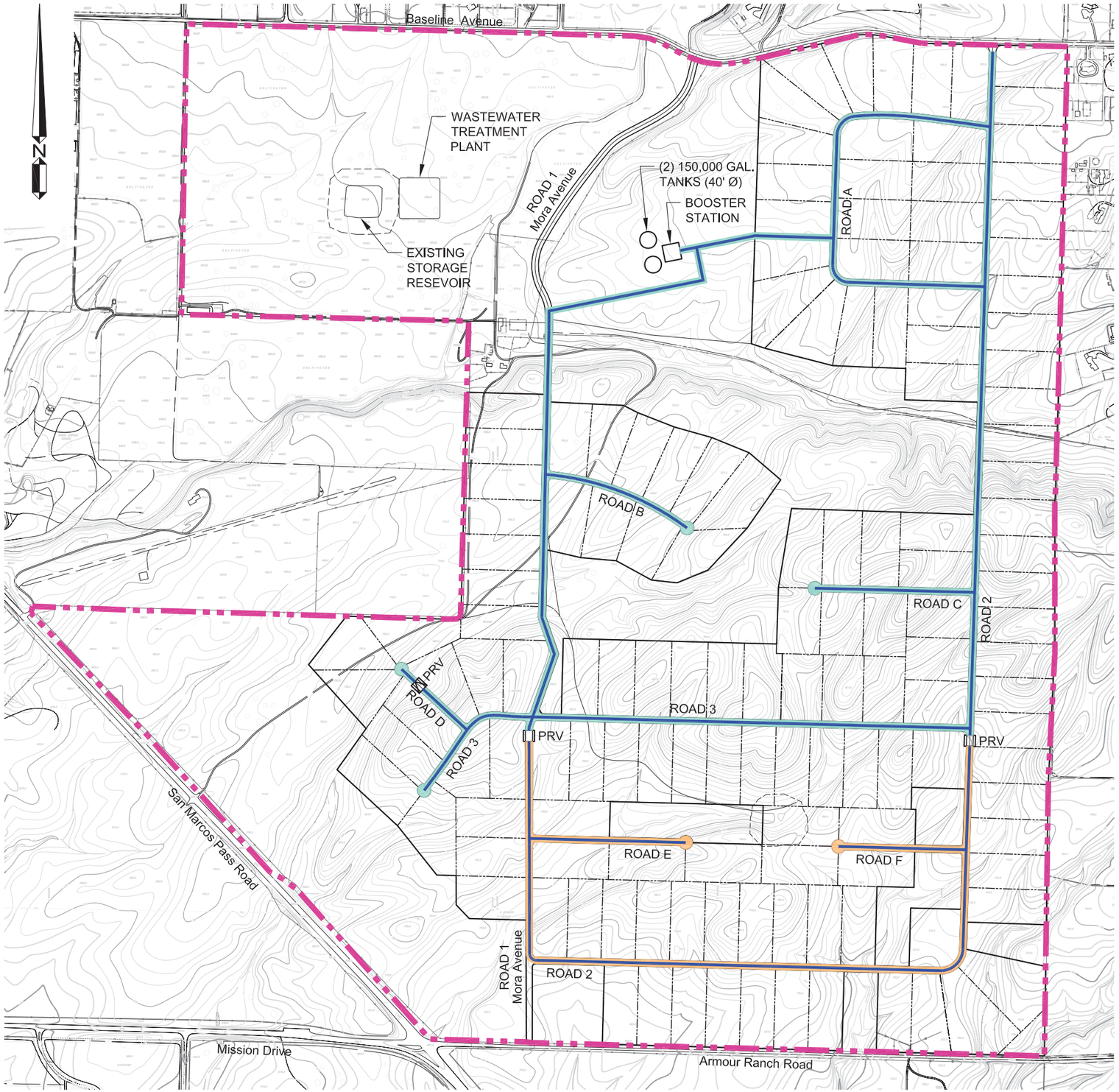
The following subsection describes water distribution system requirements for Alternatives A and B. The distribution system should be designed to ensure 40 psi pressure is delivered to each home during average demand conditions, and no less than 30 psi during peak hour flows. The distribution system should also be designed to ensure that during fire flows, a minimum residual pressure of 20 psi is achieved. If fire sprinklers are provided through the Project, higher residual pressures may be required. Again, this would be determined as part of detailed design.

Alternative A. Interior roads and residential lots will have an elevation ranging from 695 feet to 850 feet above mean sea level (MSL). With this range in elevations, the water distribution system should include at least two separate pressure zones, to avoid unusually high and low static pressures in the distribution system. Even with two pressure zones, it is likely that several of the residences may require individual pressure reducing valves at their residential water connection to the water main. A minimum pipe diameter of 8-inches is recommended throughout the water system. However, consideration to provide a 12-inch diameter main “loop” should be given, to reduce the potential for undesirable surge pressures (by reducing line velocities). A detailed hydraulic analysis of the water system is beyond the scope of this study; however, given the nature of the layout of the roads, high and low points, the need for several dead-end mains, and the required fire flows, hydraulic design considerations will be important in the overall design of the water system.

It is envisioned that there will be two pressure zones for this Alternative A, to best serve all of the residences, avoid excessive water line pressures, and to meet minimum service pressures. Refer to Figure 2-8 for a hydraulic profile graphically depicting the envisioned water pressure zones, in relationship to residences, water storage and pumping facilities, and water supply wells.

Alternative B. Similar to Alternative A, interior roads and residential lots will have an elevation ranging from 695 feet to 850 feet above mean sea level (MSL). Hydraulically, the water system will be very similar to Alternative A, with a reduced pressure zone for the southern “loop”. It is likely that the Government Center will be part of the main zone (higher pressure and hydraulic grade line). The pressure reducing station would be immediately downstream of the Government Center. The Government Center would have a water service lateral feeding off of the main line from the street, possibly with its own on-site water loop for water service and fire suppression, depending on layout of the Government Center facilities. This water system layout has no dead end mains like Alternative A; however, water main sizing should follow the same recommendations described for Alternative A. Details of the hydraulics would be determined

during detailed design of the Project. The water system hydraulics are shown, similar to Alternative A, in Figure 2-8.



LEGEND:

- PROJECT BOUNDARY
- WATER STORAGE TANK
- BOOSTER STATION
- WATER DISTRIBUTION PIPELINE, MAIN PRESSURE ZONE
- WATER DISTRIBUTION PIPELINE, PRV ZONE 1
- PRV — PRESSURE REDUCING STATION
- PRV — INDIVIDUAL RESIDENTIAL PRESSURE REDUCING VALVE

NOTES:

1. REFER TO FIGURE 1-1 FOR LAND USE DESIGNATIONS.
2. DOMESTIC WATER WELLS ARE NOT SHOWN AS LOCATIONS WILL NEED TO BE DETERMINED AT A LATER TIME.

