Appendix G

Hydrology, Water Quality, and Utilities

February 12, 2020

Under Canvas, Inc
Bozeman, Montana 59715

Attn: Mr. Daniel McBrearty

RE: **HYDROGEOLOGIC REPORT**
UNDER CANVAS – GROVELAND, CALIFORNIA

Mr. McBrearty:

Water Resources Associates, Inc. (WRA) is pleased to present this hydrogeologic report regarding the Under Canvas project located just outside of Groveland, California. We completed this work in support of State of California requirements to obtain a State Small Water Systems permit, along with supporting a Tuolumne County-required Environmental Impact Report (EIR).

If you should have any questions, comments, or require further explanation of any of the information presented herein, please do not hesitate to contact the undersigned at (805) 901-2505.

Sincerely,

WATER RESOURCES ASSOCIATES, INC.

Devon F. Ayres

Devon F. Ayres, P.G. 7197
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EXECUTIVE SUMMARY

Water Resources Associates, Inc. (WRA) was retained by Under Canvas, Inc. (Client) to conduct a hydrogeologic assessment on the proposed project, located near Groveland, California. The work included as part of this assessment:

- Preparation of State and County-required hydrogeologic workplans, SB1263 technical report documents, along with drilling and pumping test discharge permitting.
- Assessment and siting of test holes, based on hydrogeologic assessments, background hydrogeologic data collection, and onsite baseline monitoring of groundwater elevations.
- Project management and technical oversight/direction throughout completion of test hole drilling and subsequent aquifer pumping tests and water quality sampling.
- Assessment of well and aquifer hydraulics, groundwater supply and demand, and groundwater quality in support of both the project Envinronmental Impact Report (EIR) and the application for a State of California Small Water System Permit.

Project Location

The project site is located east of the town of Groveland and west of the Big Oak Flats Entrance to Yosemite National Park in southern Tuolumne County, California. The project is located on the Ascension Mountain, CA 7.5' U.S. Geological Survey (USGS) Quadrangle and falls within the southeastern portion of Section 26, Township 1 South, Range 18 East, Mount Diablo Baseline and Meridian. The project location is within unincorporated Tuolumne County and is approximately 120.7 acres. Access to the site is provided by Hardin Flat Road via State Route (SR) 120. The project site consists of open land that was previously used for forestry and logging. Adjacent land uses include scattered private residences, recreational facilities, and open space. The nearest building is a Caltrans snow plow garage approximately 1,250 feet north of the nearest corner of the project site. The nearest residence is located approximately 1,300 feet southeast of the southern boundary of the project site. Ground surace elevations at the project site range from approximately 3,740 feet above mean sea level on the eastern portion of the site to 4,050 feet above mean sea level on the western portion.

Project Goal

Our goal was to assess and report on the hydrogeologic conditions of the project site, relative to the the aquifer systems ability to support the planned development, while also not posing a significant risk to the fractured groundwater aquifer system(s) and/or other domestic wells in the site vicinity.

Project Objectives

Based on our stated project goal, WRA established the following project objectives:

- Complete and submit for approval the State- and County- required hydrogeologic workplans that were required to obtain approved permits for drilling operations. This included the State Water Board's required Hydrogeologic Workplan and Tuolumne County Drilling Permits.
- Complete and submit for approval a State Regional Water Quality Control Board Discharge permit for the temporary discharge of clean groundwater to land.
- Conduct geologic and hydrogeologic assessments for the purpose of identifying potential test hole drilling locations.
- Conduct aquifer testing on two (2) of the three (3)water supply wells completed during this work.. This included aquifer pumping tests and recovery monitoring, and collection and analysis of water quality samples. These water samples were to be analyzed for State of California Title 22 Drinking Water constituents
- Prepare and submit a technical report, describing our findings, conclusions and recommendations. The report will be suitable for updating the preliminary SB1263 report, with respect to water supply assessments, water balance, and interpretations of recharge values, potential operational conditions, potential offsite impacts, and water chemistry as it is related to State water quality standards for small water systems and the possible need for water treatment.
- Our technical report will be coordinated with the ESA (the environmental impact report [EIR] consultant), to provide them with sufficient data and technical language to support the EIR effort.

Project tasks

Based on the project objectives, the scope of work was divided into seven primary phases, which were subsequently subdivieded into specific tasks. The tasks were identified based on our understanding of the project goal and objectives.

<u>Water Supply and Demand</u>

Groundwater, recharged from rainfall, run on and snowmelt, is the only source of water for the project. To be conservative, only rainfall is considered in our recharge assessments. The source/supply of recharged water to the drainage basin ranges from 25 to 80 acre-feet of per year, on average, depending on the assumed size of the drainage basin. The groundwater extracted in support of the project, ranges from about 8 to 12.5 acre-feet per year, or conservatively 10% to 55% of the recharge source, based on rainfall alone. The impact of other withdrawals from the immediate area are assumed to be about the same for all the residential wells with the assumed drainage basin, and perhaps as much as 25% for the reported development north of the project site.

The project water demand estimates are based on other similar Under Canvas project and are summarized as follows:

- Average seasonal occupancy, in days: 250.
- Average number of daily seasonal visitors: 250.
- Average daily water use per seasonal visitor, gallons per day (GPD): 20 GPD.
- Average site water use, per day: 10,000 GPD (conservative, slightly increased).
- Average annual water use: 2,500,000 gallons, or approximately 7.7 acre-feet per year.

Aquifer pumping test results suggest that even in a worst-case-senario, which almost certainly will never occur, if the project were at maximum occupancy, using maximum water demand estimates, during the driest period of the year, water levels would likely remain stable.

<u>Water Quality</u>

Based on the results of the samples collected during aquifer pumping tests, the groundwater at the project location is classified as a “calcium bicarbonate” in nature and is of excellent quality. Gross alpha activity (as reported by in the laboratory analysis) which is an indicator of dissolved uranium in groundwater, was reported as either far below the Action Level that would trigger a subsequent uranium analysis, or at non-detectable levels. The results of the Title 22 Drinking Water analyses suite indicated that no constituents of concern were detected in the samples collected from Well 1 and Well 2.

Environmental Impact

The hydrogeologic assessment conducted by WRA related to environmental concerns, as documented in the CEQA and EIR process documents, suggest no negative impacts are likely to the environment (onsite or offsite), based on the provided project water use and wastewater parameters.

State Small Water System Permitting

The aquifer pumping tests conducted indicated minimal drawdown and the subsequent recovery rates in both wells tested (Well 1 and Well 2). Based on these findings, the water supply and demand requirements stipulated in the State Small Water System (SSWS) application seem to be satisfied. Based on the anticipated recharge rates, groundwater supply appears to be adequate, even during periods of extended droughts of two to three years duration, relative to the quantity of water consumed onsite.

Conclusions and Recommendations

Based on our background research, field work and findings, we offer the following conclusions:

- Pressurized groundwater was documented in all three of the onsite wells completed by WRA.
- Similarities documented in the groundwater samples collected from Well 1 and Well 2 suggest that both of these wells are likely drawing groundwater from a shared fracture system.
- The limited draw down recorded in the three Wells, when either Well 1 or Well 2 were being individually pumped, indicates that the there is some hydraulic communication between these wells. No indications of hydraulic communication were observed between Wells 1 and 2 with Well 3.
- Some additional drawdown was documented in Well 1 while aquifer pumping tests occurred at the project site to the immediate north. This additional drawdown indicates that some hydraulic communication likely exists between Well 1 and the well(s) at the neighboring project.
- Based on the water demand estimates provided to WRA and our conservative recharge estimates, the proposed project development does not appear to place a burden on the

available groundwater supply in the project vicinity, even during periods of extended drought.

- The findings of the aquifer pumping testsand groundwater sample results indicate that at this time there is sufficient capacity in the fractured aquifer system to support planned water use, and that the quality of the water meets all Federal and State drinking water requirements for potatability.
- The project, as described and assessed, does not appear to pose a significant risk to the environment, with respect to the use of groundwater to support the project. Facility water demand estimates, based on very conservative assumptions, are low with respect to the conservative estimates of the water supply available to the project site.

The data necessary to evaluate the magnitude of potential impact(s) to groundwater supply resulting from the reported development north of the site has not been made available to WRA and therefore cannot be adequately evaluated. However, it can be reasonably concluded that some influence on water levels are likely in at least Well 1. The magnitude of this influence cannot be estimated at this time. Depending on the neighboring projects water demand and pumping schedule, some influence to groundwater availability to the project site is possible.

Based on our findings and conclusions, we offer the following recommendations:

- A permanent weather station should be established on the project site to facilitate more accurate precipitation data.
- Each of the wells completed be should equipped with electronic logging equipment that is capable of recording and reporting water levels (static and pumping), discharge rates (instantaneous and cumulative) and power consumption.
- Each should be equipped with dedicated electronic equipment and sample collection ports.
- To maintain well performance, an operational pumping schedule should be developed to regularly pump both Well 1 and Well 2.
- As part of the State Small Water System operational requirements, routine monitoring and recording of all pumping operations should be conducted. These records should be reviewed by the System Engineer, and as needed WRA.
- At least one of the wells should be equipped with a backup generator to maintain a power supply in the event of power outage. Alternatively, sufficient onsite water storage should be maintained to meet the maximum estimated water demand for at least two (2) days (approximately 20,000 gallons).

- A "Low Water Usage" operational plan should be prepared to address reasonable reductions in groundwater use during periods of drought.

Report Organization

This report is organized into 7 sections, as follows:

- Executive Summary
- General Introduction and Background
- Project Specific Services
- Findings
- Conclusions and Reccomendations
- Environmental Impact Report
- State Small Water System

To simplify nomenclature into the future, the three test holes (TH1, 2 and 4) have been designated as Wells 1, 2 and 3 throughout the report, accept on original graphics and permit applications.

INTRODUCTION

Water Resource Associates, Inc. (WRA) was retained by Under Canvas, Inc. (Client) to conduct a hydrogeologic assessment on the proposed project, located near Groveland, California (project site). The purpose of the the work conducted was to assess and report on the hydrogeologic conditions of the project site, relative to the ability of the aquifer systems' ability to support the planned development, while not posing a significant risk to the groundwater environment.

Background

The project site is located in Tuolumne County, 15 miles east of Groveland, California, on the south side of Highway 120. The project site is approximately 120.7 acres in size, composed of high Sierran forest. Figure 1 presents the general site vicinity, and a location map for the project site. Property boundaries are presented in Figure 2.

Land Use

The project site is currently undeveloped forest and rural land. Land uses within the immediate vicinity are predominately rural in nature, consisting of open land, recreation facilities, and dispersed rural residences to the west, south and east of the project site. The project is located on lands zoned Commercial Recreation (C-K) under the Tuolumne County Ordinance Code and designated as Parks and Recreation (R/P) by the Tuolumne County General Plan (the project site also includes land zoned Open Space-1 under the Tuolumne County Ordinance Code; however, no development associated with the project will occur on land with Open Space-1 designation). Commercial Recreation and Parks and Recreation both include hotels and motels and recreational facilities such as campgrounds as an allowable land use, subject to the approval of a Site Development Permit. The site is surrounded by undeveloped land with no residences in the immediate vicinity. The project would have no impact related to physically dividing an established community.

The purpose of the R/P land use designation is to provide for recreational uses of commercial nature to serve the tourist industry as well as provide leisure activities to the County's residents. (Tuolumne County, 1996). Additionally, development in the C-K district must comply with fire safety standards, as per Title 15 of the Tuolumne County Ordinance Code.

As described in the project description, the project proposes to develop 99 luxury campsites and associated infrastructure. Accordingly, the project does not involve a change in land use and is consistent with the County General Plan land use designations as well as the County Ordinance Code zoning designations. Additionally, the project would not conflict with any policies or regulations and therefore, the project would have a less than significant impact relating to applicable land use plans, policies, and regulations.

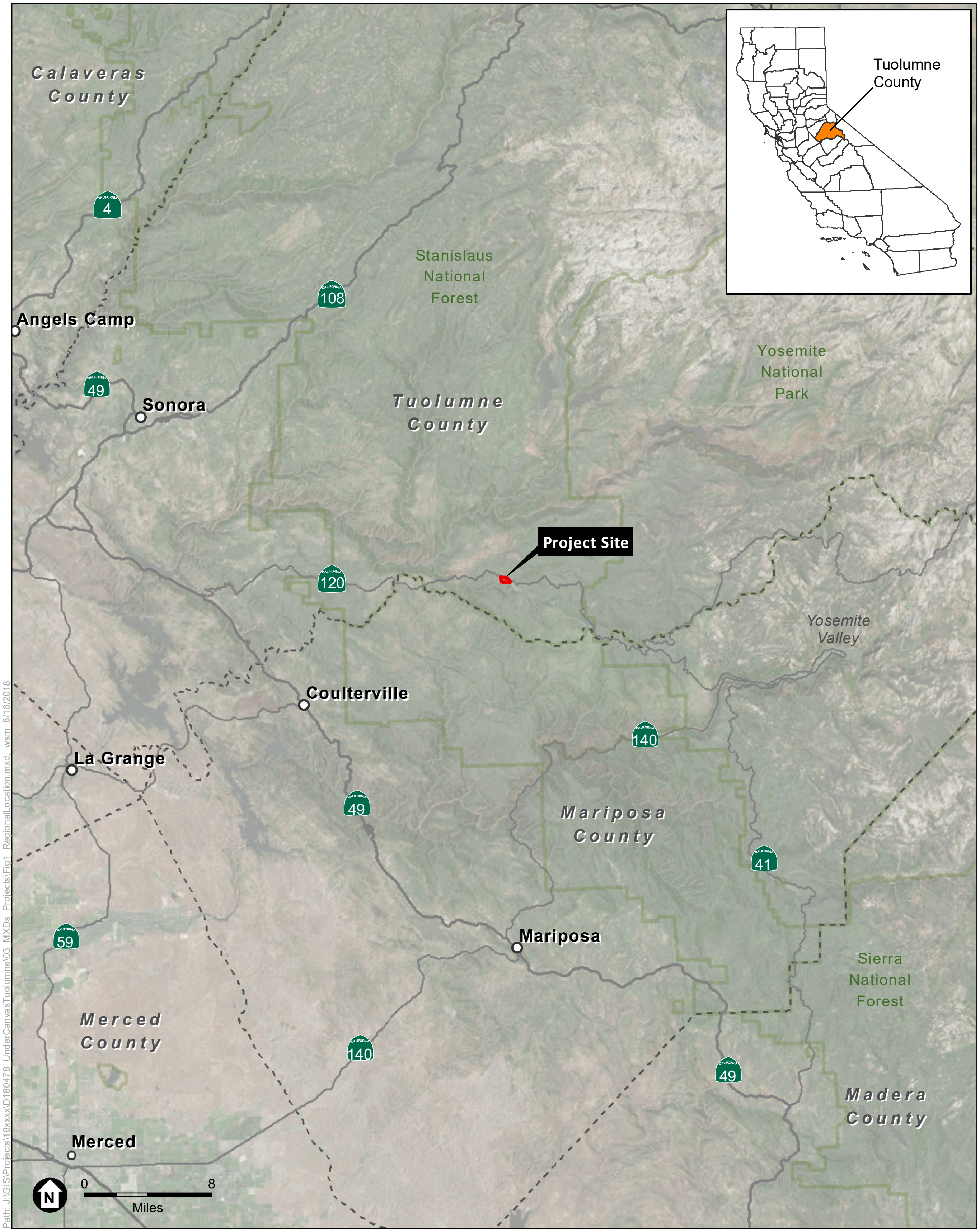
The project is not within a habitat conservation plan (HCP) or natural community conservation plan (NCCP). The nearest HCP is the PG&E San Joaquin Valley Operation and Maintenance Habitat Conservation Plan, located approximately ten miles south, in Mariposa County (CDFW, 2017). Therefore, the project would result in no impact.

Services

Based on our proposal, WRA performed the following services in support of the hydrogeologic assessment of the client's Groveland project, with respect to the proposed project goal and objectives.

State and County Liaison

In April 2019 WRA identified neighboring water systems to request approval for connecting to or a denial of the request to connect, as mandated by the State Water Board SB1263. After a Google Earth Search with minimal sucess, WRA contacted the State Water Resources Control Board (SWRCB). The SWRCB directed WRA to the businesses that are in the required (three mile) search radius. An internet search of the business names gave us basic contact information. Subsequently, we requested a list of names and direct



SOURCE: Esri, 2015; ESA, 2018

Yosemite Under Canvas Project

Figure 1
Regional Location

ESA

SOURCE: USDA, 2016; ESA, 2018

Yosemite Under Canvas Project

Figure 2
Project Site

phone numbers from SWRCB. All four water systems within the three mile radius were subsequently contacted by email, US postal service and phone. At that time we had a definitive "No" from three of the four water systems. Table 1 below lists the systems and contact information for each, along with the system response to our request for extension of their service to UC.

Table 1 – Existing Small Water Systems in the Project Vicinity.

Water System	Contact	Phone Number	Answer
San Jose Family Camp	Jay Phillips	408.390.7578	No
Camp Tawonga	Rebecca Meyer	415.543.2267	No
Yosemite Lakes Campground	Miguel Ortiz	209.962.0102	No
Yosemite Riverside Inn	Roland Hilardes	209.962.7408	

The aforementioned water systems are unable to extend service to the project site, so we spoke with the State who guided us to submit applications for the project to be considered "Transient/ Non-community" (TNC) system, as the project is to be operated on a seasonal basis.

The SB1263 Technical report consisted of: 1) a preliminary (client supplied) water demand, 2) construction cost estimate, 3) a five year expense budget, 4) a capital improvement plan, 5) ownership, rental and deed/ trust information for the property, and 6) the required State mandated forms. This information was then all submitted to the State for approval in early June 2019.

On June 13, 2019 the State gave approval to WRA to drill exploration test holes, and communicated their approval to Tuolumne County. Along with the approval the State requested additional information including, contact data, dates to establish the operating "season", a description of the orgazational structure of Under Canvas, as the project develops the name and contact information of the Operations Manager for the site, and data that would come from the pumping tests at the conclusion of the drilling and well construction operations. This information was forwarded to the Division of Drinking Water by the State.

Geologic Setting

The Natural Resources Conservation Service (NRCS) mapped two soils units within the project site boundaries (NRCS, 2018). A description of each soil unit is provided below.

