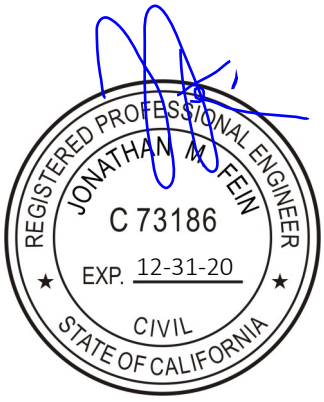
Ridgeline Engineering
421 East Loyola Drive
Tempe Arizona 85282
(602) 698-8883
www.RidgelineEngineering.com

Onsite Wastewater System Design

7548 Henness Ridge

Yosemite Village, CA



6-9-19

June 9, 2019

Prepared For: Yosemite Mountain Builders

Attn: Jeff Hornacek

Contents

1. Project Information

The proposed project is located at 7548 Henness Ridge in Yosemite National Park in California. The property consists of approximately 7 acres of undeveloped land located to the south of Henness Ridge Road (see Figure 1 below).

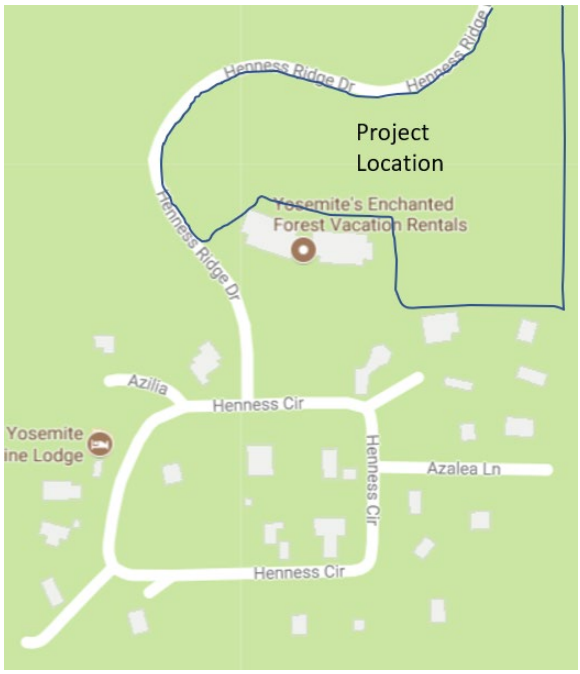


Figure 1: Project Location

The planned development consists of the following facilities:

- Main Residence
- Auxiliary Residence
- Commercial Garage
- Commercial Laundry
- Staff Residential Units (16)

The project is planned to be executed in two (2) distinct phases. Phase 1 will consist of the main residence, the auxiliary residence, the commercial garage and the commercial laundry. Phase 2 will include the addition of the 16 staff units. These calculations pertain to the Phase I development.

2. Site Characteristics

A site soils evaluation was performed by Salem Engineering Group, Inc. on June 16th. Percolation test pits were dug to evaluate the infiltration rate of the potential onsite effluent disposal fields. Based on the shallow depth to bedrock and the fast percolation rates measured onsite (> 5 in/min) a pretreatment system has been recommended by Mariposa County (see Figure 2 below for soil absorption properties).

PERCOLATION TEST RESULTS

Test No.	Depth (feet)	ULTIMATE* Percolation Rate (min/inch)	Absorption Capacity (gallon/square foot/day)	Soil Type
P-5	4.3	5.6	32.0	Silty SAND (SM)
P-6	2.3	1.7	107.3	Silty SAND (SM)
P-7	4.9	2.1	87.8	Silty SAND (SM)
P-8	2.2	2.2	80.0	Silty SAND (SM)
P-9	1.7	1.5	117.6	Silty SAND (SM)

*Appropriate safety factor not applied

Figure 2: Site Percolation / Absorption Characteristics

A map of the percolation test sites is provided below in Figure 3.

