Daniel B. Stephens & Associates, Inc.

MEMORANDUM

TO: Robert Eranio, Crestview Mutual Water Company

FROM: Gregory Schnaar, PhD, PG (VA)
Tony Morgan, PG (CA), CHG (CA)

DATE: July 14, 2021

SUBJECT: Well #7 Pathogen Transport Evaluation

EXECUTIVE SUMMARY

Modeling and review of the scientific literature estimate that pathogens will not reach groundwater supplies for proposed Crestview Mutual Water Company (CMWC) Well No. 7. **Our analyses indicate that Well No. 7 will safely produce water without pathogen contamination from septic systems.**

This technical memorandum evaluates potential pathogen impacts to groundwater at proposed CMWC Well No. 7 from septic systems within 200 feet (ft.) of the proposed well based on (1) a literature review of pathogen transport and (2) standard modeling techniques using site-specific information. Review of the scientific literature indicates that vertical transport of pathogens in unsaturated soils is generally limited to distances less than 10 ft., which is much less than the distance to groundwater at the proposed well site. Modeling methods considered processes that reduce pathogen concentrations from septic system effluent, including degradation, sorption and straining, and consistent with the scientific literature review indicate very limited pathogen transport that will not reach groundwater.

CONCLUSIONS

Literature review and vadose-zone modeling indicate the following conclusions:

- Pathogens, including bacteria and viruses, do not transport greater than 10 ft. in unsaturated soils due to sorption, filtration, and decay.

- Septic system construction specifications in various jurisdictions, including in Ventura County, allow for vertical separation between the septic system seepage pit and the groundwater table of less than or equal to 10 ft. in recognition that pathogens do not pose a risk to groundwater at greater distances.

- Site-specific vadose-zone modeling confirms that pathogen transport from a residential septic seepage pit is expected to be less than 10 ft. considering a range of potential scenarios and transport parameters.
- Based on these results, we anticipate that pathogens from residential septic systems in the vicinity of Crestview Well #7 will not contaminate groundwater pumped from the well.

INTRODUCTION AND BACKGROUND

Daniel B. Stephens & Associates, Inc. (DBS&A) has evaluated pathogen transport in the vicinity of the planned Crestview Mutual Water Company ("Crestview") Well #7 at Alviso Drive and La Patera Drive ("well site"). The objective of this study was to estimate the threat of pathogen groundwater contamination in the vicinity of the planned well site from percolation of wastewater effluent from residential septic systems. This was completed by (1) performing a review of the scientific literature on pathogen transport in the vadose zone, and (2) performing vadose zone modeling to estimate pathogen transport for conditions representative of the well site.

Based on the results of the literature review and vadose-zone modeling, as explained below, we conclude that pathogens from residential septic systems will not be transported more than approximately 10 feet (ft.) below the seepage pits and will not reach the groundwater table that is approximately 540 ft. below the seepage pits. Therefore, we anticipate that pathogens from residential septic systems in the vicinity of Crestview Well #7 will not contaminate groundwater pumped from the well.

Crestview Well #7 is planned to be located within the Las Posas Valley groundwater basin, and be screened from 1,040 to 1,080 feet below ground surface (ft. bgs) and from 1,240 to 1,420 ft. bgs (HGC, 2019). Static water-level in the vicinity of the well site is approximately 580 ft. bgs based on measurements at Crestview Well #4 located 1,550 ft. to the east-northeast.

Residential septic-system records for the vicinity of the well site were obtained from the Ventura County Environmental Health Division. Septic systems in this area are designed with seepage pit installations (as opposed to leach line installations). Seepage pit depths range from approximately 35 to 50 ft., and diameter ranges from approximately 4 to 5 ft. Given the depth of the septic system seepage pits is approximately 40 ft. bgs and the depth to the groundwater table is 580 ft. bgs, the distance from the bottom of the seepage pits to the groundwater table is approximately 540 ft.

Effluent from the septic tank passes into the seepage pit. The purpose of the pit is to filter and then disperse the effluent before it percolates into the soil. Seepage pits are filled with clean rock (0.75 to 2.5 inch), and a perforated 4 inch PVC pipe, from which the effluent exits. Effluent from the seepage pit percolates into the vadose zone. Percolating water that comes from the bottom of the seepage pits can carry pathogens (i.e., viruses, bacteria) down through the soil, but to what extent this threatens the quality of groundwater in the area depends on several factors, including the distance between the bottom of the seepage pit and the top of the groundwater table.

LITERATURE REVIEW

Vertical separation is the depth of the vadose zone between the bottom of a seepage pit and a restrictive zone (i.e., fine-textured soil) or the groundwater table. This vertical separation is required in order to remove pathogens from the percolating water via oxidation, biodegradation, and straining. Washington State Department of Health (Hall, 1990) researched the vertical separation requirements of 14 other states, all of which require only 0.5-6 ft. between septic systems and the groundwater table. For example, in North Carolina the minimum requirement is 1 ft., and in New Jersey it is 4 ft. Vertical separation to the groundwater table in the western states reviewed (Colorado, Idaho, Oregon, Utah, Wyoming) ranged from 2 to 4 ft. In Los Angeles County, a non-conventional onsite wastewater treatment system (NOWTS) is required if the vertical separation is less than 5 ft. (LACDPH, 2018). In Ventura County, current seepage pit specifications require a 10 foot vertical separation (VCEHD, 2018). These shallow requirements are due to the acknowledgement that pathogens do not travel very deeply in the vadose zone.

Decreasing moisture content results in a decrease in the depth to which pathogens penetrate (Gargiulo et al. 2008). Pathogens will travel farther under saturated flow conditions (McCoy and Ziebell, 1975; Hansel and Machmeier, 1980; Reneau et al., 1989; Gargiulo et al. 2008). Unsaturated conditions also facilitate aerobic microbiological decay (Karathanasis et al., 2006).

Lance and Gerba (1984) found that viruses traveled to a depth of 1.3 ft. in soil columns under unsaturated conditions and 5.25 ft. under saturated conditions (Figure 1). They found by controlling the application rate of effluent they could reduce the distance that pathogens penetrate soil. Lance et al. (1976) found that virus removal was not affected by varying the infiltration rate between 6-22 inches/day. They filled columns with 8.20 ft. of sandy soil and monitored the distance viruses traveled. Only 3 of the 43 columns had viruses penetrate to a depth of 5.25 ft., no viruses were detected from 7.9-8.2 ft. depths, and most viruses were removed within the first 0.16 ft. of soil.

Gilbert et al. (1976) monitored the Flushing Meadows Wastewater Renovation Project near Phoenix Arizona, in which for 8 years effluent from a secondary sewage treatment plant was applied with an average hydraulic lauding rate of 295 ft./year. The suspended solid concentration of the effluent was below 20 milligrams per liter (mg/l) in the summer and fall but between 50 and 100 mg/l in the winter. Here the water table was 9.84 ft. deep and observation wells were put in place to a depth of 20 ft., 30 ft., and 100 ft. deep. Researchers sampled the wells every two months, and did not detect viruses or salmonella in any of the well water samples. They found that fecal coliforms, fecal streptococci and total bacteria were reduced by approximately 99.9%.

Gargiulo et al. (2007a) conducted experiments on 80% water saturated packed columns of soil in order to study the influence of grain size and bacteria surface macromolecules on bacteria transport. Effluent was added to columns at a rate of 165-170 milliliters per hour (ml/hour). Straining was the primary mechanism by which bacteria was removed and accounted for 78 - 99.6 % of removal, and that removal increased with decreasing median grain size.

Many laboratories studies note that in most cases filtration media will consist of rough grains rather than smooth spherical, grains, and thus, collection efficiency is likely to be higher in field settings (Harvey et al 1993; Ryan and Elimelech, 1996; Bradford et al. 2003). Brown et al. (1979) conducted a study of unsaturated coliform bacteria and coliphage virus transport under unsaturated conditions and found that within 0.98 ft. most colloids were removed and only rarely did they observe coliforms at a max depth of 3.9 ft. Brown et al (1977) found that 3.3 ft. was sufficient vertical separation to remove coliphage and fecal coliforms from three soil types. Karathanasis et al. (2006) found that fecal bacteria removal was lower in coarse textured soils, but that the formation of biomass over time would decrease pore throats and increase soil sorption and deactivation. They found that 2 ft. was sufficient to fully remove fecal bacteria from coarse-textured soils. They concluded that the presence of fine textured sediments greatly increased treatment efficiency by decreasing pore throat size.

Ryneveld et al. (2016) studied the movement of contaminants in Johannesburg where the groundwater table was greater than 49.2 ft. bgs. They found that within a horizontal distance of 9.8 ft. from the leach field bacteria levels had dropped off to values consistent with the background values. These researchers noted that in the reviewed literature subsurface movement of fecal bacteria was on the order of 3.3 - 6.6 ft.

VADOSE ZONE MODELING

The studies cited above agree that pathogen transport in unsaturated soils is generally a maximum of 10 ft. Given that the depth to the groundwater table is 580 ft. bgs and soils beneath the seepage pits are sandy (as discussed below), soils beneath the seepage pits are unsaturated. We expect pathogen transport will therefore be limited to approximately 10 ft., which is much less than the separation of approximately 540 ft. between the bottom of the seepage pits (approximately 40 ft. bgs) and the groundwater table (approximately 580 ft. bgs). Vadose zone modeling was conducted to further evaluate pathogen transport for conditions representative of the well site.

Modeling Platform and Theory

Vadose zone pathogen transport modeling was conducted with the model platform HYDRUS-1D (Simunek et al., 2005). HYDRUS-1D is a one-dimensional finite element model that simulates the movement of water, heat, and solutes in variably saturated media. The model numerically solves the Richards' equation for saturated-unsaturated water flow and Fickian-based advection dispersion equations for heat and solute transport. The water flow equation can account for time-variable precipitation, evaporation, and transpiration at the top of the domain, root water uptake within the domain, and free drainage or a water table pressure condition at the bottom of the domain. The model can account for a one-dimensional heterogeneous distribution of soil material types in the subsurface.

Discussion of the theoretical basis of the pathogen transport model used is adopted from Gargiulo et al. (2008). Pathogen transport was modeled using a modified form of the advection-dispersion equation that includes two kinetic deposition sites:

$$\frac{\partial \theta c}{\partial t} + \rho_b \frac{\partial s_1}{\partial t} + \rho_b \frac{\partial s_2}{\partial t} = \frac{\partial}{\partial x}\left(\theta D \frac{\partial c}{\partial x}\right) - \frac{\partial q c}{\partial x} \qquad \text{Eq-1}$$

where θ is the volumetric water content, ρ_b is the soil bulk density [M L^{-3}], t is the time [T], q is the flow rate [L T^{-1}], x is the spatial coordinate [L], D is the dispersion coefficient [L^2 T^{-1}], c is the pathogen concentration in the aqueous phase [N_c L^{-3}, where N_c is the number of pathogens], and s_1 [Nc M^{-1}] and s_2 [Nc M^{-1}] are the solid phase concentrations associated with deposition sites 1 and 2, respectively.

Pathogen retention is separated into two fractions ($s_1 + s_2$) and assumes different rates and processes occurring for each. The first kinetic site (Site 1) employs the conventional attachment/detachment model to describe pathogen transfer between the aqueous and solid phases:

$$\rho_b \frac{\partial s_1}{\partial t} = \theta \psi_t k_a c - k_d \rho_b s_1 \qquad \text{Eq-2}$$

where k_a is the first-order attachment coefficient [T^{-1}], k_d is the first-order detachment coefficient [T^{-1}], and ψ_t is a dimensionless colloid retention function that accounts for time-dependent deposition described with a Langmurian approach:

$$\psi_t = 1 - \frac{s_1}{s_{\max 1}} \qquad \text{Eq-3}$$

in which s_{max1} is the maximum solid phase concentration [Nc M^{-1}] of retained pathogens on Site 1. Under unsaturated conditions, attachment to the solid phase and the air–water interface are lumped in the k_a term. For most simulations conducted here s_{max1} was assigned a very large value to represent a no-blocking scenario for Site 1 consistent with Gargiulo et al. (2007b).

The second solid-phase site (Site 2) represents pathogen straining. Scientific studies, such as those cited above, observe decreasing straining with depth. Depth-dependent mass-transfer associated with Site 2 are described by the following equation:

$$\rho_b \frac{\partial s_2}{\partial t} = \theta \psi_x k_2 c \qquad \text{Eq-4}$$

where k_2 [T^{-1}] is the deposition coefficient on Site 2, and ψ_x is the dimensionless colloid retention function that accounts for depth-dependent deposition as

$$\psi_x = \left(\frac{d_c + x - x_0}{d_c} \right)^{-\beta} \qquad \text{Eq-5}$$

where d_c is the median diameter of the sand grains [L], x_0 is the coordinate [L] of the location where the depth-dependent deposition process starts (in this case, the surface of the soil profile), and β is an empirical factor controlling the shape of the spatial distribution (assigned a value of 0.43 based on results of previous studies).

Modeling Scenarios

A HYDRUS-1D model framework was established to be consistent with vadose zone properties in the vicinity of the well site. Assumed model lithology is given in Table 1. Lithology from ground surface to 50 ft. bgs was identified based on a Soil Identification Report associated with the septic system records at 179 Alviso (Fry, 1985) and a Geotechnical Engineering Report conducted for the 191 Alviso Drive property (Earth Systems Pacific, 2019). Below 50 ft. the Well Completion Report for Well #3 (approximately 500 ft. away from Well #7) and associated borehole geophysical logs were evaluated to create the conceptual lithology for use in modeling. Boring and geophysical logs are included in Appendix A. The borehole geophysical logs were used to define the top and bottom depths of discrete lithologic packages with the descriptions of those packages provided from the Well Completion Report. Borehole geophysical logs often provide more definitive determinations of lithologic boundaries. Lithologic description of “Clay” in the Well Completion Report were assigned as Silty Clay in Hydrus, “Sand” as Loamy Sand, and “Grey sand and rock” as Sand (García-Gaines and Frankenstein, 2015).

Vadose zone properties for each material type (Van-Genuchten water retention parameters and hydraulic conductivity) were assigned based on soil texture using the ROSETTA-3 model (Zhang and Schaap, 2017) and are given in Table 2. The model domain begins at 40 ft. bgs (assumed as the bottom of the seepage pit) and extends to 580 ft. bgs (the groundwater table). Vadose-zone properties were assigned based on the measured soil texture in the 179 Alviso Soil Identification Report for the 40 to 85 ft. bgs interval, and were assigned based on defaults for each material type in ROSETTA-3 for the deeper intervals.

Figure 2 displays the HYDRUS model layers and steady-state water saturation profile assuming continuous wastewater seepage at the bottom of the pit. Residential wastewater discharge was assumed to be 150 gallons per day per unit (Hantzsche and Finnemore, 1992). For most simulations it was assumed that the seepage spreads somewhat horizontally, so was applied over a 10-foot diameter area, resulting in a recharge rate of 0.26 ft./day (7.78 cm/day). One additional scenario was considered where all seepage occurs over a 6-foot diameter area, resulting in a recharge rate of 0.7 ft./day (21.3 cm/day).

Pathogen transport parameters were assigned based on the range of values in the scientific literature, and several scenarios were run to test the impact of these parameters within the range of values reported in the literature. Table 3 summarizes the model scenarios. In all cases the upper pathogen boundary condition was assigned as 4,200 pathogens per milliliter (N_c/ml), representative of typical assumptions of fecal coliform concentrations in septic system effluent (Reneau et al., 1989).

Scenario 3 is the base-case scenario, with all transport parameters assigned based on the median values from the scientific literature and considering no pathogen decay (die-off). Site 1 and Site 2 attachment/detachment rates were based on the median values from Gargiulo et al. (2007b, 2008), and s_{max1} was assigned as very large in order to consider essentially no blocking of sorption on Site 1 (Gargiulo et al., 2007b). Scenario 1 and Scenario 2 considered pathogen die-off in the aqueous and sorbed phases, respectively. The assumed aqueous decay rate was based on observed viral decay rates (Sasidharan et al., 2017), and the sorbed decay rate was taken as the median observed from a review of various pathogens in soils (Reddy et al., 1981).

Scenario 4 was conducted with a lower value of the Site 2 attachment rate based on the minimum value reported by Gargiulo et al. (2007b, 2008). Scenario 5 tested a greater value of dispersivity (Vanderborght and Vereecken, 2007; Rubin et al., 1999), and Scenario 6 used a larger flux rate representative of seepage over a 6 ft. diameter as opposed to 10 ft. diameter area. Scenario 7 tested a smaller value of s_{max1} and Scenario 8 tested a larger value of the Site 2 detachment rate based on values reported in Gargiulo et al. (2007b, 2008). Lastly, Scenario 9 tested a smaller value of the representative grain diameter.

Modeling Results

In all cases pathogen transport was limited to the first 10 ft. below the seepage pit, consistent with the literature studies cited above. For all scenarios pathogen concentrations were reduced from 4,200 to 200 N_c/ml within the first 2 ft., and to 1 N_c/ml within the first 10 ft. A steady-state condition was reached within 1,000 days after seepage began for all scenarios (i.e., no further change in modeled pathogen travel distance was observed over time). Therefore, the modeled pathogen transport distance at 1,000 days after seepage begins are representative of long-term conditions and are the basis for all model results discussed here.

Figure 3 displays a graph of aqueous pathogen concentration versus depth for Scenarios 3 and 4. Scenario 4, which had smaller Site 2 attachment rate, exhibited the largest vertical aqueous pathogen movement of all scenarios. Table 4 summarizes the depth of vertical pathogen movement for all scenarios. Attachment to Site 2 dominated total pathogen attachment (greater than 99%), consistent with previous scientific studies (e.g., Gargiulo et al., 2008).

SIGNATURES

Gregory Schnaar, PhD, PG (VA)
Principal Environmental Scientist/Hydrologist

Tony Morgan, PG #4178 (CA), CHG #159 (CA)
Vice President/Principal Hydrogeologist

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Figures

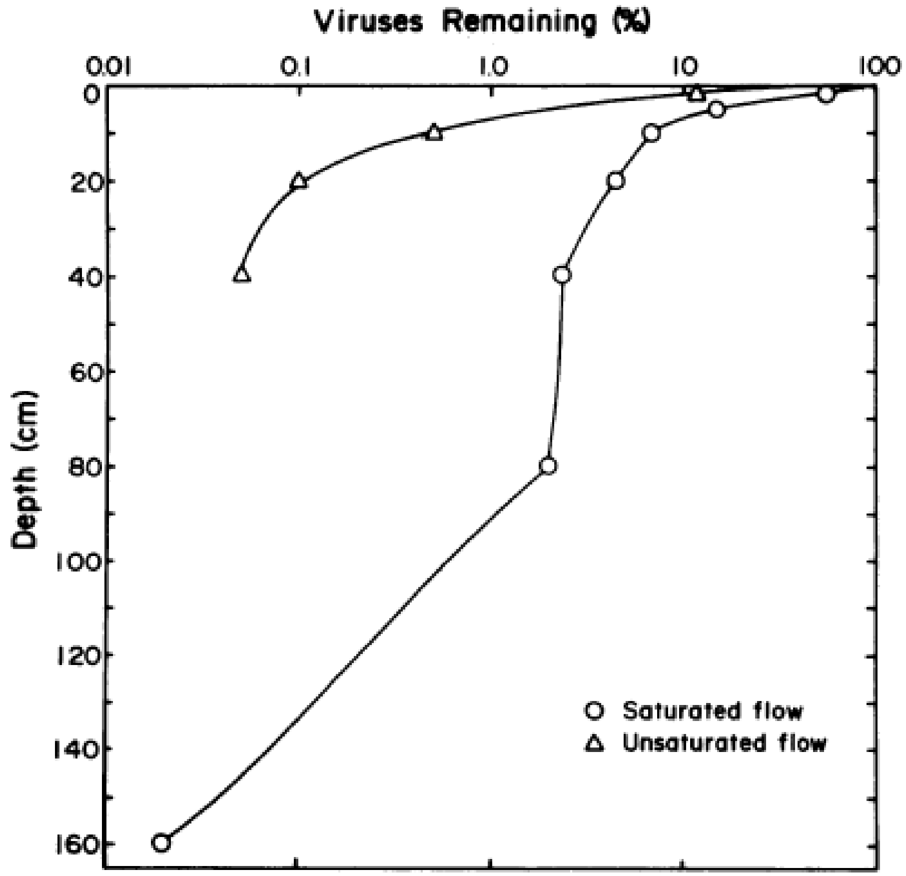

FIG. 1. Virus adsorption by a soil column during saturated and unsaturated flow. Saturated flow points are averages for three infiltration rates, and unsaturated flow points are averages for two infiltration rates.