• Holland family, deep- moderately deep complex, 5 to 35 percent slopes (map unit symbol 130), is not listed as hydric by the NRCS. Included in this soil map unit are minor components of Lithic xerumbrepts, Rock outcrop, and Dystric xerochrepts. The map unit composition is 80 percent Holland family and similar soils and 20 percent minor components. The unit consists of well drained soils.
• Josephine family, moderately deep, deep complex, 5 to 35 percent slopes (map unit symbol 159), is not listed as hydric by the NRCS. Included in this soil map unit are minor components of Dystric lithic xerochrepts and Sites family. The map unit composition is 70 percent Josephine family and similar soils and 30 percent minor components. The unit consists of well drained soils.

Faults, Seismicity, and Landslides

A fault is defined as a "fracture or fracture zone in the earth's crust along with which there has been displacement of the sides relative to one another." For the purpose of planning there are two types of faults, active and inactive. Active faults have experienced displacement in historic time, suggesting that future displacement may be expected. Inactive faults show no evidence of movement in recent geologic time, suggesting that these faults are dormant. Ground-shaking is motion that occurs as a result of energy released during faulting. The damage or collapse of buildings and other structures caused by ground-shaking is among the most serious seismic hazards. The project site lies in the foothills of the western Sierra Nevada Mountains, an area experiencing relatively low seismic activity. No active faults or Earthquake Fault Zones (Special Studies Zones) are located within or adjacent to the project area (CDC, 2018).

According to the California Department of Conservation Division of Mines and Geology, the project site is not located within a delineated Alquist-Priolo Earthquake Fault Zone or Landslide and Liquefaction Zone (CDC, 2018). Because the project is not located in an area considered at high seismic risk, it is not expected to expose people or structures to earthquake risk, including strong seismic ground shaking, seismic-related ground failure, liquefaction, or landslides. In addition, slopes in the project area are relatively modest and pose no threat of landslides.

Liquefaction

Potential Liquefaction is a type of ground failure most likely to occur in water-saturated silts, sands, and gravels, having low to medium density. When a soil of this type is subjected to vibration, it tends to compact and decrease in volume. If the groundwater is unable to drain during the vibration, the tendency of the soil to decrease in volume results in an increase in pore-water pressure. When the pore-water pressure builds up to the point where it is equal to the over-burden pressure (effective weight of overlying soil), the effective stress becomes zero. In this condition, the soil loses its shear strength and assumes the properties of a heavy liquid. Based on the lack of published historic evidence of liquefaction in the area, the liquefaction potential of the site soils is considered low.

Tsunami, Seiche, and Volcanic Hazards

Tsunamis are a series of waves in a water body caused by the displacement of a large volume of water, generally in an ocean or a large lake. Earthquakes, volcanic eruptions and other underwater explosions above or below water all have the potential to generate a tsunami. Seiches are waves generated by earthquakes, winds, or landslides that set up oscillatory waves in an enclosed basin (i.e. lake or reservoir). The project site is not located near any enclosed bodies of water, large bodies of water, or oceans; therefore, there is no reasonable danger from tsunamis or seiches at the project site. There is no significant source of volcanism in proximity to the project site; therefore, there is no reasonable danger from volcanic eruption hazards at the project site.

As more fully described above, the project is not located within a delineated Alquist-Priolo Earthquake Fault Zone. Additionally, the probability of soil liquefaction actually taking place on the project area is considered to be low. With adherence to all applicable codes and regulations, geologic hazard impacts associated with on-or offsite landslide, lateral spreading, subsidence, liquefaction, or collapse are minimal and result in a less-than-significant impact.

Subsidence

Subsidence is the gradual settling or sinking of the earth's surface with little or no horizontal motion. Subsidence is caused by groundwater withdrawal, gas withdrawal, hydrocompaction or peat oxidation. Subsidence would not be expected to occur in the bedrock geology that characterizes the project site.

Expansive Soils

Expansive soils are largely comprised of clays, which greatly increase in volume when water is absorbed and shrink when dried. When buildings are placed on expansive soils, foundations may rise each wet season and fall each dry season. This movement may result in cracking foundations, distortion of structures and warping of doors and windows. The soil at the project site has a low shrink-swell potential (NRCS, 2018). Consequently, expansive soils are not likely an issue at the project site.

Although no subsurface exploration has been conducted to confirm the relative absence or presence of expansive soil materials, the soils types found on-site would be expected to contain higher clay content than that of the surface. Expansive soil materials are encountered throughout the state and are generally addressed through standardized foundation engineering practices.

Mineralogy

Tuolumne County contains a wide variety of mineral resources. Both the United States Geological Survey (USGS) and the California Geological Survey (CGS) have evaluated the potential locations and production capacity of various types of extractive resources throughout the area. No known mineral resource recovery sites have been identified in the immediate project vicinity (USGS, 2017). Additionally, policy 4.E.1 of the Conservation Element of the Tuolumne County General Plan directs the County to protect lands classified as significant Mineral Resource Zone-2 (MRZ-2) by the State Department of Conservation Division of Mines and Geology, and to meet the criteria established in the General Plan for Mineral Preserve Zone (-MPZ) overlay, from conflicts, such as incompatible development on surrounding land, which might prevent future mining activities. The State of California Division of Mines and Geology surveyed Tuolumne County for the presence of economically important mineral resources. The project site does not contain areas classified as MRZ-2 and therefore, the project will not result in the loss of the availability of a known mineral resource or affect a locally-important mineral resource recovery site delineated on a local general plan, specific plan or other land use plan, resulting in no impact to mineral resources.

Hydrogeologic Setting

An ephemeral drainage system occurs within the project site. The main ephemeral drainage on-site is tributary to the South Fork Tuolumne River. The South Fork Tuolumne River lies approximately 0.6 miles to the south of the project site and is part of the Upper Tuolumne River

Watershed. The South Fork Tuolumne River drains a small portion of the western edge of Yosemite National Park. The headwaters begin between White Wolf and Yosemite Valley at elevations between 8,000 feet and 8,500 feet. The South Fork Tuolumne River exits the park at an elevation of 4,500 feet, just north of Hodgdon Meadow and upstream of its confluence with the main Tuolumne River. The confluence of the Middle Fork and South Fork occur approximately five miles downstream of the project.

Surface water quality in the region is generally considered very good. For example, most of the water from the Tuolumne River is usable for human consumption with disinfection alone, although additional treatment is required by law (Tuolumne-Stanislaus IRWM Plan, August 2013). The majority of the surface water quality issues identified within the County can be linked back to current or historical land use practices such as mining, septic systems, livestock grazing and water based recreation activities.

The County is located within the foothills and higher elevations of the Sierra Nevada where the subsurface material consists primarily of impermeable granitic and greenstone bedrock which can result in a low groundwater yield. The California Department of Water Resources (DWR) Bulletin 118 provides a detailed description of groundwater basins in California; however, the bulletin does not identify any groundwater basins within Tuolumne County. Groundwater is the primary source of water for most small water systems in Tuolumne County and the characteristics of the fractured bedrock and precipitation variations have led to some wells providing unreliable sources of water in this area.

The project is not located in an area designated as a 100-year flood zone. As described in the Tuolumne County Multi-Jurisdictional Hazard Mitigation Plan, the physical geography of the County impacts and limits the flooding potential. The overall slope of the watersheds is relatively steep and the river and stream flows typically run off quickly and therefore very little flood plain has been formed (Tuolumne County, 2017). In addition, the Tuolumne County Multi-Jurisdictional Hazard Mitigation Plan lists the project area as Zone X which is a designation for areas of minimal flood hazard.

Dam failure, which is the collapse or failure of an impoundment that causes significant downstream flooding, is not a concern for the project area. Although Tuolumne County has multiple large and small dams, only the O'Shaughnessy Dam poses a risk for significant flooding. However, this dam is located on the Middle Fork Tuolumne River and the project is located near the South Fork Tuolumne River and any inundation from a catistropic failure would not reach the project area.

Project Specific Tasks

WRA performed the following tasks, in support of the project:

WRA completed and submitted for review a draft workplan for the project team. The workplan described the well siting process, test hole/well construction, pumping tests and water quality testing. The work-plan helped establish more precise preliminary cost estimate and ensure project team have the same basic plan.

Fracture trace analysis

WRA conducted both an office-based and field assessment of readily apparent fracture traces around and upon the project site. Hard rock aquifers primarily occur in the secondary porosity of fractures. Groundwater is recharged, stored and moves through these fractures, to wells, and reach the surface in some instances as springs. WRA was specifically targeting these fractures as locations for test well drilling, which might result in successful water supply wells for the project.

The fracture trace analysis began with reviewing readily available satellite imagery, reviewing Tuolumne County geologic and land-owner data, querying Department of Water Resources database to gather surrounding properties well completion reports. Based on this work, WRA prepared a preliminary fracture trace map, which indicated our interpreted locations for fractures and fracture system traces. From this we identified six locations that warranted further assessment, as possible locations for test hole drilling. Of the original six locations, we requested permits for drilling three (3) of the locations, from both the County and the State. Figure 3 shows the first six preliminary test hole locations.

WRA then conducted “ground truthing” (field verification of aerial photo interpretation, along with geologic mapping) and assement of the feasibility of the six locations initially identified, with particular emphasis on the three most promising locations. These locations were the most promising based on their proximity to suspected fractures/fracture systems, separation from known components of development planning, offset from sensitive areas, and lastly access for drilling equipment. Ground truthing included assessing the veracity of apparent fracture systems, i.e. distinguishing them from other surface features (e.g. power lines, fences, etc.). Strike/dips of the apparent fractures, when found, were measured. Too few in-place bedrock outcrops were found, and as such the stereonet mapping of the fracture systems was not attempted because locations based on these stereonets would have been less than reliable.

Based on the ground truthing and field mapping, four locations were selected, out of the six, and three were considered most promising for the previously stated reasons. Figure 4 shows the locations of the six original and final four locations. Well siting involved a more in-depth review of each site, and assessing proximity to the inferred fracture system. Figure 5 indicates the results of the well siting effort, and the three most promising are numbered in the order of precedence for drilling.

Well Completion Operations

WRA collaborated with Canepa and Sons Well Drilling of Sonora, California, (Canepa) in applying for Tuolumne County drilling permits. Additionally, WRA interacted with the State Water Board in gaining approval for proceeding with drilling operations. The County and State concurred and permitted three of the four identified locations. The fourth location was considered in close proximity to two of the other sites, and as such, was not permitted initially. Had either Well 2 or Well 3 proven unacceptable to WRA, then the County and State would have been contacted, for approval to drill TH3. Copies of the drilling permits are included in Appendix A.

Drilling operations began with site staking, pad preparation, and general equipment staging. WRA coordinated with Canepa to ensure accessibility for drilling equipment, prior to staking the final well locations, and discussed guidelines on how/where to discharge encountered fracture water during drilling. Canepa subsequently applied to the county for a drilling permit before mobilizing drilling equipment to the site.

Drilling was conducted using an air-rotary, down hole hammer drill bit, utilizing compressed air as a drilling fluid. Significant issues were not encountered in any of the three drilled test holes. WRA representatives were onsite during drilling operations to conduct the following:

- Technical direction and oversight throughout drilling operations.
- Collection and logging of cuttings/chip samples.
- Observation of encountered water.
- Assessment of geologic and hydrogeologic conditions related to final drilled depth, and encountered water.

Canepa began drilling the first test hole (Well 1) on August 28, 2019. The county arrived on August 29, 2019 to observe installation of the 100 foot surface seal at Well 1 and, to approve Well 2 and Well 3 locations. The County reported no issues with the surface seal or the selected locations for additional test holes. Well 1 encountered its first fracture (producing water) at a depth of about 115 to 116 feet below ground surface (bgs) reportedly producing upwards of 50 gallons per minute (gpm). Lesser fractures were discovered at 195-196 feet and 305-306 feet producing 15 GPM for each depth. The test hole was drilled to about 1,000 feet bgs, completed on September 5, 2019.

The State required that drilling fluid and water produced from Well 1 be kept isolated, and as far removed as possible from a spring fed stream, approximately 30 yards north from the drilling location. Canepa constructed a pit, surrounded by earth berms, to prevent water from entering the stream. The pit was located south of both the stream, and main road on the property. WRA monitored the integrity of the berm and pit to ensure no generated water flowed or leaked into/near the stream. Drilling of Well 1 concluded without any delays or unforeseen problems and site conditions and the discharge pit were restored to pre-drilling conditions.

Upon completion of Well 1, Canepa mobilized to Well 2. On September 9, 2019, after road clearing and pad prepation were complete, Canepa drilled 100 feet to install the County-required surface seal. The county arrived to observe installation of the surface seal on September 10, 2019. The County reported no issues with the surface seal. Air-rotary drilling commenced on September 11, 2019 and Well 2 encountered its first fracture at a depth of 119-120 feet bgs producing about 15 gpm. Additional fractures were discovered at 132-134 feet (15 gpm), 138-139 feet (10 gpm), 159-182 feet (10 gpm), 182-183 feet (20 gpm), and 295-296 feet (5 gpm). The well was drilled to 980 feet bgs, completed on September 16, 2019. Anticipated yield, as reported by the drilling contractor, was about 75 gpm.

The intended depth for Well 2 was 1,000 feet however, difficulties were encountered during drilling. The bit penetration rate slowed significantly below a depth of approximately 830 feet bgs. Two factors influenced the decision to terminate drilling at 980 feet:

- The prolific, fractured-rock aquifer was producing more water than could be adequately circulated and discharged.

- Overlying unstable, fractured, and weathered granite, i.e. decomposed granite (DG) was perceived to "slough off" during drilling and circulation activities, falling on the top of the drill bit.

Due to the overlying DG within Well 2, a PVC liner was installed from a depth of 40 feet bgs to 280 feet bgs. The liner is constructed of 4.950 inches OD, schedule 40 PVC with a wall thickness of 0.248 inches. The liner was milled slot perforated from a depth of 40 to 280 feet. From 280 feet to 600 feet, blank PVC of the same diameter/thickness was installed. Site conditions were restored to pre-drilling condition upon completion.

During Well 2 construction, final road work and pad prep were completed at the Well 3 site. Canepa mobilized to Well 3 on September 17, 2019, and drilled 100 feet to install the surface seal on September 19, 2019. The County arrived to observe installation of the surface seal the following day. The county reported no issues with the surface seal. During drilling of Well 3 first water was encountered in the initial fracture at a depth of 119-120 feet bgs producing about 2 GPM. Additional fractures were discovered at 208-209 feet (3 gpm), 385-395 feet (7 gpm), 535-536 feet (2 gpm), 580-581 feet (4 gpm), and 664-668 feet (2 gpm). The well was drilled to about 1000 feet bgs and was completed on September 26, 2019. Anticipated yield, as reported by the drilling contractor, was estimated at about 18 gpm. Drilling of Well 3 concluded without any delays or unforeseen problems and site conditions were restored to pre-drilling conditions upon completion.

Following completion of the three wells, to the site was "winterized". This involved the placement of additional rock road base to provide access during subsequent pump testing operations on Well 1 and Well 2.

Table 2, below, summarizes drilling operations for the three completed wells:

Table 2 – Summary of Test Hole Drilling

	Start	Stop	Diameter, inches	Depth (Ft.)	Airlift flow, GPM	First water	Static water level, feet		
Well 2	29 Aug 2019	9 Sep 2019	6.125	1000	80	115	12.5		
Well 1	6 Sep 2019	16 Sep 2019	6.125	980	75	119	84		
Well 3	17 Sep 2019	27 Sep 2019	6.125	1000	18	119	46.2		

Copies of the State Well Completion reports are included in Appendix B.

Background Water Level Measurements

WRA temporarily installed pressure transducers in each of the three wells completed to electronically measure background (i.e. ambient) static water levels (swl). This data was important to establish the background water levels in each well in the absence of onsite pumping, to assess water levels for possible offsite influences, and finally as a reference point for assessing groundwater elevation recovery upon completion the aquifer pumping tests.

Before placing the transducers into each well, manual water level measurements werecollected using a water level meter. Once the manual measurements were obtained the transducers were programmed to report depth to water every five minutes based on pressure readings from the overlying groundwater and verified with the manually derived measurements. A single water level meter was used throughout the process (background, 2-hour pumping, 10-day aquifer pumping tests, and recovery) to reduce the risk of inconsistent measurements.

The electronic equipment (In-Situ vented pressure transducers, Level TROLL 700; Part Number LT700, Item Number, R0089160) were installed at an approximate depth of 580 feet bgs on Friday, October 18, 2019. Installing transducers on a Friday facilitated measurements of groundwater elevations through the weekend (when the closesthomeowners would likely be home, using their domestic wells). Neighboring pumping tests on the north side of the road were also reportedly scheduled to occur on or about the time our testing was occurring.

During the monitoring period (October 18 through 25, 2019), WRA removed the transducers in Well 1 and Well 2 for a short period so the pump contractor could install the sounding tubes, submersible pumps, and discharge components. Below lists the date and time where data is

Figure 3
Purposed Well Locations

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Figure 4
Proposed Discharge Area

Water Resources Associates, Inc.