GRAPHIC SCALE

0 1000' 2000'

(IN FEET)
1 IN = 1000 FT

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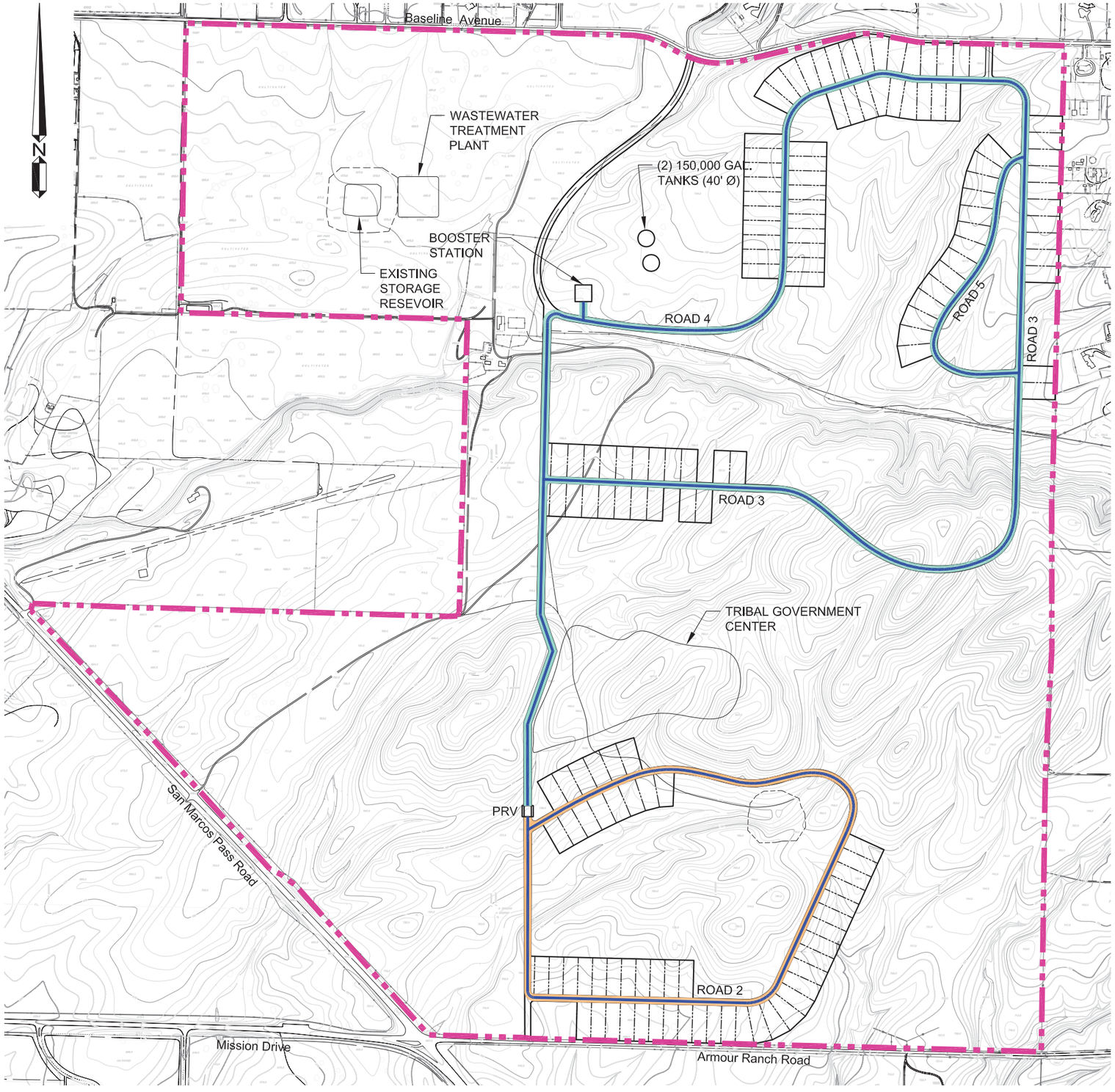
Figure 2-6
Waterline Layout - Alternative A



LEGEND:

- PROJECT BOUNDARY
- WATER STORAGE TANK
- BOOSTER STATION
- WATER DISTRIBUTION PIPELINE, MAIN PRESSURE ZONE
- WATER DISTRIBUTION PIPELINE, PRV ZONE 1
- PRV — PRESSURE REDUCING STATION

NOTES:

1. REFER TO FIGURE 1-2 FOR LAND USE DESIGNATIONS.
2. DOMESTIC WATER WELLS ARE NOT SHOWN AS LOCATIONS WILL NEED TO BE DETERMINED AT A LATER TIME.

GRAPHIC SCALE

0 1000' 2000'

(IN FEET)

1 IN = 1000 FT

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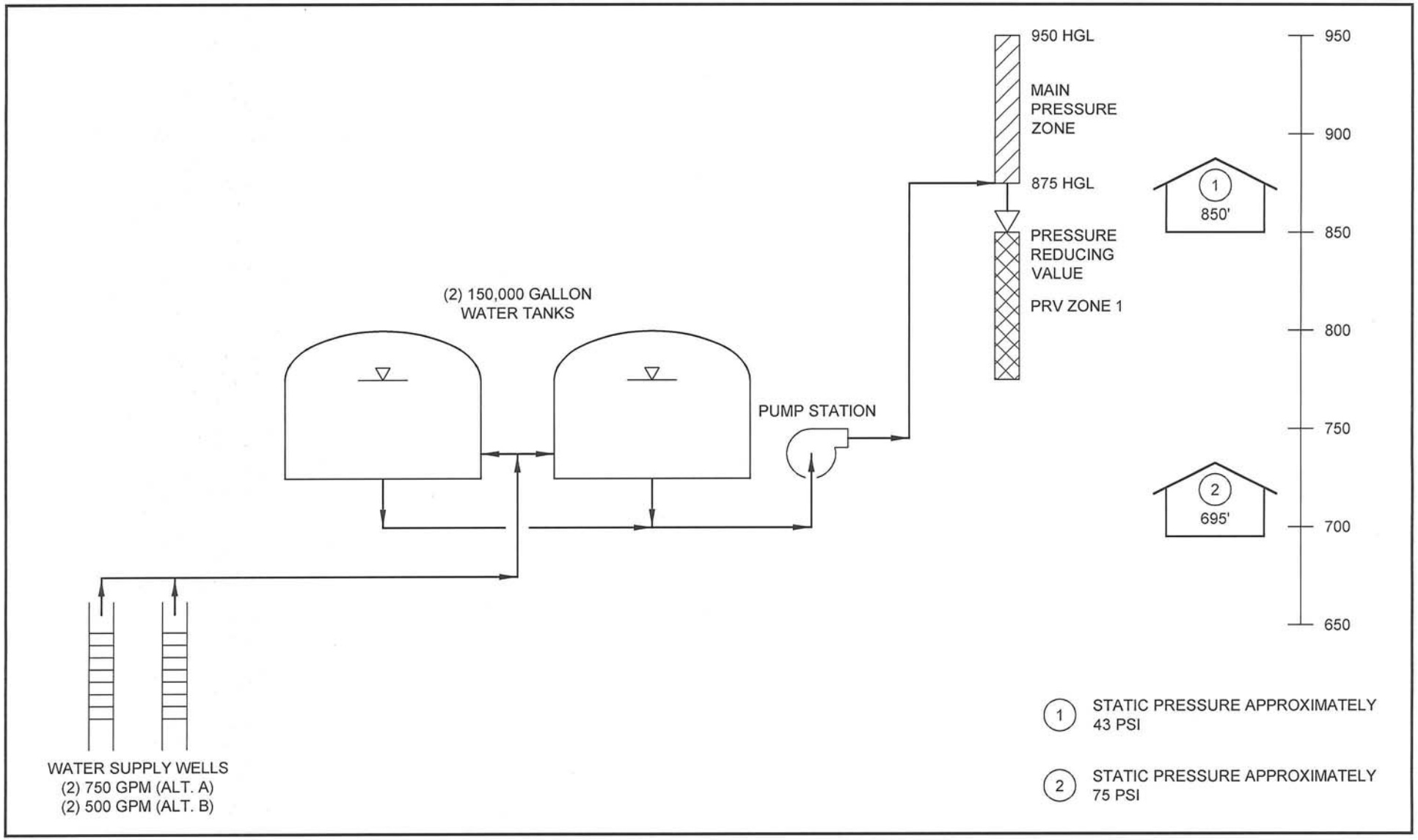
Figure 2-7
Waterline Layout - Alternative B



CHUMASH CAMP 4 FEE-TO-TRUST

Figure 2-8
Hydraulic Profile Of Water Distribution System

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Water Storage and Pumping

This section describes the proposed water storage and pumping facilities needed to serve the Project, Alternatives A and B. Storage requirements are summarized in Table 2-8.