Source: Lance and Gerba, 1984

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Virus Transport Under Saturated and Unsaturated Flow Conditions

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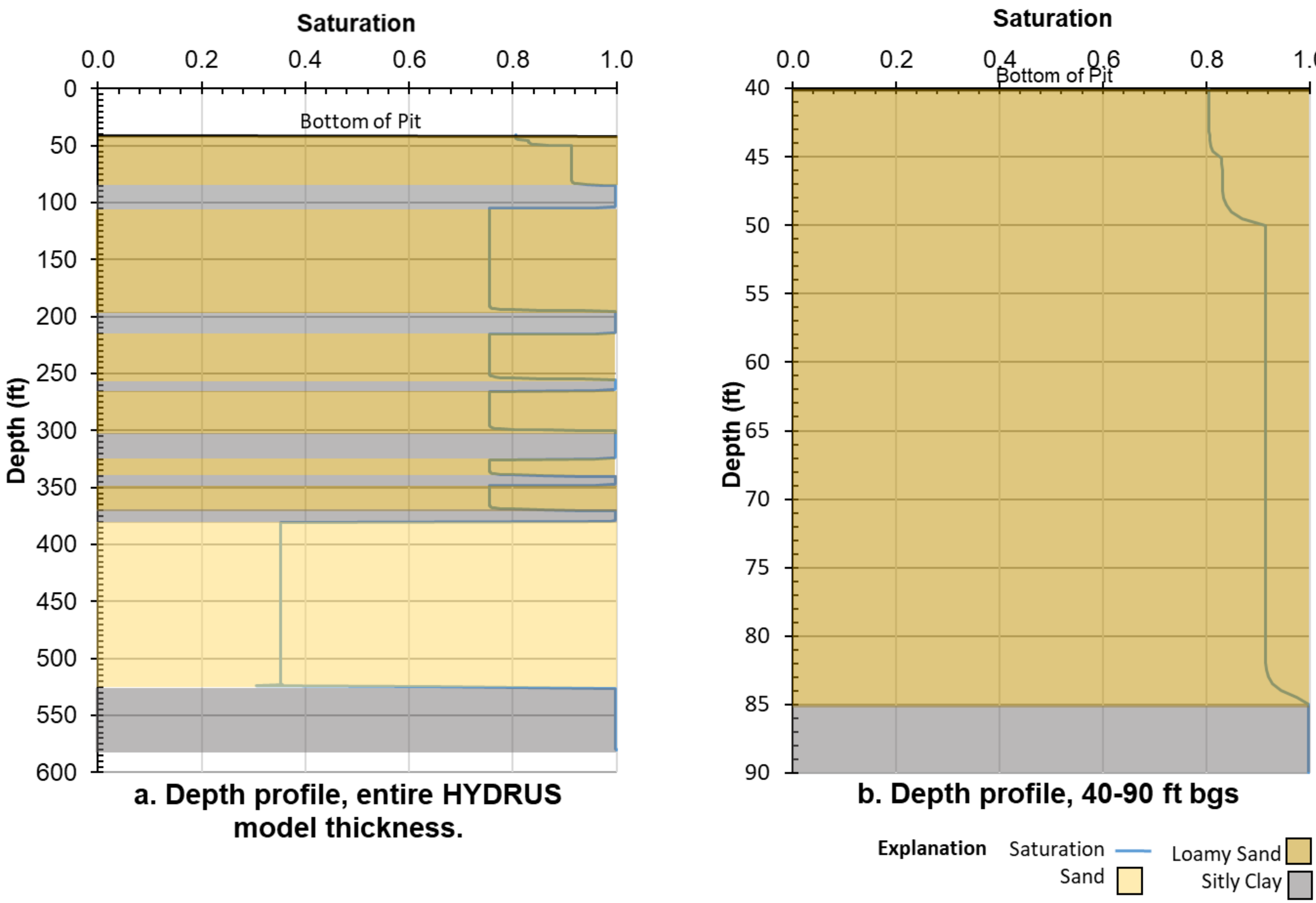

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HYDRUS Model Layers and Steady-State Water Saturation Profile

DBS&A
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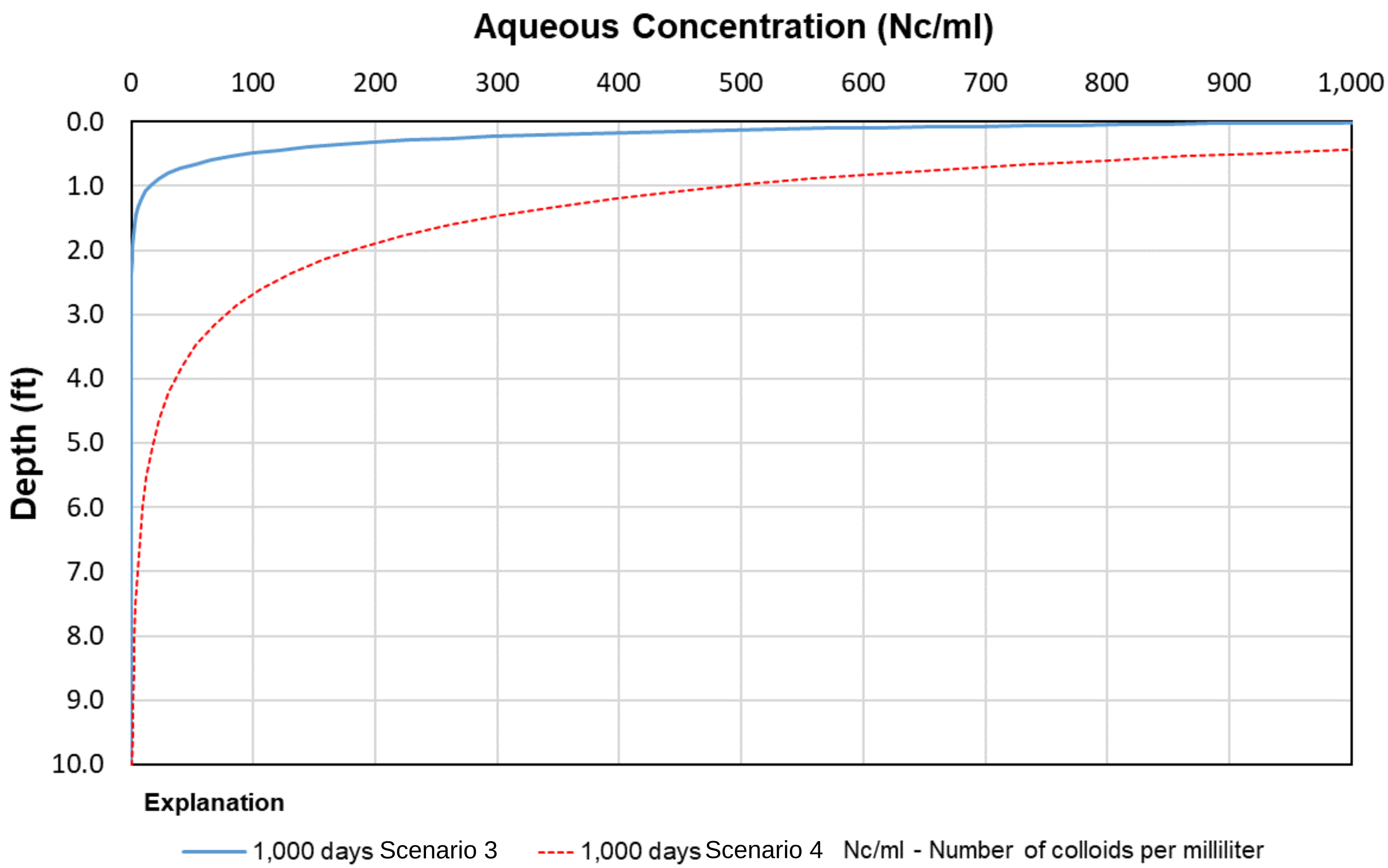

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Summary Concentration Profile

Tables

Table 1. HYDRUS Model Lithology

Starting Depth, ft	Ending Depth, ft	Lithologic Description	Hydrus Simulated Lithology (Rosetta 3)
0	5	Clay Loam	Not modeled
5	10	Sandy Loam	Not modeled
10	15	Clay Loam	Not modeled
15	20	Silt Loam	Not modeled
20	25	Clay Loam	Not modeled
25	30	Sandy Clay Loam	Not modeled
30	35	Clay Loam	Not modeled
35	40	Sandy Loam	Not modeled
40	45	Sandy Loam	Sandy Loam
45	50	Sandy Loam	Sandy Loam
50	85	Loamy Sand	Loamy Sand
85	105	Clay	Silty Clay
105	195	Sand	Loamy Sand
195	215	Clay	Silty Clay
215	255	Sand	Loamy Sand
255	265	Clay	Silty Clay
265	300	Sand	Loamy Sand
300	325	Clay	Silty Clay
325	340	Sand	Loamy Sand
340	348	Clay	Silty Clay
348	370	Sand	Loamy Sand
370	380	Blue clay	Silty Clay
380	525	Gray sand & rock	Sand
525	580	Blue clay	Silty Clay

Notes:

Bold black line denotes the bottom of the pit.

Table 2. HYDRUS Vadose Zone Properties

Lithologic Description	θ_r	θ_s	α (cm^{-1})	n	K_s (cm/day)
Sandy Loam (45-50ft)	0.059	0.374	0.024	1.62	83.0
Sandy Loam (40-50ft)	0.059	0.376	0.023	1.57	72.5
Loamy Sand (50-85ft)	0.067	0.378	0.020	1.45	40.3
Silty Clay	0.123	0.473	0.010	1.27	9.6
Loamy Sand	0.058	0.383	0.025	1.70	108.2
Sand	0.055	0.363	0.033	2.90	643.0

Notes:

θ_r - Residual water content

θ_s - Saturated water content

α - Related to the inverse of air suction

n - measure of pore size distribution

K_s - Saturated hydraulic conductivity

Table 3. HYDRUS Model Scenarios

Scenarios	Flux (cm/day)	Boundary Concentration (N_c/ml)	Dispersivity (cm)	μ_w (day^{-1})	μ_s (day^{-1})	Grain diameter (cm)	Site 2 attachment rate (day^{-1})	Site 1 attachment rate (day^{-1})	Site 1 detachment rate (day^{-1})	S_{max1} (N_c/g)
1	7.8	4,200	10	0.036	0	0.1	68.5	10.9	12	1,000,000
2	7.8	4,200	10	0	0.014	0.1	68.5	10.9	12	1,000,000
3	7.8	4,200	10	0	0	0.1	68.5	10.9	12	1,000,000
4	7.8	4,200	10	0	0	0.1	15.8	10.9	12	1,000,000
5	7.8	4,200	76	0	0	0.1	68.5	10.9	12	1,000,000
6	21.3	4,200	10	0	0	0.1	68.5	10.9	12	1,000,000
7	7.8	4,200	10	0	0	0.1	68.5	10.9	12	100
8	7.8	4,200	10	0	0	0.1	68.5	10.9	21.4	1,000,000
9	7.8	4,200	10	0	0	0.025	68.5	10.9	12	1,000,000

Notes:

μ_w - decay rate of pathogens in water

μ_s - decay rate of pathogens in soil

S_{max1} - Maximum solid phase concnetration

N_c/g - number of colloids per gram of soil

N_c/ml - number of colloids per milliliter of pore water

Table 4. Colloid transport for all scenarios

Test	Depth to 200 Nc/ml (ft)	Depth to 1 Nc/ml (ft)
1	0.32	1.94
2	0.32	1.94
3	0.32	1.94
4	1.94	10.00
5	0.60	5.08
6	0.98	5.54
7	0.32	1.94
8	0.32	1.94
9	0.66	3.82

Notes

N_c/ml - number of colloids per milliliter of pore water.

Appendix A: Boring and Geophysical Logs

Anacapa Imaging **Scan Control Sheet**

ISDS Site Files

Client: Ventura County Environmental Health Division

Street Address: 179 ALVISO DR

Area: CAMARILLO

Owner/Builder: KARL FRY

Assessor Parcel No.: ______

Initials SG BG

Box No 94349 6284

RESOURCE MANAGEMENT AGENCY
ENVIRONMENTAL HEALTH DIVISION

APPLICATION FOR INDIVIDUAL SEWAGE DISPOSAL SYSTEM (ISDS)

(Application expires 180 days from date of submittal)

NOTE: Three copies of Soils Report and Plot Plan with System Design Specifications to be attached AND submitted with this application.

FOR OFFICE USE ONLY
Receipt No.: 9101
Date Received: 8-21-85
Plan Check No.:

179

1. Job Address: Alviso Drive (Las Posas Hills) Camarillo
   NUMBER STREET CITY/TOWN/AREA
2. Assessor's Parcel Number: lot 15, Tr. 2706 487-4108
3. Owner's Name: Karl Fry c/o Alvie Thompson 4. Telephone No.: ( ) 483-5230 HOME ( ) OFFICE
5. Mailing Address: 3777 PCH, Oxnard 93033
   NUMBER STREET CITY ZIP CODE
6. Type of Development: [x] Residential: Number of Bedrooms 4
   Number of Fixture Units 35
   [ ] Commercial: Number of Fixture Units
   Maximum number of employees and visitors
7. Water Supply: [x] Public: Name of Water Company
   [ ] Private
8. Distance from nearest water well: na
   Septic Tank ____ feet
   Sewage Disposal System ____ feet
9. Distance from springs, streams, lakes, ocean waters & natural drainage courses: na
   Septic Tank ____ feet
   Sewage Disposal System ____ feet
10. Type of absorption system: [ ] Leach Line [x] Seepage Pit [ ] Mound System [ ] Subsurface Filter
11. Size of Septic Tank: 1500 Gallons 12. Surface Slope 5-7 percent

13. Leach Line Installation:

Number of trenches		Length of each trench	feet
Depth of each trench	inches	Bottom width of trench	inches
Earth cover over drainline	inches	Filter material under drain line	inches
Square Feet/Linear Feet of trench		Absorption area	square feet

14. Seepage Pit Installations:

Number of pits	2	Diameter of each pit	4 feet
Earth cover over pits	36 inches	Depth of each pit	40 feet
Absorption area provided	929		

Signature of Applicant or Representative: Alvie Thompson Date: 8-22-85

OFFICE USE ONLY

Application: [x] Approved [ ] Denied By: W. C. Stratton (SANITARIAN) Date: 10-21-85

Conditions of Approval:

Installation Approved By: F. Gerly (COUNTY INSPECTOR) Date: 6-16-86

DISTRIBUTION: **White**-Building & Safety Division **Canary**-Environmental Health Division **Pink**-Environmental Health Division **Goldenrod**-Applicant

EH-77-8 (Rev. 8/84)

BUILDING & SAFETY DIVISION

SOIL IDENTIFICATION REPORT

Location of Property: lot 15, Tr. 2706 Alviso Drive Nearest Cross Street: La Patera
(Job Address)

Owner / Builder: Karl Fry c/o Alvie Thompson Address: 3777 PCH, Oxnard, 93030

Method of Drilling: 6" auger Drilled By: Tierra Tech

Date Tested: 8-7-85 Weather Conditions: cloudy 68°±

LOG OF BORING No: DP-1

DEPTH (Feet)	UNIFIED SOIL CLASSIFICATION	G.R. SYMBOL	SOIL TYPE TABLE I-4 UPC	MOISTURE CONTENT %	HYDROMETER ANALYSES % SAND	% SILT	% CLAY	SOIL DESCRIPTION
0								
5			IV	13.9	28	36	38	sandy silty clay
10			III	5.5	58	36	6	silty sand
15			IV	20.1	25	48	27	clayey silt
20			IV	18.3	26	54	20	sandy silty clay
25			IV	21.5	26	46	28	clayey silt
30			III	13.3	46	26	28	sandy clay
35			V	18.6	22	44	34	silty clay
40			II	8.1	72	16	12	sand
45				8.4	78	14	8	sand
50				6.8	80	12	8	sand
55								
60								

Try 5' diam. pit
D = 40'
A = 15.7 ft²
Eff D = 37'

II 4(4.0)(15.7) = 251
III 10(2.5)(15.7) = 393
IV 18(1.1)(15.7) = 311
V 5(.83)(15.7) = 65
Q = 1020 gal

4' diam. Q = 816 gal/day

PIT PERFORMANCE TEST DATA WORKSHEET

Location Alviso Dr Date Tested 8-7-85

Test performed by B. K. Depth of hole 50 (ft)

Date presaturated 8-6-85 Diameter 0.5 (ft)

Sloughed to 31'-1"

Time (Min)	Elapsed Time (ET) (Min)	Depth to Top of Water (TP) (Inches)	Water Drop (Δh) (ft)	Depth of Water Left in Hole (d) (ft)	Average Head (Have) (ft)	Absorption Rate (A) gal/ft²/day	Comments
					$A = \frac{1348.3\ (\Delta H)}{H_A \cdot ET}$		
1 11:45		3'-1" = 37		28.00			
2 11:50	5	7'-1" = 85	4.0	24.00	26.00	41.48	
3 11:55	5	11'-1" = 133	4.0	20.00	22.00	49.02	
4 12:03	8	15'-1" = 181	4.0	16.00	18.00	37.45	
5 12:30	27	19'-1" = 229	4.0	12.00	14.00	14.26	
6 1:00	30	23'-6" = 282	4.42	7.58	9.79	20.29	
7 1:30	30	27'-6" = 330	4.00	3.58	5.58	32.22	
8 2:00	30	29'-7" = 355	2.08	1.50	2.54	36.80	
9 2:30	30	31'-1" = 373 (Dry)	1.50	~~0~~	0.75	—	
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							

TIERRA TECH. TESTING LAB INC.
388 Dawson Drive,
P.O. Box 3240
Camarillo, CA 93011

Leonard C. Hayes
(805) 484-3657
(805) 482-7626

August 20, 1985
File No. 85-4664
Report No. 13350

Env. Health

KARL ~~FRY~~
c/o Alvie Thompson
3777 Pacific Coast Highway
Oxnard, CA. 93033

COUNTY COPY

SUBJECT: Lot 15, Tr. 2706, Alviso Drive
Las Posas Hills

PERCOLATION DATA (Deep Seepage Pit)

In response to your request and as required by Ventura County, we have explored the subsurface soils regarding suitability for sewage effluent disposal by the deep seepage pit method. Based upon the configuration of the site, the deep pit method is considered significantly preferable.

Hydrometer classification tests were performed on representative soil samples obtained from the 6-inch diameter x 50-foot deep test boring. The location of the test boring and the proposed seepage pit is shown on the attached map. The log of boring is also attached.

Based upon our field observations, test results and calculations, it is our determination that two 4-foot diameter x 40-foot deep seepage pits will be adequate.

Since an on-site sewage disposal system may be new to you, we would advise you that proper maintenance of the system is vital to ensure a long lasting, trouble-free system. One important maintenance item is to pump the septic tank once every two or three years.