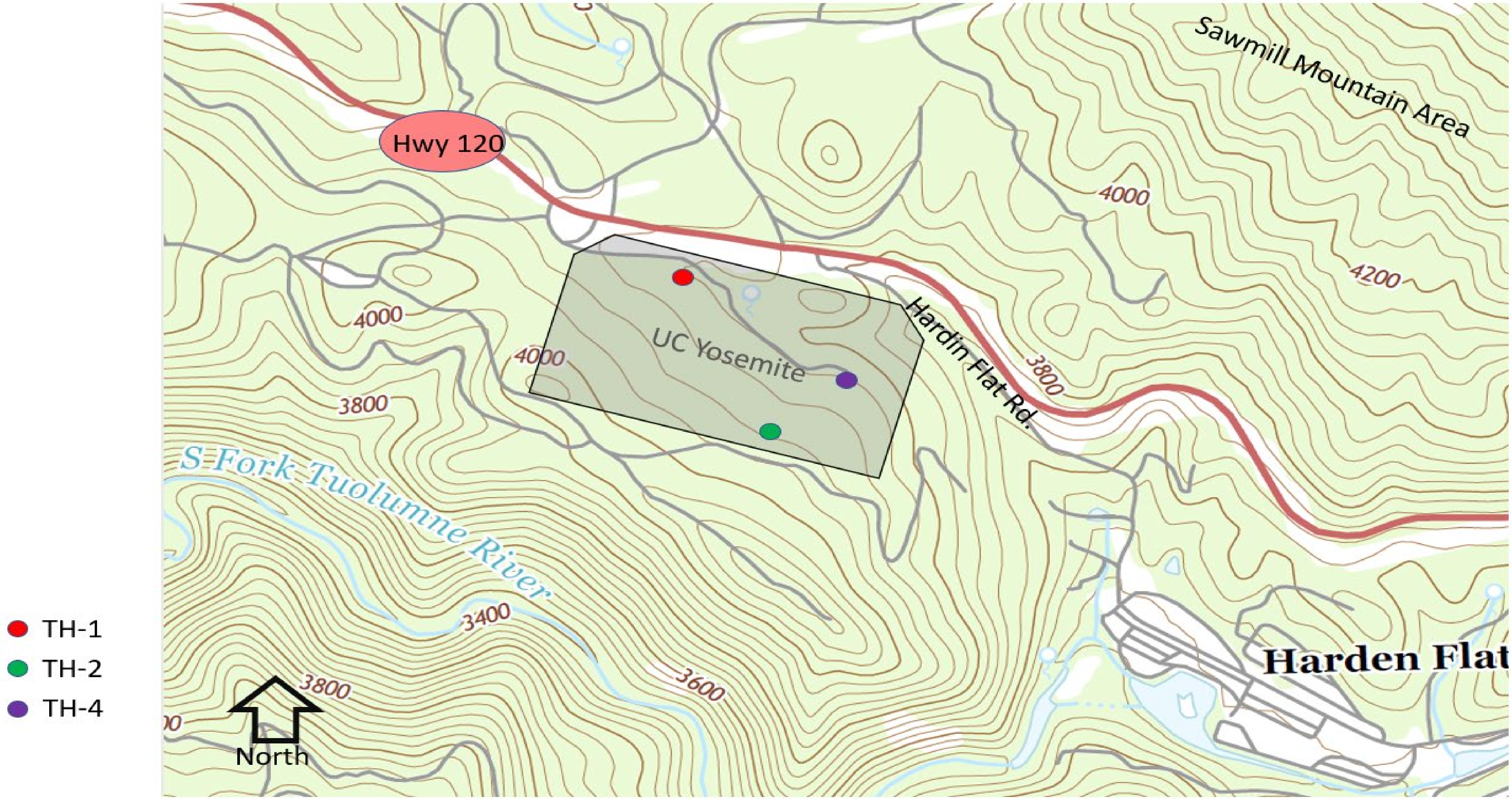

Figure 5
Final Sited Well Locations
Water Resources Associates, Inc

missing during the background monitoring period. Additionally, when WRA field personnel reset the transducer at Well 2, the depth placement was approximately 4 feet deeper, than previously. This is reflected in the raw data, directly before removal and after reinstallation.

Well 1
Removed pressure transducer: 10/22/2019 12:13:40
Reinstalled pressure transducer: 10/23/2019 16:59:50

Well 2
Removed pressure transducer: 10/22/2019 12:27:24
Reinstalled pressure transducer: 10/24/2019 19:22:24

Monitoring data was uninterrupted in Well 3 because pump and components were not installed. Upon successful installation of the pumps, the pressure transducers were reinstalled in Well 1 and Well 2 and monitoring resumed.

Pumping Tests

Prior to conducting the aquifer pumping tests , a discharge area for the discharge water was established and a discharge waiver was applied for, and received from the Regional Water Quality Control Board for the discharge water. Figure 4 presents the location of the aquifer pumping tests discharge area, and Appendix 3 contains a copy of the Approved Waste Discharge Permit.

Constant discharge rate pumping tests were conducted first on Well 2 and then on Well 1. Water levels were continuously monitored in all three completed wells throughout all testing and recovery monitoring. Groundwater samples were collected from both Well 1 and Well 2 at the end of each wells ten-day duration test.

The constant rate pumping tests were conducted at discharge rates that were considered in excess of normal pumping rates anticipated to support the planned operations on site. The constant discharge rate of forty (40) gallons per minute (gpm) was selected for both wells. The objective was to stress the aquifer system supporting each well. The elevated discharge rates, relative to projected demand, and the duration of the tests were used to assess the adequacy and resiliency of the fractured aquifer system.

Discharge rates were controlled throughout testing using a check valve downstream of the flowmeter. Flow meters were installed approximately 10 feet from the wellheads of both Well 1 and Well 2 to minimize turbulence in the discharge piping to ensure accurate readings could be collected. The discharge rates were collected throughout testing using dedicated flowmeters and periodically verified with a stopwatch and totalizer readings.

Aquifer pumping testswere conducted on one well at a time to avoid potential interference between the wells. It is not anticipated, within the operational planning of this project, that conditions are likely to require both wells to be pumped simultaneously to meet projected water demands. As such, pumping occurred in one well at a time while water levels were monitored in all three wells.

Ground water samples were collected from Well 1 and Well 2 at the end of each ten-day pumping test. No precipitation events occurred throughout the duration of the testing and recovery monitoring.

The discharged water from Well 1 and Well 2 was piped into a 5,000 gallon capacity storage tank positioned near Well 2. Inside the water tank was a float mechanism. When the float was actuated an external-auxillary booster pump powered on and sent the stored water to the discharge area (via PVC piping). The discharged water was dispersed through holes drilled in the most distal PVC sections within the discharge area. Water was spread (sprayed) across ~140 linear feet within the discharge area approved in the site-specific Waste Discharge Permit. The discharge area was continually monitored for ground saturation or any indications of washout. No saturation or washouts were observed throughout the testing.

The booster pump and submersible pumps used during testing were powered by diesel generators stationed adjacent to Well 2. One generator was sufficient to power the needs of the site however, to ensure testing integrity a second generator was wired to the primary to automatically take over should the primary generator fail.

Prior to beginning the ten-day pumping portion of the testing, each well was individually pumped for two (2) hours, per State Board testing procedures, to establish a secondary static water level. This procedure is required to assess adequate recovery at the conclusion of the pumping portion of each test.

Field personel remained at the project site around the clock throughout all testing. Field operations were conducted in 12 hour shifts and included monitoring the discharge area, documenting water levels in all three wells, regulating discharge flow rates, fueling the generators, collect water samples, document all site operations, and provide general site security.

The following are brief descriptions of each pumping test, in the order in which the tests were conducted, i.e. Well 2 then Well 1.

Well 2

On October 25, 2019, at 09:03 hours the pump was run for two (2) hours at an approximate rate at 40 GPM. The manually obtained static water level before pumping the well was 86.7 bgs and the pump was set at a depth of 600 feet bgs. For the purposes of further discussion, all water elevations will be referenced to above mean sea level (AMSL). In Well 2 the static water level was recorded at 3828.3 feet AMSL. The transducer recorded water elevation every 2 minutes. WRA monitored Well 1 and Well 3 during the 2-hour run and found little to no influence in their water elevations. After pumping the well for 2 hours, draw-down was measured at approximately 7.5 feet and upon terminating the 2-hour run, recovery was documented. Well 2 recovered to 3823.9 feet AMSL (95% of it's original static water level) within seven (7) minutes. The water elevation continued to recover for one hour to about 3827.2 feet AMSL before the 10-day test period was started.

On October 25, 2019, at 13:05 hours the pump was run for ten days at a rate of approximately 40 gpm. The transducer was set to record measurements in 1 minute intervals. Field personal regulated the valving throughout the duration of the test to maintain an average flow rate of 40 gpm. After the first day, the drawdown was measured at 10 feet and specific capacity was calculated at 3.7 gallons per ft. drawdown (gal/ft. DD.).

The pumping test was completed on November 4, 2019, at 12:15. The total drawdown at the end of the 10 day pump testing was 18.2 feet with a pumping groundwater elevation of approximately 3810.1 AMSL and a specific capacity of 2.1 gal/ft. DD. Before terminating the pumping test, groundwater samples were collected from the dedicated sample port on the discharge piping.

At the conclusion of pumping, the water level recovery rate was monitored for a period of approximately 17 days. During the recovery period for Well 2, Well 1 was pumped and little to no change in water elevation was observed in either Well 2 or Well 3. The water level in Well 2 recovered within 3 days to 91.035 feet bgs (3823.9 feet AMSL). After 10 days of recovery, Well 2 recovered to 88.8 feet bgs (3826.2 feet AMSL) or, 97% of its static water level. After 17 days, recovery reached 99% with a water elevation of 87.9 feet bgs (3827 feet AMSL).

The following table summarizes the Well 2 pumping test:

Table 3 – Well 2 Pump Test Summary

Start	End	Avg. Flow, GPM	SWL, feet	PWL, feet	Drawdown, feet	Specific capacity(End)
Oct. 25, 12:05	Nov. 4, 12:15	40	3828.3	3810.1	18.2	2.1

The following table summarizes the Well 2 water level recovery measurements:

Table 4 – Well 2 Recovery Test Summary

Start	End	SWL,feet – start	SWL, feet – End	Time to 95% recovery
Nov. 4, 12:16	Nov. 21, 14:59	3810.1	3827.0	4097 Min./ 2.85 Days

Well 1

WRA pumped Well 1 for two (2) hours prior to initiating the ten-day pumping test. The purpose of the two-hour test was to establish the static water level (SWL), per State Water Board guidelines, to be used for assessing water level recovery at the conclusion of the ten-day test.

On November 8, 2019, at 12:06 hours the pump was run for two hours at 40 gpm. The SWL was measured at 3821.3 feet AMSL and the pump was set at a depth of 600 feet bgs. The transducer was programmed to record water elevation every minute. Well 2 and Well 3 water elevations appeared un-affected during the initial 2-hour pumping of Well 1. Well 1 drew down approximately 22.45 feet and upon termination of the 2-hour run, water level recovery was measured. Well 2 recovered to 13.335 feet bgs (95% of it's original SWL or 3820.6 feet AMSL) within one hundred seventy five (175) minutes.

On November 8, 2019, at 17:15 hours the pump was run for ten days at a rate at 40 gpm. The transducer recorded water elevation every minute. Field personal maintained an average flow rate of 40 gpm. Drawdown after the first 24 hours was measured at 25.6 feet with a specific capacity of 1.5 gal/ft. DD.

The pumping test on Well 1 was completed on November 18, 2019, at 12:58. The total drawdown at the end of the 10-day pumping test was 28.5 feet with a pumping water level of approximately 3792.75 feet AMSL, and a specific capacity of 2.1 gal/ft. DD. Groundwater samples were collected from the dedicated sample port and submitted to BSK Labortories for State monitored chemical constituents (Title 22 analysis) and Per- and polyfluoroalkyl substances (PFAS).

Upon completion of the pumping test, recovery measurements were collected in Well 1 for a period of approximately 3.2 days via the transducer. Subsequently, the transducer was removed by WRA. After five (5) days, WRA returned to the site and manually gathered a final water level measurement. The final water level measurement on November 26, 2019 was 3821.0 feet AMSL, reaching slightly less than 95% recovery (3820.665 ft AMSL).

The following table summarizes the Well 1 pumping test:

Table 5 – Well 1 Pumping Test Summary

Start	End	Avg. Flow, GPM	SWL, feet	PWL, feet	Drawdown, feet	Specific capacity(End)
Nov. 08, 17:15	Nov. 18, 12:58	40	3821.3	3792.7	28.6	1.4

The following table summarizes the Well 1 water level recovery test, post ten-day pumping

Table 6 Well 1 Recovery Test Summary

Start	End	PWL,feet – End	SWL, feet – End	Time to 95% recovery
Nov. 18, 12:58	Nov. 26, 14:30	3792.75	3821.0	11612 Min./ 8.06 Days

Well 3

Groundwater elevations were measured for approximately 34 days in Well 3 beginning on October 18, 2019, at 16:23 hours. Well 3 groundwater elevations were recorded every 2 minutes throughout the testing and recovery monitoring of Well 2 and Well 1. The manually recorded static water level was measured at 3755.8 AMSL on October 18, 2019. Transducer readings documented variation in groundwater elevation of 1.3 feet (greatest water elevation – lowest water elevation) from October 18 through November 21, 2019.

On October 25, 2019, beginning at 09:03 the 2 hour run began at Well 2. At Well 3, between 08:28 and 12:00 (just before, during, and after recovery), the water elevation flucuated 0.71 feet. During 10 day pump testing at Well 2 the water elevation in Well 3 flucuated 0.97 feet. During the recovery period of Well 2, the water elevation in Well 3 flucuated 0.90 feet.

On November 8, 2019 , beginning at 12:06, the 2 hour run began at Well 1. At Well 3, between 11:35 and 17:15 (just before, during, and up to recovery), the water elevation flucuated 0.72 feet. During 10 day pump testing at Well 1, the water elevation in Well 3 flucuated 0.88 feet. During the recovery period of Well 1, the water elevation in Well 3 flucuated 0.95 feet.

The last transducer measurement in Well 3 was collected on November 21, 2019, at 13:53 and was 3756.1 feet AMSL. A final, manual static water level measurement of 45.0 feet was collected on November 26, 2019.

Water Sampling and Analysis

WRA collected groundwater samples from each of the pumped wells shortly before completion of test pumping operations. The samples were collected in clean, laboratory supplied bottles from BSK Associates (BSK). Sampling took place using the dedicated sample port located on the discharge piping of Well 1 and Well 2, approximately 12 inches downstream of the flow meter. All sampling was conducted using laytex glovesthat were supplied by the lab to reduce the risk of potential sample contaminantion.

Samples were analyzed for Title 22 and PFAS constituents as specified by the California State Water Quality Control Board (CSWQCB). Submission of the samples to the lab occurred under standard chain of custody protocol, including the quality assurance/quality control sample blanks. The sample bottles were delivered to the labaratory facility in chilled coolers and kept under

WRA's custody the entire time. Labatory test methods utilzed are summarized in Table 7 below and meet all requirements and standards established by both State and Federal regulations.

Table 7 Water Sample Analytical Methods

General Chemistry	**Method**	**Organics**	**Method**
General	SM 2320B	*EDB and DBCP*	EPA 504.1
	EPA 300.0	*Organohalide Pesticides and PCBs*	EPA 505
	SM 2120B	*Chlorinated Acid Herbicides*	EPA 515.4
	SM 4500-H+ B	*VOC (Volatile Organic Compounds)*	EPA 524.2
	SM 2510B	*Semi-VOC*	EPA 525.3
	SM 2330B	*Carbamates*	EPA 531.1
	SM 5540C	*Glyphosate*	EPA 547
	SM 2150B	*Endothall*	EPA 548.1
	EPA 314.0	*Diquat*	EPA 549.2
	SM 2540C	*1,2,3-Trichloropropane*	SRL 524M-TCP
	SM 2130B		
Metals	EPA 200.7	*Per- and Polyfluoroalkyl Substances (PFAS)*	EPA 537.1
	EPA 200.8		
	SM 2340B		
	EPA 900.0		

On November 4, 2019, at about 12:00 hours, water quality samples were collected from Well 2 and transported to BSK. The samples were submitted to the lab at 16:30 hours on November 4, 2019. Samples were analyzed for all parameters required by the CSWQCB (Title 22 constituents) and PFAS, a constituent of growing concern.

The Well 1 samples were collected on November 18, 2019, at 13:00 hours. The samples were transported to BSK on November 19, 2019, and submitted to the lab at 12:50 hours. Samples were analyzed for all parameters required by the CSWQCB (Title 22 constituents) and PFAS.

FINDINGS

The following section presents a discussion of our findings, with respect to the hydrogeologic assessment of the fractured bedrock aquifer beneath the project site. The discussion focusess on groundwater elevations and quality, along with water supply and demand for the project.

Fracture trace mapping

Four apparent fracture traces were identified within the property boundary of the project site. Very little indications of these fractures were readily apparent on the ground in the form of in-place bedrock outcrops, and as such test hole locations were approximated. Where possible, test holes were sited in locations that appeared to be the intersection of at least two of the fracture traces.

Test Hole Drilling & Well Completion

Drill cuttings and drilling conditions reflected the anticipated localized geologic and drilling conditions. Approximately 100 feet of unconsolidated decomposed granitic material was encountered at all three test hole locations. Permanent steel conductor casings were placed to about 100 feet (per County requirements) in Well 1, Well 2, and Well 3 respectively.

At least three fracture sets were encountered during the drilling of each test hole. The shallower fracture sets encountered in Well 1 and Well 2 appeared to likely be more productive than the deeper fractures, whereas in Well 3 the fractures appeared to likely to produce about equal quantities of groundwater.

First encountered groundwater in the test holes was roughly at the interface between the decomposed granitic overburden and the more competent fractured bedrock. The manually measured static water levels in the completed wells were recorded as considerably shallower in all three wells. This suggests that groundwater rose up inside the cased portion of the well, which was sealed off from the surrounding decompose granite by the concrete seals. This suggests that the portion of the fractured bedrock aquifer system in which the three wells are completed are at least partially pressurized (artesian).

Background static water levels

Once the three wells were surface cased and completed to final depth, background static water levels were measured and recorded electronically. The background water levels were necessary to assess the potential for offsite influences (e.g. other pumped wells) to onsite water levels, particularly during the two planned pump tests. The following summarizes those measurements:

- Well 1: approximately 3,834 feet AMSL, or 12.0 feet as depth to water
- Well 2: approximately 3,915 feet AMSL, or 86.7 feet as depth to water
- Well 3: approximately 3, 801 feet AMSL, or 45.2 feet as depth to water

The recovery and re-installation of the electronic monitoring equipment influenced these background measurements, in that there was approximatle four (4) feet of apparent offset between the original water background static water levels and those recorded after the test pumps were placed into Well 2 and the instruments returned down the well. Such offset did not occur in Well 1. We are confident the offset was correctly accounted for in our assessment of groundwater elevations.

As discussed previously, fluctuations of the background static water levels were recorded during the pump testing. The fluctuations appear to be uniformwith relatively constant average water levels. This finding suggests these fluctuations are normal variations within the fracture system.

Pumping Tests

Most available groundwater resources in the region (encapsulating the project site) are contained within the Sierra Nevada Geomorphic province. Groundwater is found in fractures within the granitic, volcanic, and metamorphic rocks of this region.

It is extremely difficult to predict sustainable yield and storage capacity for fractured bedrock aquifers, which are the sole source of groundwater for the project. The lack of regional information on the fractured bedrock aquifers, the absence of readily available well hydrographs for the area, and a reliable data base of groundwater conditions in this area, all contribute to reducing effective estimates of sustainability from groundwater sources.