Storage Requirements. Storage requirements for community systems are generally comprised of three components:

- Emergency Storage
- Fire Storage
- Operational Storage

Emergency Storage

Emergency storage is intended to provide for conditions such as extended power outages, pump failures, and similar problems. Most water planners accept that during emergencies, supply per capita may be reduced to minimum levels. Typically, on that basis, an emergency storage volume of 50 gpcd for three days is accepted as a reasonable value. Emergency storage for this Project is thus based on 500 permanent residents, for both Alternatives A and B.

Fire Storage

Fire storage is the volume of water needed to control an anticipated fire in a building or group of buildings. The determination of this storage is based upon a recommended flow rate, its duration, and a minimum residual pressure as established by the agency of interest. Based on experience in working with the County of Santa Barbara Fire Department, it is anticipated that with sprinklered buildings, the recommended fire flow will be 1,500 gpm for a duration of 2 hours.

Operational Storage

Operational storage is the amount of water needed to equalize the daily supply and demand. Without this storage, water production facilities large enough to meet the instantaneous peak demands of the system would be required. With adequate operational storage, well pumps can operate at the daily average rate, while storage facilities meet the hourly peaks. This operating method also prevents the unnecessary use of additional well pumps at times when electrical rates are the highest. Based on the typical daily water use patterns of most communities, it is recommended that the required operational storage be approximately 25 percent of the total water use for any given day. The American Waterworks Association (AWWA) Manual of Practice M-32 recommends operational storage of 20 to 25 percent of build-out average day demand for the given zone, or up to 15 percent of the ultimate maximum day demand. Storage recommendations for Alternatives A and B are based on storage for 25% of maximum day demand.

Table 2-8. Summary of Storage Requirements – Alternatives A and B

Alternative	Storage Component (rounded numbers)			
	Emergency	Fire	Operational	Total (rounded up)
A	75,000	180,000	23,000	300,000
B	75,000	180,000	25,000	300,000

It is noted that detailed siting of water storage tanks is beyond the scope of this study. However, based on site topography, and location of proposed residential lots, both Alternatives will require some degree of pumping from storage reservoirs. Full gravity flow and pressure from storage reservoirs cannot be accommodated due to the higher elevations of the lots. For both Alternatives, water storage was envisioned to be at approximately elevation 725. The tanks should be located where relatively accessible for maintenance, while protecting the existing view sheds.

The water storage reservoirs are envisioned to be welded steel tanks (at-grade), meeting current standards for tank design and seismic requirements. Alternatively, should it be desired to further screen or hide the tanks from view, the tanks could be pre-stressed concrete tanks that can be partially or fully buried.

Booster Station – Alternative A. The water system will require a booster station, rated at 2,250 to 2,500 gpm to achieve fire flow demand of 1,500 gpm plus domestic demands. Given site and storage reservoir elevations, the pumps will need to be rated for approximately 250 feet total dynamic head (TDH). Consideration could be given to locating the storage reservoirs at a higher elevation and thus reduce the pumping head requirements of the booster station. This would increase the head requirements for pumping from the wells to the storage reservoirs, however. The pump station should be designed to also operate efficiently at low flows, thus provision for a small “jockey” pump to handle night-time flows should be considered. The booster station should be equipped with emergency standby power provisions (generator) to ensure uninterrupted service in the event of power outages.

Booster Station – Alternative B. Booster Station recommendations for this Alternative would be very similar to that described for Alternative A, except that the total pumping capacity could be reduced slightly to 2,000 gpm.

CHAPTER 3

WASTEWATER SYSTEM

The wastewater system includes collection, treatment and reuse/disposal of all products of the treatment processes including:

- Effluent water
- Bio-solids
- Screenings

The intent is to provide a high quality system so that the treatment meets the water quality for unrestricted reuse on the property, that bio-solids can be disposed economically, and screenings can be disposed in a conventional (publicly-owned and operated) landfill.

WASTEWATER FLOWS

Based on Tables 2-1, 2-2 and 2-3 showing domestic water demands (and Tribal Government Center water demands for Alternative B), wastewater flows were calculated based on:

- 90% of domestic water demand generates wastewater flow
- Permanent population of 500 (3.5 persons per household)
- 75 employees at Tribal Government Center
- 100 Tribal Government Center events per year, drawing up to 1,000 people per event, including food preparation for these events

Based on the above assumptions and factors, wastewater flows to the wastewater treatment plant (WWTP) are summarized as follows:

- Alternative A – 41,000 gpd average dry weather flow (ADWF)
- Alternative B – 44,000 gpd ADWF

WASTEWATER COLLECTION SYSTEM

The collection system will provide capacity to convey the ultimate wastewater flow at peak hydraulic conditions. These conditions include the potential for a diurnal maximum flow when all development is complete and the flows may include an allowance for nominal pipeline infiltration as considered feasible according to maximum EPA limitation (200 gallons per inch diameter per mile per day).

The collection system will also include a number of gravity sewer manholes, 48” diameter, and spaced at intervals for ease of access for maintenance. Typically, manhole spacing for the size pipelines considered will be on the order of 300 feet, and also where significant grade breaks and bends occur in the gravity sewer system.

Gravity Sewer System

The gravity sewer system will be comprised of four-inch diameter laterals between buildings and street mains, eight-inch diameter street mains, and interceptor pipelines ranging in diameter from 8-inches to 15-inches.

The minimum diameter for street mains and interceptors is eight inches for ease of maintenance. The actual design will be based on two criteria:

- Capacity is required for peak hydraulic flow and this determines the combination of diameter and slope
- Because slope is also a result of the topography, the pipe diameter may be controlled as a function of slope and the ability to meet a minimum velocity which would be defined as meeting a velocity of 1.3 feet per second at a depth of flow ratio to pipe diameter of 0.2. (Steeply sloping topography makes this issue disappear. When the topography is relatively flat, to maintain an adequate slope may require inclining pipeline profiles deeply below grade in order to satisfy the minimum velocity requirement

Sewage Lift Stations

Sewage lift stations will be constructed as submersible pumping unit stations utilizing circular precast concrete vaults, lined for corrosion protection, equipped with duplex pumping units for redundancy and backup, and level controls for starting, stopping pumping units and signaling alarms for failure conditions.

If the depth of pipelines becomes excessive (more than 20 feet below grade), then consideration is given to the benefit of raising the pipeline profile to a shallower level by the installation, and use of a lift station. Where individual homes may require a solitary pumping unit to pump residential wastewater into the collection system, a sub-grade sump with an automatically- controlled grinder pump will be installed on the owner's property where the home waste plumbing will directly discharge into the sump and the sump discharge will go into a street main.

The collection system is envisioned to be a combination of eight-inch diameter PVC gravity pipelines, four-inch diameter PVC force mains, and small pumping stations and individual home grinder pump sump installations. Details of pipeline diameter and material selection would be confirmed during detailed design.

Collection System – Alternative Layout A. The composition of this system is based on the Program definition described in Chapter 1 of this report, and Figure 1-1. The recommended sewer collection system layout for Alternative A is presented in Figure 3-1.

LEGEND:

PROJECT BOUNDARY

GRAVITY SEWER

(X) REFER TO TABLE 3-1 FOR CORRESPONDING SEWER PIPE DESCRIPTIONS

NOTE:

REFER TO FIGURE 1-1 FOR LAND USE DESIGNATIONS

GRAPHIC SCALE

0 1000' 2000'

(IN FEET)

1 IN = 1000 FT

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Figure 3-1
Sewer Collection System Layout - Alternative A

The pipelines are summarized in Table 3-1. For Alternative A, the topography, if not modified, would lead to the installation of as many as 10 pumping stations and short force mains or inverted siphons to cross beneath washes. However, for planning purposes, it will be assumed that these wash crossings will be graded for the dual objectives of providing a manageable roadway profile for ease of transportation and to allow a continuous profile of gravity sewer pipelines.