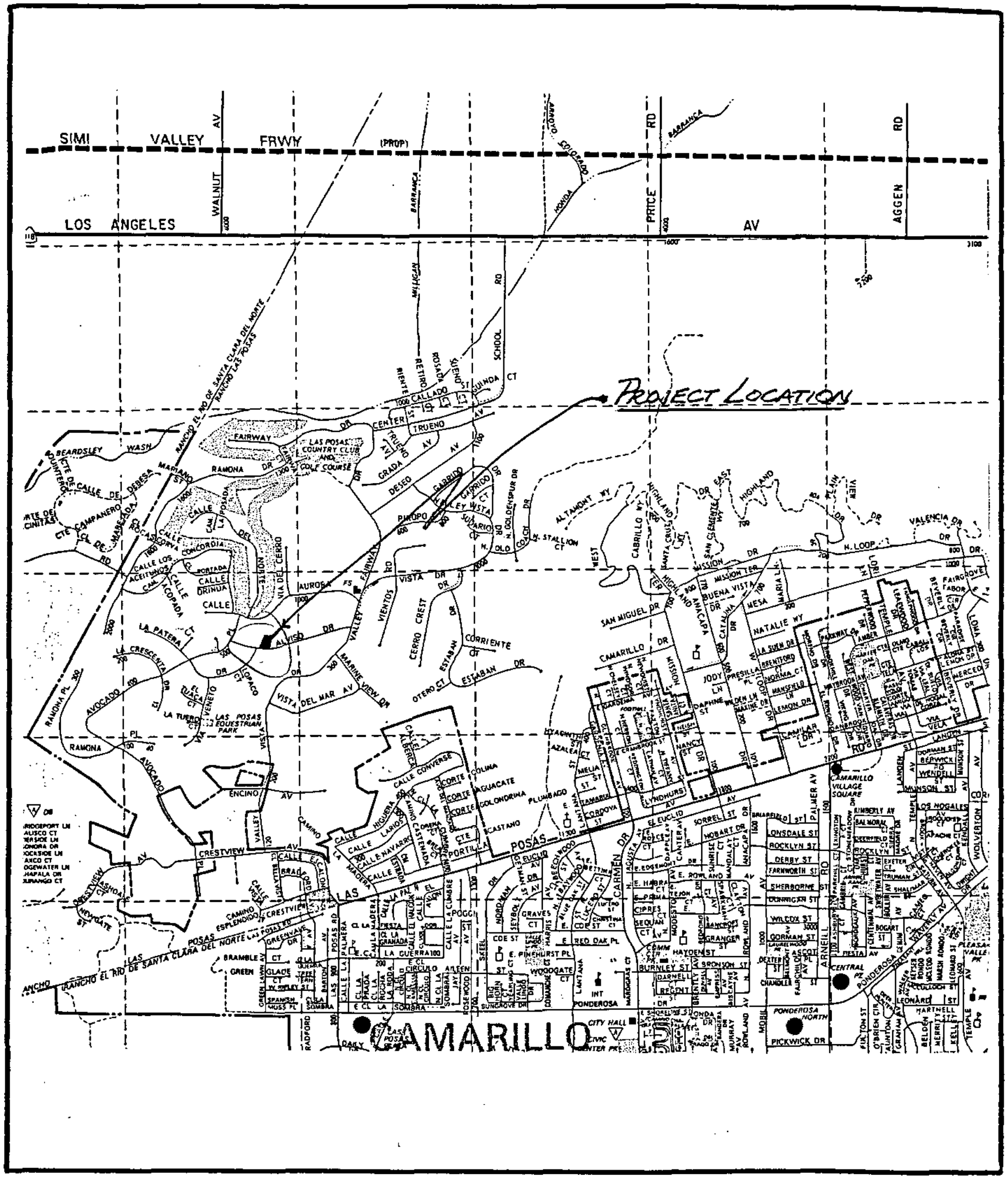
Respectfully submitted,

TIERRA TECH Testing Laboratory, Inc.

Leonard C. Hayes, RCE 21561

LCH:cl
Fry(5)

FIGURE I



SCALE: 1" = 2800'

N

LOCATION MAP

SOIL IDENTIFICATION REPORT

Location of Property: lot 15, Tr. 2706 Nearest Cross Street: Alviso Drive La Patera
(Job Address)

Owner / Builder: Karl Fry c/o Alvie Thompson Address: 3777 PCH, Oxnard, 93030

Method of Drilling: 6" auger Drilled By: Tierra Tech

Date Tested: 8-7-85 Weather Conditions: cloudy 68°±

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10	4'		III	5.5	58	36	6	silty sand
15			IV	20.1	25	48	27	clayey silt
20			IV	18.3	26	54	20	sandy silty clay
25	14'		IV	21.5	26	46	28	clayey silt
30	6'		III	13.3	46	26	28	sandy clay
35	4'		V	18.6	22	44	34	silty clay
40	4'		II	8.1	72	16	12	sand — Bottom of Pit
45				8.4	78	14	8	sand
50				6.8	80	12	8	sand
55								
60								

36.82, 43.63, 33.33, 12.66, 18.06, 26.68, 32.75

Try 5' diam. pit
D = 40'
A = 15.7 ft²
Eff D = 37'

II 4(4.0)(15.7) = 251
III 10(2.5)(15.7) = 393
IV 18(1.1)(15.7) = 311
V 5(.83)(15.7) = 65
Q = 1020 gal

4' diam. Q = 816 gal/day

PIT PERFORMANCE TEST DATA WORKSHEET

Location: Alviso Dr Date Tested: 8-7-85

Test performed by: B. K. Depth of hole: 50 (ft)

Date presaturated: 8-6-85 Diameter: 0.5 (ft)

Sloughed to 31'-1"

Time (Min)	Elapsed Time (ET) (Min)	Depth to Top of Water (TP) (Inches)	Water Drop (Δh) (ft)	Depth of Water Left in Hole (d) (ft)	Average Head (Have) (ft)	Absorption Rate (A) gal/ft²/day	Comments
						$A = \frac{1348.3\,(\Delta H)}{H_A \cdot ET}$ .89	
1 11:45		3'-1" = 37		23.00			
2 11:50	5	7'-1" = 85	4.0	24.00	26.00	41.48	36.82
3 11:55	5	11'-1" = 133	4.0	20.00	22.00	49.02	43.63
4 12:03	8	15'-1" = 181	4.0	16.00	18.00	37.45	33.33
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7 1:30	30	27'-6" = 330	4.00	3.58	5.58	32.22	26.68
8 2:00	30	29'-7" = 355	2.08	1.50	2.54	36.80	32.75
9 2:30	30	31'-1" = 373 (Dry)	1.50	~~0~~	0.75	—	
10							
11							
12							
13	1,701	21,542					
14		585 = 36.82 / 41.48 = .89 Correction					
15							
16		12.66					
17							
18							
19							
20							

county of ventura

SPEED LETTER

TO: BILL STRATTON

RETURN TO:

SUBJECT: C.S.A. 32

Message: Clear Fray Property

Signature: Tony   Date: 12-30-85

Reply:

Signature: __________   Date: __________

9-29-85 Initial Review
1) CSA 32 Required
2) Fee Required

W.C.S.

10-2-85 Site Visit: Looks adequate
10-3-85 T.C. Alvie Thompson - Re - CSA 32 ✗ -
Fee ~~$42.00~~ OK -
12-30-85 CSA32 - Clearance

Address 179 Aluiso Dr.
Lot #16
Owner Karl Fry - C/o Alvie Thompson.

Description

#bedrooms - - 4
#F.U. - - - 35
Tank - - - 1500
} OK

Proposed System.
Two pits 2 x 4' x 40'

4' x 40'

UPC Soil absorbtion System.

II =	4'	x 12.57 =	50	x 4.0 =	201	
III =	10'	" =	126	x 2.5 =	315	
IV =	19'	" =	239	x 1.1 =	263	
V =	4'	" =	50.28	x .83 =	42	
	37'		465 ft²		821	gal/ft²/day

Design Rate
821 gal/ft²/day x 2 pits = 1,642 gal/ft²/day

1500 gal < 1,642 — appear adequate

465 ft² x 2 = 930 ft² — Loading Rate

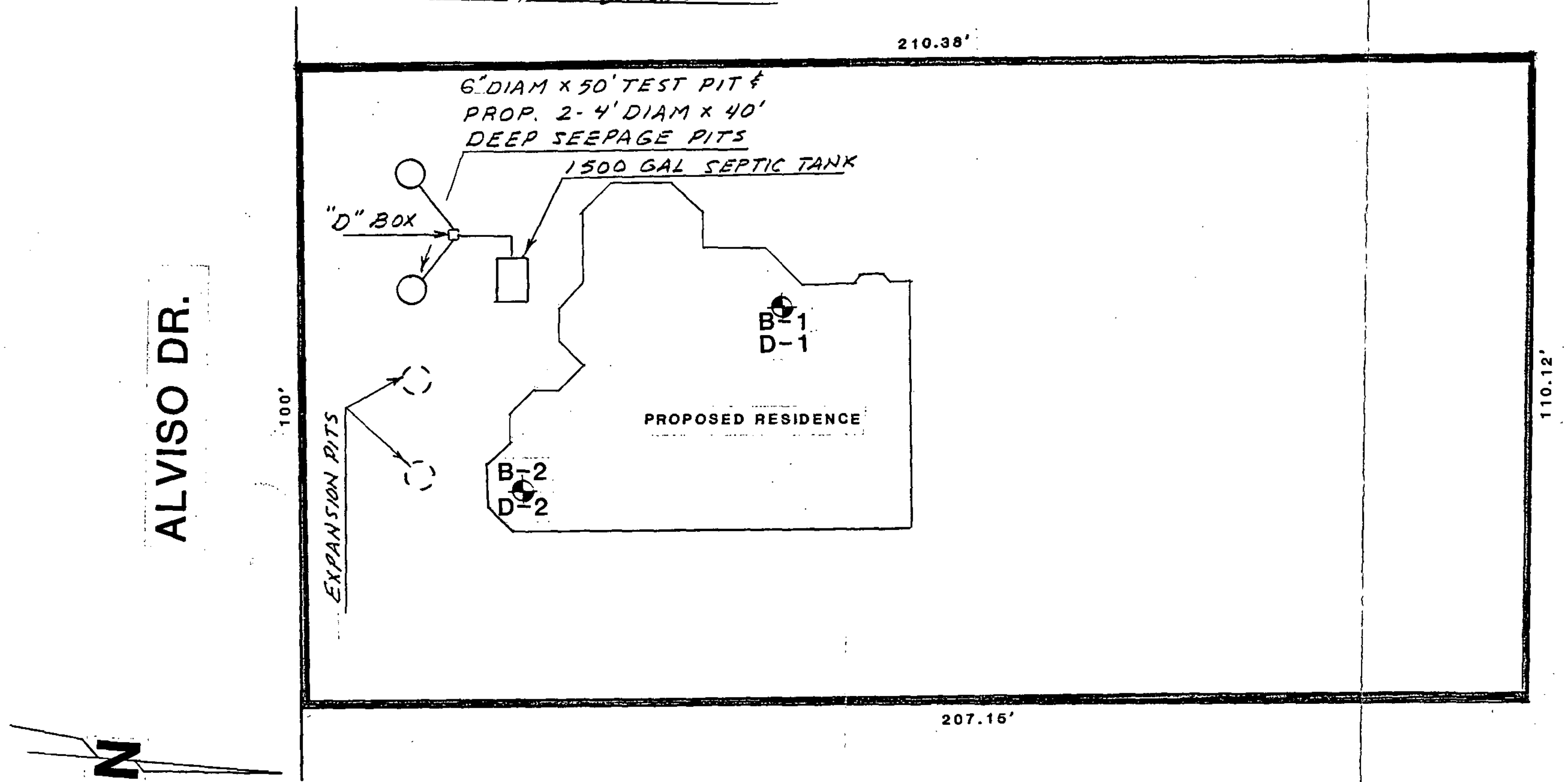
a) $\frac{1500 \text{ gal}}{930 \text{ ft}^2}$ = 1.61 gal/ft² — maybe adequate

b) 150 x 4 = $\frac{600 \text{ gal/ft}^2\text{/bed}}{930 \text{ ft}^2}$ = .65

Performance ? Rates Exceed UPC absorbtion Rates — therefore — Performance test Rates will not be used.

APPROVED

Approval extends only to items covered by **Sanitation Codes** (State laws and local ordinances) and **does not pertain** to construction details except as related to such **Codes.**
VENTURA COUNTY ENVIRONMENTAL HEALTH DEPARTMENT
Date 10-21-85 By W C Shatta



SCALE: 1" = 20'

tierra tech	KARL FRY LOT 15, TRACT 2706 ALVISO DRIVE CAMARILLO	date: 8/14/85 **Plate 2**

GEOTECHNICAL ENGINEERING REPORT

191 ALVISO DRIVE

CAMARILLO AREA OF VENTURA COUNTY, CALIFORNIA

PROJECT NO.: 302698-001

JANUARY 4, 2019

PREPARED FOR

CRESTVIEW MUTUAL WATER COMPANY

ATTENTION: ROBERT ERANIO

BY

EARTH SYSTEMS PACIFIC

1731-A WALTER STREET

VENTURA, CALIFORNIA 93003

Earth Systems
1731 Walter Street, Suite A | Ventura, CA 93003 | Ph: 805.642.6727 | www.earthsystems.com

January 4, 2019

Project No.: 302698-001
Report No.: 19-1-4

Crestview Mutual Water Company
Attention: Robert Eranio
328 Valley Vista Drive
Camarillo, CA 93010

Project: 191 Alviso Drive
Camarillo Area of Ventura County, California
Subject: Geotechnical Engineering Report

As authorized, Earth Systems Pacific (Earth Systems) has performed a geotechnical study for proposed construction at 191 Alviso Drive in the Camarillo area of Ventura County, California. The accompanying Geotechnical Engineering Report presents the results of our subsurface exploration and laboratory testing programs, and our conclusions and recommendations pertaining to geotechnical aspects of project design. This report completes Phase 1 of the scope of services described within our Proposal VEN-18-08-010 dated August 20, 2018; revised October 10, 2018; and authorized by you on November 1, 2018.

We have appreciated the opportunity to be of service to you on this project. Please call if you have any questions, or if we can be of further service.

Respectfully submitted,

EARTH SYSTEMS PACIFIC

1-4-2019
Meng Wei Lu
Civil Engineer

REGISTERED PROFESSIONAL ENGINEER MENG WEI LU C 89106 Exp 9/30/20 CIVIL STATE OF CALIFORNIA

Reviewed and Approved

Richard M. Beard
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER RICHARD MICHAEL BEARD GE 128 Exp. 12-31-19 GEOTECHNICAL STATE OF CALIFORNIA

1/4/19

Todd J. Tranby
Engineering Geologist

PROFESSIONAL GEOLOGIST TODD J. TRANBY No. 2078 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Copies: 4 - Client (3 hardcopies, 1 email)
1 - Project File

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

INTRODUCTION

Project Description

This report presents results of a Geotechnical Engineering study performed for proposed construction at 191 Alviso Drive in the Camarillo area of Ventura County, California (see Vicinity Map in Appendix A). It is anticipated that the proposed construction will be support structures around a proposed water well including a building pad, a materials pad, a generator pad, and a retaining wall.

Structural considerations for building column loads of up to 10 kips with maximum wall loads of 1 kip per lineal foot were used as a basis for the recommendations of this report. If actual loads vary significantly from these assumed loads, Earth Systems should be notified since reevaluation of the recommendations contained in this report may be required.

Purpose and Scope of Work

The purpose of the geotechnical study that led to this report was to analyze the soil/bedrock conditions of the project site and to provide geotechnical recommendations for construction. The soil conditions include surface and subsurface soil types, expansion potential, soil strength, settlement potential, bearing capacity, and the presence or absence of subsurface water. The scope of work included:

- Performing a reconnaissance of the project site.
- Drilling, sampling, and logging 2 hollow-stem-auger borings to study bedrock, soil, and groundwater conditions.
- Laboratory testing soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
- Consulting with owner representatives and design professionals.
- Analyzing the geotechnical data obtained.
- Preparing this report.

Contained in this report are:

- Descriptions and results of field and laboratory tests that were performed.
- Conclusions and recommendations pertaining to site grading and structural design.

Site Setting

The project site is currently a vacant lot that covered by short grass. Chain-link fencing is installed abound the southern boundary of the site. The project site is bounded by Alviso Drive to the south, a natural drainage to the north, and residential lots to the west and east. The project site appears to drain to the northwards. The geographic coordinates of the project site are 34.2424˚ North Latitude and 119.0749° West Longitude.

REGIONAL GEOLOGY

The property lies within the western portion of the Transverse Ranges geologic province. Numerous east-west trending folds and reverse faults indicative of active north-south transpressional tectonics characterize the region.

Regional Geologic Map 1 (T.W. Dibblee, Jr, Geologic Map of the Camarillo and Newbury Park Quadrangles, 1990 indicates the northeast-southwest trending Springville Fault Zone to be about 3,800 feet southeast of the site (see Appendix A).

Regional Geologic Map 2 (USGS/CGS, SCAMP Geologic Map of the Camarillo 7.5' Quadrangle, 2004) indicates the northeast-southwest trending Springville Fault Zone to be about 3,200 feet southeast of the site (see Appendix A).

The site is mapped by T.W. Dibblee, Jr. as underlain by Saugus Formation Bedrock, and mapped by the USGS/CGS as underlain by both Saugus Formation Bedrock and Las Posas Formation Bedrock. Our field study encountered a layer of soil (thickness of up to about 3 feet) overlying Saugus Formation Bedrock.

SEISMICITY AND SEISMIC DESIGN

Although the project site is not within a State-designated "fault rupture hazard zone", it is located in an active seismic region where large numbers of earthquakes are recorded each year. Historically, major earthquakes felt in the vicinity of the project site have originated from faults near the area. These include the 1857 Fort Tejon earthquake, the 1872 Owens Valley earthquake, and the 1952 Arvin-Tehachapi earthquake.

It is assumed that the 2016 CBC and ASCE 7-10 guidelines will apply for the seismic design parameters. The 2016 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein were determined by the United States Seismic Design Maps "risk-targeted" calculator on the USGS website for the project site coordinates (34.2424˚ North Latitude and 119.0749° West Longitude). The calculator adjusts for Soil Site Class C, and for Occupancy (Risk) Category I/II/III.

The calculated 2016 California Building Code (CBC) and ASCE 7-10 seismic parameters typically used for structural design are included in Appendix D and summarized in the following table.

Summary of Seismic Parameters (2016 CBC)

Seismic Design Category	E
Site Class (Table 20.3-1 of ASCE 7-10 with 2013 update)	C
Occupancy (Risk) Category	I/II/III
Maximum Considered Earthquake (MCE) Ground Motion	
Peak Modified Ground Acceleration – PGA_m	1.105 g
Spectral Response Acceleration, Short Period – S_s	2.785 g
Spectral Response Acceleration at 1 sec. – S_1	0.995 g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.30
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.785 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.294 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.857 g
One Second Spectral Response – S_{D1}	0.862 g

The values presented in the table above are appropriate for a 2 percent probability of exceedance in 50 years. A listing of the calculated 2016 CBC and ASCE 7-10 seismic parameters is included in Appendix D.

The Fault Parameters table in Appendix D lists the significant "active" and "potentially active" faults within a 34-mile (55-kilometer) radius of the project site. The distance between the project site and the nearest portion of each fault is shown, as well as the respective estimated maximum earthquake magnitudes, and the deterministic mean site peak ground accelerations.

SOIL/BEDROCK AND GROUNDWATER CONDITIONS

Evaluation of the subsurface indicates that much of the project site is blanketed by a layer of soil (clayey silt, thickness of up to about 3 feet) which is underlain by Saugus Formation Bedrock.

Testing indicates that anticipated bearing soils lie in the "High" expansion range based on a measured expansion index of 102. A locally adopted version of this classification of soil expansion, Table 1809.7, is included in Appendix C of this report. It appears that soils can be cut by normal grading equipment.

Groundwater was not encountered in either boring to a maximum depth of about 31.5 feet below ground surface. According to the Seismic Hazard Zones Report for the Camarillo 7.5-Minute Quadrangle, Ventura County, California (CGS, 2002), the project site is within a valley/mountain boundary zone. See Historical High Groundwater Map in Appendix A. It should be noted that fluctuations in groundwater levels may occur because of variations in rainfall, regional climate, and other factors.

A sample of near-surface soil was tested for pH, resistivity, soluble sulfates, and soluble chlorides. The test results provided in Appendix B should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils. It should be noted that sulfate content (15 mg/Kg) is in the "S0" exposure class (i.e. "Negligible" severity range) of Table 19.3.1.1 of ACI 318-14. Therefore, special concrete designs will not be necessary for the measured sulfate content according to Table 19.3.2.1 of ACI 318-14.

Based on criteria established by the County of Los Angeles, the measured resistivity of a near-surface soil sample (6,700 ohms-cm) indicates that near-surface soils are "Moderately Corrosive" to ferrous metal (i.e. cast iron, etc.) pipes. It should be noted that Earth Systems does not practice soil corrosion engineering.

HYDROCOLLAPSE POTENTIAL

Hydrocollapse is a phenomenon in which naturally occurring soil deposits, or non-engineered fill soils, collapse when wetted. Natural soils that are susceptible to this phenomenon are typically aeolian, debris flow, alluvial, or colluvial deposits with high apparent strength when dry. Loosely compacted fills can also be susceptible to this phenomenon. The dry strength is attributed to salts, clays, silts, and in some cases capillary tension, "bonding" larger soil grains together. So long as these soils remain dry, their strength and resistance to compression are retained. However, when wetted, the salt, clay, or silt bonding agent is weakened or dissolved, or capillary tension reduced, eventually leading to collapse. Soils susceptible to this phenomenon are found throughout the southwestern United States.