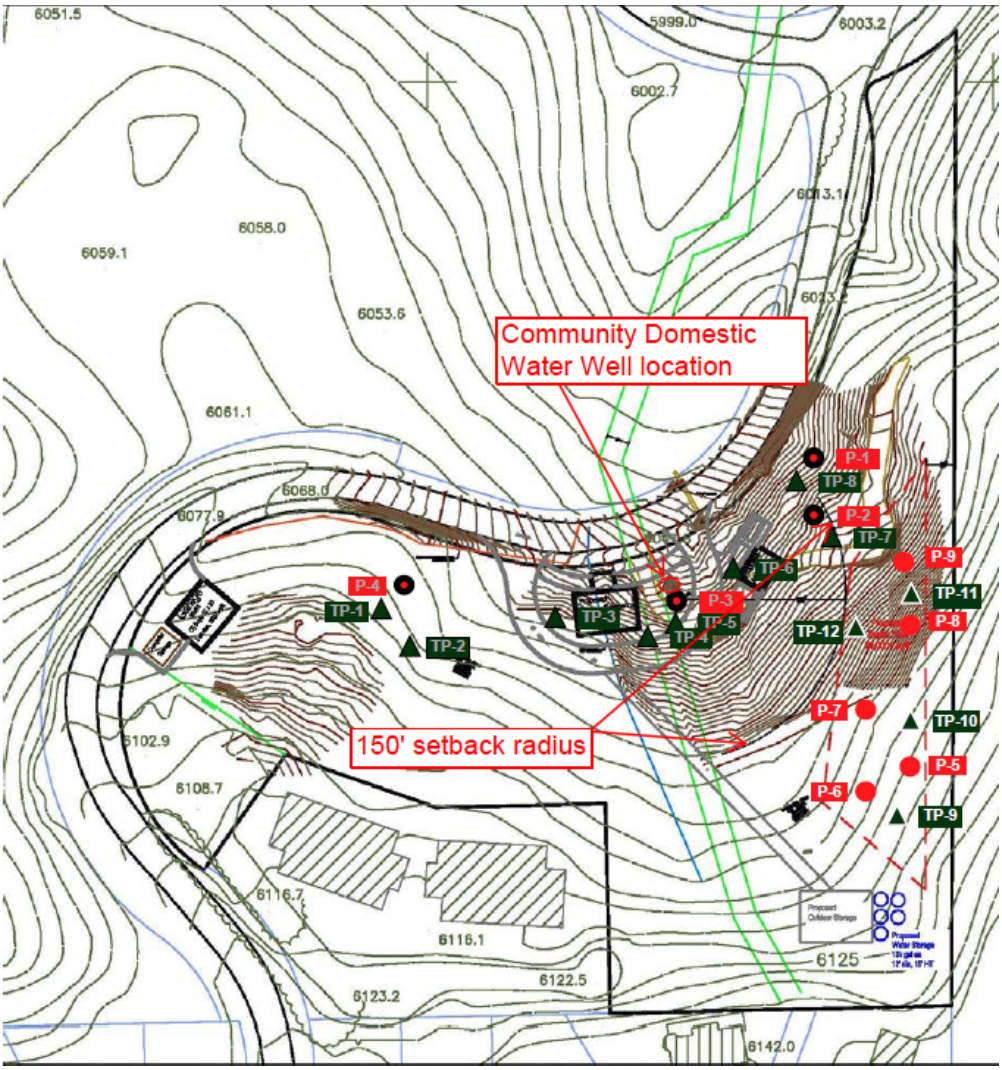


Figure 3: Percolation Test Pit Locations

3. System Design Overview

Wastewater will be conveyed from each building to a dedicated septic tank for primary treatment. Due to the nature of the area (shallow soils, steep slopes, fast percolation) secondary treatment will be provided to septic tank effluent. Orenco Advantex textile filtration systems will be utilized to provide secondary treatment prior to disposal.

Due to the steep slopes in the area and the relative location of the disposal field uphill from the development, a pressurized septic disposal system is necessary. Orenco Biotube pumps will be utilized to pump effluent from secondary treatment to the disposal field dosing tanks. Pressurized distribution will also be utilized to ensure uniform application of the effluent to the disposal area.

4. Expected Occupancy and System Design Volume

4.1. Expected Design Volume

The expected flowrates are determined by estimating 150 GPD / Bedroom for the development, as recommended by Mariposa County. The average daily volume of graywater originating from the commercial laundry (Phase I) is calculated assuming 6 washes / hour at 40 Gal/Wash for 10 hr/day.