Table 3-1. Summary of Sewer Pipelines – Alternative A

Line Number	Description	Length, feet
1	8" gravity sewer, Road 2 connecting with Road A, south of Baseline Ave. to intersection with Sewer Pipeline 14	2,500
2	8" gravity sewer, Road A loop along a loop, connecting to Road 2	3,500
3	8" gravity sewer, Road 4 to Sewer Pipeline 14	1,900
4	8" gravity sewer, Road C connecting to Road 2	1,300
5	8" gravity sewer, Road 2 to Road 1 loop, extending from Road C to Road 1, to Sewer Pipeline 15	12,500
6	8" gravity sewer, eastern portion of Road 3	1,600
7	8" gravity sewer, Road F	1,000
8	8" gravity sewer, Road E	1,200
9	8" gravity sewer, western portion of Road 3 connecting through property to Road 1	1,200
10	8" gravity sewer, Road 3 west of Road 1	500
11	8" gravity sewer, western portion of Road 3	1,500
12	8" gravity sewer, aligned west and north of Road 3 properties, west of Road 1	1,300
13	8" gravity sewer, Road B	1,750
14	8" gravity sewer, connecting Road 2 to Road 1 paralleling large east-west drainage wash	3,600
15	12" to 15" gravity interceptor line to WWTP from Road 1	1,500
TOTAL		36,850

See Figure 3-2 for layout of sewer collection system.

Collection System – Alternative Layout B. The composition of this system is based on the Program definition described in Chapter 1 of this report, and Figure 1-2. The recommended sewer collection system layout for Alternative A is presented in Figure 3-2. The pipelines and a single pumping station are summarized in Table 3-2 for Alternative B. Just as in Alternative A, the assumption is made that numerous washes will be graded to facilitate both ease of roadway driving and to allow a continuous gravity sewer pipeline through the washes. There will be one unavoidable exception in Alternative B where a pumping station will be required.

WASTEWATER TREATMENT SYSTEM

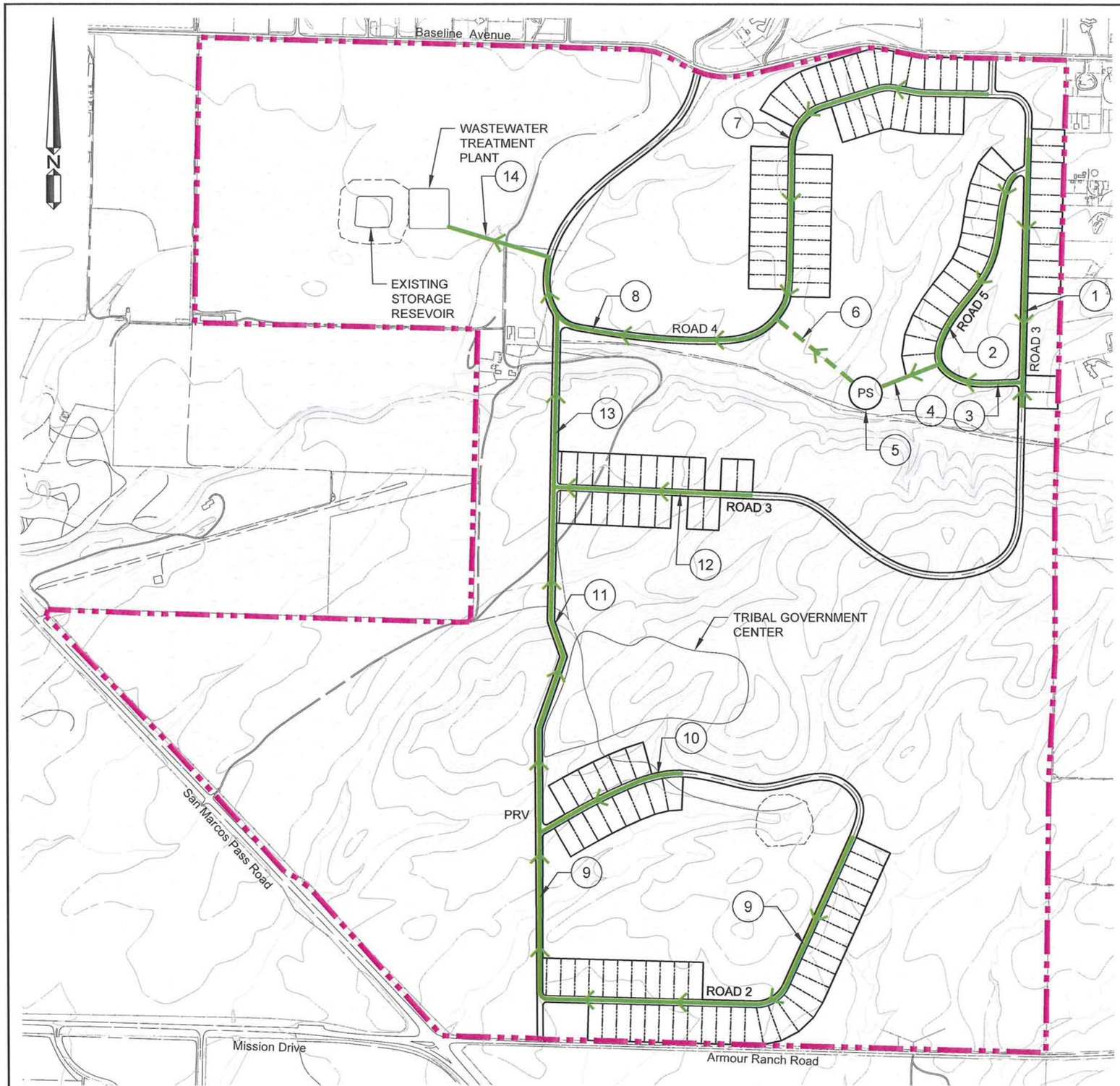
The wastewater treatment system will be designed to receive a high strength domestic waste based on experience of this type of customer base including residential, some office, and Tribal Government Center wastewater. It is envisioned that the Government Center will include food preparation facilities, and thus a higher strength wastewater is anticipated. For planning, the strength of wastewater will be assumed to be as presented in Table 3-3. The table also presents the required effluent quality for the purpose of recycling as an unrestricted (meeting same requirements as California Code, Title 22) reuse.

Wastewater Process Selection

The objective is to produce a reusable effluent that does not create a restrictive constraint on the area of beneficial use. This means that in the issue of health and safety, the industry defined quality is known as a “tertiary - 2.2 coliform” effluent. It means the effluent is the highest level of treatment currently practiced and offers the widest possible array of reuse options. From a practical standpoint, it means that the effluent can be spray irrigated in agricultural areas without restriction of humans and animals in the area of application.

In order to process the expected wastewater, based on volume, strength, and the variations of flow, the number of options for treatment has been narrowed to three that are deemed both feasible and cost-effective:

1. Sequencing batch reactor with tertiary filtration, ultraviolet (UV) disinfection, and sludge holding (and probably with dewatering)
2. Multiple Stage Activated Biological Process (MSABP), with tertiary filtration, and UV disinfection
3. Modified Ludzack-Ettinger (mLE) process reactor with tertiary filtration, UV disinfection, and sludge holding (and probably with dewatering)



LEGEND:

- PROJECT BOUNDARY
- (PS) SEWAGE PUMP STATION
- GRAVITY SEWER
- FORCE MAIN
- (X) REFER TO TABLE 3-2 FOR CORRESPONDING SEWER PIPE DESCRIPTIONS

NOTE:

REFER TO FIGURE 1-2 FOR LAND USE DESIGNATIONS

GRAPHIC SCALE

0 1000' 2000'

(IN FEET)

1 IN = 1000 FT

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Figure 3-2
Sewer Collection System Layout - Alternative B

Table 3-2. Summary of Sewer Pipelines – Alternative B

Line Number	Description	Length, feet
1	8" gravity sewer, Road 3 south of Baseline Avenue	1,800
2	8" gravity sewer, Road 5 (flowing southerly) to Sewer Pipeline 4	1,800
3	8" gravity sewer, Road 5 (flowing northerly) to Sewer Pipeline 4	600
4	8" gravity sewer from Road 5 to Pumping Station	500
5	Pumping Station	
6	4" force main from Pumping Station to Road 4	1,300
7	8" gravity sewer, Road 4 to tie-in with force main	4,000
8	8" gravity sewer, Road 4 from force main to Road 2	2,300
9	8" gravity sewer, eastern loop of Road 2 to Sewer Pipeline 10	5,200
10	8" gravity sewer, northwestern loop of Road to north/south leg of Road 2	1,300
11	8" gravity sewer, Road 2 between Sewer Pipeline 10 and Road 3	2,500
12	8" gravity sewer, western leg of Road 3	1,700
13	8" gravity sewer, Road 2 between Road 3 and Road 4	1,500
14	12" to 15" interceptor sewer, Road 4 to WWTP	1,500
TOTAL		26,000

See Figure 3-2 for layout of sewer collection system.