The potential of this phenomenon is considered to be low at the project site because the project site is underlain at shallow depths by Saugus Formation Bedrock that is typically not susceptible to hydrocollapse.

LIQUEFACTION POTENTIAL

Earthquake-induced cyclic loading can be the cause of several significant phenomena, including liquefaction in fine sands and silty sands. Liquefaction results in a loss of soil strength and can cause structures to settle and, in extreme cases, to experience bearing failure.

The potential hazard posed by liquefaction is considered to be low at the project site because:

- The project site does not lie within a potentially liquefiable zone (see Seismic Hazard Zones Map in Appendix A).
- The project site is underlain at shallow depths (about 3 feet) by Saugus Formation Bedrock that is typically not susceptible to liquefaction.

SEISMIC-INDUCED SETTLEMENT OF DRY SANDS

Dry (unsaturated) soils tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, cyclic shear strain magnitude, and the number of strain cycles. A procedure to evaluate this type of settlement was developed by

Seed and Silver (1972) and later modified by Pyke, et al. (1975). Tokimatsu and Seed (1987) presented a simplified procedure that has been reduced to a series of equations by Pradel (1998). Research on this subject is continuing (Stewart, et al., 2004).

The potential of this phenomenon is considered to be low at the project site because the project site is underlain at shallow depths by Saugus Formation Bedrock that is typically not susceptible to seismic-induced settlement of dry sands.

FAULT RUPTURE HAZARD

A fault is a break in the earth's crust upon which movement has occurred in the recent geologic past and future movement is expected. A summary of nearby active faults is presented in Appendix D under Table 1 Fault Parameters.

The project site does not lie within a State of California designated active fault hazard zone. The activity of faults is classified by the State of California based on the Alquist-Priolo Earthquake Fault Zoning Act (1972). An active fault has had surface rupture with Holocene time (the past 11,000 years). A potentially active fault shows evidence of surface displacement during Quaternary time (last 1.6 million years). An inactive fault has no evidence of movement within the Quaternary time.

As previously discussed in the Regional Geology section of this report, all nearby faults according to both reviewed Regional Geologic Maps are no closer than about 3,200 feet from the project site. Therefore, the potential for fault rupture at the project site is considered low.

LANDSLIDES

Landsliding is a process where a distinct mass of rock or soil moves downslope because of gravity. No landslides are mapped on the project site by Dibblee or USGS (see Regional Geologic Maps in Appendix A). Because there are no identified landslides either on or trending into the project site, hazards associated with these phenomena are considered low.

ROCKFALL

Loose boulder-sized rocks and/or weathering bedrock outcrops located upslope from construction can lead to a rockfall hazard. Because of the project site's location on top of a slope area, the potential for rockfall onto the project site appears to be low.

EARTHQUAKE-INDUCED FLOODING

Earthquake-induced flooding types include tsunamis, seiches, and reservoir failure. Because of the inland location of the project site, hazards from tsunamis and seiches are considered unlikely. Additionally, there are no reservoirs upstream of the project site. Therefore, earthquake-induced flooding is not considered a potential hazard at the project site.

OTHER FLOODING

The project site is not within any of the flood hazard areas mapped by Federal Emergency Management Agency (FEMA), FEMA Flood Map for Ventura County Unincorporated Areas, effective January 7, 2015, Map No. 06111C0927F.

CONCLUSIONS AND RECOMMENDATIONS

Based on the data provided in this report, it appears that the project site is suitable for the proposed improvements from a geotechnical engineering standpoint provided that the recommendations provided herein are properly implemented into the project.

Earth Systems recommends conventional footings and/or pad footings to be used to support the proposed improvements. Given the site conditions encountered, we conclude that remedial grading will be needed to provide a more uniform bearing condition (i.e., the footings should be supported only by recompacted fill, not by native soil and/or native Saugus Formation Bedrock).

Specific conclusions and recommendations addressing these geotechnical considerations, as well as general recommendations regarding the geotechnical aspects of design and construction, are presented in the following sections.

A. Grading

1. Pre-Grading Considerations

a. Roof draining systems should be designed so that water is not discharged into bearing soils or near structures.

b. Final site grade should be designed so that all water is diverted away from the structures over paved surfaces, or over landscaped surfaces in accordance with current codes. Water should not be allowed to pond anywhere on the pad.

c. Shrinkage of soils (uncertified fills) affected by compaction is estimated to be about 5 percent based on an anticipated average compaction of 92 percent.

d. Earth Systems should be retained to provide geotechnical engineering services during site development and grading, and foundation construction phases of the work to observe compliance with the design concepts, specifications and recommendations. This will allow for timely design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

e. Plans and specifications should be provided to Earth Systems prior to grading. Plans should include the grading plans, foundation plans, and foundation details. Earth Systems will review these plans only for conformity with geotechnical parameters not including drainage. It is the responsibility of the Client and other Engineers to review and approve designs and plans for conformity with all engineering and design requirements necessary to the proper function and performance of the structure.

f. Compaction tests should be made to determine the relative compaction of the fills in accordance with the following minimum guidelines: two tests for each 1.5-foot vertical lift in every isolated area graded; two tests for each 500 cubic yards of material placed; and two tests at finished subgrade elevation in the areas of remedial grading.

2. Rough Grading/Areas of Development

a. Grading at a minimum should conform to the 2016 California Building Code.

b. The existing ground surface should be initially prepared for grading by removing all vegetation, trees, large roots, debris, other organic material and non-complying fill. Organics and debris should be stockpiled away from areas to be graded, and ultimately removed from the project site to prevent their inclusion in fills. Voids created by removal of such material should be properly backfilled and compacted. No compacted fill should be placed unless the underlying soil has been observed by the Geotechnical Engineer.

c. To provide a uniform and constructible pad, overexcavation and recompaction of soils in these construction areas will be necessary. Soils should be overexcavated to at least 1.5 feet below the bottom of footings (or through soil). Overexcavation should be extended to a distance of at least 5 feet laterally, but not less than a distance equal to the depth of removal, beyond the outside edge of the foundation elements.

d. The bottoms of all excavations should be observed by a representative of Earth Systems prior to processing or placing fill.

e. The resulting surface(s) should then be scarified an additional 6 inches, uniformly moisture conditioned to about 3 percent over the optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D 1557 maximum dry density. Compaction of the prepared subgrade should be verified by testing prior to the placement of engineered fill.

f. To control differential settlement and provide a more uniform bearing condition, foundations should bear completely onto recompacted soil

g. On-site soils may be used for fill once they are cleaned of all organic material, rocks, debris, and irreducible material larger than 6 inches.

h. Fill and backfill placed 3% over the optimum moisture in layers with a loose thickness not greater than 8 inches should be compacted to a minimum of 90 percent of the maximum dry density obtainable by the ASTM D 1557 test method unless otherwise recommended or specified by the Geotechnical Engineer or his/her representative. Random compaction tests by Earth Systems can assist the Grading Contractor in evaluating whether the Grading Contractor is meeting compaction requirements. However, compaction tests pertain only to a specific location and do not guaranty that all fill has been compacted to the prescribed percentage of maximum density. It is the ultimate responsibility of the Grading Contractor to achieve uniform compaction in accordance with the requirements of this report and the grading ordinance.

i. Import soils used (if any) to raise site grade should be equal to, or better than, on-site soils in strength, expansion, and compressibility characteristics. Import soil can be evaluated, but will not be prequalified by the Geotechnical

Engineer. Final comments on the characteristics of the import will be given after the material is at the project site.

j. Periodic wetting of the soils after grading would be beneficial in regard to presaturation.

3. Utility Trenches

a. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with native soils compacted to 90 percent of maximum density. Backfill of offsite service lines will be subject to the specifications of the jurisdictional agency or this report, whichever are greater.

b. Utility trenches running parallel to footings should be located at least 5 feet outside the footing line, or above a 1:1 (horizontal to vertical) projection downward from the outside edge of the bottom of the footing.

c. Compacted on-site native soils should be utilized for backfill below structures. Clean sand backfill should be avoided under structures because it provides a conduit for water to migrate under foundations.

d. Backfill operations should be observed and tested by the Geotechnical Engineer to monitor compliance with these recommendations.

e. Rocks greater than 6 inches in diameter should not be placed in trench zones (from 12 inches below pavement subgrade or ground surface to 12 inches above top of pipe or box); rocks greater than 2.5 inches in diameter should not be placed in pipe zones (from 12 inches above top of pipe or box to 6 inches below bottom of pipe or box exterior).

f. Jetting should not be utilized for compaction in utility trenches.

B. Structural Design

1. Conventional Shallow Foundations

a. Conventional continuous footings and/or interior pad footings can be used to support structures. It should be noted that if pad footings are to be used, they must be tied together by grade beams (each way) or by slabs. Based on the tested expansion index of 102, perimeter continuous and/or pad footings should have a minimum embedment depth of 27 inches, and interior pad footings should have a minimum embedment depth of 12 inches. The expansion index should be re-evaluated at the completion of rough grading to confirm that these minimum footing depths are appropriate.

b. Footings should bear into firm recompacted fill as recommended elsewhere in this report. Foundation excavations should be observed by a representative of this firm after excavation, but prior to placing of reinforcing steel or concrete, to verify bearing conditions.

c. Perimeter footings embedded 27 inches deep may be designed based on an allowable bearing value of 2,200 psf. This value includes a safety factor of 3. This allowable bearing value is net (weight of footing and soil surcharge may be neglected) and is applicable for dead plus reasonable live loads.

d. Interior footings embedded 12 inches deep may be designed based on an allowable bearing value of 1,800 psf. This value includes a safety factor of 3. This allowable bearing value is net (weight of footing and soil surcharge may be neglected) and is applicable for dead plus reasonable live loads.

e. Bearing values may be increased by one-third when transient loads such as wind and/or seismicity are included.

f. Lateral loads may be resisted by soil friction on floor slabs and foundations and by passive resistance of the soils acting on foundation stem walls. Lateral capacity is based on the assumption that any required backfill adjacent to foundations and grade beams is properly compacted.

g. The information that follows regarding reinforcement and premoistening for footings is the same as that given in Table 1809.7 for the "High" expansion range. Actual footing designs should be provided by the project Structural Engineer, but the dimensions and reinforcement he recommends should not be less than the criteria set forth in Table 1809.7 for the appropriate expansion range.

h. Continuous footings bottomed in soils in the "High" expansion range should be reinforced, at a minimum, with two No. 4 bars along the bottom and two No. 4 bars along the top. In addition, bent No. 3 bars on 24-inch centers should extend from within the footings to a minimum of 3 feet into adjacent slabs.

i. Bearing soils in the "High" expansion range should be premoistened to about 3 percent above optimum moisture content to a depth of 33 inches below lowest adjacent grade. Premoistening should be confirmed by testing.

2. Slabs-on-Grade

a. Concrete slabs on grade should be supported by firm recompacted fills as recommended elsewhere in this report. Because the soils of the project site are in the “High” expansion range, it should be anticipated that exterior concrete supported on grade will be susceptible to movement with seasonal change in soil moisture content. The following recommendations for concrete slabs on grade can help mitigate, but not eliminate, such movement.

b. It is recommended that perimeter slabs (walkways, patios, etc.) be designed relatively independent of footing stems (i.e. free floating) so foundation adjustment will be less likely to cause cracking. Because the on-site soils are highly expansive, the exterior concrete slabs on grade should have turned-down edges of at least 8 inches into the soil.

c. The information that follows regarding design criteria for slabs is generally the same as that given in Table 1809.7 for the "High" expansion range. Actual slab designs should be provided by the project Structural Engineer, but the reinforcement and slab thicknesses he recommends should not be less than the criteria set forth in Table 1809.7 for the appropriate expansion range, or as recommended below, whichever is more stringent.

d. Slabs bottomed on soils in the "High" expansion range should be underlaid with a minimum of 4 inches of sand. Areas where floor wetness would be undesirable should be underlaid with a vapor retarder (as specified by the Project Architect or Civil Engineer) to reduce moisture transmission from the subgrade soils to the slab. The retarder should be placed as specified by the project Structural Engineer or Architect.

e. Slabs bottomed on soils in the "High" expansion range should at a minimum be reinforced at mid-slab with No. 3 bars on 24-inch centers, each way. No. 3 bars acting as dowels should also extend out of the perimeter footings, and should be bent so that they extend a minimum of 3 feet into adjacent slabs.

f. Soils underlying slabs that are in the "High" expansion range should be premoistened to about 3 percent above optimum moisture content to a depth of 33 inches below lowest adjacent grade.

g. Premoistening of slab areas should be observed and tested by this firm for compliance with these recommendations prior to placing of sand, reinforcing steel, or concrete.

3. Frictional and Lateral Coefficients

a. Resistance to lateral loading may be provided by soil friction acting on the base of foundations. A coefficient of friction of 0.53 may be applied to dead load forces. This value does not include a safety factor.

b. Passive resistance acting on the sides of foundation stems equal to 310 pcf of equivalent fluid weight may be included for resistance to lateral load. This value does not include a safety factor.

c. A minimum safety factor of 1.5 should be used when designing for sliding or overturning.

d. Passive resistance may be combined with frictional resistance provided that a one-third reduction in the coefficient of friction is used.

4. Retaining Walls

a. Conventional cantilever retaining walls should not be backfilled with on-site soils because of the expansion potential of those soils. Walls that are backfilled at a 1:1 projection upward from the heels of the wall footings with crushed rock or non-expansive sand, may be designed for active pressures of 38 pcf of equivalent fluid weight for well-drained, level backfill. An 18-inch thick cap of compacted native soils should be placed above the rock or sand. Filter fabric should be placed between the rock or sand and native soils and/or backfill over the top.

b. The pressures listed above were based on the assumption that backfill soils will be compacted to 90 percent of maximum dry density as determined by the ASTM D 1557 Test Method.

c. Retaining walls may need to be designed for a seismic loading force that is applied in addition to the static forces when seismic shaking occurs. A seismic increment of earth pressure determined using 34 pcf of additional equivalent fluid weight needs to be considered for cantilever retaining walls that retain more than 6 feet of soil. This pressure has been determined by a procedure presented by Al Atik and Sitar (2010). The seismic increment of pressure can be assumed to be distributed so that the centroid of pressure acts at 0.33H above the base of a retaining wall, where H is the wall height in feet. Because this seismic force is transient, and in accordance with CBC Section 1807.2.3, a minimum safety factor of 1.1 may be used for sliding and overturning when seismic loads are included.

d. The lateral earth pressure to be resisted by the retaining walls or similar structures should also be increased to allow for any other applicable surcharge loads. The surcharges considered should include forces generated by any structures or temporary loads that would influence the wall design.
e. A system of backfill drainage should be incorporated into retaining wall designs. Backfill comprising the drainage system immediately behind retaining structures should be free-draining granular material with a filter fabric between it and the rest of the backfill soils. As an alternative, the backs of walls could be lined with geodrain systems. The backdrains should extend from the bottoms of the walls to about 18 inches from finished backfill grade. Waterproofing may aid in reducing the potential for efflorescence on the faces of retaining walls.
f. Compaction on the uphill sides of walls within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential "locked-in" lateral pressures caused by compaction with heavy grading equipment.
g. Water should not be allowed to pond near the tops of retaining walls. To accomplish this, final backfill site grades should be such that all water is diverted away from retaining walls.

5. Settlement Considerations
   a. A maximum settlement (static and seismic combined) of about half of an inch (0.5”) is anticipated for foundations and slabs designed as recommended.
   b. Differential settlement between adjacent load bearing members could be about one-half the maximum settlement.
   c. The Project Structural Engineer will need to design the foundation system to accommodate the potential settlement values.

ADDITIONAL SERVICES

This report is based on the assumption that an adequate program of monitoring and testing will be performed by Earth Systems during construction to check compliance with the recommendations given in this report. The recommended tests and observations include, but are not necessarily limited to the following:

- Review of the structural and grading plans during the design phase of the project.
- Observation and testing during site preparation, grading, placing of engineered fill, and foundation construction.
- Consultation as required during construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analyses and recommendations submitted in this report are based in part upon the data obtained from the on-site borings. The nature and extent of variations beyond the points of exploration may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

The scope of services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements in this report or on the soil boring logs regarding odors noted, unusual or suspicious items or conditions observed, are strictly for the information of the client.

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they are because of natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of 1 year.

In the event that any changes in the nature, design, or location of the proposed structures and other improvements are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to ensure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth Systems has striven to provide services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of the Client for the purposes stated in this document for the referenced project only. No third party may use or rely on this report without express written authorization from Earth Systems for such use or reliance.

It is recommended that Earth Systems be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded the privilege of making this recommended review, it can assume no responsibility for misinterpretation of the recommendations contained herein.

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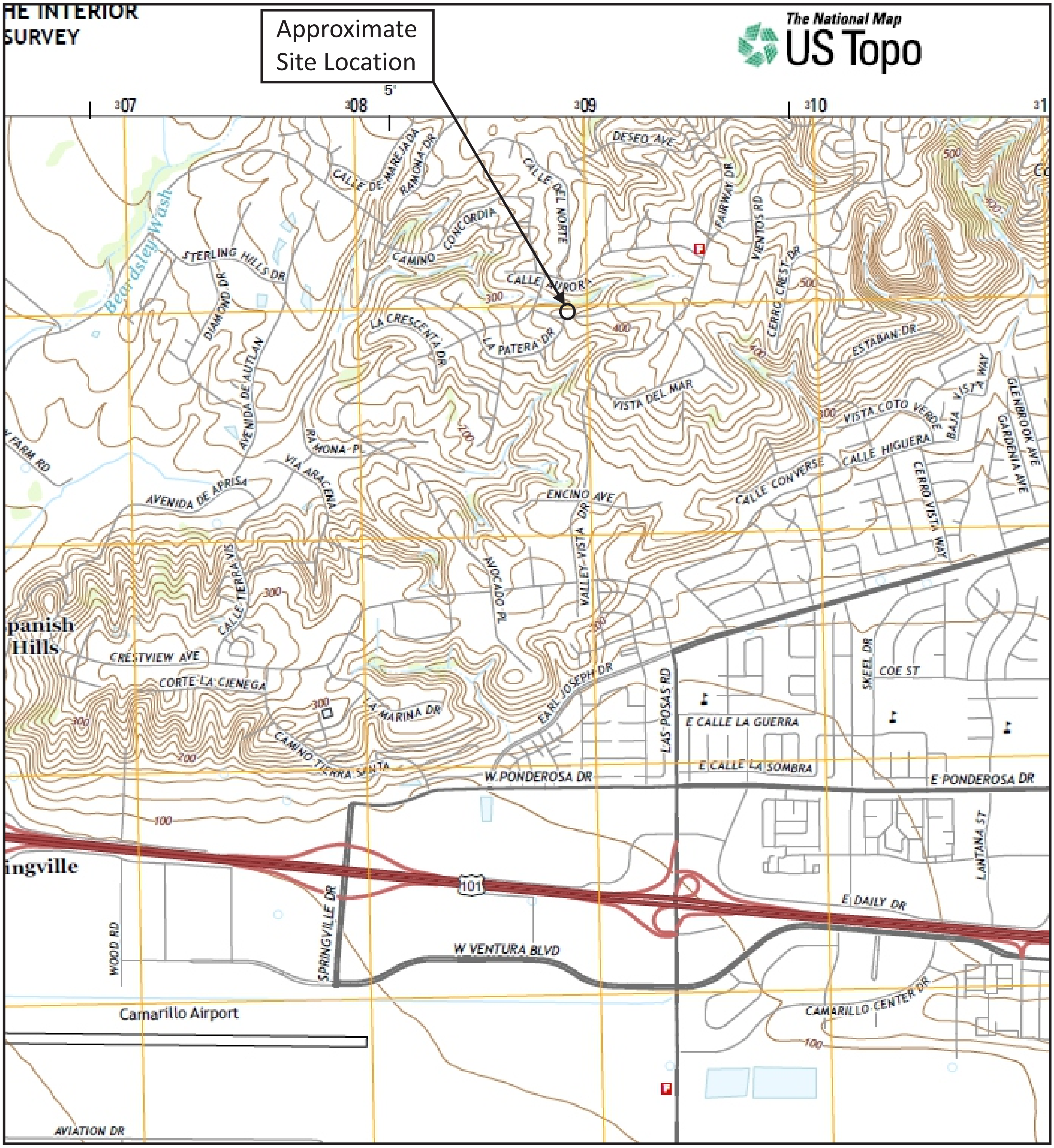
APPENDIX A

Vicinity Map
Regional Geologic Map 1 (Dibblee)
Regional Geologic Map 2 (USGS/CGS [SCAMP])
Seismic Hazard Zones Map
Historical High Groundwater Map
Site Plan
Field Study
Logs of Borings
Boring Log Symbols
Unified Soil Classification System



*Taken from USGS Topo Map, Camarillo Quadrangle, California, 2015.