	BLDG NO.	DESCRIPTION	# UNITS	FLOW/UNIT	TOTAL
PHASE I	1	MAIN	6	150	900
	2	AUXILIARY	4	150	600
	3	GARAGE	1	150	150
GRAY WATER		LAUNDRY (10 HR)	6	40	2400

Table 1: Design Effluent Flow Rates

5. Septic Tank Sizing

The recommended septic tank sizing to treat the calculated effluent flow based on the 2007 CPC, for flows above 1500 GPD is calculated as Flow x 0.75 + 1,125 GPD (tank size for flows below 1500 GPD recommended at 1.5 x daily average volume). The recommended size septic tank to treat the design flow is provided below in Table 3.

Building Serviced	**Ave. Daily Vol. (GPD)**	**Recommended Tank Vol. (gal)**	**Provided Tank Volume (gal)**
Main	900	1350	1500
Auxiliary	600	900	1500
Garage	150	225	1000

Table 2: Recommended Septic Tank Sizing

6. Peak Volume

Peak flow volumes are calculated using the following equation (Lindeburg, CERM Section 28-2). Gravity line size and pump sizing for the project is based on peak design volumes. See Table 3, below.

$$Qpeak / Qave = (18 + (P)^{1/2})/(4 + (P)^{1/2})$$

Building Serviced	Ave. Daily Vol. (GPD)	Peaking Factor	Design Flowrates (gpm)
Main	900	1.41	0.88
Auxiliary	600	1.49	0.62
Garage	150	1.86	0.19

Table 3: Peak Design Flowrates

7. Proposed Disposal Fields

To ensure conservatism in the system design and account for eventual clogging and deterioration of performance of the disposal area, a reduced Absorption Capacity of 1.2 GPD/ft^2 was used to size the disposal fields (see Figure 4 below).

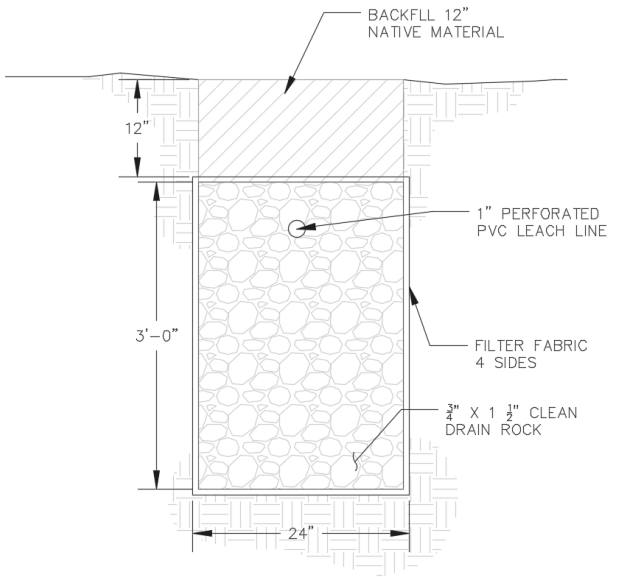
Soil Texture	Percolation Rate (min/in/ min/cm)	Application Rate (gpd/ft²/ Lpd/m²)
Gravel, coarse sand	<1/ <0.4	not suitable
Coarse to medium sand	1 - 5/ 0.4 - 2.0	1.2/ 0.049
Fine to loamy sand	6 - 15/ 2.4 - 5.9	0.8/ 0.033
Sandy loam to loam	16 - 30/ 6.3 - 11.8	0.6/ 0.024
Loam, porous silt	31 - 60/ 12.2 - 23.6	0.45/ 0.018
Silty clay loam, clay loam	61 - 120/ 24.0 - 47.2	0.2/ 0.008
Clay, colloidal clay	>120 / >47.2	not suitable

Notes: 1) min/in x 0.4 = min/cm
2) gpd/ft² x 40.8 = Lpd/m²

Source: Crites & Tchobanoglous, 1998.

Figure 4: Recommended Effluent Application Rates

A cross section of the proposed disposal field trench is provided in Figure 5 below. The bottom area and 24" of side wall on either side of the trench are counted as participating in the effluent percolation (OWTS Appendix Table K3).



DETAIL A: TYPICAL LEACH TRENCH SECTION

Figure 5: Proposed Disposal Field Trench Section

The required absorption areas for the disposal fields are as follows in Table 4:

LEACH FIELD NO.	PHASE I FLOWRATE (GPD)	Max Application Rate (GSF)	PHASE I REQ'D "L" (FT)
A	1050	1.2	146
B	600	1.2	83
C	2400	1.2	333

Table 4: Required Leach Field Sizing