Table 3-3. Influent and Effluent Wastewater Characteristics

	Constituent	Units	Value
Influent:			
(BOD)	Biochemical Oxygen Demand	mg/l	600
(TSS)	Total Suspended Solids	mg/l	350
(TKN)	Total Kjeldahl Demand	mg/l	75
(TP)	Total Phosphorus	mg/l	20
Effluent:			
(BOD)	Biochemical Oxygen Demand	mg/l	10
(TSS)	Total Suspended Solids	mg/l	10
NH3	Ammonia	mg/l	1
T.I.N.	Total inorganic nitrogen	mg/l	8
T.N.	Total nitrogen	mg/l	8
T.P.	Total phosphorus	mg/l	3
Coliform	Total coliform	mpn/100 ml	2.2
	Turbidity	NTU	2

Each of these combinations is capable of meeting the project needs; each has some specific advantages which will be summarily presented in this chapter. For purposes of the recommendations in the wastewater system, only one of these three will be presented in detail.

Wastewater Treatment Plant Capacity – Alternative A. Based on the development plans proposed for Alternative A, as summarized in Table 1-1, the capacity required in the treatment plant at build out will be 41,000 gallons per day of average dry weather flow. The distinctions of Alternative A that separate it from Alternative B are:

- 143 Five acre lots
- 206 acres of open space/recreational use and trails

For the normal design practice, a peaking factor must be estimated to apply to the average dry weather flow. Small systems require high peaking factors. For this system, a peaking factor of 3.0 will be assumed.

Wastewater Treatment Plant Capacity – Alternative B. Based on the development plans proposed for Alternative B, as summarized in Table 1-1, the capacity required in the treatment

plant at build out will be 45,000 gallons per day of average dry weather flow. The distinctions of Alternative B that separate it from Alternative A are:

- 143 One- acre lots
- 755 acres of open space/recreational use and trails
- Addition of Tribal Government Center

For the normal design practice, a peaking factor must be estimated to apply to the average dry weather flow. Small systems require high peaking factors. For this system, a peaking factor of 3.0 will be assumed.

Wastewater Treatment Plant Description

The wastewater treatment plant will be capable of processing the influent wastewater and producing the effluent by means of the plant process units outlined below in Table 3-4.

Table 3-4. Description of Wastewater Treatment Process Units

Item No.	Process Unit	Number of Units	Type	Criteria
	Headworks:			
1	Screens	2	Auto	1 mm
2	Grit cyclone *	2		PWWF
3	Grit classifier *	1		PWWF
	Biological Treatment:			
4	Treatment Basins	12	A	ADWF
5	Effluent Equalization Basin	1	A or B	PWWF
	Final Effluent:			
6	Filtration	2	Cloth	ADWF
7	Disinfection	2	UV	ADWF
	Equipment:			
8	Influent Pumping Units	3	Submersible	PWWF
9	Internal Recyle Pumping	4	Centr.	ADWF
10	Effluent Pumping	2	Centr.	ADWF
11	Aeration Blowers	3	Pos Displ.	Max Mo.

A=Concrete
B=Steel
** - Optional - assessment of grit potential during design*

Influent Pumping and Transfer Pumping. There will be two categories of pumping units; submersible type pumping units for influent pumping, and end suction dry pit units for transfer puming. The submersible units are installed with an easily removable connection with cable for ease of pulling pumps for maintenance.

Headworks (Screening and De-gritting). Screening is a highly variable choice of equipment. The size of treatment plant, the nature of waste material, and objective for removal of material are equally important factors in choosing the right screen. Maintenance requirements are dependent on the choice made. For the plant proposed, there are at least three possible choices that would be appropriate. The three most logical types for the proposed plant are:

- A cylindrical metal (stainless steel) screen with small (1 mm) opening size
- A continuous belt type unit with medium opening size range (6 to 8 mm)
- A step screen also with medium opening size range (6 to 8 mm)

Detailed selection of such equipment is beyond the scope of this study.

Biological Treatment**.** The three types of biological treatment processes described above (SBR, MSABP, and mLE) will be described in detail below: Each of these processes will produce a secondary effluent of a quality so that a conventional tertiary filtration process, followed by disinfection, will produce a final effluent meeting the requirements of effluent for unrestricted reuse.

1. Sequencing batch reactor – this process has certain unique characteristics as presented in the Table 3-5.

Table 3-5. Characteristics of Sequencing Batch Reactor

Factor	Unique Characteristics of the Overall Process Train
Tanks	Tanks are large - based on 24-hour detention time
Energy	Not energy efficient due to variable water level
Flexibility	Very flexible - capable of high turndown ratio
Operation	Simple operation - controlled by single PLC
Screenings	Normal amount based on 6-8 mm opening size
Effluent	Excellent quality
Bio-solids	Large quantity
Concept	Alternate tanks fill and process in batches based on 5 cycles per 24 hrs

2. Multi-stage activated biological reactor – this process' characteristics are presented in Table 3- 6.

Table 3-6. Characteristics of Multi-Stage Biological Reactor

Factor	Unique Characteristics of the Overall Process Train
Tanks	Tanks are large - based on 40-hour detention time
Energy	Energy efficient due to constant water surface
Flexibility	Moderately flexible
Operation	Simple operation - controlled by single pLC
Screening	Large amount based on 1 mm screen size
Effluent	Superior quality - clarifier is just a backup
Bio-solids	Minimal quantity - mostly reduced in secondary process
Concept	12 stages of bioreactor - multiple stages of oxic and anoxic biology

3. Modified Ludzack Ettinger - this process' characteristics are presented in Table 3-7.

Table 3-7. Characteristics of Modified Ludzack Reactor

Factor	Unique Characteristics of the Overall Process Train
Tanks	Tanks are small - based on 12 hour detention time
Energy	Energy efficient due to constant water surface
Flexibility	Moderately flexible
Operation	Requires daily interaction by operator
Screenings	Normal amount based on 6-8 mm opening size
Effluent	Excellent quality
Bio-solids	Large quantity
Concept	High rate with multiple recycle streams in 2-stage reactor

Flow Equalization. Effluent equalization is valuable ahead of the tertiary and disinfection processes because it eliminates the hydraulic flow variability, and thus minimizes size/capacity requirements of downstream process units. The design capacity then gets reduced to the maximum month flow at the build out stage of development, as opposed to peak hourly flow. The equalization basin is usually sized for a fraction (20 to 40 percent) of the daily flow at the rated average capacity of the plant. In this case, the volume would be between 10,000 and 20,000 gallons. Because the plant is small (only 50,000 gallons per day) the more conservative volume of 20,000 gallons is recommended.

The basin should be designed for the supply side to be provided by gravity and the discharge side by pumping if the site topography and geology restricts the ability to allow a continuous gravity system to the final effluent discharge point.

Tertiary Treatment**.** **T**he most cost effective type of system for this plant would be a cloth disk filter unit. The units are constructed to have a basin enclosure that contains the secondary effluent submerging a number of cloth covered disks that rotate and allow the water to penetrate the cloth, leaving behind the filter reject materials, primarily particulate material that represents suspended solids, and turbidity. The cloth disks are periodically sprayed with a clean water stream to remove accumulated particulates from the individual disks. Also, a periodic backwash cycle pumps water fro the basin as the cumulated reject material increases within the basin. However, a steady supply of secondary effluent enters the basin allowing the disk filters to continuously filter uninterrupted.