Classical pump testing data analysis is based on methods established primarily for unconsolidated aquifers and not for fractured bedrock aquifers. Some analysis for pump testing does exist for fractured aquifers, but even then are often for cases where observation wells are available. We have made efforts to interpret the data relative to well behavior and aquifer behavior with respect to groundwater sustainability.

Ten (10) day long pumping tests wereconducted on Well 2 and Well 1, individually, in that order. All three wells were electronically monitored throughout the pumping and recovery periods. The discharge rate (40 gpm) was selected as twice the "acceptable" discharge rate to be included in the testing program. This rate is in excess of twice what the project would need to sustain water demand for the completed project. In addition, we considered a discharge rate of 40 gpm to be strenuous with respect to the anticipated aquifer performance in the area.

Each test revealed the following:

- Both wells were able to sustain discharge rates of about 40 GPM without significant adjustments to the flow rates. This was because we did not encounter rapidly dropping pumping water levels, and as such the wells (and aquifer) were able to readily sustain this flow rate.
- Maximum drawdowns of about 18 and 28 feet were recorded and reported for Well 2 and Well 1 respectively. The drawdown reached "stable" conditions, i.e. a predictable decline in the pumping water level of a fixed period of time, within about 100 minutes at Well 2, and 300 minutes at Well 1.
- Stable drawdown continued through the course of each ten-day test period.

Fractured bedrock aquifers behave differently than alluvial aquifers. Traditional analysis of well and aquifer hydraulics, based on unconsolidated aquifer analysis methods, are not easily transferrable to fractured bedrock aquifers. The lack of direct hydraulic connection between wells, or at least an easily definable connection, makes observations between wells questionable with respect to calculating aquifer behavior.

Figure 6 presents the semilogarithmic plot of drawdown over time in Well 1. Pumping in Well 1 began after the completion of pumping and recovery in Well 2 and after pumping and recovery of wells at the neighboring project site to the north. The shift in the measured water level at about 270 minutes is interpreted as an adjustment in the flow rate from about 42 gpm to 40 gpm and was confirmed with WRA staff's field notes.

Figure 7 shows a highlight of time between 1,000 and 10,000 minutes with calculations of the transmissivity values based on the traditional Cooper-Jacob straight line method (Cooper, Jacob

1946), and the Huntley method (Huntley et al, 1992). The two values, while slightly different, are generally consistent. Huntly offered that the CJSM tends to overestimate the transmissivity values and that his method is based on specific capacity comparisons, yields better (lower) values for in fractured bedrock. In the case of Well 1 and Well 2 the transmissivity values are roughly the same. In the case of Well 1 transmissivity values are considered "good" for domestic use, based on unconfined aquifer conditions (Ground Water Manual, 1981).

Figure 8 shows water level recovery measurements for Well 1. The recovery "target" groundwater elevation for the ten days of recovery was 3820.6 feet AMSL. Full recovery to this elevation occurred about 8 days after the cessation of pumping operations. Again, the complex nature of fracture flow makes assessing and estimating aquifer hydraulics difficult. This is particularly true for recovery in these types of aquifers. The apparent transmissivity calculated for Well 1 during recovery seems reasonable and within range of the values calculated from pumping. Often, recovery values are more indicative than pumping values of transmissivity. However, in fractured bedrock, the interconnectivity of the fractures acts as secondary porosity and makes it very difficult to accurately assess the relationship between the two values.

Figure 9 shows a highlight of recovery time in Well 1 between 400 and 4,000 minutes along with calculations of the transmissivity based on the traditional Cooper-Jacob straight line method (Cooper, Jacob 1946),

Figure 10 presents the semilogarithmic plot of drawdown over time in Well 2. Pumping in Well 2 occurred prior to pumping in Well 1 and was essentially coincident with pumping on the neighboring project to the north.

Figure 11 shows a highlight of time between 1,000 and 10,000 minutes in Well 2 with calculations of the transmissivity based on the traditional Cooper-Jacob straight line method (Cooper, Jacob 1946), and the Huntley method (Huntley, 1994). The two values, while different, are similar to transmissivities found at Well 1. In Well 2 the T values are considered "good" for domestic use based on unconfined aquifers conditiions (Ground Water Manual, 1981).

Figure 12 shows water level recovery measurements after the Well 2 pumping was completed. The recovery "target" groundwater elevation for the ten days of recovery was 3823.96 feet AMSL. Full recovery to this elevation occurred about 3 days after the cessation of pumping operations.

Figure 13 shows a highlight of recovery time in Well 2 between 1,000 and 10,000 minutes with calculations of the transmissivity based on the traditional Cooper-Jacob straight line method (Cooper, Jacob 1946).

Our assessment of the well and aquifer hydraulics suggest:

- The specific capacities calcuated are not unusual for fractured bedrock aquifers but are on the high end of the range we anticipated. The specific capacity of a well is the flow rate divided by the drawdown, and in general it can be expected for the specific capacity to decrease over the period of the pumping test. Typical specific capacity values for fractured bedrock aquifers would range from 0.1 to 1.0 gallons per minute per foot of drawdown. The range of specific capacity for Well 1 and Well 2 were 0.78 to 1.34, respectively.
- Transmissivities calculated from the pumping portion of the test (1,000 to 10,000 minutes), and using classical analysis from Cooper Jacob revealed a range from 205.2 (Well 2) to 354.7 (Well 1) in cubic feet/day; with Huntley the transmissivity values range from 310.2 (Well 2) to 591.7 in cubic feet/day. Storativity could not be calculated for either well due to the absent to neglible effects of onsite pumping on nearby wells.

Well 1 and Well 2 both experienced significantly less drawdown than was initially anticipated for fractured bedrock aquifers. The static water levels, as mentioned above, had risen above their first encounted depth during drilling operations. As such, as we have discussed, it is possible that these wells are located in a paritally pressurized portion of the overall fracture system. The pump testing revealed that there is very little hydraulic communication between Well 1 and Well 2, and none between either of these wells and Well 3.

A concern always present in aquifer tests is the possibility that water from the discharge area has somehow artificially recharged the aquifer, making it appear that the drawdown is less pronounced, and the flow rate higher than it would actually be in the absence of the artificial recharge. Our assessment is that there was no influence resulting from artificial recharge for the following reasons:

- There were no erratic changes in water levels during the pumping test on either Well 1 or Well 2. The consistent drawdown was established early on in each test, and no indication of recharge (e.g. sudden rise in the pumping water level, suggestive of a influx of water to the well system or fracture) was measured.

- There were no erratic changes in the flow rate from the pumps, again something that would have suggested a sudden influx of water to the well or fracture system from the discharge area.
- It is unlikely that if discharged groundwater was returning to the area of the pumping well that all of it would find its way to the pumping well and only the pumping well. As such, if some kind of artificial recharge was actually occurring then we would have expected to see some indication (e.g. a rise in the static water level) in one of the non-pumping wells, which we did not.

Water levels in Well 3 were unaffected by the pumping in either the two onsite wells (Well 1 and Well 2) or from pumping at the project to the north. Figure 13 shows the groundwater elevation in Well 3 during both the onsite and offsite pumping tests. There is no discernable fluctuation in the groundwater elevation as a result of the pumping tests, either onsite or offsite (to the north), as the maximum recorded fluctuation appears to be only slightly greater than one (1) foot.

Figure 14 shows the groundwater elevation for both the project wells (Wells 1, 2, & 3) and for the wells on the project to the north, as a group, during the course of the pumping tests on both project sites. The following are our findings when specifically considering this figure:

- Groundwater elevation measurements in Well 3 appear to be unchanged, either by the onsite or offsite pumping.
- Groundwater elevation measurements in Well 1 appear unaffected, in general, when pumping occurred in Well 2. However, a slight decline can be seen when the wells on the project to the north begin pumping, and then a slight rise in the groundwater elevation occurred while Well 2 was still pumping, when the wells on the project to the north stopped pumping, suggesting that there was some influence from those offsite wells.
- Minor influence on Well 1 groundwater elevations were also observed upon terminating the pumping test at Well 2; the water elevation in Well 1 rises, seemly in conjunction with the end of testing.
- When Well 2 and the wells for the project to the north are all into recovery, Well 1 begins pumping with no real discernable influence on any of these three wells.
- Overall, there appears to be little or no influence between the onsite project wells on each other, but some slight influence on Well 1 as a result of pumping of the offsite wells for the project to the north.

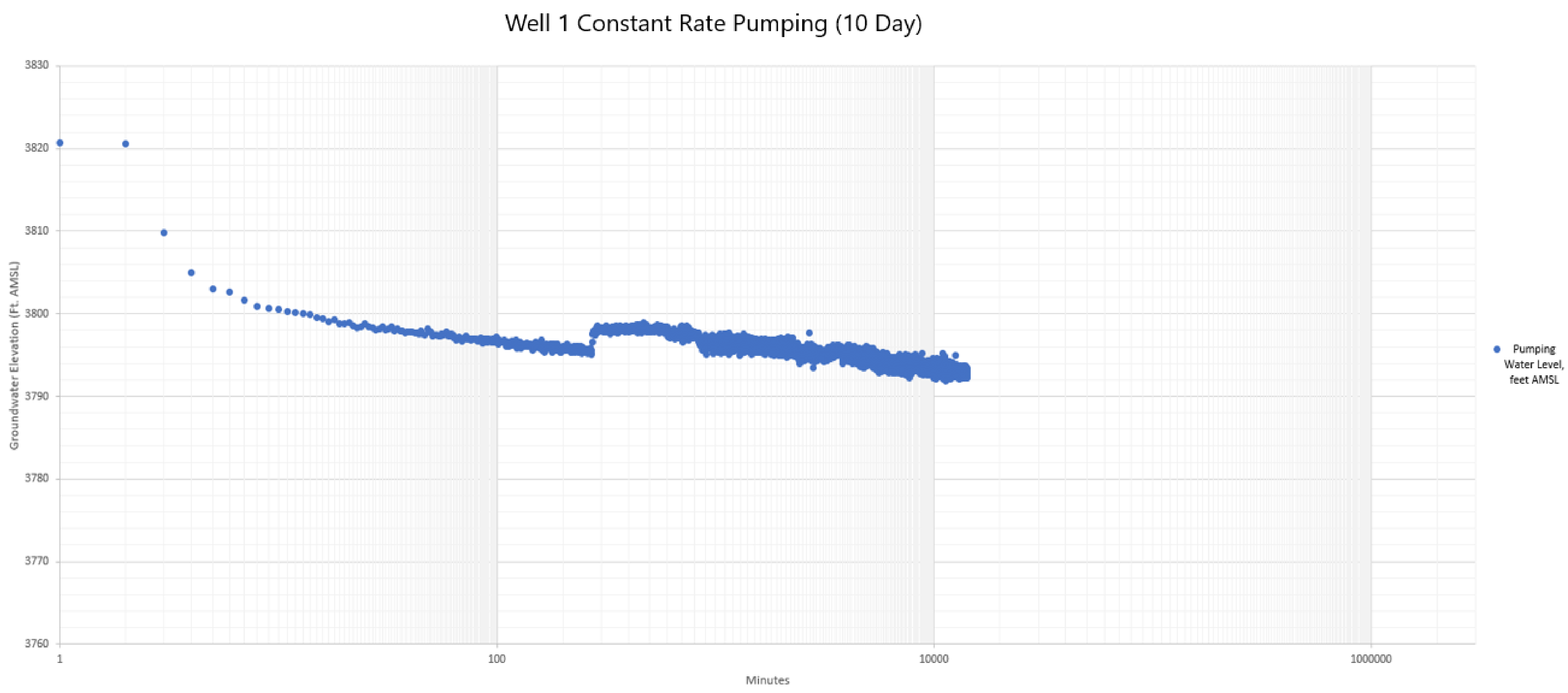

Figure 6
Well 1 Constant Rate Pumping Test 10 Day

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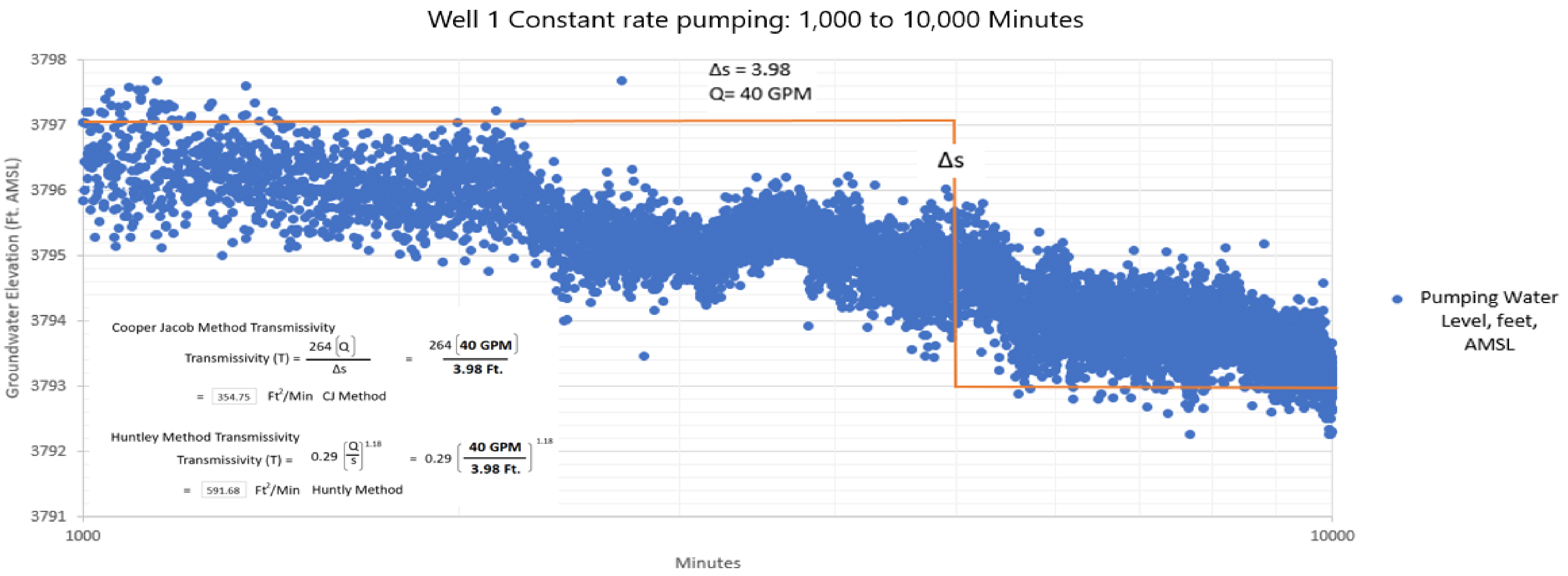

Figure 7
Well 1 Constant Rate Pumping test 1,000-10,000 min

Water Resources Associates, Inc

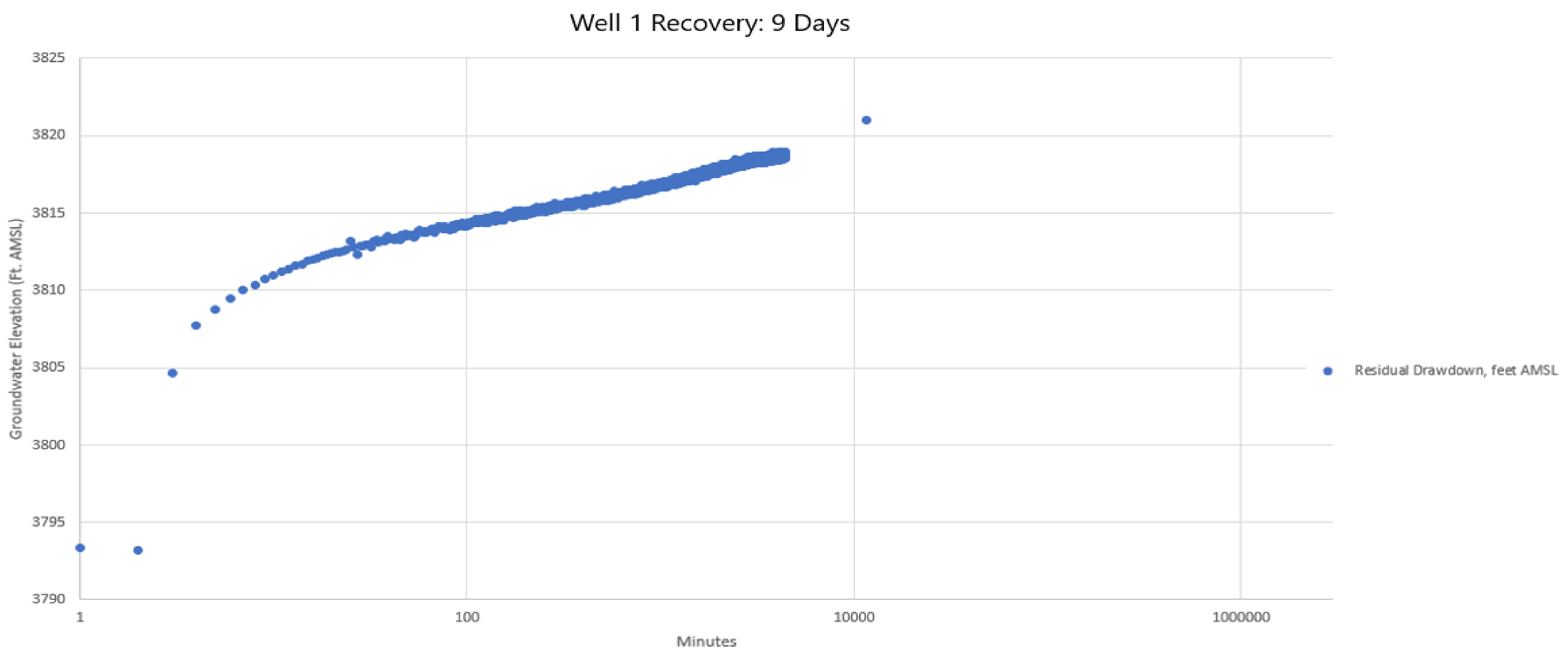

Figure 8
Well 1: 9 Day Recovery

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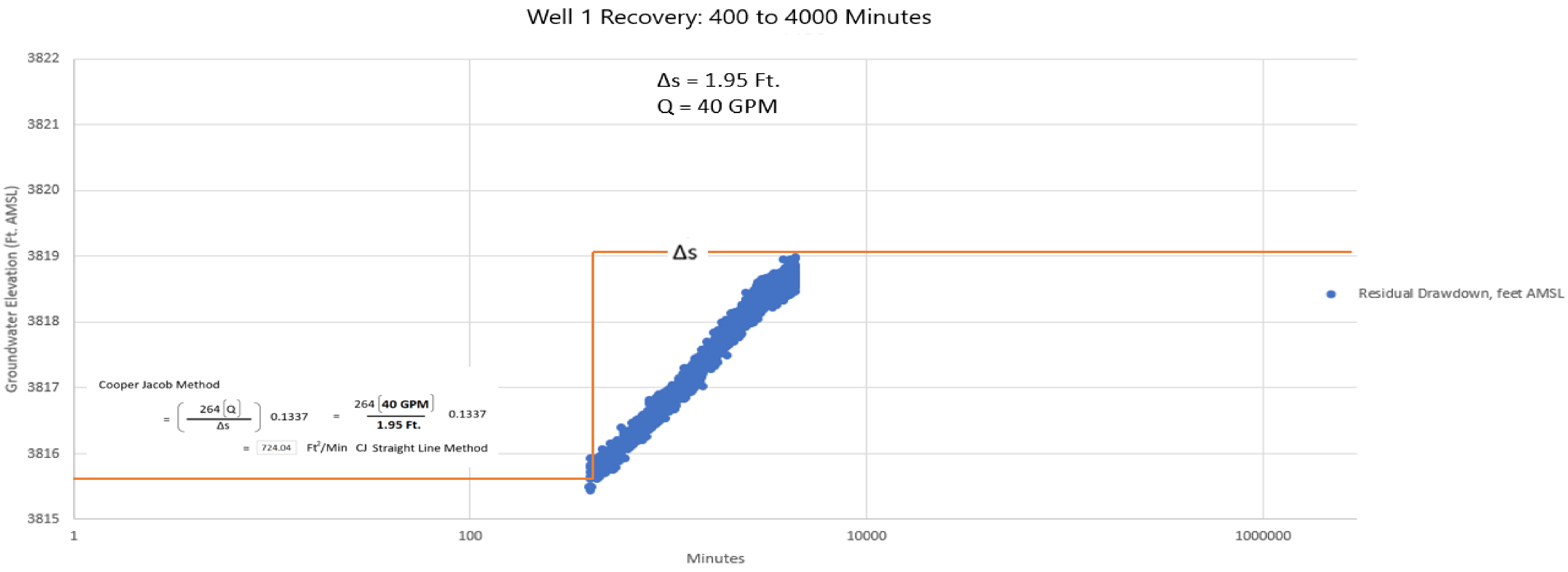