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

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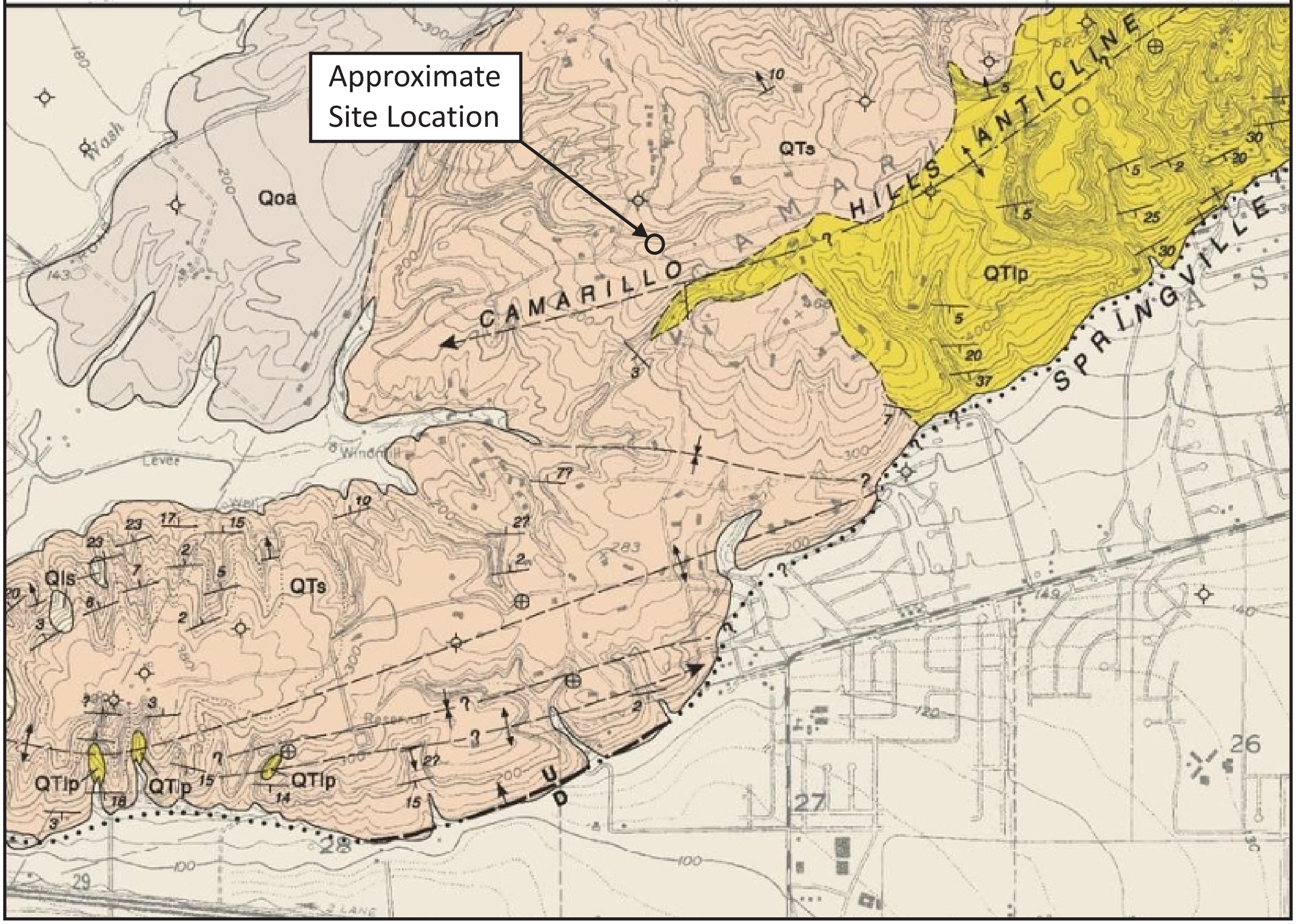
VICINITY MAP

191 Alviso Drive
Camarillo Area of Ventura County, California

Earth Systems

January 2019	302698-001

DF-28 GEOLOGY CAMARILLO AND NEWBURY PARK QUADRANGLES



*Taken from Dibblee, Jr., Geologic Map of The Camarillo and Newbury Park Quadrangles, Ventura County, California, 1990, DF-28.

GEOLOGIC SYMBOLS

not all symbols shown on each map

FORMATION CONTACT *dashed where inferred or indefinite, dotted where concealed*

MEMBER CONTACT *between units of a formation* .... Prominent bed

CONTACT BETWEEN SURFICIAL SEDIMENTS *located only approximately in places*

FAULT: *Dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful. Parallel arrows indicate inferred relative lateral movement. Relative vertical movement is shown by U/D (U=upthrown side, D=downthrown side). Short arrow indicates dip of fault plane. Sawteeth are on upper plate of low angle thrust fault.*

FOLDS: overturned — ANTICLINE — SYNCLINE

arrow on axial trace of fold indicates direction of plunge; dotted where concealed by surficial sediments

Strike and dip of sedimentary rocks: inclined; inclined (approximate); overturned; horizontal; vertical

Strike and dip of metamorphic or igneous rock foliation or flow banding or compositional layers: inclined; inclined (approximate); vertical; overturned

OTHER SYMBOLS: Direction of landslide movement; outline of water bodies shown on map; water well; oil well; springs

QTs

SAUGUS FORMATION
(Of Hershey, 1909; Kew, 1924, Weber et al, 1973; Jakes, 1979)
Nonmarine fluviatile; probably Pleistocene age in this area

QTs *Weakly indurated, light gray to light brown pebble-cobble gravel, sand and clay; includes indurated paleo-soil layers locally; gravel contains clasts of granitic and metavolcanic rocks, quartzite and siliceous shale (Monterey Formation); in eastern Los Posas Hills contains lenses of volcanic detritus from Conejo Volcanics at and near base; grades downward and in part eastward into Las Posas Sand*

Qoa

OLDER DISSECTED SURFICIAL SEDIMENTS

Qoa *Dissected, weakly indurated alluvial gravel, sand and clay*

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

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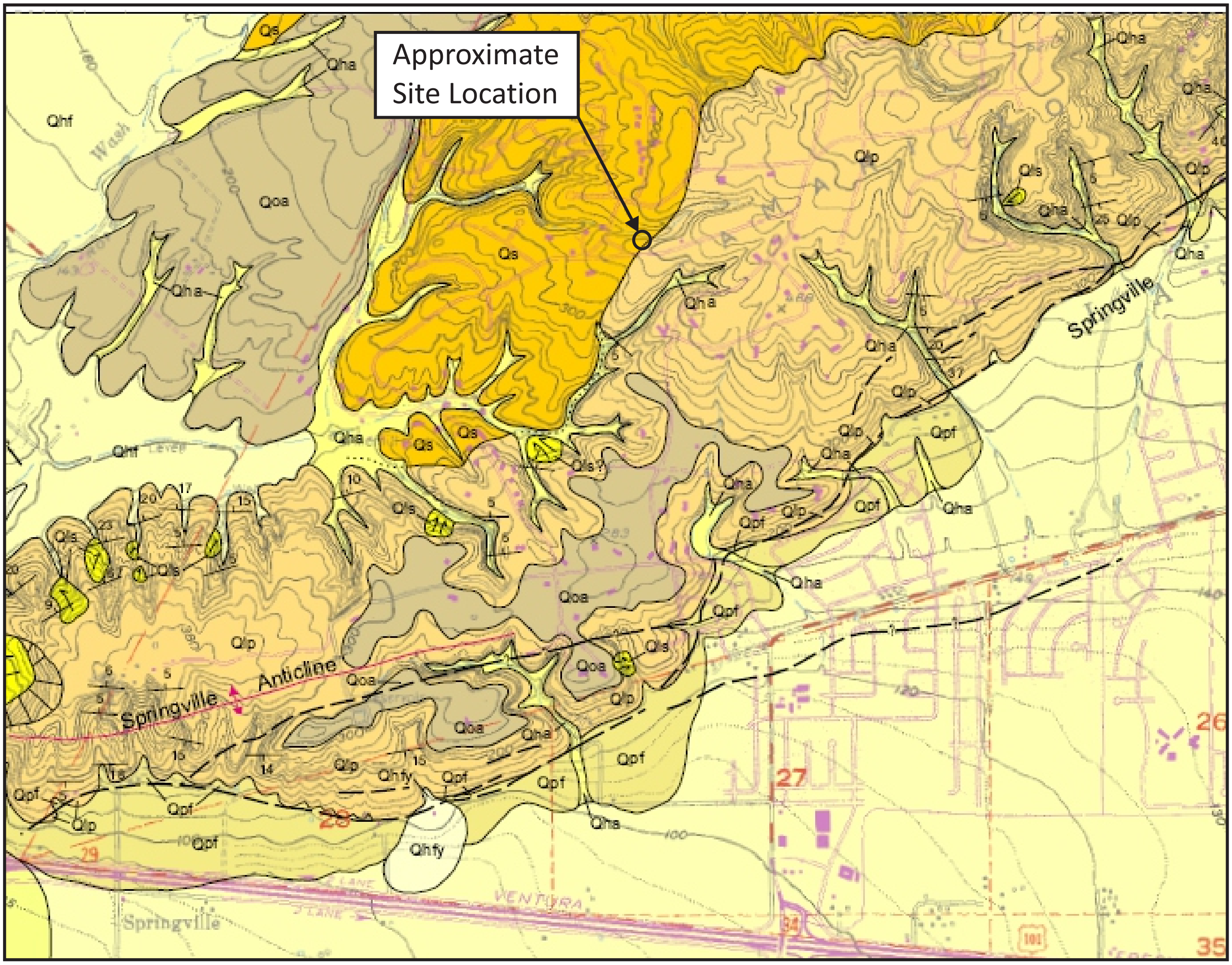
REGIONAL GEOLOGIC MAP 1

191 Alviso Drive
Camarillo Area of Ventura County, California

Earth Systems

January 2019	302698-001



*Taken from USGS, SCAMP Geologic Map of the Camarillo 7.5' Quadrangle, Ventura County, California, 2004.

Southern California USGS SCAMP CDMG Areal Mapping Project

MAP SYMBOLS

Contact between map units - Generally approximately located or inferred, dotted where concealed.

Contact between similar map units of different relative age - Recognized by scour and incised channelling features. Generally approximately located.

Fault - Generally approximately located or inferred, dotted where concealed, queried where location is uncertain.

Axis of anticline

Axis of syncline

Strike and dip of bedding.

Landslide - Arrows indicate principal direction of movement, queried where existence is questionable (some geologic features are drawn within questionable landslides); hachured where headscarp is mappable.

Qlp: Las Posas Formation (Pleistocene)

Qs: Saugus Formation (Pleistocene)

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

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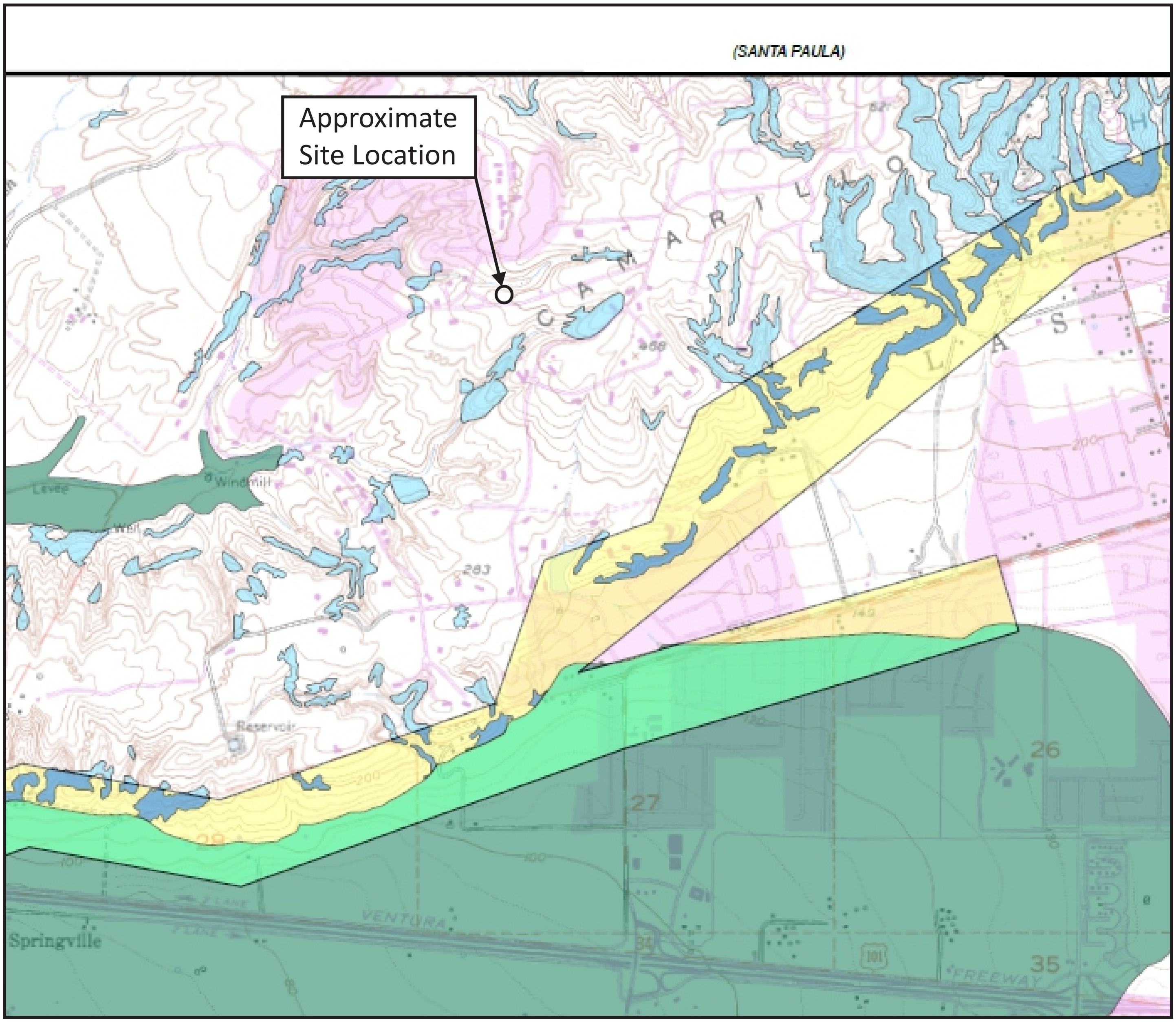
REGIONAL GEOLOGIC MAP 2

191 Alviso Drive
Camarillo Area of Ventura County, California

Earth Systems

January 2019	302698-001



MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

CAMARILLO QUADRANGLE

EARTHQUAKE FAULT ZONES	SEISMIC HAZARD ZONES
Delineated in compliance with Chapter 7.5 Division 2 of the California Public Resources Code *(Alquist-Priolo Earthquake Fault Zoning Act)*	Delineated in compliance with Chapter 7.8 Division 2 of the California Public Resources Code *(Seismic Hazards Mapping Act)*
OFFICIAL MAP	**OFFICIAL MAP**
Released: May 1, 1998	**Released: February 7, 2002**

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

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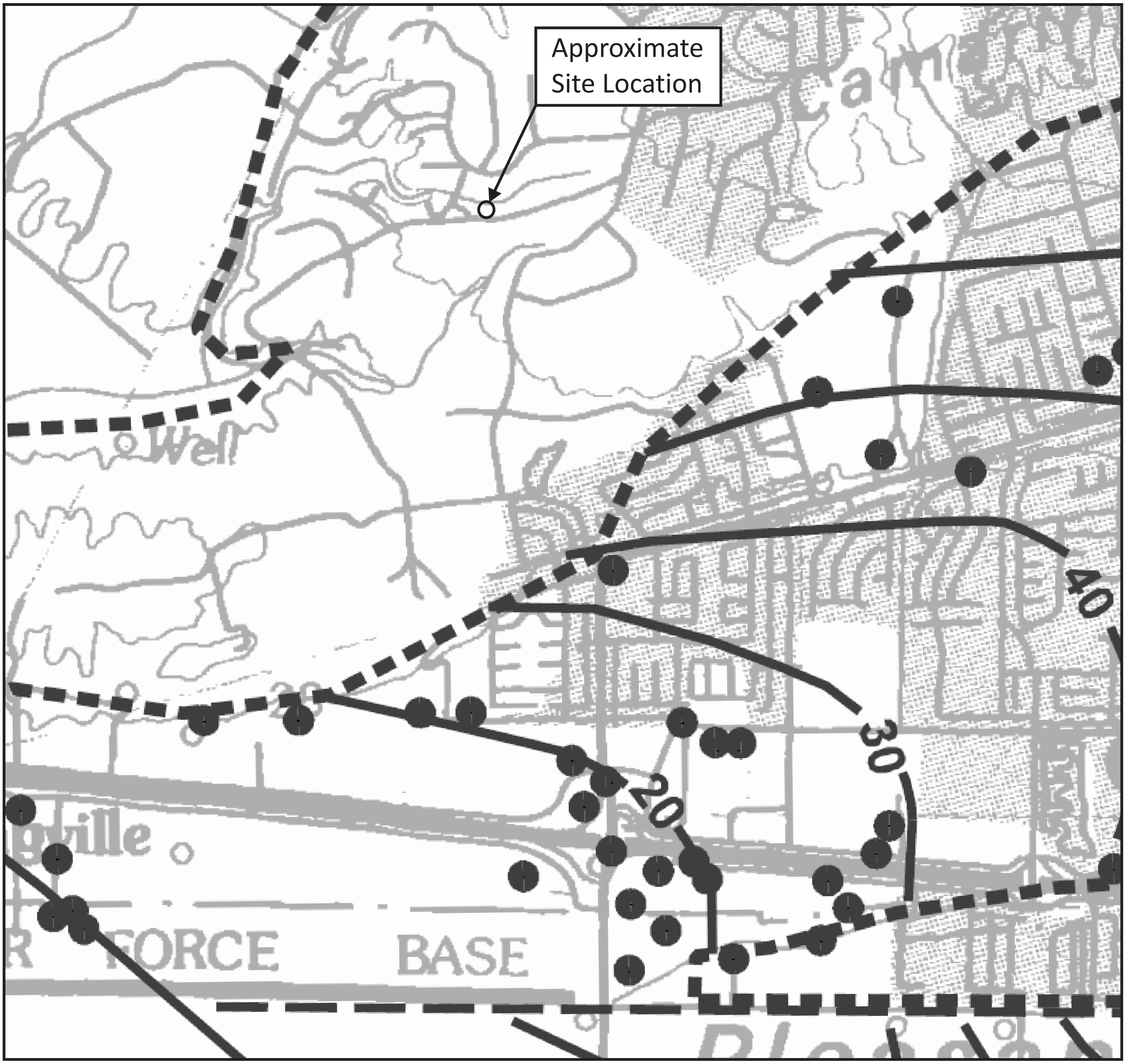
SEISMIC HAZARD ZONES MAP

191 Alviso Drive
Camarillo Area of Ventura County, California

Earth Systems

January 2019	302698-001



*Taken from CGS, Seismic Hazard Zone Report For The Camarillo 7.5-Minute Quadrangle, Ventura County, California, 2002.

30 Depth to historically highest ground water level in feet	Valley / Mountains Boundary	Groundwater Barrier	
Borehole Site	Approximate location of sand boils and lurch cracks observed following the Point Mugu Earthquake of February 21, 1973.		