Disinfection. Two methods of disinfection are practical for the 50,000 gpd size plant. The easiest to operate is the ultraviolet (UV) type. For such a small size, the low pressure, low intensity type is the most cost-effective. Several manufacturers make this type of UV system. They are available in both open channel and closed conduit configuration. The advantage of open channel is ease of access and the advantage of closed conduit is compactness.

Generally speaking, a UV system is more expensive to operate than a sodium or calcium hypochlorite disinfection system because the cost of electrical power used is more expensive than the cost of chemicals. However, on a small system, the cost of maintenance becomes a more significant factor. Maintaining a chemical feed system such as hypochlorite requires more diligence and attention to equipment condition than does a UV system. Also, hypochlorite systems are intricate in the feed and control systems and generally require periodic surveillance to make sure all parts are functioning properly.

SUMMARY OF ALTERNATIVE PROCESS TRAINS

The complete process trains for three alternatives are compared in Table 3-8. A process schematic of the entire recommended process train is presented in Figure 3-3. This schematic illustrates the sequence of the primary liquid stream though each step of the treatment process. It illustrates the disposition of the end products of the treatment (effluent for reuse, bio-solids, and screenings). It does not illustrate the numerous internal process loops or the various additive process inputs such as the aeration or mixing air, the chemical feed systems, or instrumentation and controls. During design, individuals process and instrumentation drawings will present that level of detail.

Table 3-8. Comparison of Treatment Process Schemes

Factor	MSABP	SBR	MLE
Screenings Volume	Very high due to small screen opening size (1 mm)	Moderate due to size of screen opening (6-8 mm)	Moderate due to size of screen opening (6-8 mm)
Power Usage	Moderate usage overall; MSABP high; sludge zero	High due to varible depth and low transfer efficiency	Moderate; low on mLE but also needed for sludge
Tank Volume	High	Moderate	Low
Sludge Volume	Low	Moderate	Moderate
Effluent Quality	High	High	High
Ease of Operation	Dictated by tertiary and disinfection	Dictated by tertiary and disinfection	Dictated by tertiary and disinfection
Maintenance	Moderate	Moderate	Moderate
Process Flexibility	Highly flexible (turndown ratio is high)	Highly flexible (turndown ratio is high)	Moderately flexible (turndown ratio is moderate)

A layout for the process units and ancillary facilities is presented in Figure 3-4. In this figure, the approximate sizes and locations of the process units and the ancillary buildings are shown. The plant will have a fenced area of approximately one acre, allowing for a 50-foot buffer around all buildings, and providing access for all building and process areas that will require ingress for maintenance. This site would be secured by an 8-foot high fence and accessed through one of two 20-foot gated openings. The site would be partially asphalt paved and partially landscaped with low maintenance cover such as gravel and shrubbery.

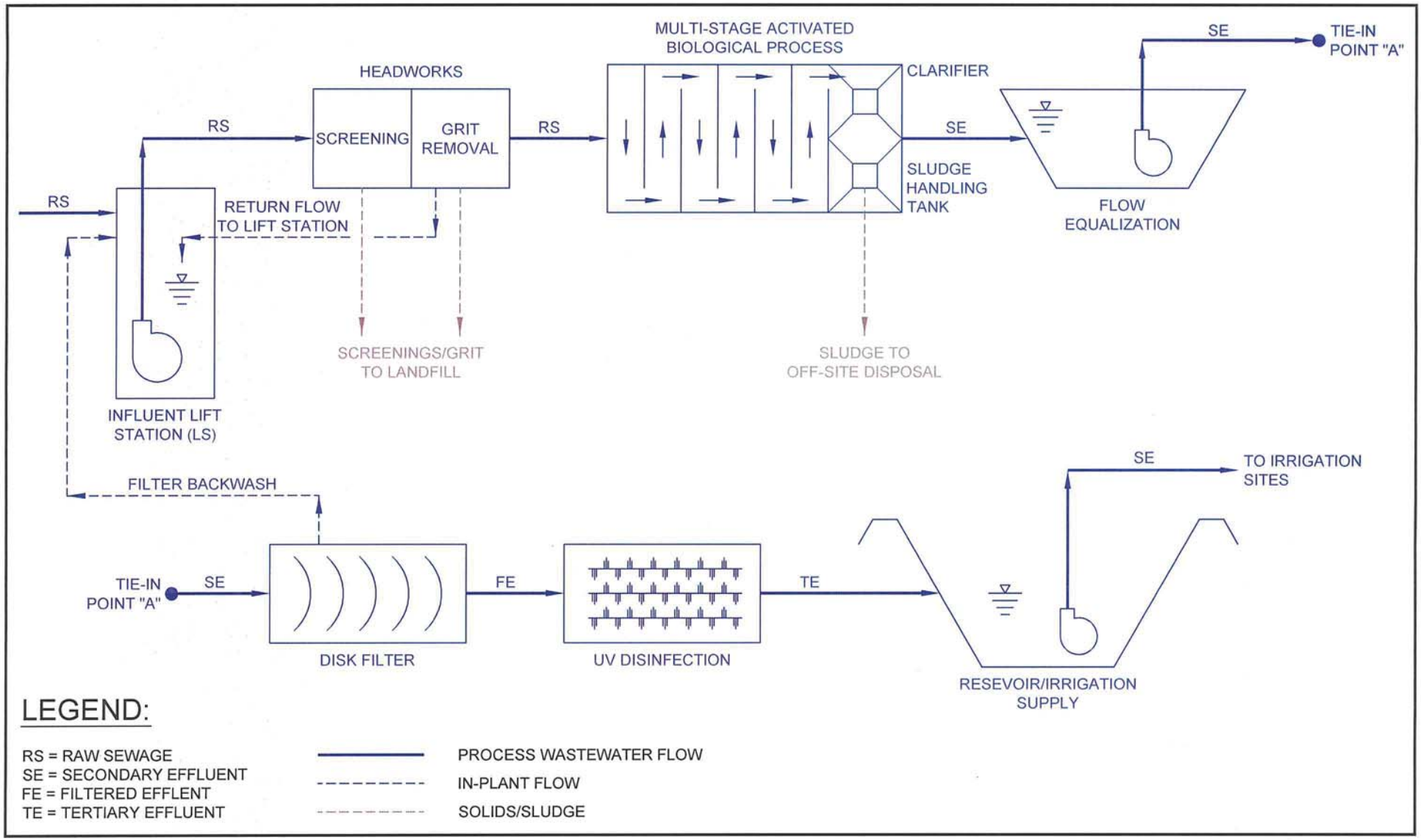

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Figure 3-3
Recommended Wastewater Process Flow Schematic