Figure 9
Well 1 400 to 4,000 minute Recovery

Water Resources Associates, Inc

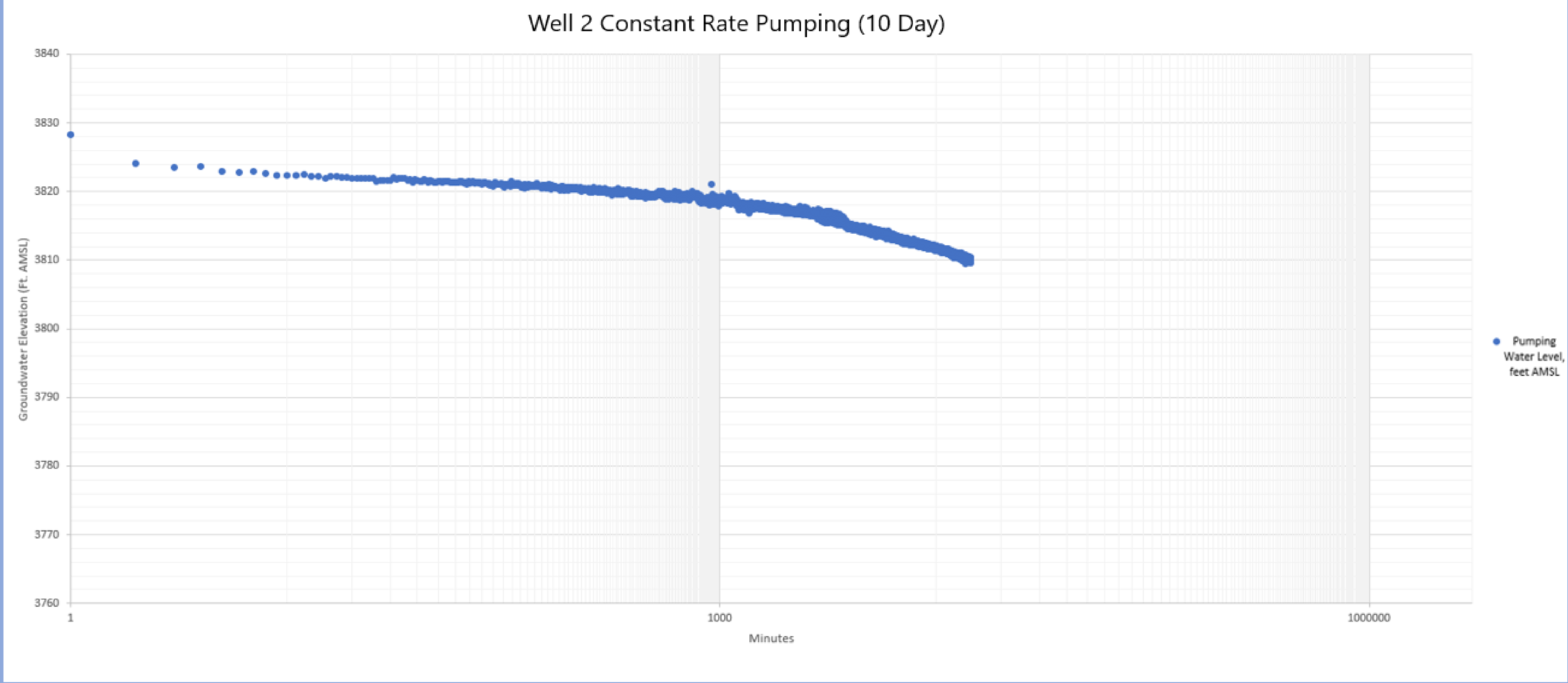

Figure 10
Well 2: 10 Day Constant Rate Pumping

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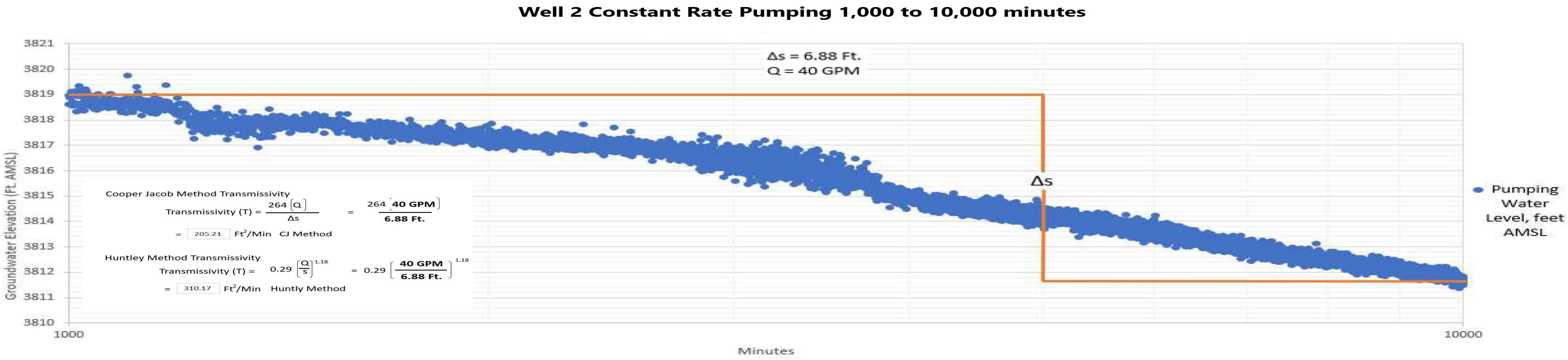

Figure 11
Well 2 Constant Rate Pumping 1,000 to 10,000 min

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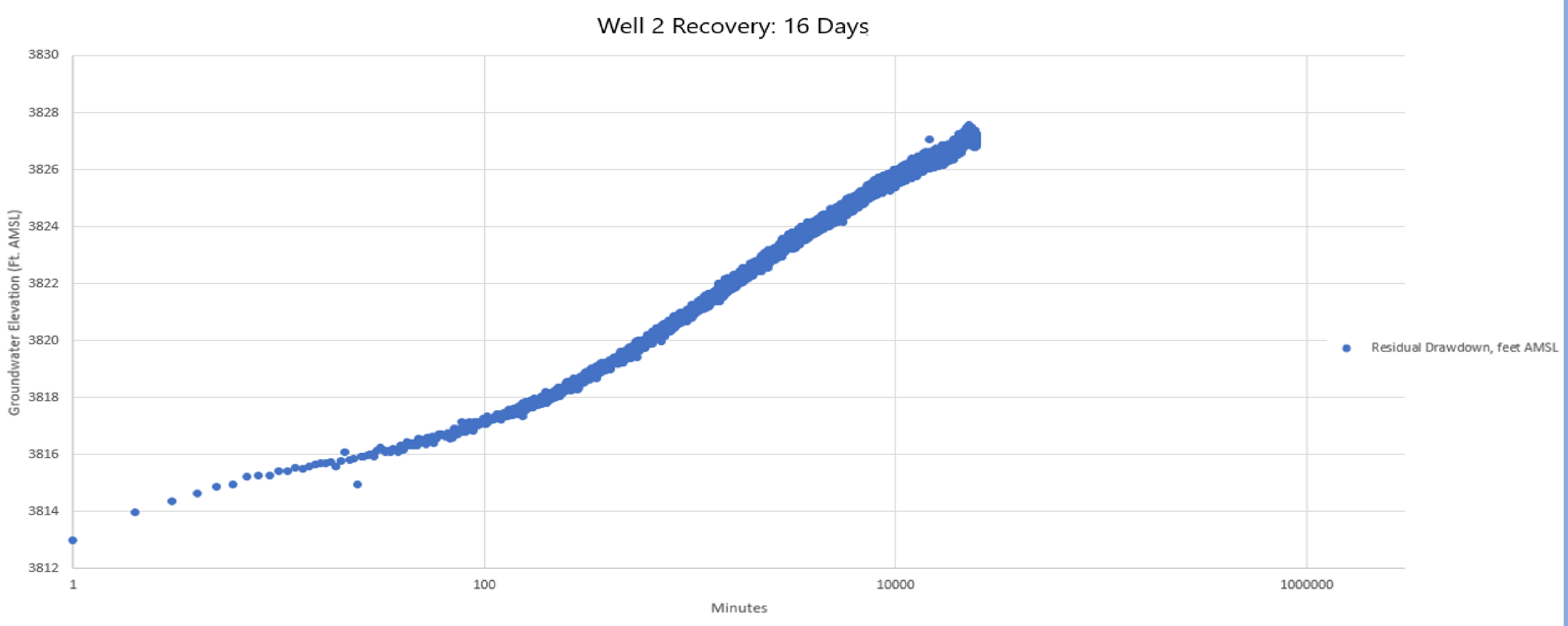

Figure 12
Well 2: 16 Day Recovery

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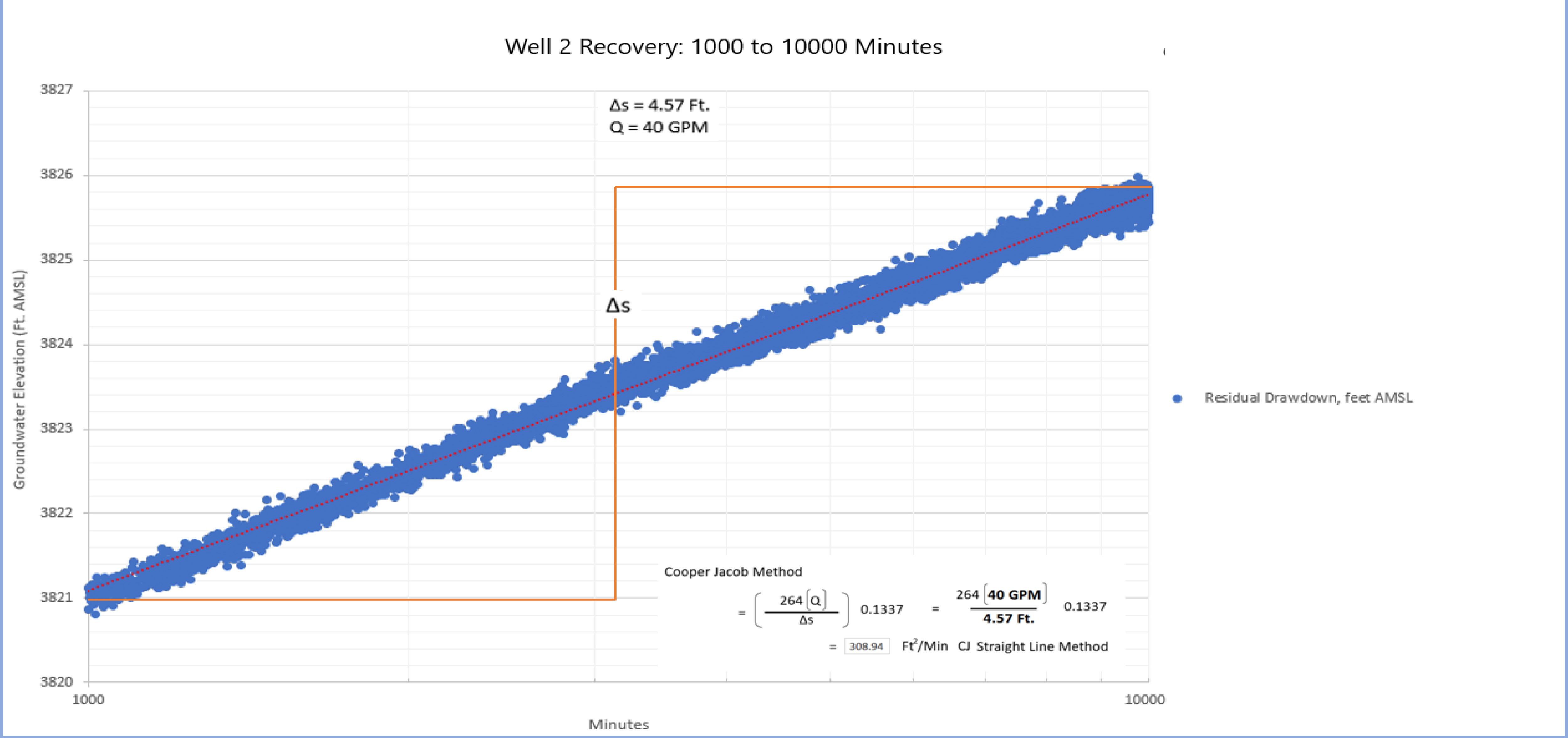