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

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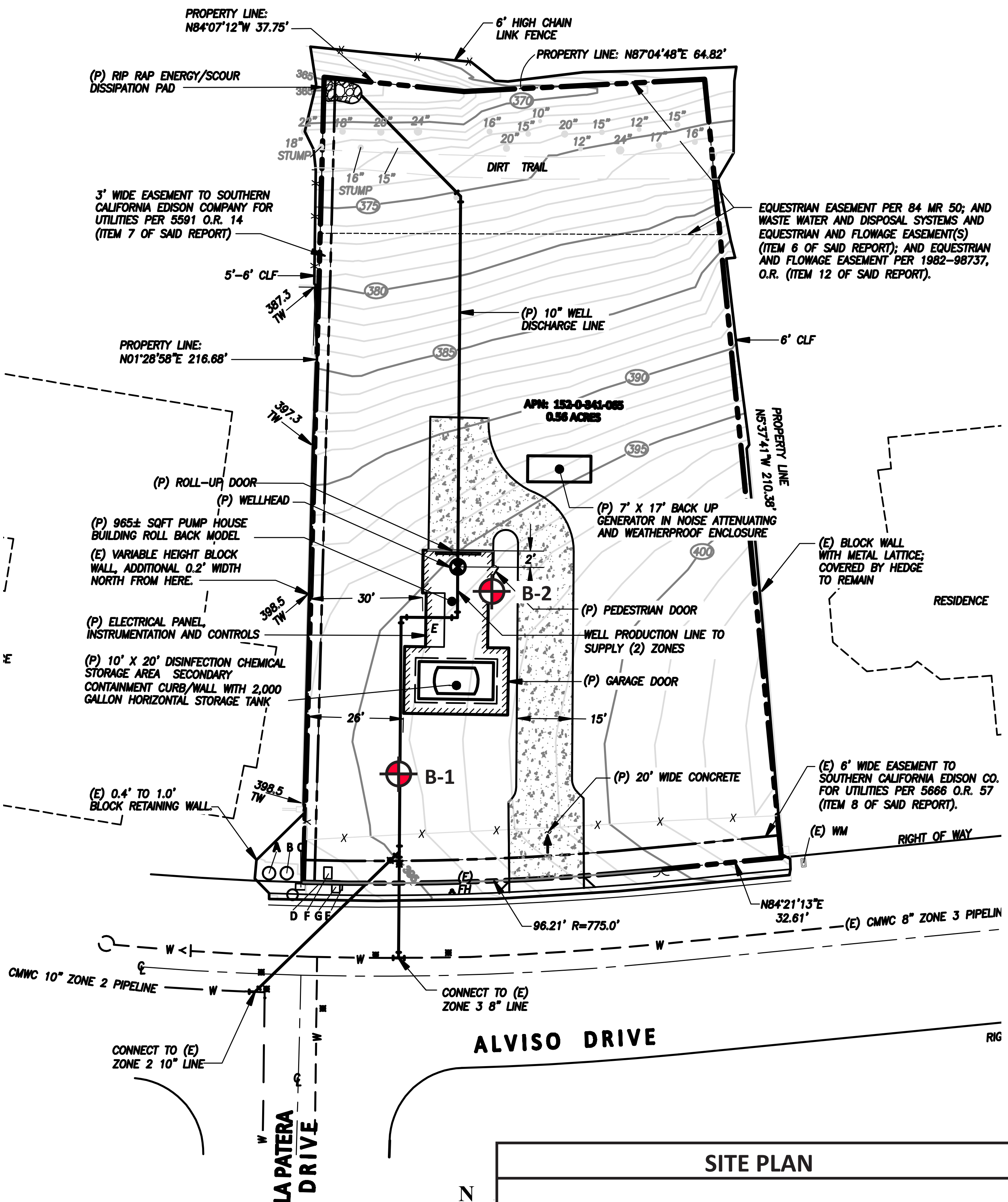
HISTORICAL HIGH GROUNDWATER MAP

191 Alviso Drive
Camarillo Area of Ventura County, California

Earth Systems

January 2019	302698-001



Approximate Boring Location

Approximate Scale: 1" = 60'

SITE PLAN

191 Alviso Drive
Camarillo Area of Ventura County, California

Earth Systems

January 2019	302698-001

FIELD STUDY

A. Two borings (B-1 and B-2) were drilled to a maximum depth of about 31.5 feet below the existing ground surface to observe the soil/bedrock profile and to obtain samples for laboratory analyses. The borings were drilled on November 15, 2018, using 6-inch diameter hollow-stem continuous flight auger powered by a Mobile Drill B-61 truck mounted drilling rig. The approximate locations of the borings were determined in the field by pacing and sighting, and are shown on the Site Plan in this Appendix.

B. Samples were obtained within the borings with a Modified California (M.C.) ring sampler (ASTM D 3550 with shoe similar to ASTM D 1586). The M.C. sampler has a 3-inch outside diameter, and a 2.42-inch inside diameter when used with brass ring liners (as it was during this study). The samples were obtained by driving the sampler with a 140-pound hammer dropping 30 inches in accordance with ASTM D 1586. The hammer was operated with an automatic trip mechanism.

C. Two bulk samples were collected from the cuttings of the soils encountered between the depths of 0 to 3 feet in Boring B-1, and 1.5 to 6 feet in Boring B-2.

D. The final logs of the borings represent interpretations of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface study. The final logs are included in this Appendix.

Logs of Borings

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-1
PROJECT NAME: 191 Alviso Drive
PROJECT NUMBER: 302698-001
BORING LOCATION: Per Plan

DRILLING DATE: November 15, 2018
DRILL RIG: Mobile Drill B-61
DRILLING METHOD: Six-Inch Hollow Stem Auger
LOGGED BY: SC

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0	X		■	9/14/27		ML	99.5	17.7	**SOIL:** Dark brown clayey silt; stiff; dry to damp.
5			■	11/23/32		QTs	107.6	8.7	**SAUGUS FORMATION:** Yellowish brown silty fine sand with abundant seashells; dense; damp.
10			■	11/18/31		Qts	100.4	16.4	**SAUGUS FORMATION:** Mottled yellowish brown silty fine sand; bedded; iron staining; dense; moist.
15			■	15/24/25		QTs	98.9	24.6	**SAUGUS FORMATION:** Interbedded yellowish brown and gray fine sandstone and siltstone; bedded; dense; moist.
20			■	14/20/21		QTs	91.2	6.8	Same as above.
25			■	12/25/35		QTs	106.4	16.6	**SAUGUS FORMATION:** Interbedded yellowish brown and gray fine sandstone and siltstone; bedded; dense; moist.
30			■	19/21/35		QTs	116.7	6.9	**SAUGUS FORMATION:** Interbedded yellowish brown and gray fine sandstone and siltstone; bedded; blocky; dense; moist.
35									Total Depth: 31.5 feet. No Groundwater Encountered.

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-2	DRILLING DATE: November 15, 2018
PROJECT NAME: 191 Alviso Drive	DRILL RIG: Mobile Drill B-61
PROJECT NUMBER: 302698-001	DRILLING METHOD: Six-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: SC

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0						ML			**SOIL:** Dark brown clayey silt; stiff; dry to damp.
	X		■	7/9/11		QTs	107.1	8.7	**SAUGUS FORMATION:** Dark yellowish brown silty sand seashell conglomerate; medium dense; damp.
5	X		■	7/14/18		QTs	95.6	7.0	**SAUGUS FORMATION:** Dark yellowish brown silty sand; bedded; medium dense; damp to moist.
10			■	7/15/24		Qts	97.8	25.0	**SAUGUS FORMATION:** Interbedded yellowish brown and gray fine sandstone and siltstone; bedded; medium dense; moist.
15			■	6/12/22		QTs	99.4	26.6	**SAUGUS FORMATION:** Interbedded yellowish brown and gray fine sandstone and siltstone; bedded; medium dense; moist.
20			■	8/14/25		QTs	99.7	23.4	Same as above.
25			■	10/15/27		QTs	98.3	27.8	**SAUGUS FORMATION:** Interbedded yellowish brown and gray fine sandstone and siltstone; bedded; blocky; iron staining; medium dense; moist.
30			■	19/21/35		QTs	114.9	14.7	Same as above; dense.
35									Total Depth: 31.5 feet. No Groundwater Encountered.

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

BORING LOG SYMBOLS

- Modified California Split Barrel Sampler
- Modified California Split Barrel Sampler - No Recovery
- Standard Penetration Test (SPT) Sampler
- Standard Penetration Test (SPT) Sampler - No Recovery
- Perched Water Level
- Water Level First Encountered
- Water Level After Drilling
- Pocket Penetrometer (tsf)
- Vane Shear (ksf)

1. The location of borings were approximately determined by pacing and/or siting from visible features. Elevations of borings are approximately determined by interpolating between plan contours. The location and elevation of the borings should be considered.

2. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

3. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. This data has been reviewed and interpretations made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, tides, temperature, and other factors at the time measurements were made.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			*GRAPH SYMBOL*	*LETTER SYMBOL*	*TYPICAL DESCRIPTIONS*
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNTOF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENT

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

Earth Systems

APPENDIX B

Laboratory Testing

Tabulated Laboratory Test Results

Individual Laboratory Test Results

LABORATORY TESTING

A. Samples were reviewed along with field logs to determine which would be analyzed further. Those chosen for laboratory analyses were considered representative of soils that would be exposed and/or used during grading, and those deemed to be within the influence of proposed structures. Test results are presented in graphic and tabular form in this Appendix.

B. In-situ moisture content and dry unit weight for the ring samples were determined in general accordance with ASTM D 2937.

C. A maximum density test was performed to estimate the moisture-density relationship of typical soil materials. The test was performed in accordance with ASTM D 1557.

D. The relative strength characteristics of soils were determined from the results of a direct shear test on a remolded sample. The specimen was placed in contact with water at least 24 hours before testing, and was then sheared under normal loads ranging from 1 to 3 ksf in general accordance with ASTM D 3080.

E. An expansion index test was performed on a bulk soil sample in accordance with ASTM D 4829. The sample was surcharged under 144 pounds per square foot at moisture content of near 50 percent saturation. The sample was then submerged in water for 24 hours, and the amount of expansion was recorded with a dial indicator.

F. A portion of the bulk sample was sent to another laboratory for analyses of soil pH, resistivity, chloride contents, and sulfate contents. Soluble chloride and sulfate contents were determined on a dry weight basis. Resistivity testing was performed in accordance with California Test Method 424, wherein the ratio of soil to water was 1:3.

TABULATED LABORATORY TEST RESULTS

REMOLDED SAMPLE

BORING AND DEPTH	B-1@0'-3'
USCS	ML
MAXIMUM DRY DENSITY (pcf)	109.5
OPTIMUM MOISTURE (%)	14
PEAK COHESION (psf)	470
PEAK FRICTION ANGLE	23°
ULTIMATE COHESION (psf)	110
ULTIMATE FRICTION ANGLE	28°
EXPANSION INDEX	102
pH	7.9
RESISTIVITY (ohms-cm)	6,700
SOLUBLE CHLORIDES (mg/Kg)	8.2
SOLUBLE SULFATES (mg/Kg)	15

Individual Laboratory Test Results

File Number: 302698-001 Lab Number: 098035

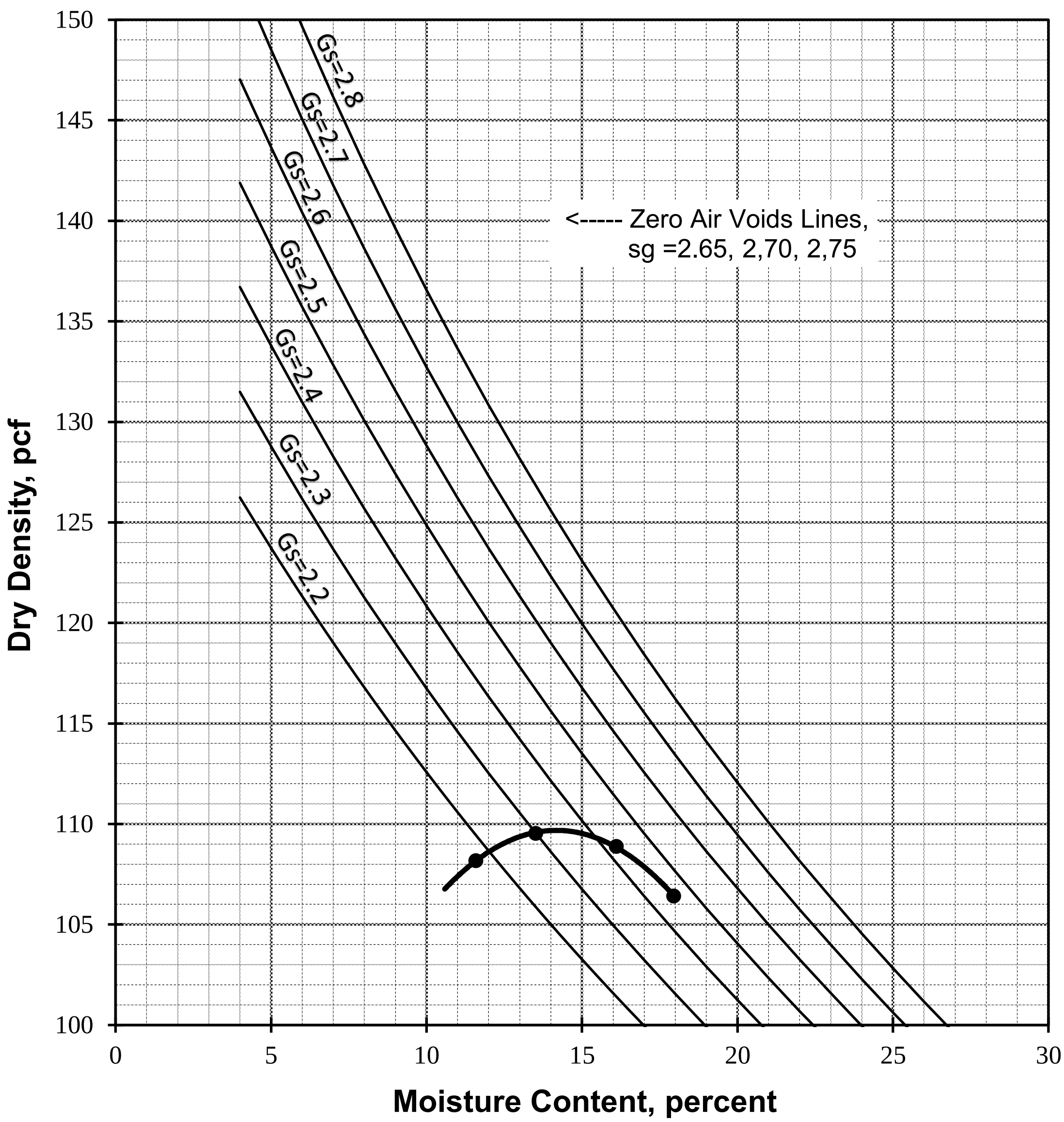
MAXIMUM DENSITY / OPTIMUM MOISTURE ASTM D 1557-12 (Modified)

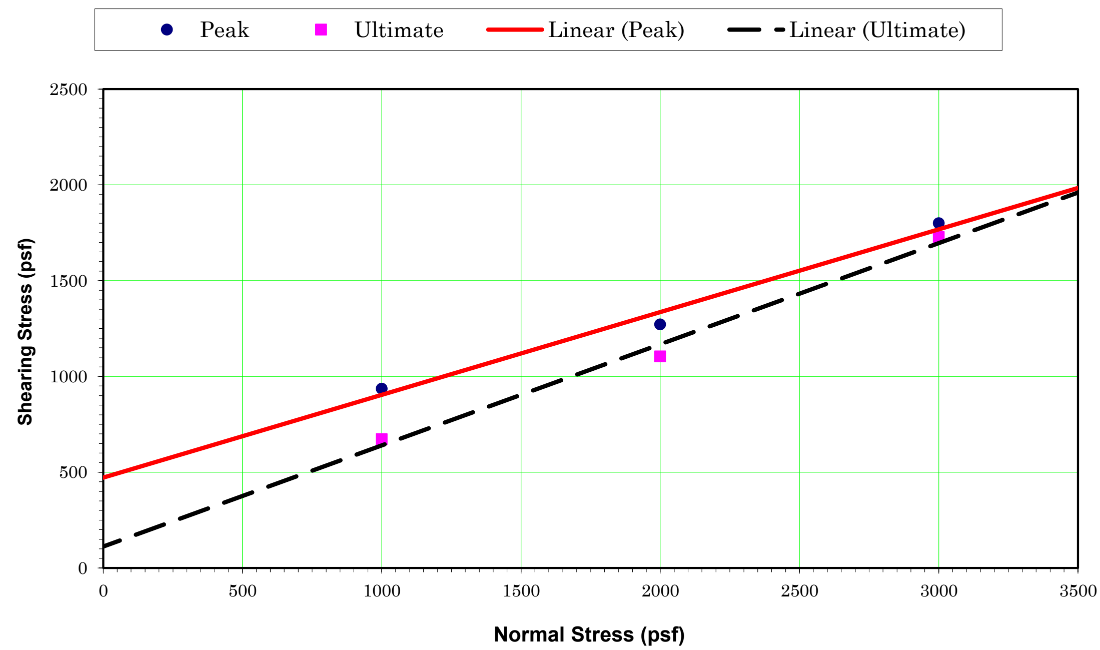
Job Name: 191 Alviso Drive
Sample ID: B 1 @ 0-3'
Date: 12/26/2018
Description: Dark Brown Clayey Silt
SG: 2.34

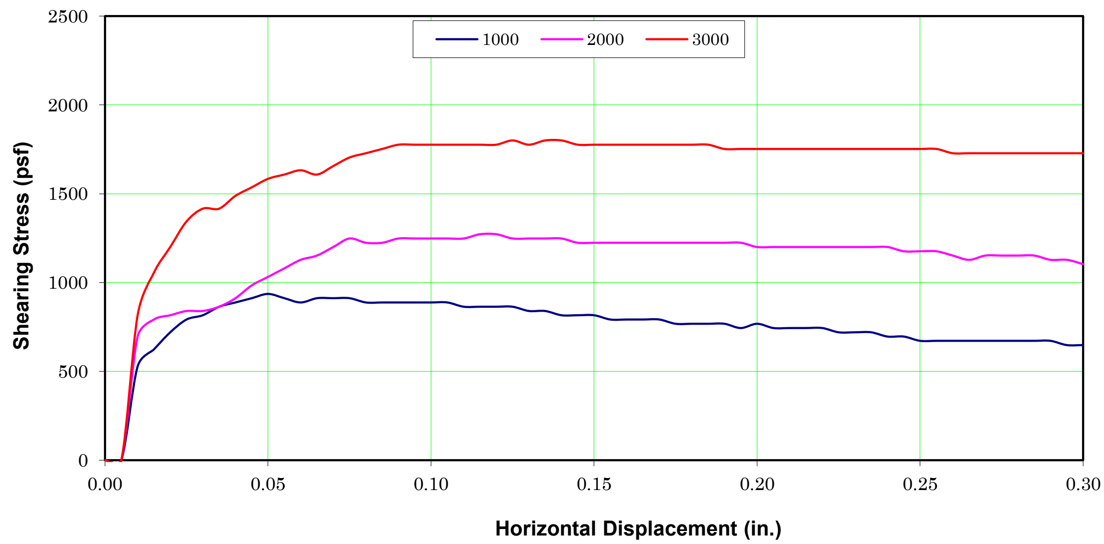
Procedure Used: A
Prep. Method: Moist
Rammer Type: Automatic

Maximum Density: **109.5 pcf**
Optimum Moisture: **14%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	2.6







DIRECT SHEAR DATA*

Sample Location: B 1 @ 0-3'
Sample Description: Clayey Silt
Dry Density (pcf): 98.5
Intial % Moisture: 14.2
Average Degree of Saturation: 100.0
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	936	1272	1800
Ultimate stress (psf)	672	1104	1728

	Peak	Ultimate
ϕ Angle of Friction (degrees):	23	28
c Cohesive Strength (psf):	470	110

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

191 Alviso Drive

Earth Systems

12/29/2019 | 302698-001

File No.: 302698-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

Job Name: 191 Alviso Drive
Sample ID: B 1 @ 0-3'
Soil Description: ML

Initial Moisture, %:	11.1
Initial Compacted Dry Density, pcf:	105.1
Initial Saturation, %:	50
Final Moisture, %:	33.7
Volumetric Swell, %:	10.2

Expansion Index: 102 High

EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

CAPCO Analytical Services, Inc.