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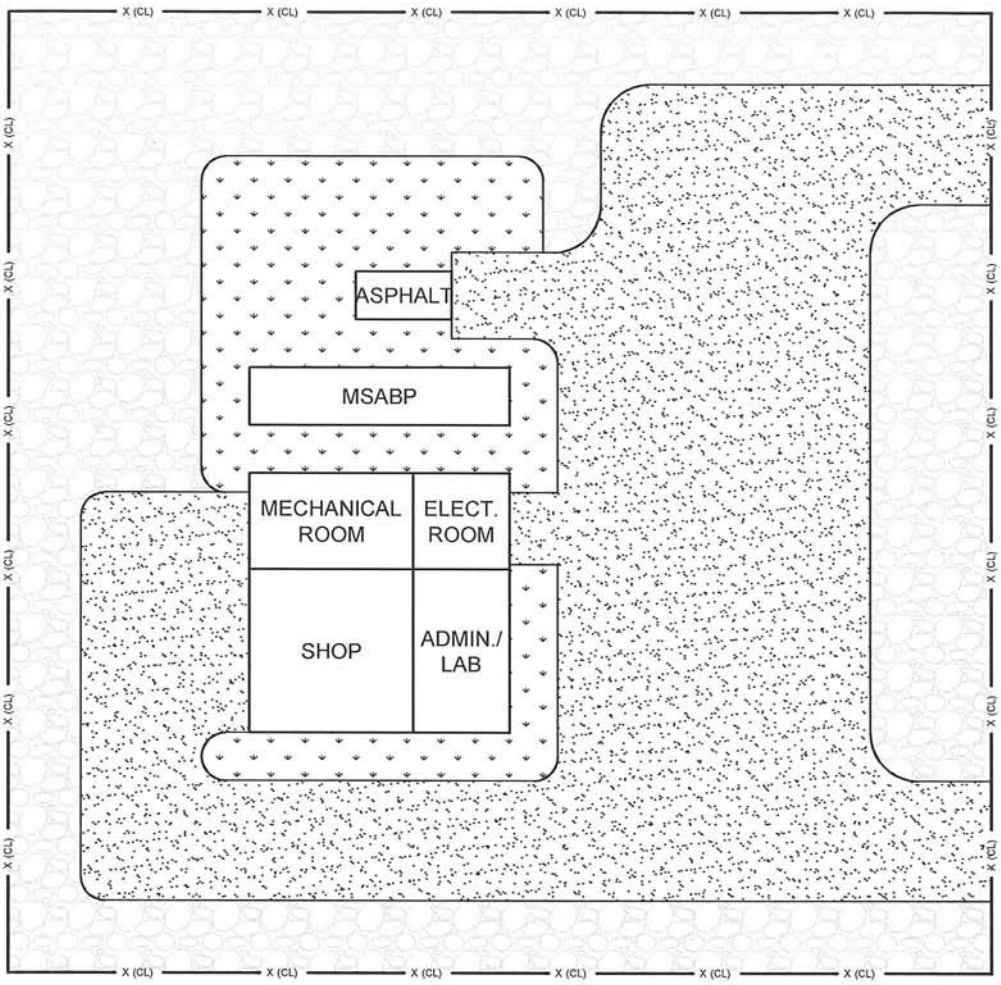

LEGEND:

GRAVEL, SHRUBBERY MAY BE INCORPORATED

GRASS/LANDSCAPING

ASPHALT

X (CL) — CHAIN LINK FENCE

MSABP — MULTIPLE STAGE ACTIVATED BIOLOGICAL PROCESS

GRAPHIC SCALE

0 20' 40' 80'

(IN FEET)

1 IN = 40 FT

N

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Figure 3-4
Wastewater Treatment Plant Layout

RECYCLED WATER DISTRIBUTION AND STORAGE

The purpose of recycling is twofold; to supplement the use of groundwater in irrigation, thus conserving the potable water supply (and reduce groundwater pumping demand), and secondly to beneficially reuse the effluent in a manner that avoids creating a point discharge to a known waterway.

In order to manage the entire annual effluent, a comprehensive plan for reuse, storage, and distribution becomes necessary. In the southwestern United States, the most critical element of a comprehensive plan is formulating a sound storage system that addresses daily, monthly and seasonal storage requirements.

The first decision is to make the storage a joint use process; a storage facility that can allow water to be withdrawn without upsetting the biology and secondly a facility that can maintain a stable, balanced biological system of clean water, microorganisms for sustaining an aquatic food chain, and a water chemistry that does not accumulate inorganic salts.

From a practical perspective, this means the storage must be deep (more than 20 feet deep), cold (allow a high dissolved oxygen concentration), and conducive to plant growth (a soil-based bottom). Inevitably, some change will occur over time and this means that some maintenance will become necessary. However, that maintenance level of effort is minimized when the water is allowed to develop a balanced biological system of plants and aquatic animals.

Determining the size of the storage is based on two principals; a pre-determined depth for successful operation and a volume that allows the required seasonal storage in addition to a permanent volume for sustainability of the pond itself. The seasonal storage requirement is a mathematically-determined volume that represents the volume that would be accumulated during the colder months of the year when demand is diminished and the effluent continues to supply the pond on a regular basis.

Recycled Water Irrigation System

The design of a recycled water irrigation system shall provide adequate distribution to apply the water based on a combination of water duty factor (acre-feet per acre per year, or simply feet per year) and when using recycled water, an agronomic application must be considered based on specific vegetation assimilation rates. For example, duty factors can vary from less than one foot per year up to seven feet per year for certain tree grown nut crops (i.e. pecans). It is understood that the project site will use recycled water for both an existing vineyard and for more common grass and shrubbery type landscaping. These types of irrigation requirements can vary from less than one foot per year to approximately three feet per year, depending on the system efficiency. A summary of the expected irrigation supply of recycled water is presented in Table 3-9.

Table 3-9. Monthly Water Balance

Month	WWTP	Pond			Evap, Inches - c	Pond	Vineyard	NET	Potable Supple-ment	Storage Volume	Vol. - Yr. N+1	Vol. - Yr. N+2
	Effluent, Af-a	Rainfall, inches -b	Rainfall , AF	Supply, AF		Evap., AF	Irrigatio n, AF	Surplus, AF				
January	4.28	3.78	0.32	4.60	1.68	0.14	0	4.46	0.00	4.46	16.98	16.98
February	3.87	4.18	0.35	4.22	2.21	0.18	0	4.03	0.00	8.49	21.02	21.01
March	4.28	3.07	0.26	4.54	3.52	0.29	20	-15.75	7.26	0.00	5.26	5.26
April	4.14	1.28	0.11	4.25	5.01	0.42	30	-26.17	26.17	0.00	0.00	0.00
May	4.28	0.30	0.03	4.31	5.78	0.48	30	-26.17	26.18	0.00	0.00	0.00
June	4.14	0.09	0.01	4.15	6.18	0.52	40	-36.36	36.36	0.00	0.00	0.00
July	4.28	0.03	0.00	4.29	6.40	0.53	40	-36.25	36.25	0.00	0.00	0.00
August	4.28	0.06	0.01	4.29	6.01	0.50	40	-36.21	36.21	0.00	0.00	0.00
September	4.14	0.25	0.02	4.17	4.46	0.37	25	-21.21	21.21	0.00	0.00	0.00
October	4.28	0.50	0.04	4.32	3.57	0.30	0	4.03	0.00	4.03	4.03	4.03
November	4.14	1.76	0.15	4.29	2.19	0.18	0	4.11	0.00	8.14	8.14	8.14
December	4.28	2.91	0.24	4.53	1.67	0.14	0	4.39	0.00	12.53	12.52	12.52
Total	50.42	18.21	1.52	51.94	48.68	4.06	225.00	-177.11	189.64	---	---	---

A - Based on the plant capacity and 10 percent loss in process and in internal consumption
B - Monthly averages from 1931 through 2003
C - Data from C.I.M.I.S.; based on the evaporative losses from storage pond

Daily Storage Requirements**.** Daily storage is not a factor for the system. The seasonal storage volume will far exceed the need for daily volume. Daily/diurnal storage volume will be accounted for in the 20,000 gallon equalization storage tank within the plant process train.