Figure 13
Well 2: Recovery 1,000 to 10,000 Minutes

Water Resources Associates, Inc

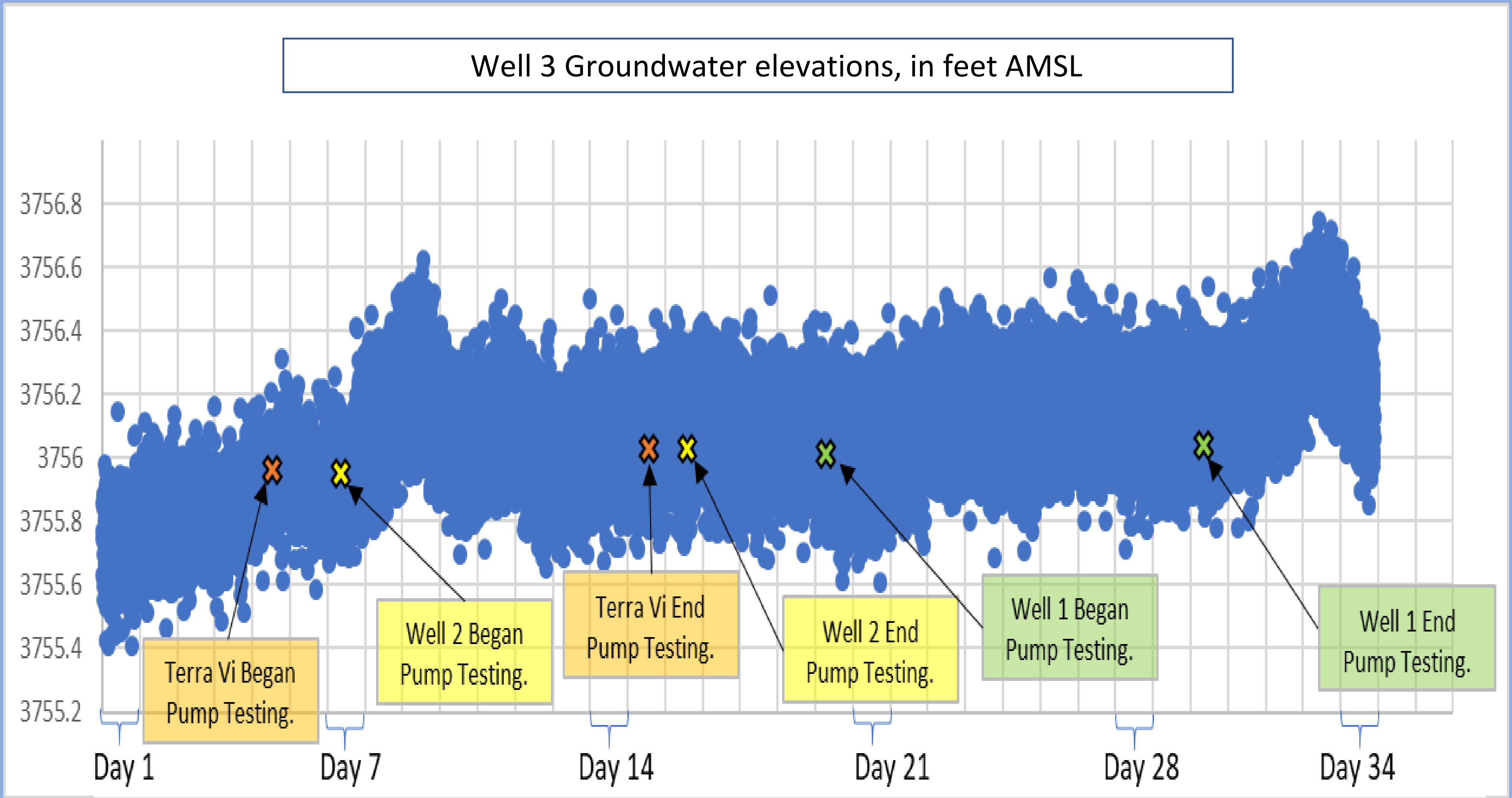

Figure 14
Well 3 Groundwater Elevations

Water Resources Associates, Inc

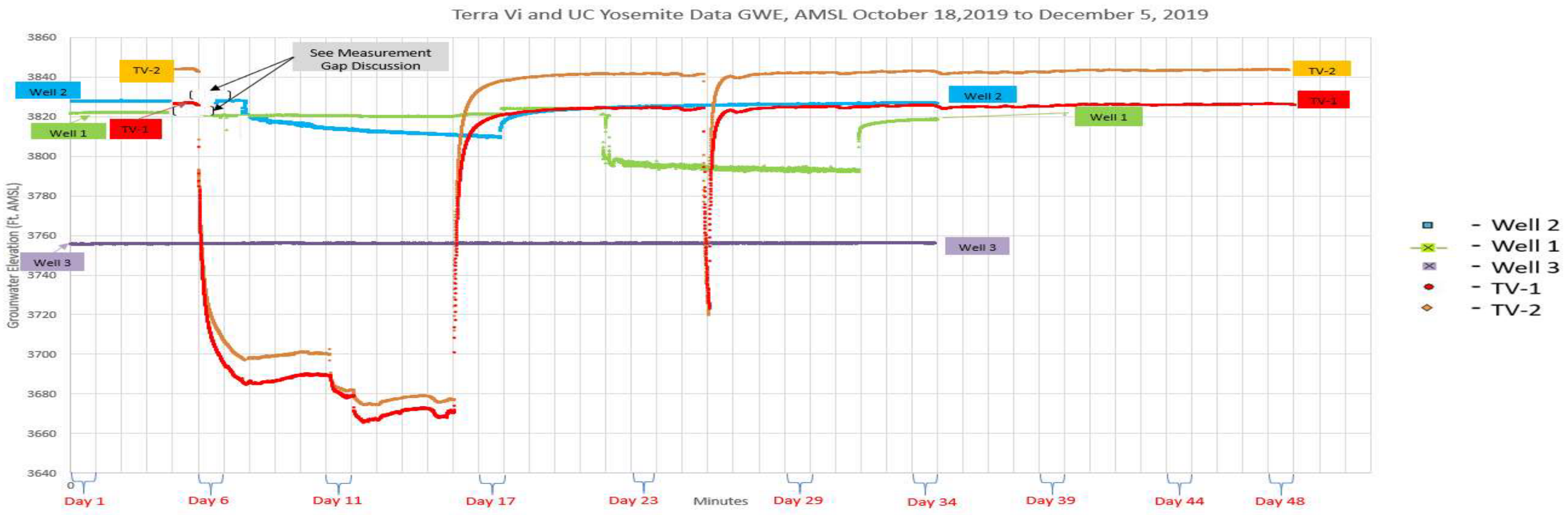

Figure 15
Terra Vi and UC Yosemite Data Oct-Dec 2019
Water Resources Associates, Inc

Table 8 – Aquifer Test Analysis Summary

	Water Elevation (SWL) Ft.	GPM	Drawdown (Ft.)	Q/ds (Ft^2/min)	Transmissivity(cj) (Ft.^{2}/day)	Transmissivity(h) (Ft.^{2}/day)	Residual Drawdown (Ft.)	Recovery Transmissivi (Ft.^{2}/day)
UC								
TH-1	3821.30	40	3.98	1.34	354.74	591.68	1.95	724.04
TH-2	3828.30	40	6.88	0.78	205.21	310.17	4.57	308.94
TV								
TV1	3826.85	27.4	30.51	0.12	31.70	34.23	11.84	81.68
TV2	3843.86	25.4	35.91	0.09	24.97	25.83	12.08	74.22

Water quality

Onsite groundwater can be described as a calcium bicarbonate type water, calcium being the most prevalent inorganic cation, and bicarbonate being the most prevalent anion in the two water samples collected from Well 1 and Well 2, respectively. This is typical water chemistry for groundwater in fractured granitic bedrock aquifers in the high Sierra Mountains. Table 7 below, summarizes the reported constituents in the two submitted water samples. A summary of our water quality findings is as follows:

- Well 1 is more mineralized than Well 2, but not significantly more. Overall, the water chemistry for both wells is quite similar, almost identical.
- No manmade constituents were reported, other that the detection of toluene in the Well 2 sample but not in Well 1. We account for this by noting that the diesel-powered generator was positioned adjacent to Well 2 and it is almost certain that the toluene detected in that sample is the result of diesel exhaust in the air.
- The gross alpha activity was below the current action level, which indicates that dissolved uranium analysis is not required by the State, and as such uranium analyses were not conducted. Generally, when the gross alpha particle activity "threshold" of 0.67 pCi/L is exceeded, this automatically triggers a uranium analysis. However, if the gross alpha particle activity is below that threshold, then the analysis is not conducted.

Figure 15 is a "Stiff" diagramthat graphically represents water chemistry as polygons, using similar cations and anions, along with the concentrations represented in milliequivalents per liter (meq/l). The shape of the polygon is a visual representation of the water chemistry, the size of the polygon a visual representation of the mineralization of the water, i.e. larger polygons have higher total dissolved solids concentrations. Figure 14 compares the polygons for Well 1 and Well 2, and it is obvious that both are of a nearly identical shape, and that the Well 1 poloygon is slightly largerdue to the slightly greater mineralization in Well 1.

Figure 16 is a "Piper" trilinear diagram that is used to group water samples visually for easier comparison. The two triangles on the lower left and right represent cations and anions, respectively, as a percentage of the sample mineralization. The cation triangle shows the larger variation between magnesium and calcium for the two samples. The anion triangle shows little variation between the two samples. The central diamond shape is a combination of the two lower triangles and shows that while the two samples plot slightly apart they are still close enough to qualify as nearly the same water chemistry.

Table 9 – Summary of Water Quality Laboratory Analyses

Analyte	Units	Well 1	Well 2	State of Califonia Drinking Water Standard
Aggressive Index	--	10	10	~
Alkalinity (CaCO3)	*mg/L*	76	56	~
Aluminum	*mg/L*	ND	ND	0.2**, 1.0*
Antimony (Sb)	*ug/L*	ND	ND	6*
Arsenic (Total)	*ug/L*	ND	ND	10*
Barium (Ba)	*mg/L*	ND	ND	1*
Beryllium (Be)	*ug/L*	ND	ND	4*
Bicarbonate (CaCO3)	*mg/L*	76	56	~
Cadmium	*ug/L*	ND	ND	5*
Calcium	*mg/L*	18	12	~
Carbonate (CaCO3)	*mg/L*	ND	ND	~
Chloride	*mg/L*	1.1	ND	250***
Chromium (total Cr)	*ug/L*	ND	ND	50*
Color, Apparent	*CU*	ND	5.0	15
Color, pH (1)	*pH Units*	6.7	6.4	
Conductivity	*umho/cm*	140	110	900***
Copper	*ug/L*	ND	ND	1300*, 1000**
Cyanide (Cn)	*mg/L*			0.15*
Fluoride	*mg/L*	ND	ND	2.0
Hardness (CaCO3)	*mg/L*	63	45	~
Hydroxide(CaCO3)	*mg/L*	ND	ND	~
Iron (Total)	*mg/L*	ND	ND	0.3**
Langelier Index	--	-1.6	-1.9	--
Analyte	**Units**	**Well 1 results**	**Well 2 results**	**State of Califonia Drinking Water Standard**
Lead (Pb)	*ug/L*	ND	ND	15*
Magnesium	*mg/L*	4.3	3.6	~
Manganese (Total)	*mg/L*	ND	ND	0.05**
MBAS	*mg/L*	ND	ND	0.5
Mercury (Hg)	*ug/L*	ND	ND	2.0*

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Consulting Hydrogeology

Nickel (Ni)	*ug/L*	ND	ND	100*
Nitrate + Nitrite as N	*mg/L*	ND	ND	
Nitrate (NO3)	*mg/L*	ND	ND	45*
Nitrite (N)	*mg/L*	ND	ND	1*
Odor	*TON*	ND	ND	3**
Perchlorate	*ug/L*	ND	ND	
pH	*units*	6.8	6.8	6.5-8.5**
ph Temperature in *C		21.7	21.9	
Potassium	*mg/L*	ND	ND	~
Selenium- Total (Se)	*ug/L*	ND	ND	50*
Silver	*mg/L*	ND	ND	.1**
Sodium	*mg/L*	6.9	5.3	~
Sulfate (SO4)	*mg/L*	ND	ND	250***
Thallium (TI)	*ug/L*	ND	ND	2.0*
Total Dissolved Solids	*mg/L*	120	87	
TDS	*mg/L*			500***
Toluene	*ug/L*	ND	1.0	
Turbidity	*NTU*	0.22	0.11	5.0**
Zinc (Zn)	*mg/L*	1.0	0.9	5.0**
Organics	*ug/L*			~

Analyte	Units	Well 1 results	Well 2 results	State of Califonia Drinking Water Standard
Uranium (Total)	*ug/L*	*NA*	NA	30*
Uranium (Dissolved)	*ug/L*	*NA*	NA	30*
Uranium, Radiological	*pCi/L*	*VALUE*	<0.67	
Gross Alpha	*pCi/L*	0.741	7.06 +/- 2.05	15*
EDB and DBCP by GC-ECD	*ug/L*	ND(1)	ND(1)	
Organohaide Pesticides and PCBs by GC-ECD	*ug/L*	ND(1)	ND(1)	
Chlorinated Acid Herbicides by GC-ECD	*ug/L*	ND(1)	ND(1)	
Volatile Organics by GC- MS	*ug/L*	ND(1)	ND(1)	
Semi-Volatile Organics by GC-MS	*ug/L*	ND(1)	ND(1)	
Carbamates by HPLC		ND(1)	ND(1)	
Glyphosate by HPLC	*ug/L*	ND(1)	ND(1)	
Endothall by GC-MS	*ug/L*	ND(1)	ND(1)	
Diquat by HPLC	*ug/L*	ND(1)	ND(1)	
1,2,3-Trichloropropane by GC-MS SIM	*ug/L*	ND(1)	ND(1)	
EPA Method 1613B		ND	ND	
Asbestos		ND	ND	

ND: Non-Detect- Below Laboratory detection limits

ND(1): Non-Detect for all constitutents

* Primary Drinking Water Standard

** Secondary Drinking Water Standard

*** Secondary Drinking Water Standard, Recommended

~ Standard not established

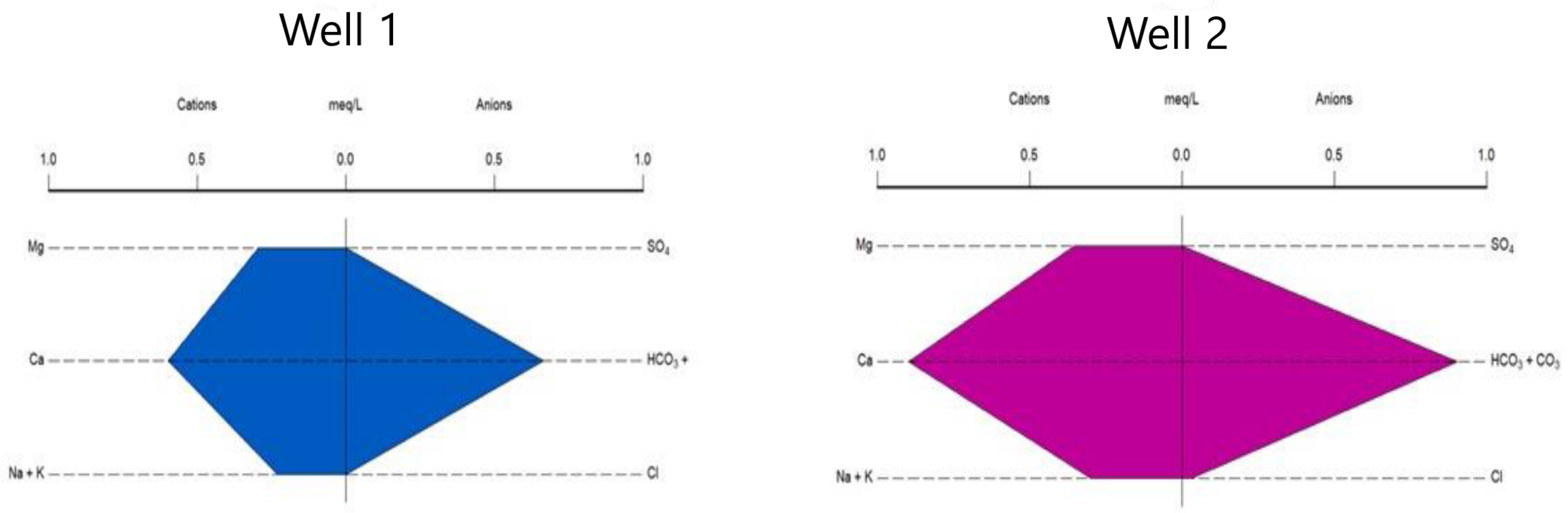

Figure 16
Wells 1 and 2 Geochemical Diagram

Water Resources Associates, Inc.

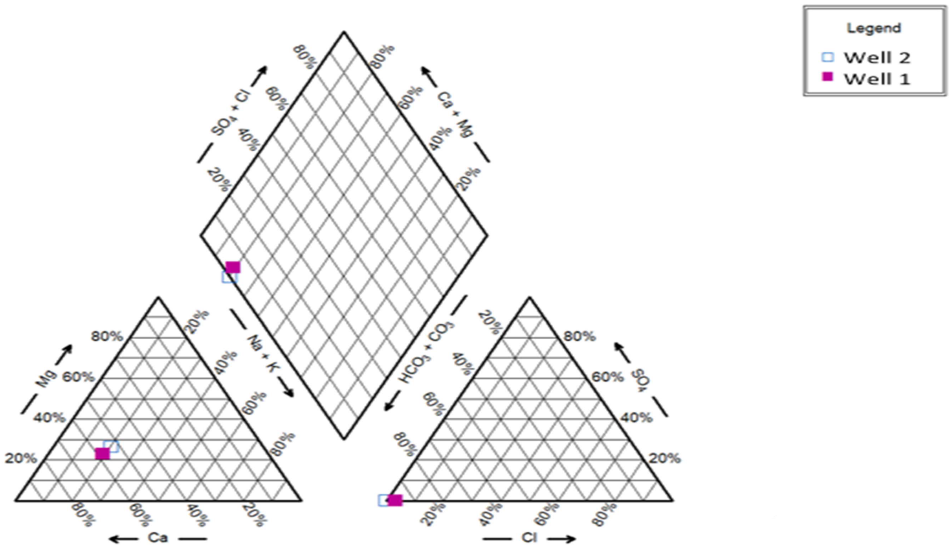

Figure 16
Piper Diagram

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The South Fork Tuolumne River watershed is defined by the national watershed classification system (USDA 2013). This system is a spatial hierarchy of eight nesting watershed size classes ranging from very large (greater than 250,000 acres) to very small (less than 2,000 acres) (Weddle and Frazier, 2014). The South Fork Tuolumne River comprises 57,855 acres, classifying it as a Hydrologic Unit Code (HUC) Level 6. The South Fork Tuolumne watershed starts in the high country of Yosemite National Park above 8,500 feet and terminates at the confluence of the South Fork with the Middlefork Tuolumne River.

The estimated groundwater recharge watershed for the project site is reprented in Figure 16, below. The overall estimated size of the recharge watershed is estimated at approximately 462 acres. There are two small local intermittent streams that flow over the project site and drain into the South Fork Tuolumne River below. These are:

- An unnamed intermittent drainage/stream apparently originating onsite, perhaps as underflow from higher in the watershed, or from an intermittent spring, herein referred to as “the Westside Drainage”. There are no known gauging stations or readily available data regarding flow for this drainage and as such this is not considered a source for groundwater recharge in our estimates of water supply onsite. This is a conservative approach that we feel supports our assessment based on the limited to non-existent data for this stream.

- An unnamed intermittent drainage starting from across Highway 120, herein referred to as “the Eastside Drainage”. There are no known gauging stations or readily available data regarding flow for this drainage and as such this is not considered a source for groundwater recharge in our estimates of water supply onsite. This is a conservative approach that we feel supports our assessment based on the limited to non-existent data for this stream.

The average annual precipitation at the project site is estimated to range between 35 to 40 inches, however the watershed has extensive areas above snowline, meaning that rainfall is not the only source of runoff from the watershed. Table 10 presents a summary of normal year precipitation data for the project site.

Table 10 Estimated "Normal Year" precipitation data

Source:	usclimatedata.com [1]	Water.ca.gov [2]	Berkley Study [3]	worldclimate.com [4]	wrcc.dri.edu [5]
	Groveland, CA (2009 - 2019)	*Groveland, CA (2009 - 2019)*	*Adjacent water shed*	*Groveland, Ca (2009 - 2019)*	*Groveland, Ca (2009 - 2019)*
Average Inches of Precip.	*36.12*	*35.54*	*46.65*	*40.55*	*38.69*
Approximate Drainage Basin Size	Precipitation (AC/Ft.)				
462 Acres	1390.55	1368.26	1795.90	1561.18	1489.57
231 Acres	695.28	684.13	897.95	780.59	744.78
120.7 Acres	363.29	357.46	469.19	407.87	389.16

In general, "normal" yearly rainfall appears to range between 35.54 and 46.65 inches per year, depending on the source of the data. Excluding those low and high rainfall values, respectively, suggests an average annual rainfall of about 38.5 inches, and as such, an apparent range between 35 and 40 inches per year, as previously noted.

"Drought" years, or those years with under average rainfall, as reported for the Groveland area in 2012, and again in the years 2012 to 2014, ranged from 8.83 to 16.42 inches, or an average of about 12.6 inches annually, depending on the source of the data. For the purposes of assessing water supply, in single and two-year drought scenarios, we are assuming average annual rainfall of twelve (12) inches per year. Table 11 presents a summary of drought year precipitation data for the area around the project site.