Environmental and Analytical Services-Since 1994
California State Accredited Laboratory in Accordance with ELAP Certificate # 2332

Prepared for: Earth Systems Pacific
1731 A Walter Street
Ventura, CA 93003
Attn: Todd Tranby

Report Date: December 13, 2018
Laboratory Number: 182202
Project Name: 191 Alviso Drive
Project No: 302698-001
Sampled by: Client

Enclosed are the analysis results for samples received December 5, 2018 with the Chain of Custody document. The samples were received in good condition, at 16.2°C, and it was identified and assigned the laboratory ID number listed below:

SAMPLE DESCRIPTION	CAS LAB NUMBER ID
B-1@0-3'	182202-01

By my signature below, I certify that the results contained in this laboratory report comply with applicable standards for certification by the California Department of Public Health's Environmental Laboratories Accreditation Program (ELAP), both technically and for completeness, and that, based on my inquiry of the person or persons directly responsible for performing the analyses, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete.

Lance Lewy-Laboratory Director

If you have any further questions or concerns, please contact me at your convenience. This report consists of 2 pages excluding the cover letter and the Chain of Custody.

This report shall not be reproduced except in full without the written approval of CAS. The test results reported represent only the item being tested and may not represent the entire material from which the sample was taken.

CAPCO Analytical Services, Inc.

Environmental and Analytical Services-Since 1994
California State Accredited Laboratory in Accordance with ELAP Certificate # 2332

CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 182202-01
Sample ID: B-1@0-3'
Analyst: GP

Date Sampled: 12/04/18
Date Received: 12/05/18
Sample Matrix: Soil

WET CHEMISTRY ANALYSIS SUMMARY

COMPOUND	RESULTS	UNITS	DF	PQL	METHOD	ANALYZED
pH (Corrosivity)	7.9	S.U.	1	---	9045	12/06/18
Resistivity*	6700	Ohms-cm	1	---	SM 120.1M	12/06/18
Chloride	8.2	mg/Kg	1	0.6	300.0M	12/06/18
Sulfate	15	mg/Kg	1	0.6	300.0M	12/06/18

*Sample was extracted using a 1:3 ratio of soil and DI water.

DF: Dilution Factor
PQL: Practical Quantitation Limit
BQL: Below Quantitation Limit
mg/Kg: Milligrams/Kilograms(ppm)

CAPCO Analytical Services, Inc.

Environmental and Analytical Services-Since 1994

Quality Control Report

Client:	EARTH SYSTEMS	Date Sampled:	12/04/18
Sample ID:		Date Received:	12/05/18
CAS LAB NO:	182202	Date Analyzed:	12/06/18
Sample Matrix:	SOIL	Analyst:	GP

Sample Name	Qualifier	Sample Result	QC Result	Unit	Spike Level	%REC	Control Limits
	Chloride (by EPA 300)						
Method Blank			**BQL**	mg/L			
Lab Control Sample			**28.68**	mg/L	30	96	90-110
181206 Blank Spike		0.00	**28.65**	mg/L	30	96	80-120
181206 Blank Spike Duplicate		0.00	**28.73**	mg/L	30	96	80-120
	Sulfate (by EPA 300)						
Method Blank			**BQL**	mg/L			
Lab Control Sample			**28.83**	mg/L	30	96	90-110
181206 Blank Spike		0.00	**28.76**	mg/L	30	96	80-120
181206 Blank Spike Duplicate		0.00	**28.90**	mg/L	30	96	80-120

*ALL QC SAMPLES ARE PREPARED IN LIQUID PHASE

mg/L:Milligrams/Liter(ppm)

%Rec:Percent Recovered

BQL:Below Practical Quantitation Limit

APPENDIX C

Table 1809.7 Minimum Foundation Design Table

TABLE 1809.7
PRESCRIPTIVE FOOTINGS FOR SUPPORTING WALLS OF LIGHT FRAME CONSTRUCTION*

WEIGHTED EXPANSION INDEX (13)	FOUNDATION FOR SLAB & RAISED FLOOR SYSTEM (4) (8)							CONCRETE SLABS (8) (12)		PREMOISTENING OF SOILS UNDER FOOTINGS, PIERS AND SLABS (4) (5)	RESTRICTION ON PIERS UNDER RAISED FLOORS
	NUMBER OF STORIES	STEM THICKNESS	FOOTING WIDTH	FOOTING THICKNESS	ALL PERIMETER FOOTINGS (5)	INTERIOR FOOTINGS FOR SLAB AND RAISED FLOORS (5)	REINFORCEMENT FOR CONTINUOUS FOUNDATIONS (2) (6)	3-1/2" MINIMUM THICKNESS			
					DEPTH BELOW NATURAL SURFACE OF GROUND AND FINISH GRADE			REINFORCEMENT (3)	TOTAL THICKNESS OF SAND (10)		
		(INCHES)									
0 - 20 Very Low (non-expansive)	1 2 3	6 8 10	12 15 18	6 6 8	12 18 24	12 18 24	1-#4 top and bottom	#4 @ 48" o.c. each way, or #3 @ 36" o.c. each way	2"	Moistening of ground recommended prior to placing concrete	Piers allowed for single floor loads only
21-50 Low	1 2 3	6 8 10	12 15 18	6 6 8	15 18 24	12 18 24	1-#4 top and bottom	#4 @ 48" o.c. each way, or #3 @ 36" o.c. each way	4"	120% of optimum moisture required to a depth of 21" below lowest adjacent grade. Testing required.	Piers allowed for single floor loads only
51-90 Medium	1 2	6 8	12 15	6 6	21 21	12 18	1-#4 top and bottom	#3 @ 24" o.c. each way	4"	130% of optimum moisture required to a depth of 27" below lowest adjacent grade. Testing required	Piers not allowed
	3	10	18	8	24	24	#3 bars @ 24" in ext. footing Bend 3' into slab (7)				
91-130 High	1 2	6 8	12 15	6 6	27 27	12 18	2-#4 Top and Bottom	#3 @ 24" o.c. each way	4"	140% of optimum moisture required to a depth of 33" below lowest adjacent grade. Testing required.	Piers not allowed
	3	10	18	8	27	24	#3 bars @ 24" in ext. footing Bend 3' into slab (7)				
Above 130 Very High	Special design by licensed engineer/architect										

*Refer to next page for footnotes (1) through (14).

FOOTNOTES TO TABLE 1809.7

1. Premoistening is required where specified in Table 1809.7 in order to achieve maximum and uniform expansion of the soil prior to construction and thus limit structural distress caused by uneven expansion and shrinkage. Other systems which do not include premoistening may be approved by the Building Official when such alternatives are shown to provide equivalent safeguards against the adverse effects of expansive soil.

2. Reinforcement for continuous foundations shall be placed not less than 3" above the bottom of the footing and not less than 3" below the top of the stem.

3. Reinforcement shall be placed at mid-depth of slab.

4. After premoistening, the specified moisture content of soils shall be maintained until concrete is placed. Required moisture content shall be verified by an approved testing laboratory not more than 24 hours prior to placement of concrete.

5. Crawl spaces under raised floors need not be pre-moistened except under interior footings. Interior footings which are not enclosed by a continuous perimeter foundation system or equivalent concrete or masonry moisture barrier complying with Footnote # 12 of Table 1809.7 shall be designed and constructed as specified for perimeter footings in Table 1809.7.

6. Foundation stem walls which exceed a height of three times the stem thickness above lowest adjacent grade shall be reinforced in accordance with Chapter 21 and Section 1914 in the IBC, or as required by engineering design, whichever is more restrictive.

7. Bent reinforcing bars between exterior footing and slab shall be omitted when floor is designed as an independent, "floating' slab.

8. Where frost conditions or unusual conditions beyond the scope of this table are found, design shall be in accordance with recommendations of a foundation investigation. Concrete slabs shall have a minimum thickness of 4 inches when the expansion index exceeds 50.

9. The ground under a raised floor system may be excavated to the elevation of the top of the perimeter footing, except where otherwise required by engineering design or to mitigate groundwater conditions.

10. GRADE BEAM, GARAGE OPENING. A grade beam not less than 12" x 12" in cross section, or 12" x depth required by Table 1809.7, whichever is deeper, reinforced as specified for continuous foundations in Table 1809.7, shall be provided at garage door openings..

11. Where a post-tensioning slab system is used, the width and depth of the perimeter footings shall meet the requirements of this table.

12. An approved vapor barrier shall be installed below concrete slab-on-grade floors of all residential occupancies in such a manner as to form an effective barrier against the migration of moisture into the slab. When sheet plastic material is employed for this purpose it shall be not less than 6 mils (.006 inch) in thickness. The installation of a vapor barrier shall not impair the effectiveness of required anchor bolts or other structural parts of a building. Foundations at the perimeter of concrete floor slabs shall form a continuous moisture barrier of Portland cement concrete or solid grouted masonry to the depths required by Table 1809.7.

13. When buildings are located on expansive soil having an expansion index greater than 50, gutters, downspouts, piping, and/or other non-erosive devices shall be provided to collect and conduct rainwater to a street, storm drain, or other approved watercourse or disposal area.

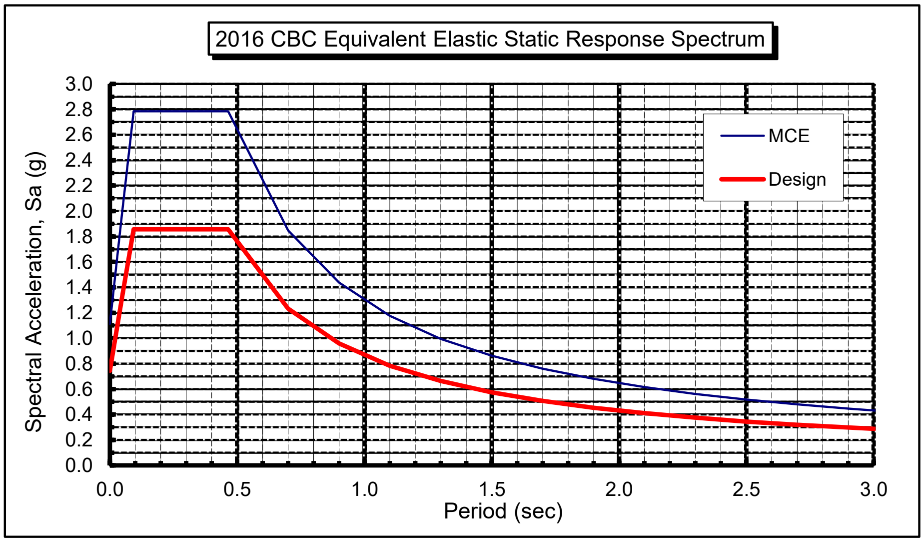
14. Fireplace footings shall be reinforced with a horizontal grid located 3" above the bottom of the footing and consisting of not less than No. 4 Bars at 12" on center each way. Vertical chimney reinforcing bars shall be hooked under the grid. Depth of fireplace chimney footings shall be no less than that required by Table 1809.7.

APPENDIX D

2016 CBC & ASCE 7-10 Seismic Parameters
USGS Design Maps Reports
Fault Parameters

2016 California Building Code (CBC) (ASCE 7-10) Seismic Design Parameters

				CBC Reference	ASCE 7-10 Reference
Seismic Design Category		**E**		Table 1613.5.6	Table 11.6-2
Site Class		**C**		Table 1613.5.2	Table 20.3-1
Latitude:		34.242	N		
Longitude:		-119.075	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Reponse	$\mathbf{S_S}$	**2.785**	**g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	$\mathbf{S_1}$	**0.995**	**g**	Figure 1613.5	Figure 22.4
Site Coefficient	F_a	1.00		Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	1.30		Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	2.785	g	$= F_a*S_S$	
	S_{M1}	1.294	g	$= F_v*S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	$\mathbf{S_{DS}}$	**1.857**	**g**	$= 2/3*S_{MS}$	
1 second Spectral Response	$\mathbf{S_{D1}}$	**0.862**	**g**	$= 2/3*S_{M1}$	
	To	0.09	sec	$= 0.2*S_{D1}/S_{DS}$	
	Ts	0.46	sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.00		Table 1604.5	Table 11.5-1
	F_{PGA}	1.00			

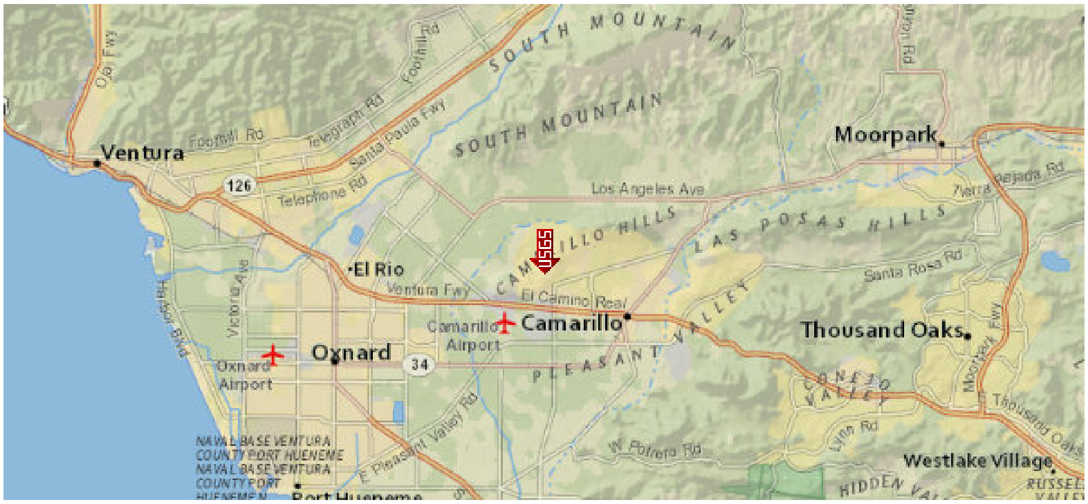


Period T (sec)	Design Sa (g)
0.00	0.743
0.05	1.342
0.09	1.857
0.46	1.857
0.70	1.232
0.90	0.958
1.10	0.784
1.30	0.663
1.50	0.575
1.70	0.507
1.90	0.454
2.10	0.411
2.30	0.375
2.50	0.345
2.70	0.319
2.90	0.297

Design Maps Summary Report

User–Specified Input

Report Title	191 Alviso Drive Tue November 6, 2018 22:32:52 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.2424°N, 119.0749°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

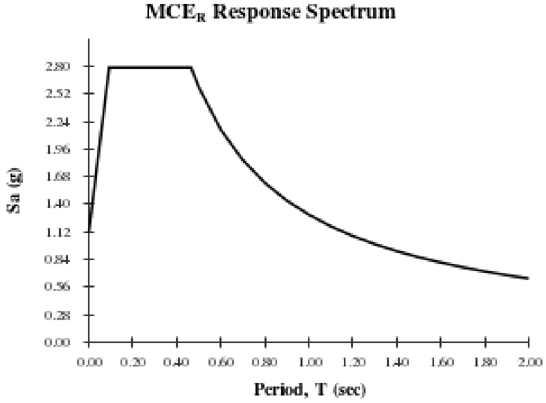


USGS–Provided Output

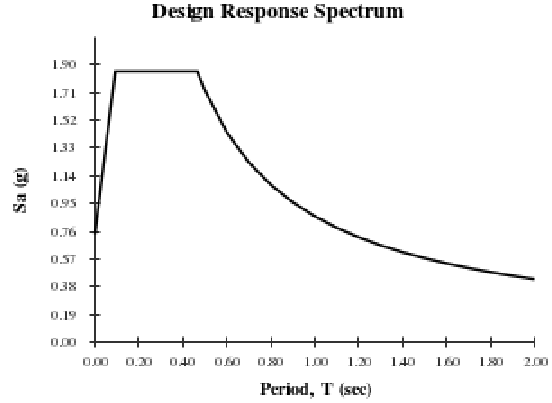
$S_S =$ 2.785 g	$S_{MS} =$ 2.785 g	$S_{DS} =$ 1.856 g
$S_1 =$ 0.995 g	$S_{M1} =$ 1.294 g	$S_{D1} =$ 0.862 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.2424°N, 119.0749°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 2.785 g
From Figure 22-2 [2]	S_1 = 0.995 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = C and S_s = 2.785 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.995 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.785 = 2.785$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 0.995 = 1.294$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

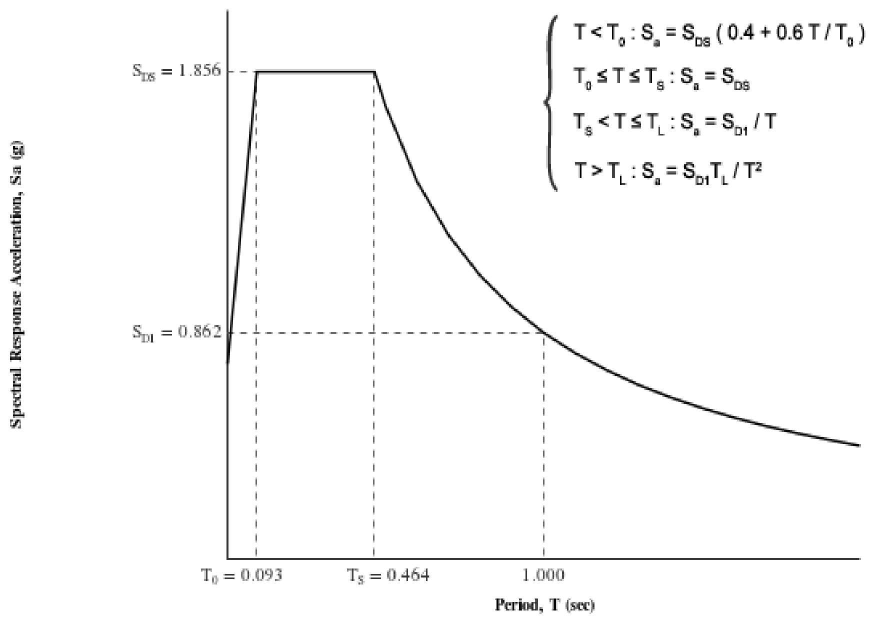
Equation (11.4–3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 2.785 = 1.856$ g

Equation (11.4–4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 1.294 = 0.862$ g

Section 11.4.5 — Design Response Spectrum

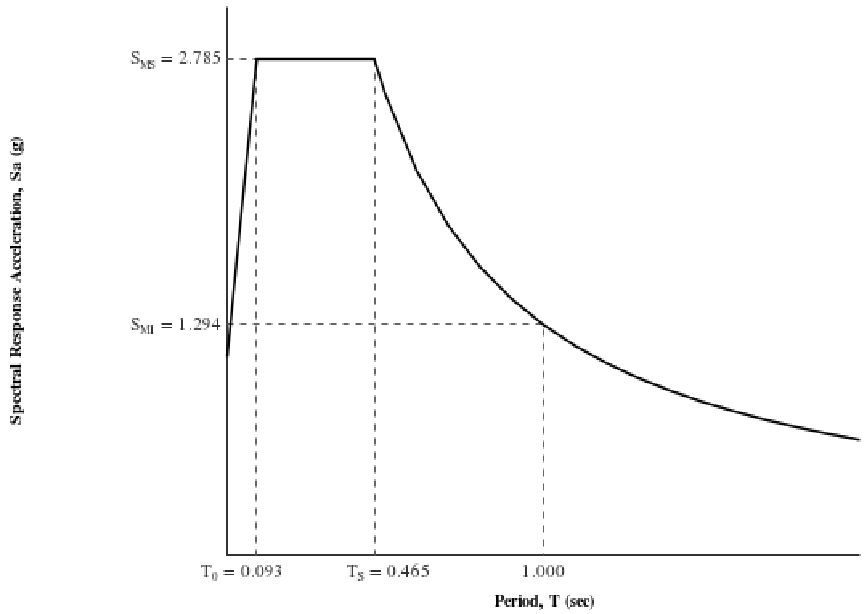
From Figure 22-12 [3] T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 1.105

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 1.105 = 1.105$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = C and PGA = 1.105 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.902

From Figure 22-18 [6] C_{R1} = 0.908

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.856 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.862 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