Water Balance and Seasonal Storage Requirements. Seasonal storage is required to account for the annual variations between effluent supply, potable use for irrigation, irrigation demand, evaporation, and evapo-transpiration. The scenarios can be summarized as follows:

- In the summer, demand exceeds supply. The water from the plant will be supplied directly to a storage pond on-site where the water level will vary.
- In the winter, demand will infrequently exceed supply however, most of the time supply will exceed demand.
- Quantitatively, a month by month assessment of the supply and demand, accounting for vineyard crop irrigation, other landscape irrigation, and the effects of average rainfall and evaporation from the storage pond.
- For an analysis of storage requirements and to prepare a mass balance of treatment plant effluent, rainfall (on the vineyard area), evaporation from the storage pond surface, and irrigation for the vineyard, these assumptions were used:

➢ The required total duty factor for the vineyard, in addition to rainfall, is 0.75 feet per year which results in 225 AF/year on a 300-acre vineyard

➢ The monthly allocation of the presumed 225 AF/year will be distributed from March through September with the emphasis on the summer months

➢ The rainfall will be only accounted for in the mass balance in the amount of rainfall that contributes directly to the storage pond

➢ Evaporation from the storage pond will also be accounted for the in the mass balance

➢ The supply of available non-potable water will be the sum of effluent plus rainfall (pond only) minus evaporation (pond only)

The data presentation in Table 3-9 illustrates several key results of the analysis:

- The net annual balance is a deficit (the total irrigation for the vineyard, at 225 AF/year exceeds the production of effluent and the net loss of evaporation from rainfall incident to the storage pond)
- The precise net balance will vary slightly depending on how large the surface are for the pond is chosen to be
- The duty factor of 0.75 Feet per year for irrigation was chosen neglecting the rainfall that occurs during the winter months and is directly applicable to the vineyard; if this assumption is altered and the rainfall is accounted for, and then the effluent could be available for irrigation of other areas. For this to occur, rainfall harvesting and capture would be required
- Storage volume would merely be 5 AF

- Each month the vineyard requires irrigation, most of the water would come from sources other than effluent

RECYCLED WATER QUALITY

The chemical make-up of water used for irrigation purposes is very important in ensuring maintenance of the quality of the landscaping and crops being irrigated. Key water quality parameters from an agronomic aspect are described in this section.

Sodium, Sodium Adsorption Ratio (SAR), and Adjusted SAR (aSAR)

Sodium is not an essential plant nutrient, yet it is always present in the irrigation waters and it can become the most important single constituent in the water if it exceeds tolerable concentrations. Acceptable levels of sodium are judged in proportion to divalent cations, principally calcium and magnesium in the water. The criteria commonly used to determine the potential effect of this critical element are sodium adsorption ratio (SAR) and adjusted SAR. Adjusted SAR accounts for the presence of carbonates and bicarbonates in the irrigation water, because of their tendency to precipitate calcium from the solution, aggravating the effect of sodium. The most widely accepted method of adjusting the SAR is the so-called Cax method, wherein the ratio of bicarbonate to calcium is used to determine the adjustment factor. Long-term use of irrigation water with high SAR can result in gradual elevation of soil solution SAR and deleterious effects on soil structure, leading to progressively reduced soil permeability, water-logging, and anaerobic (oxygen deficient) conditions in the root zone.

Calcium

Calcium is essential for all plant life. It is almost always available in abundance in the soil, as far as plant nutrition requirements are concerned. However, calcium also plays another important role in the soil solution. It can balance the adverse impacts of sodium on soil physical structure and the soil's ability to transport water. Native soils in California are generally rich in calcium compounds.

Chloride

Chloride is also essential to plant life, but sufficient in extremely low concentrations. This element is almost never deficient in the environment. Excessive concentrations of chloride (beyond 140 mg/L) can be harmful due to toxicity to the plant tissues.

Dissolved Solids, Specific Conductance

Total dissolved solids (TDS) is a direct measure of salinity in the irrigation water. An indirect index of salinity is the electrical conductance (EC, inverse of electrical resistance) of the water sample. Elevated TDS concentrations of irrigation water can cause deleterious effects to plant growth and to soil conditions and characteristics.

Boron

Boron is an essential nutrient for plant germination and growth. However, beyond a narrow band of concentrations (0.1 to 5 mg/L), it becomes toxic to plant life. Boron is not highly mobile and cannot be easily flushed out of the root zone; however, boron can be taken up by the plant roots to the leaf tips. Thus, for turf grasses, where frequent mowing generally occurs, removal of boron can be effective

Chumash Camp 4 Project Recycled Water Quality

Projected irrigation water quality for the Chumash Camp 4 Project cannot be provided, since there is not an existing WWTP from which to take samples. In addition, Boron was not analyzed as part of the water quality results presented in Table 2-6. However, based on the total dissolved solids (TDS) concentration of the local water wells, mineral quality of the recycled water is anticipated to be 700 to 800 mg/L. Table 3-10 provides a general summary of the various water quality parameters relative to their degree of restriction on use for irrigation.

Table 3-10. Guidelines for Irrigation Water Quality Impacts

Parameter	Degree of Restriction on Use		
	None	Slight to Moderate	Severe
Boron, mg/L	<0.7	0.7-1.0	>3.0
Chloride, mg/L	<140	140-350	>350
TDS, mg/L	<450	450-2,000	>2,000
EC, mmhos/cm	<0.7	0.7-3.0	>3.0
SAR = 0-3, and EC (mmhos) =	>0.7	0.7-0.2	>0.2
SAR = 3-6, and EC (mmhos) =	>1.2	1.2-0.3	<0.3
SAR = 6-12, and EC (mmhos) =	>1.9	1.9-0.5	<0.5
SAR = 12-20, and EC (mmhos) =	>2.9	2.9-1.3	<1.3
SAR = 20-40, and EC (mmhos) =	>5.0	5.0-2.9	<2.9

Based on the anticipated water quality of the Chumash Camp 4 recycled water, the recycled water is generally suitable for landscape and irrigation uses for the Project. Based on the water balance, during the warmer season, potable water will also be supplementing the recycled water. The anticipated blend of potable water and recycled water for irrigation use should maintain the mineral quality at a desirable value, in the range of 600 to 700 mg/L. Irrigation with this recycled water is expected to yield good results.

WASTEWATER COLLECTION, TREATMENT, AND DISPOSAL SUMMARY

There are two alternative possible collection systems, depending on whether the decision is made to create 143 five-acre residential lots or to create 143 one-acre residential lots. The result of that decision will affect the layout and dimensions of the collection system. The collection system, regardless of which of these two choices is made, will be greatly affected by the dendritic topography. The topography will cause the landforms to be partially re-shaped in order to control the costs of both roadways and of sewer collection pipelines. There will be

pumping stations regardless of the choice made. The number of pumping stations will be determined both by the choice of the two alternatives discussed in this section and in final grading plans which are not part of this section's discussion.

The treatment system will occupy a relatively small footprint (less than an acre) allowing for the treatment process units and a small building to house both some mechanical equipment, an electrical and control room, and an operators station for conducting routine administrative duties and some laboratory testing. The photograph at right shows an example of a small MSABP treatment process module.

The example module illustrates the position of an influent screening unit (to the right and above the module) and the blowers (to the left and on grade). The process can be either above grade, as shown, or below grade. Not shown in this picture are the other process units such as the clarifier, sludge holding, and any tertiary of disinfection.

The effluent storage proposed is a deeply excavated basin that would allow the water to draw and fill without ever completely evacuating the basin. The concept is to fill the basin during cooler, winter months when the vineyard lies dormant and un-irrigated. The spring growth in the vineyard would signal the start of irrigation and would use the stored effluent from the pond as needed and as available. The 50,000 gpd treatment plant does not generate enough effluent and thus the pond is not adequate to meet the needs of a 300-acre vineyard. Potable water will be an important source of supply once the irrigation begins. Thus, it is proposed that a potable water make-up pipeline with air-gap separation, be provided to fill and supplement the storage reservoir when irrigation demand exceeds supply and stored irrigation water supply. During the summer months, it is important for the pond to retain at least five feet of depth for the water to sustain a biological balance in aquatic plant and animal life. If the pond is NOT sustained biologically, it will eventually become more of a nuisance than a benefit.

The water would be pumped from the pond, through a filtered drip system that operates on a schedule controlled by the vineyard operator using the knowledge and understanding required of a viticulturist. An important fact to remember about recycling water is that as soon as the effluent is produced and stored in a downstream system, the effects of nature begin to modify its chemistry and biology – and that means the effort to control the water's quality does not end until it has been applied to the final use.

Wastewater Operator Considerations

A tertiary wastewater facility of this size and complexity will require a Grade III operator, similar to that of the wastewater facility currently serving the Chumash Reservation in Santa Ynez. It is envisioned that a new WWTP for this Project would be operated under the same arrangements as the Chumash Reservation WWTP, being operated under contract by a qualified public agency operator (Santa Ynez CSD) or equivalent.

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