Table 11 Estimated "Drought Year" precipitation data

Source:	usclimatedata.com [1]	Water.ca.gov [2]
	Groveland, Ca (2012)	*Groveland, Ca (2014 - 2015)*
Average Inches of Precip.	8.83	16.42
Approximate Drainage Basin Size	Precipitation (AC/Ft.)	
462 Acres	339.96	632.17
231 Acres	169.98	316.09
120.7 Acres	88.82	165.16

For the purposes of estimating water supply to the project, we are discounting the contribution of both infiltration from either of the intermittent streams, and snowmelt to recharge, as there is little data to base a reliable estimate upon. As such, we are assuming that groundwater supporting the site, is limited to rainfall within the watershed. Furthermore, we are taking a conservative approach with respect to the percentage of rainfall that actually recharges groundwater. In general, the greater the rainfall, the greater the percent recharge, because in higher rainfall conditions, the ground is saturated, facilitating greater "non-recoverable deep percolation", in essesence more water gets past the unsaturated zone and down into the fractured aquifer system. In drier conditions, such as drought, recharge can drop as low as 10% of the rainfall across the drainage area, while in normal years it can reach as high as 35% (Kirk, 2014). To carry on with our conservative approach, we are going to assume that recharge to the project site, is 10%, for assessing water demand and supply, with respect to sustainability. Furthermore, using the more conservative 10% recharge rate, allows us to discount evapotranspiration as a "loss" to the system, as estimating that value will probably change during development of the project.

Table 12 presents estimates of the aquifer recharge to the project site based of the following assumptions and variables:

- Normal versus "drought" precipitation, averages of 35.8 inches, versus 12.6 inches annually.
- A range of drainage (recharge) basin size, from 462 (largest estimate) to 120.7 (project site only).
- Only ten percent (10%) of the precipitation is assumed to actually recharge groundwater, which is on the low end (drought-like) for recharge percentage, and is very conservative.
- Discounts gains from offsite inflow, onsite septic recharge and snowmelt, and losses from evapotransipiration. We believe these are acceptable assumptions, given the conservative recharge percentage being used.

Table 12 – Estimated Annual Recharge

Normal Precipitation Year

Source:	usclimatedata.com [1]	Water.ca.gov [2]	Berkley Study [3]	worldclimate.com [4]	wrcc.dri.edu [5]
	Groveland, CA (2009 - 2019)	*Groveland, CA (2009 - 2019)*	*Adjacent water shed*	*Groveland, CA*	*Groveland, Ca (2009 - 2019)*
Average Inches of Precip.	*36.12*	*35.54*	*46.65*	*40.55*	*38.69*
	Estimated Acre Feet of Recharge				
462 Acres	139.06	136.83	179.59	156.12	148.96
231 Acres	69.53	68.41	89.79	78.06	74.48
120.7 Acres	36.33	35.75	46.92	40.79	38.92

Drought Precipitation Year

Source:	usclimatedata.com [1]	Water.ca.gov [2]
	Groveland, Ca (2012)	*Groveland, Ca (2014 - 2015)*
Average Inches of Precip.	8.83	16.42
	Estimated Acre Feet of Recharge	
462 Acres	34.00	63.22
231 Acres	17.00	31.61
120.7 Acres	8.88	16.52

Water Demand

Under Canvas has established general water demand for the project based on other similar operating facilities and has a reasonable grasp of both the annual water demands for these sites, and the water supplies that the sites require, with respect to the daily and annual demand. There is limited long term data available for similar facilities (other UC facilities as an example) to establish daily, monthly, and annual water demands. As such, conservative assumptions have been made, essentially maximizing the overall demand of the facilty to allow for a comparison to existing supplies. Table 13 presents a general synopsys of specific types of water uses, and the estimates of daily demands. Those demands are based on the following exaggerated assumptions:

- 100 percent occupancy, for the duration of the operational season.
- Two hundred and fifty (250) day long season (Eight months a year, roughly).

- The facility will achieve complete build-out at ninety nine (99) tents, and for the purpose of assessing water demand, it is assumed that one hundred (100) tents will be in use from the opening of the facility, and will remain in use the entire time.
- None of the employees will remain on site overnight, so they are not accounted for in the estimates of "guest water usage".

TABLE 13 PROPOSED WATER USE

Proposed Use	Design GPD	Unit Per	Number of Units	GPD	Notes
*Reported Average Demand Tents (99), 2.5 folks/tent	20	Person	247.5	4950	20 gpd/camper
**Estimated Maximum Demand Tents (99), 2.5 folks/tent	50	Person	247.5	12375	50 gpd/camper
Employee	10	Person	40	400	10 gpd/employee
Laundry	550	Machine	2	1100	550 gpd/single service
Food Prep	4	Service	375	1500	4.0 gpd/single service
*TOTAL (Average GPD)				7950	
**TOTAL (Estimated Maximum GPD)				15375	

*Reported Average from Under Canvas includes employee, laudry, food prep

**Estimated Maximum includes employee, laudry, food prep

The facility will need to supply an average estimated daily demand of 7,950 gpd according to Under Canvas' estimates, determined using data from their other operating properties. To provide a further conservative estimate, the average daily demand (ADD) was rounded up to 10,000 gallons per day. Water demands are not expected to increase for the facility due to the land use plan developed by Under Canvas. The following table 14 assess water demand of the facility water system versus the estimated water supply, based on the following:

- Historical rainfall totals measured for the area around the UC facility.
- Average daily demand and maximum daily demand (MDD)
  - The Under Canvas provided Daily Demand (UCDD) of about 8,000 GPD, was conservatively increased to 10,000 GPD, and will be the average daily demand (ADD) for the purposes of this water suppy and demand assessment.
  - The state recommends multiplying the ADD by a factor of 1.5 to account for stressed conditions. For the purpose of remaining conservative, the ADD was multiplied by 1.6 to obtain the value, 16,000 gallons/day, which will be used for the purposes of this assessment as the maximum daily demand (MDD). It should be noted, that the MDD is assessed, primarly to address the sustainability of the water supply, with respect to a very conservative MDD.
  - Estimates of water demand utilized to describe waste water management are consistent with our conservative values, and should be considered representative within the range of water demands we have used.
- The drainage basin supplying groundwater recharge for the facililty's water system was estimated at 462 acres within a greater drainage basin of the South Fork Tuolomne River area
  - The quantity of water available for groundwater recharge from the 462-acre drainage basin was estimated for an average year based on historical rainfall and for a drought (lasting 2 years).
  - The basin area was halved to obtain a more conservative estimate during normal and drought years.
  - Finally, the table considered inflow (i.e. groundwater recharge at 10% of rainfall) strictly from the project site, 120.7 acres.

Table 13 shows that during a normal precipitation year the ADD versus the estimated water supply varies widely, from about 4.3% up to 21.5%, depending on the size of the drainage basin. For the MDD, that range is from about 9% to 34.5%. Again, this assumes the conservative 10% recharge, as opposed to the more realistic 30% to 35% recharge that should be expected in normal precipitation years.

For years of "drought" precipitation values, the ADD versus the estimated water supply varies widely, from about 12% up to 86%, depending on the size of the drainage basin. For the MDD, that range is from about 19% to 138%.

The project facility site-only drainage basin size (about 121 acres) is the most conservative, and the least representative of actual conditions. It is unlikely that the largest drainage basin (462 acres) is the most representative as it is unlikely that all water falling within this area will accumulate in the project site area. As such, we are going to focus on the intermediate sized basin (231 acres) for our further assessment.

Table 14 Estimated Groundwater Recharge

Normal Precipitation Year

Source:	usclimatedata.com[1]	Water.ca.gov[2]	Berkley Study[3]	worldclimate.com[4]	wrcc.dri.edu[5]
	Groveland, CA (2009 - 2019)	*Groveland, CA (2009 - 2019)*	*Adjacent water shed*	*Groveland, CA*	*Groveland, Ca (2009 - 2019)*
Average Inches of Precip.	*36.12*	*35.54*	*46.65*	*40.55*	*38.69*
	Water Usage based off Recharge Totals, 10,000 Gal per day				
462 Acres	5.52%	5.61%	4.27%	4.91%	5.15%
231 Acres	11.03%	11.21%	8.54%	9.83%	10.30%
120.7 Acres	21.12%	21.46%	16.35%	18.81%	19.71%
	Water Usage based off Recharge Totals, 16,000 Gal per day				
462 Acres	8.83%	8.97%	6.84%	7.86%	8.24%
231 Acres	17.66%	17.94%	13.67%	15.73%	16.48%
120.7 Acres	33.79%	34.34%	26.16%	30.10%	31.54%

Drought Precipitation Year

Source:	usclimatedata.com[1]	Water.ca.gov[2]
	Groveland, Ca (2012)	*Groveland, Ca (2014 - 2015)*
Average Inches of Precip.	8.83	16.42

	Water Usage based off Recharge Totals, 10,000 Gal per day	
462 Acres	22.57%	12.14%
231 Acres	45.14%	24.27%
120.7 Acres	86.38%	46.45%
	Water Usage based off Recharge Totals, 16,000 Gal per day	
462 Acres	36.11%	19.42%
231 Acres	72.22%	38.84%
120.7 Acres	138.21%	74.33%

Table 15 presents the three drainage basin sizes as one component of three scenarios, and the range of water usage (10,000 versus 16,000 GPD) as the other component, and then using these values to assess water supply versus water demand. The following should be considered when using Table 14:

- The Annual estimated recharge (acre/ft per year) is the average of the recharge estimates presented in Table 12.
- The annual recharge values are based on, again, a 10% recharge factor, which is very conservative, and more representative of drought conditions, whereas 30% to 35% is more likely in normal precipitation conditions.

Focusing on the middle sized drainage basin (231 acres), Table 14 shows that the anticipated water demand versus water supply, expressed as a percentage of the estimated available water suppy, ranges from about 11% to 16% for the ADD, and MDD respectively, during normal precipitation years. During drought precipitation conditions, the anticipated water demand will range from about 35% to 55%, respectively.

TABLE 15 WATER USE ESTIMATIONS

Drainage Basin (Acres)	Planned Water Use (Gallons/ Day)	Estimated Water use (Gallons/ Yr)	Annual Estimated Water Use (Acre-Feet/Yr)	Annual Estimated Recharge AC/Ft.*	Annual Estimated Recharge AC/Ft. Drought*	Estimated % Use	Estmated % Use, Drought
462	10000	2.50E+06	7.67	152.11	48.16	5.09%	17.35%
462	16000	4.00E+06	12.28			8.15%	27.76%
231	10000	2.50E+06	7.67	76.05	24.30	10.18%	34.70%
231	16000	4.00E+06	12.28			16.30%	55.53%
120.7	10000	2.50E+06	7.67	39.74	12.70	19.49%	66.42%
120.7	16000	4.00E+06	12.28			31.19%	106.27%
*Assumes average recharge equals 10% of average rainfall							

Conclusions

Based on our background research, field work and findings, we offer the following conclusions:

- Pressurized (artesian) groundwater conditions were seen in all three of the onsite wells, which suggests that these wells are within a separate portion of the larger fracture system. They may in fact individually be in separate sections of the fracture system, based on this limited interaction with each other; and additionally the fractures they are completed in may be different than those fractures in which the well on the project to the north are completed in.
- Similarities in Well 1 and Well 2 water chemistries indicate that both of these wells are obtaining water from a shared fracture system.
- The limited draw down recorded in the three Wells, when either Well 1 or Well 2 was being individually pumped, indicates that the there is at least some hydraulic communication between Well 1 and Well 2, but it is very limited. There appears to be little to no hydraulic communication between Well 3 and Well 1 or Well 2.
- The additional drawdown recorded in Well 1 when the well on the project to the north were pumping indicates that to some degree Well 1 is in hydraulic communication with one or both of the wells on that project.

- Based on the provided water demand, our conservative assessment suggests that the project does not appear to place a burden on the available groundwater supply, even in the driest years and most exaggerated assumptions.

The results of the pumping tests and groundwater analytical results indicate that at this time there is sufficient capacity in the system to support the planned water use, and that the quality of the water meets all Federal and State drinking water standards.

The project, as described and assessed, does not appear to pose a significant risk to the environment, with respect to the use of groundwater to support the project. Facility water demand perctenages, based on very conservative estimates, are fairly low with respect to conservative estimates of the water supply generally available to the project site.

The magnitude of the impact on the projects groundwater supplies as a result of pumping from the project to the north, cannot be adequately evaluated with the existing data. However, it can be concluded that some influence on pumping water levels in at least Well 1 should be anticipated, and that some influence on groundwater availability to the project will result from operations on the project to the north.

Recommendations

Based on our findings and conclusions, we offer the following recommendations:

- A permanent weather station should be established on the project site to facilitate more accurate precipitation data.
- Each of the wells completed be should equipped with electronic logging equipment that is capable of recording and reporting water levels (static and pumping), discharge rates (instantaneous and cumulative) and power consumption.
- Each should be equipped with dedicated electronic equipment and sample collection ports.
- To maintain well performance, an operational pumping schedule should be developed to regularly pump both Well 1 and Well 2.
- As part of the State Small Water System operational requirements, routine monitoring and recording of all pumping operations should be conducted. These records should be reviewed by the System Engineer, and as needed WRA.

- At least one of the wells should be equipped with a backup generator to maintain a power supply in the event of power outage. Alternatively, sufficient onsite water storage should be maintained to meet the maximum estimated water demand for at least two (2) days (approximately 20,000 gallons).
- A "Low Water Usage" operational plan should be prepared to address reasonable reductions in groundwater use during periods of drought.

Environmental Impact

With respect to the overall environmental impact report (EIR) we have been tasked with responding to six (6) specific questions. The following are those questions, and our responses to them:

- Are sufficient water supplies available to serve the project and reasonably foreseeable future development during normal, dry and multiple dry years?

  Based on the assessment of rainfall, drainage area, and recharge, and considering the conservative approach taken to describe the available water suppy, it is considered unlikely that that project, now or in the reasonably foreseeable future will lack for a sufficient and sustainable water supply.

  The project water demand is conservative, estimated for both a slightly higher demand (10,000 GPD) than suggested by client supplieddata for other facilities, and an extreme demand (16,000) to over-emphasize the possible impact on the water supply.

  Under normal rainfall years, using the slightly higher water demand, the more conservative recharge percentage (10%), and the smallest drainage basin area (120 acres), the project will only use about 11% to 16% of the available water supply.

  Under a single drought year, using the same assumptions as previously stated, the project will use about 35% to 55% of the available water supply. Multiple dry years will exacerbate the minimal recharge; however, the minimal drawdown measured in the two onsite wells suggests that there will be more than sufficient capacity for the wells to continue to meet the projects water demand.

The conservative approach strongly suggests that there are sufficient water supplies to support the project, during both normal and multiple year droughts.

- Substantially decrease groundwater supplies or interfere substantially with groundwater recharge such that the project may impede sustainable groundwater management of the basin?

  There are no indications that current projected water demands in support of the project, which rely solely on groundwater, will substantially decrease available groundwater supplies to other known users, nor should these demands substantially decrease the available recharge to known users.

  Groundwater recharge estimates will range from 10% to 35% of the rainfall in the drainage basin, and the size of the basin capturing that rainfall. Drier periords, such as droughts, will see lower recharge because of minimal infiltration of rainfall, whereas normal (wetter) rainfall periods may see recharge rates of up to 35%. The projects water demand will range from about 5 to 65% of the available recharge, under the most conservative conditions.

- Violate any water quality standards or waste discharge requirements or otherwise substantially degrade surface or ground water quality?

  Ground water quality is excellent, and as such there is no indication based on current water quality data, that groundwater quality will be impacted or violated by using groundwater for potable purposes.

- Require or result in the relocation or construction of new or expanded water, wastewater treatment or storm water drainage, electric power, natural gas, or telecommunications facilities, the construction or relocation of which could cause significant environmental effects?

  The use of groundwater as a sole-source water supply should not cause any relocation, construction or expansion of water or waste water treatment facilities, storm water drainage facilities, or electrical power, natural gas, or telecommunications facilities.

- Have a substantial adverse effect on any riparian habitat or other sensitive natural community identified in local or regional plans, policies, regulations or by the California Department of Fish and Wildlife or U.S. Fish and Wildlife Service?

  Groundwater use, in support of the project, is projected to have a minimal impact on groundwater extraction in the area around the project, and a minimal impact on overall basin recharge. The pump testing results suggest that groundwater withdrawal will minimally lower groundwater levels in the immediate vicinity of each well, due to the behavior of fractured bedrock, and even over the duration of an operational season, we do not anticipate substantial lowering of the groundwater elevation in the vicinity.

  As such, while we cannot definitively state that no impact will occur, the minimal changes in groundwater elevations documented in our testing indicate the project is very unlikely to create any substantial adverse effects.

- Have a substantial adverse effect on state or federally protected wetlands (including, but not limited to, marsh, vernal pool, coastal, etc.) through direct removal, filling, hydrological interruption, or other means?

  Based on our assessment and findings, we do not anticipate at this time any substantial adverse effects on wetleands in the area of groundwater withdrawal.

State Small Water System

As part of the application for a State Small Water System permit, we can offer responses to the following questions, as they pertain to the hydrogeologic assessment, and water sustainability of the project site:

1. Source capacity: The project currently has two (2) wells that have been completed as potable water supply wells. Each source has demonstrated a sustainable production of about 40 gpm over ten days; and jointly would probably be able to produce up to 80 gpm. In other words, each well pumped about 576,000 gallons of water over the ten day period, which is about 1.77 acre-feet of water. This volume of water is roughly equivalent to 15% to 25% of the facility demand, based on precipitation conditions. Both wells recovered to within 95% of static water level within ten-days, and little to no interference was detected between the two onsite supply wells. Finally, the water quality is equivalent in both wells,

and either well operating independently can meet ADD in as little as four (4) hours of pumping a day.