Table 1
Fault Parameters

Fault Section Name	**Distance** (miles)	(km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
Simi-Santa Rosa	0.9	1.4	60	346	30	39	B	**6.8**		1
Oak Ridge (Onshore)	4.5	7.3	65	159	90	49	B	**7.2**		4
Ventura-Pitas Point	7.9	12.8	64	353	60	44	B	**6.9**		1
Oak Ridge (Offshore)	11.3	18.2	32	180	90	38	B	**6.9**		3
Malibu Coast (Extension), alt 1	12.7	20.4	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	12.7	20.4	74	4	30	35	B'	**6.9**		
San Cayetano	12.7	20.4	42	3	90	42	B	**7.2**		6
Sisar	13.2	21.2	29	168	na	20	B'	**7.0**		
Red Mountain	14.7	23.6	56	2	90	101	B	**7.4**		2
Malibu Coast, alt 1	15.8	25.4	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	15.8	25.4	74	3	30	38	B	**6.9**		0.3
Mission Ridge-Arroyo Parida-Santa Ana	16.1	25.9	70	176	90	69	B	**6.8**		0.4
Channel Islands Thrust	18.4	29.6	20	354	90	59	B	**7.3**		1.5
Anacapa-Dume, alt 1	18.7	30.0	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	18.7	30.0	41	352	60	65	B	**7.2**		3
Santa Susana, alt 1	19.3	31.1	55	9	90	27	B	**6.8**		5
Santa Susana, alt 2	19.5	31.4	53	10	90	43	B'	**6.8**		
Santa Ynez (East)	20.6	33.1	70	172	0	68	B	**7.2**		2
North Channel	20.7	33.3	26	10	90	51	B	**6.7**		1
Santa Cruz Island	20.8	33.5	90	188	30	69	B	**7.1**		1
Northridge Hills	21.4	34.5	31	19	90	25	B'	**7.0**		
Channel Islands Western Deep Ramp	22.4	36.1	21	204	90	62	B'	**7.3**		
Del Valle	22.4	36.1	73	195	90	9	B'	**6.3**		
Holser, alt 1	22.8	36.7	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	22.8	36.7	58	182	90	17	B'	**6.7**		
Shelf (Projection)	23.1	37.2	17	21	na	70	B'	**7.8**		
Pine Mtn	23.2	37.3	45	5	na	62	B'	**7.3**		
Pitas Point (Lower)-Montalvo	23.5	37.8	16	359	90	30	B	**7.3**		2.5
Northridge	24.0	38.6	35	201	90	33	B	**6.8**		1.5
San Pedro Basin	26.2	42.1	88	51	na	69	B'	**7.0**		
Santa Monica Bay	27.0	43.5	20	44	na	17	B'	**7.0**		
Santa Cruz Catalina Ridge	29.3	47.2	90	38	na	137	B'	**7.3**		
Pitas Point (Upper)	29.6	47.6	42	15	90	35	B	**6.8**		1
Compton	30.6	49.3	20	34	90	65	B'	**7.5**		
San Gabriel	30.8	49.6	61	39	180	71	B	**7.3**		1
Big Pine (Central)	32.4	52.2	76	167	na	23	B'	**6.3**		
Santa Monica, alt 1	33.1	53.3	75	343	30	14	B	**6.5**		1
San Pedro Escarpment	33.4	53.7	17	38	na	27	B'	**7.3**		
Santa Monica, alt 2	33.6	54.1	50	338	30	28	B	**6.7**		1
Sierra Madre (San Fernando)	34.3	55.2	45	9	90	18	B	**6.6**		2

Reference: USGS OFR 2007-1437 (CGS SP 203) Based on Site Coordinates of 34.2424 Latitude, -119.0749 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2007-1437). Mean magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

REMARKS Well #3 Pg#1

Changes in Mud Type or Additional Samples				
Date | Sample No.				
Depth—Driller				
Type Fluid in Hole				
Dens. | Visc.				
ph | Fluid Loss		ml		
Source of Sample				
R_m @ Meas. Temp.	@	°F	@	°F
R_{mf} @ Meas. Temp.	@	°F	@	°F
R_{mc} @ Meas. Temp.	@	°F	@	°F
Source: R_{mf} | R_{mc}				
R_m @ BHT	@	°F	@	°F
R_{mf} @ BHT	@	°F	@	°F
R_{mc} @ BHT	@	°F	@	°F

Scale Changes

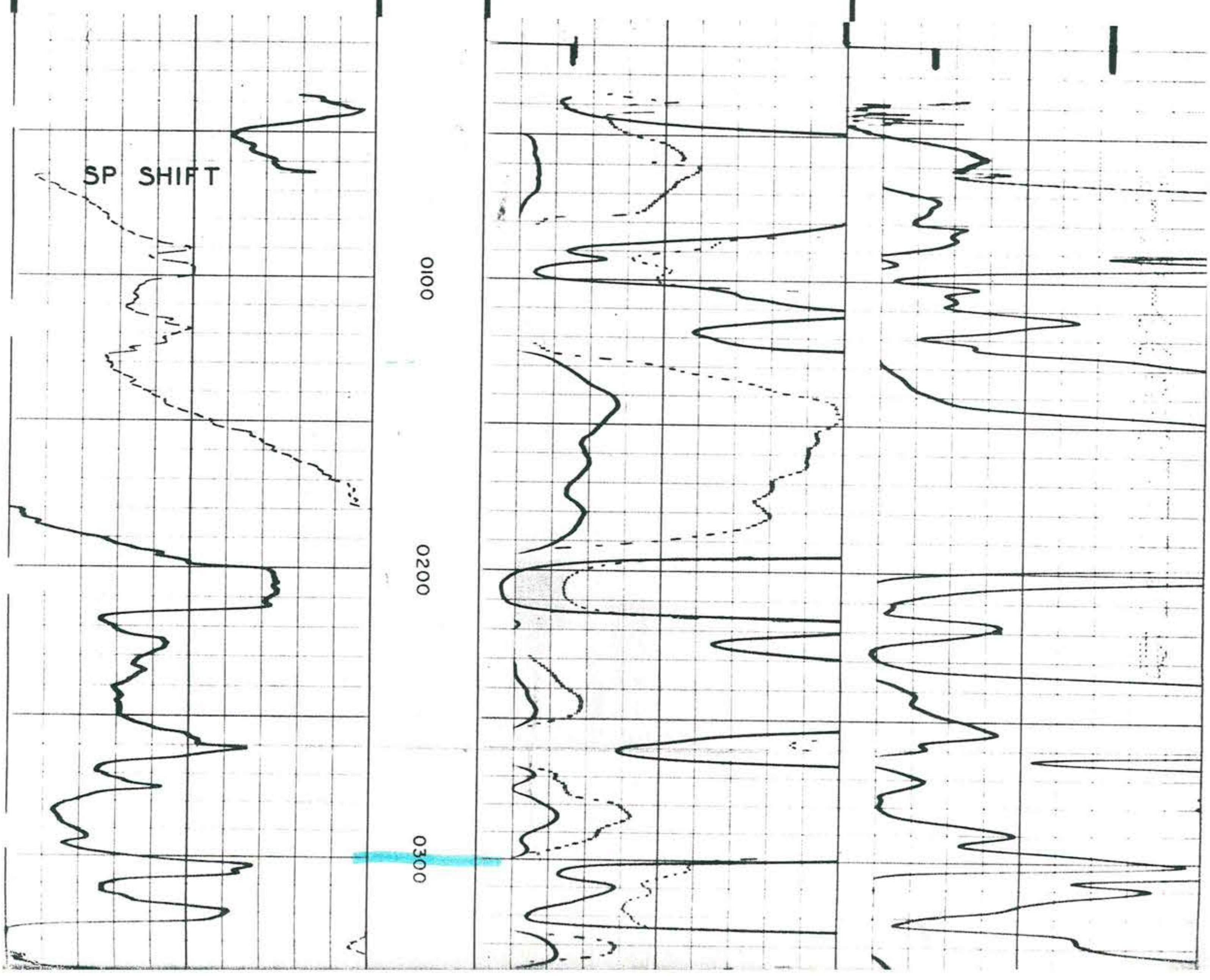
Type Log	Depth	Scale Up Hole	Scale Down Hole

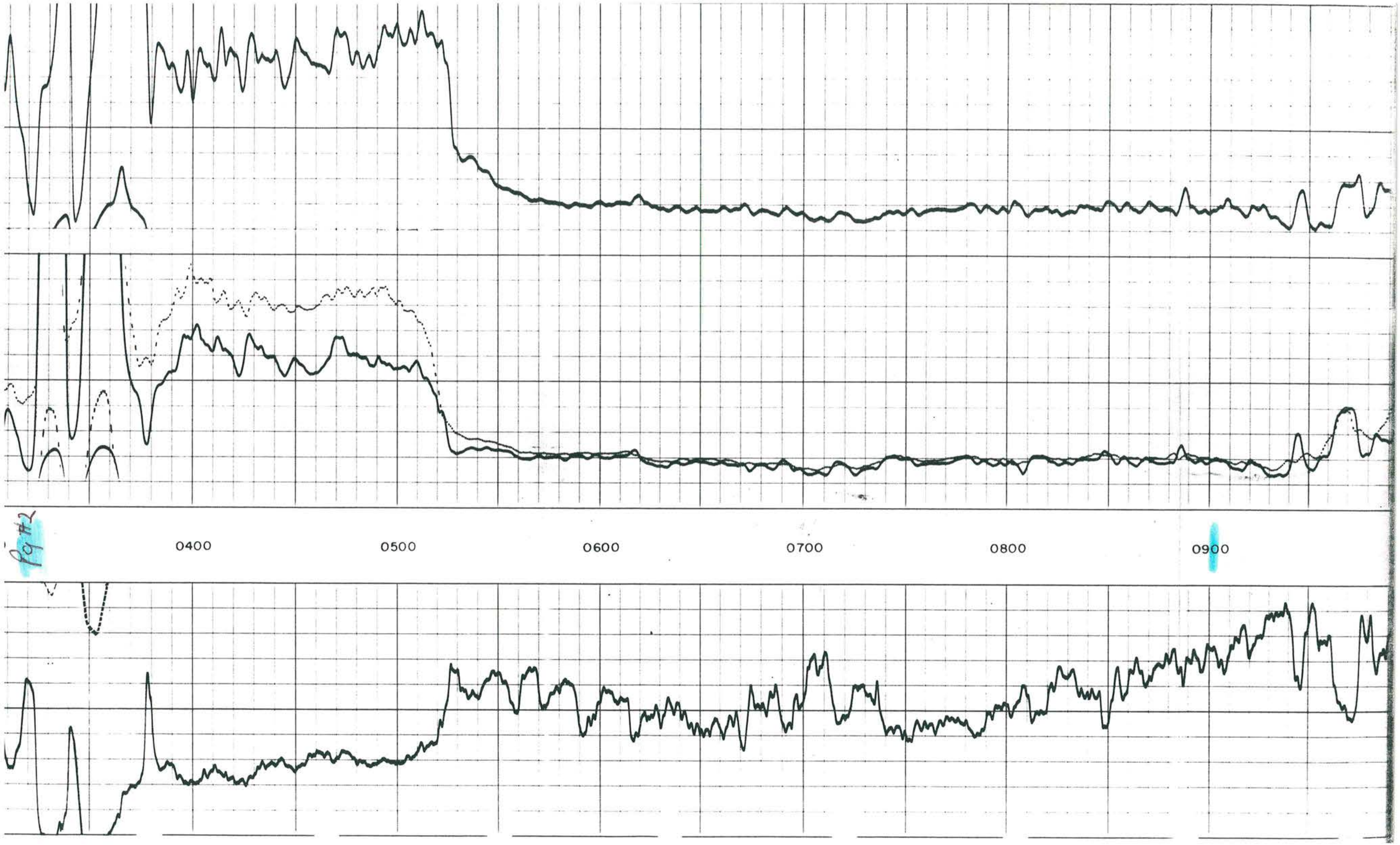
Equipment Data

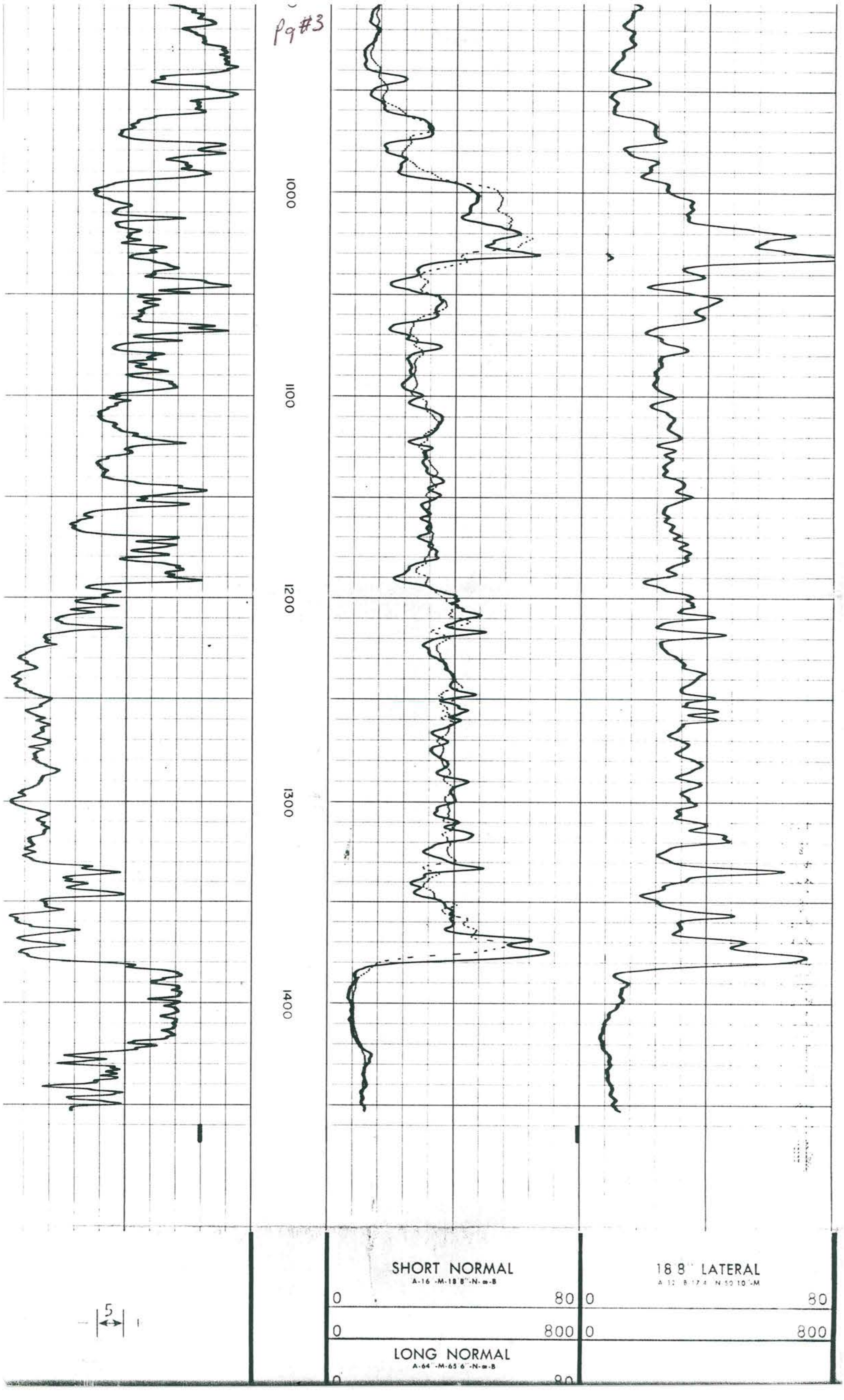
Run No.	Tool Type	Pad Type	Tool Pos.	Other
ONE	S-54	-	FREE	-

BIT SIZE: 13¾ X 12¼ @1012

SPONTANEOUS POTENTIAL millivolts	Depths	RESISTIVITY ohms. m²/m	RESISTIVITY ohms. m²/m
5 – |⟷| +		SHORT NORMAL A-16"-M-18'8"-N-∞-B 0 — 80 0 — 800 LONG NORMAL A-64"-M-65'6"-N-∞-B 0 — 80 0 — 800	18'8" LATERAL A-32"-B-17'4"-N-50'10"-M 0 — 80 0 — 800




Pg #2
0400
0500
0600
0700
0800
0900


Pg #3
1000
1100
1200
1300
1400
SHORT NORMAL
0 80
0 800
LONG NORMAL
18 8" LATERAL
0 80
0 800
5

ORIGINAL
File with DWR

No 3

WATER WELL DRILLERS REPORT
(Sections 7076, 7077, 7078, 7079, 7080, Water Code)

THE RESOURCES AGENCY OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES

Do Not Fill In
No 32472
State Well No.
Other Well No. 22E1

(1) OWNER:
Name Crestview Water Company
Address P. O. Box 369
Oxnard, Calif

(2) LOCATION OF WELL:
County Ventura Owner's number, if any
Township, Range, and Section
Distance from cities, roads, railroads, etc.

(3) TYPE OF WORK (check):
New Well ☐ Deepening ☐ Reconditioning ☐ Destroying ☐
If destruction, describe material and procedure in Item 11.

(4) PROPOSED USE (check):
Domestic ☐ Industrial ☐ Municipal ☐
Irrigation ☐ Test Well ☐ Other ☒

(5) EQUIPMENT:
Rotary ☒
Cable ☐
Other ☐

(6) CASING INSTALLED:
STEEL: SINGLE ☒ DOUBLE ☐ OTHER:

From ft.	To ft.	Diam.	Gage or Wall	Diameter of Bore	If gravel packed From ft.	To ft.
0	594	16	5/16	30	75	1393
594	1393	14	5/16			

Size of shoe or well ring: Size of gravel: 5
Describe joint welded

(7) PERFORATIONS OR SCREEN:
Type of perforation or name of screen

From ft.	To ft.	Perf. per row	Rows per ft.	Size in. x in.
1000	1046			14x5/16
1190	1370			14x5/16

(8) CONSTRUCTION: 3" gravel line 75
Was a surface sanitary seal provided? Yes ☒ No ☐ To what depth 50 ft.
Were any strata sealed against pollution? Yes ☐ No ☐ If yes, note depth of strata
From ft. to ft.
From ft. to ft.
Method of sealing cement

(9) WATER LEVELS:
Depth at which water was first found, if known ft.
Standing level before perforating, if known ft.
Standing level after perforating and developing 374 ft.

(10) WELL TESTS:
Was pump test made? Yes ☒ No ☐ If yes, by whom? Midway
Yield: 2000 gal./min. with 520 ft. drawdown after hrs.
Temperature of water Was a chemical analysis made? Yes ☒ No ☐
Was electric log made of well? Yes ☒ No ☐ If yes, attach copy

(11) WELL LOG:
Total depth 1456 ft. Depth of completed well 1393 ft.
Formation: Describe by color, character, size of material, and structure

ft. to ft.	
0 - 200	brown rock & sand & sea shell
200 - 247	Blue clay & sand
247 - 370	Blue sand, rock & sea shell
370 - 420	Blue clay
420 - 474	Gray sand & rock
474 - 560	Blue sand, rock & sea shell
560 - 680	Blue sand & blue clay
680 - 960	Blue clay
960 - 1012	Blue sand & clay
1012 - 1323	Hard sand & rocks
1323 - 1420	Sand, rock & streaks of clay
1420 - 1460	Blue clay

Pump Setting 640

Work started 12/31 1966, Completed 2/12/66

WELL DRILLER'S STATEMENT:
This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief.
NAME Midway Drilling & Pump Co.
(Person, firm, or corporation) (Typed or printed)
Address P. O. Box 4608
Saticoy, California
[SIGNED] Carl Clem owner
(Well Driller)
License No. 177084 Dated 3/14/66, 19

SKETCH LOCATION OF WELL ON REVERSE SIDE

DWR 188 (8-55)

Conceptual Lithology and SWL for Well #7

Starting Depth, ft	Ending Depth, ft	Lithologic Description	Comments	SWL Well #4
0	55	Clay	SWL in Well #3 borehole was at about 374 ft bgs, so lithologic descriptions are more generalized in the vadose zone. For example, a Sand in the vadose zone may be a silty sand. A Clay may be a clay and silt mixture.	
55	85	Sand		
85	105	Clay		
105	195	Sand		
195	215	Clay		
215	255	Sand		
255	265	Clay		
265	300	Sand		
300	325	Clay		
325	340	Sand		
340	348	Clay		
348	370	Sand		
370	380	Blue clay		
380	525	Gray sand & rock		
525	940	Blue clay		580 ft
940	1040	Hard sand, rocks & clay		
1040	1380	Sand, rock & streaks of clay		
1380	1450	Blue clay		
bottom of hole				

The conceptual lithology for Well #7 was developed by reviewing the Well Completion Report for Well #3 and the associated borehole geophysical logs. The lithologic descriptions from the WCR were assigned a depth interval based on our review of the BGLs. An assumed SWL of 580 ft bgs is based on a recent measurement from Well #4.