2. Water Demand. The proposed facility will be the first of it's kind in this particular geographic setting, and as such information based on size, elevation, climate, demography, residential property size, and metering are not available to determine the average water usage per connection. Under Canvas has the ability to limit water use on the facility, and this "water use management" is the basis of the water demand estimates presented above. Under Canvas intends the facility to be "low impact" with respect to water use, and therefore feels that permitting can go forward with the estimated usages standing in for the "peak hour demand" (PHD), "maximum day demand" (MDD). As needed, when sufficient operational data is accumulated, the PHD and MDD can be revised/updated.
3. System Growth. There are no known plans to expand the system beyond the initial infrastructure planned to support the ninety nine (99) guest tents, nor the 2.5 guests per tent occupancy, nor to extend the operational season of the facility. As such, expansion of the water supply system is not anticipated.
4. Wells. The following summarizes well information typically requested during the permitting process:
    a. The pumptesting discharge location is shown on Figure 4. The discharge area was approximately 1,000 feet from either of the two wells during testingt, and was monitored for the duration of the pumping portion of each of the tests.
    b. No surface water, staff gauges or other production wells were within 1,000 feet of the two onsitewells.
    c. Well construction information is included in Appendix B.
    d. Well completion dates are listed above, and included in the State of California, Department of Water Resources Well Completion Reports in Appendix B.
    e. Test pumps were placed at about 600 feet in the well, and were 7.5 horsepower submersible electric pumps.
    f. A single flow control valve was used to regulate flow during testing, and a mechanical flow meter, capable of instantaneous and cumulative flow measurement, was placed inline with the pump discharge piping. See the section on the pumping tests above.
    g. Water levels were electronically recorded, and manually cross-checked. See the section above on the pumping tests.
    h. There are no other known wells within the fracture system in which the UC wells were installed, with the exception of potentially wells associated with the project to

the north, that are of any demand or capacity similar to the project wells. Please see the section above describing the interactions between these wells.

i. “Casing storage” was addressed by the duration of the pumping test.
j. Annual aquifer recharge is addressed above.

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Appendix A	Under Canvas drilling permits
Appendix B	Under Canvas well completion reports
Appendix C	Waste Discharge permits
Appendix D	Pumping Test records – Under Canvas
Appendix E	BSK Laboratory Reports
Appendix F	Pumping test logs – Geoscience

General Construction Information

1) All labor used to execute the wastewater system installation must be by a California State Contractor, licensed to perform the required task, such as excavation, plumbing, or electrical work (an electrical permit is required).
2) Required inspections by the design consultant are: ground preparation/ layout, open excavation, open install and system flush (prior to covering any part of the installation), Septic Tank and treatment unit/pump basin liquid tightness test, final inspection after electrical hookup, AKA "System Startup". **NOTE: If I do not get my required inspections, I cannot certify the septic system, and the septic system cannot receive a construction final;** PLEASE CALL for INSPECTIONS
3) Inspections need to be arranged between 24 and 48 hours prior to the event.
4) Inspections that are missed due to improper or inadequate notification could jeopardize the certification of completion, and possibly delay dwelling occupancy.
5) All re-inspections for improper or incomplete work will be charged at the rate of $400.00 per re-inspection.
6) Any changes in materials and or equipment as specified by the septic system design must be pre-approved by the design consultant.
7) Any confusion regarding the installation of the septic system as it is designed, must be remedied by communication with the design consultant prior to proceeding with any further construction.

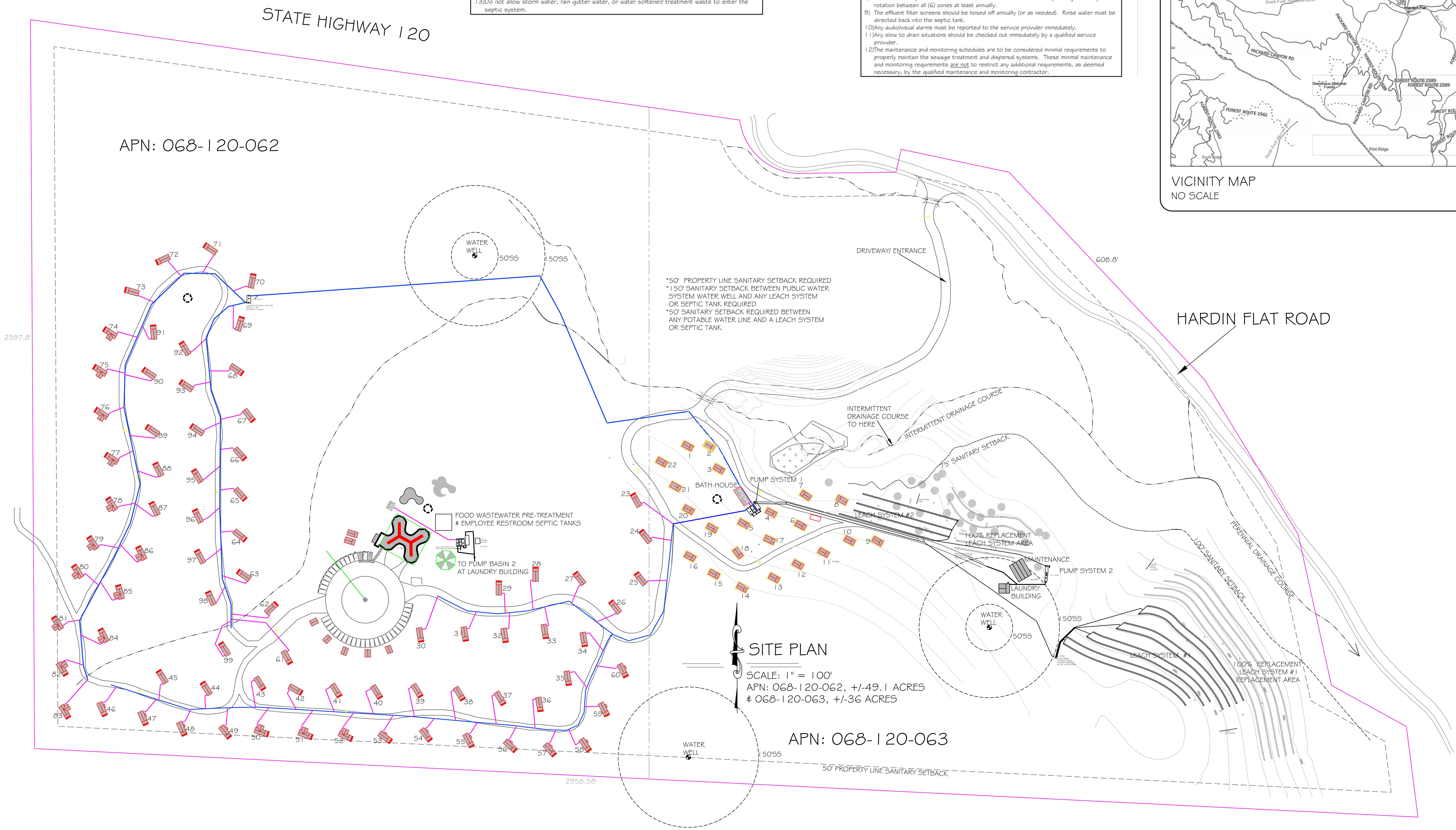
Septic System Operational Restrictions

1) No vehicle traffic should be allowed over any part of septic system.
2) Keep indoor plumbing systems in good working in order to avoid over-dosing the septic system. (no leaking faucets or toilets)
3) The septic system is designed to handle a specified number of gallons per day; avoid situations that would exceed this capacity.
4) Do not excessively irrigate over leach area.
5) Do not uses use degreasers and harsh cleaning products.
6) Do not use chemical sanitizing dishwashers (Use high-heat dishwasher or hand wash)
7) Avoid putting toxic or hazardous substances down drains (the septic system is a living system, and will only properly function if it is kept that way).
8) Do not use substances that claim to "help" or otherwise "improve" the function of your septic system. This includes but is not limited to "enzymes" and so called "grease digesters".
9) Never put materials that are difficult to decompose down drains (grease, plastics, rubber, excess paper products, etc.)
10) Avoid using powdered detergents (liquid detergents best).
11) Do not flush medications down toilets or drains.
12) Avoid excessive garbage disposal usage.
13) Do not allow storm water, rain gutter water, or water softener/ treatment waste to enter the septic system.

Commercial Wastewater System Maintenance and Monitoring Requirements

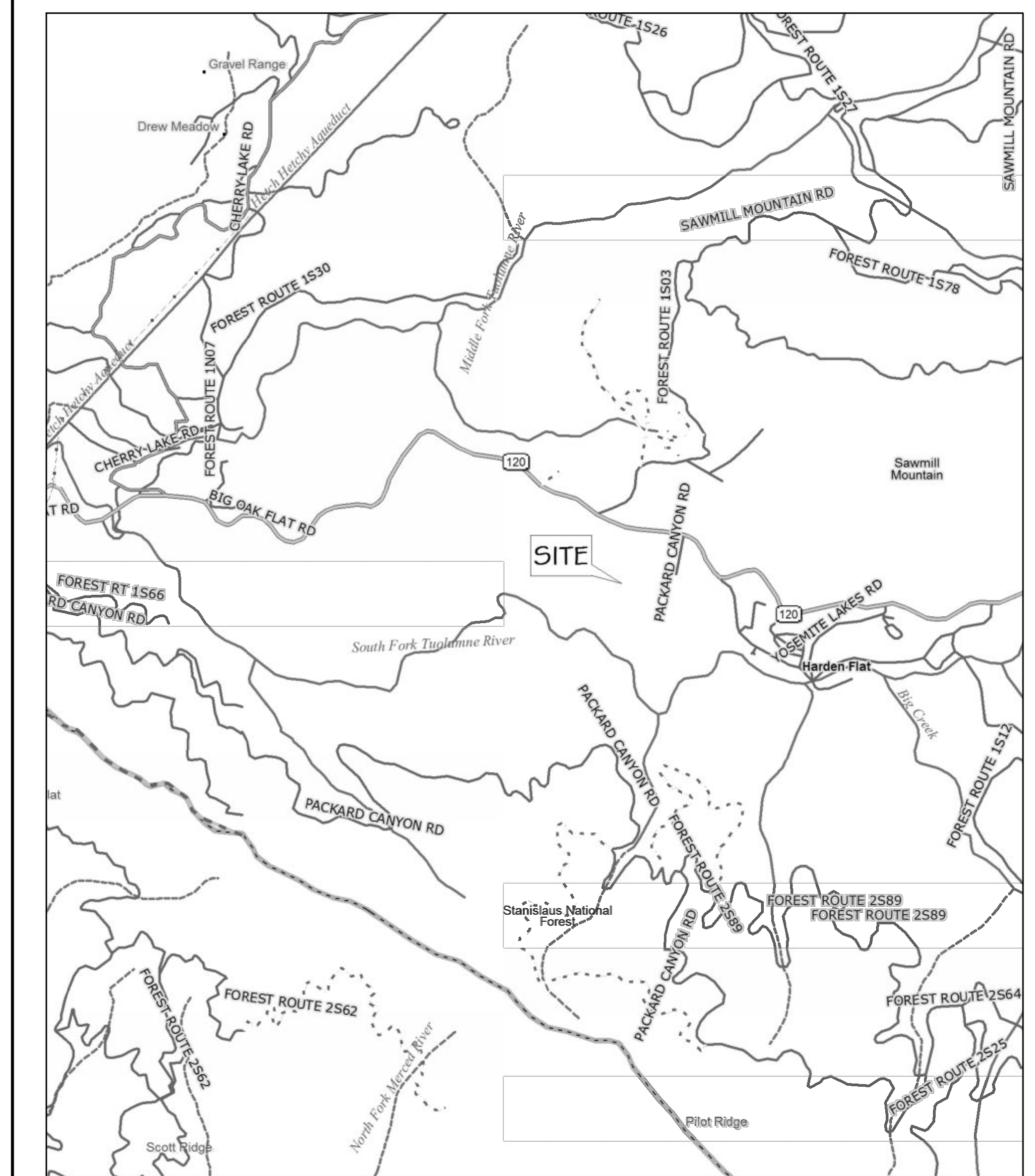
The proposed septic system design is intended to meet the "long term" need for adequate and proper sewage treatment and dispersal. Tuolumne County Ordinance Code (TCOC 130.08) requires that these wastewater systems be monitored and maintained by a qualified 3rd party service provider. The following is required for monitoring and maintaining the proposed wastewater system. (Referrals Available)

1) A third party maintenance and monitoring contract should be acquired prior to receiving a wastewater system construction final and subsequently operating the sewage treatment and dispersal system.
2) The third party maintenance and monitoring contract must be in place perpetually (must be renewed annually), and must coordinate with future regulatory Wastewater System Permit to Operate requirements.
3) Persons conducting the third party maintenance and monitoring inspections must be certified to do so by an accredited institution recognized by the State of California and or County Environmental Health Division for certifying persons for maintenance and monitoring of wastewater systems (COWA & or NAWT).
4) In addition any contracted service provider must be certified by Orenco for servicing and maintaining the TCOM control/alarm system and the Moving Bed Biofilm Reactor (MBBR)
5) Septic tanks should be monitored annually for excessive sludge or scum and serviced as required. (Never leave a tank pumped empty, fill tank with water after pumping)
6) Leach system inspection risers should be monitored at least annually for excessive effluent level, or unequal distribution. System must be serviced by a qualified person if substandard conditions are found.
7) Pressure laterals should be pumped individually (one at a time until effluent flows clear) with wastewater discharged properly back into the tank or an inspection risers using an appropriate hose. Pressure laterals must be re-adjusted to 5' of head for equal flow at all clean-outs.
8) The automatic hydraulic distribution valve must be confirmed to be operating correctly for rotation between all (6) zones at least annually.
9) The effluent filter screens should be hosed off annually (or as needed). Rinse water must be directed back into the septic tank.
10) Any audio/visual alarms must be reported to the service provider immediately.
11) Any slow to drain situations should be checked out immediately by a qualified service provider.
12) The maintenance and monitoring schedules are to be considered minimal requirements to properly maintain the sewage treatment and dispersal systems. These minimal maintenance and monitoring requirements are not to restrict any additional requirements, as deemed necessary, by the qualified maintenance and monitoring contractor.



SITE PLAN

SCALE: 1" = 100'
APN: 068-120-062, +/-49.1 ACRES
& 068-120-063, +/-36 ACRES



VICINITY MAP
NO SCALE

SEPTIC SYSTEM CONSULTING & DESIGN BY:

REGISTERED ENVIRONMENTAL HEALTH SPECIALIST
DONALD H. MYERS
No. 6587
STATE OF CALIFORNIA

DON MYERS, REHS
14801 TWIST ROAD
JAMESTOWN, CA 95327
(209) 743-9493

"#1 Consultant In The #2 Business"

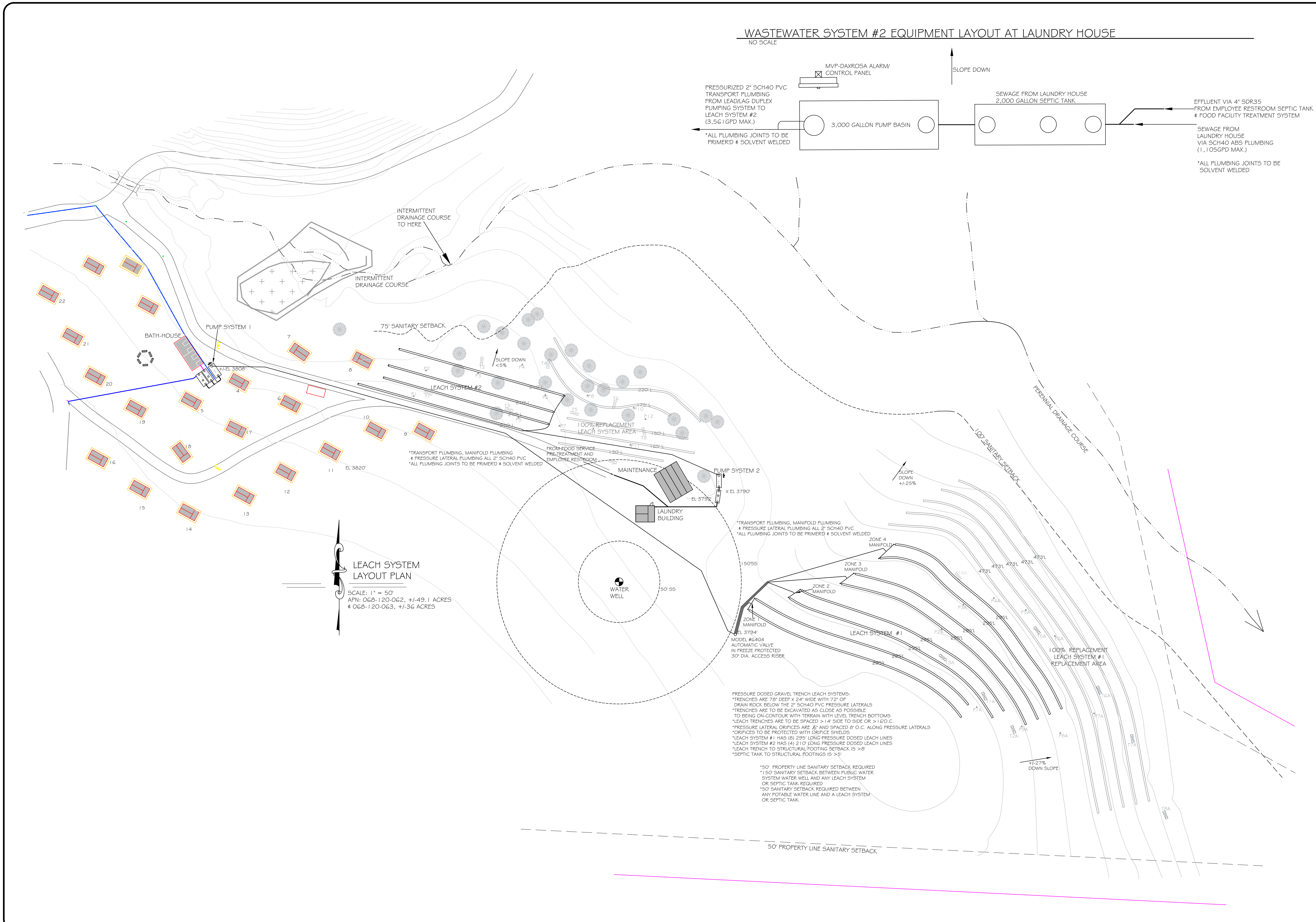
PROJECT SITE:
UNDER CANVAS
HARDIN FLAT ROAD
GROVELAND, CA

APN: 068-120-062
& 068-120-063
+/-85.1 ACRES

CURRENT PROPERTY OWNER(S):
HARDIN FLAT LLC
P.O. BOX 130
MOCCASIN, CA 95347

JULY 16, 2019 SCALE: AS SHOWN PAGE 1 OF 7
UNDER CANVAS
COMMERCIAL WASTEWATER SYSTEM

Note: Plan set For Wastewater System Approval Only



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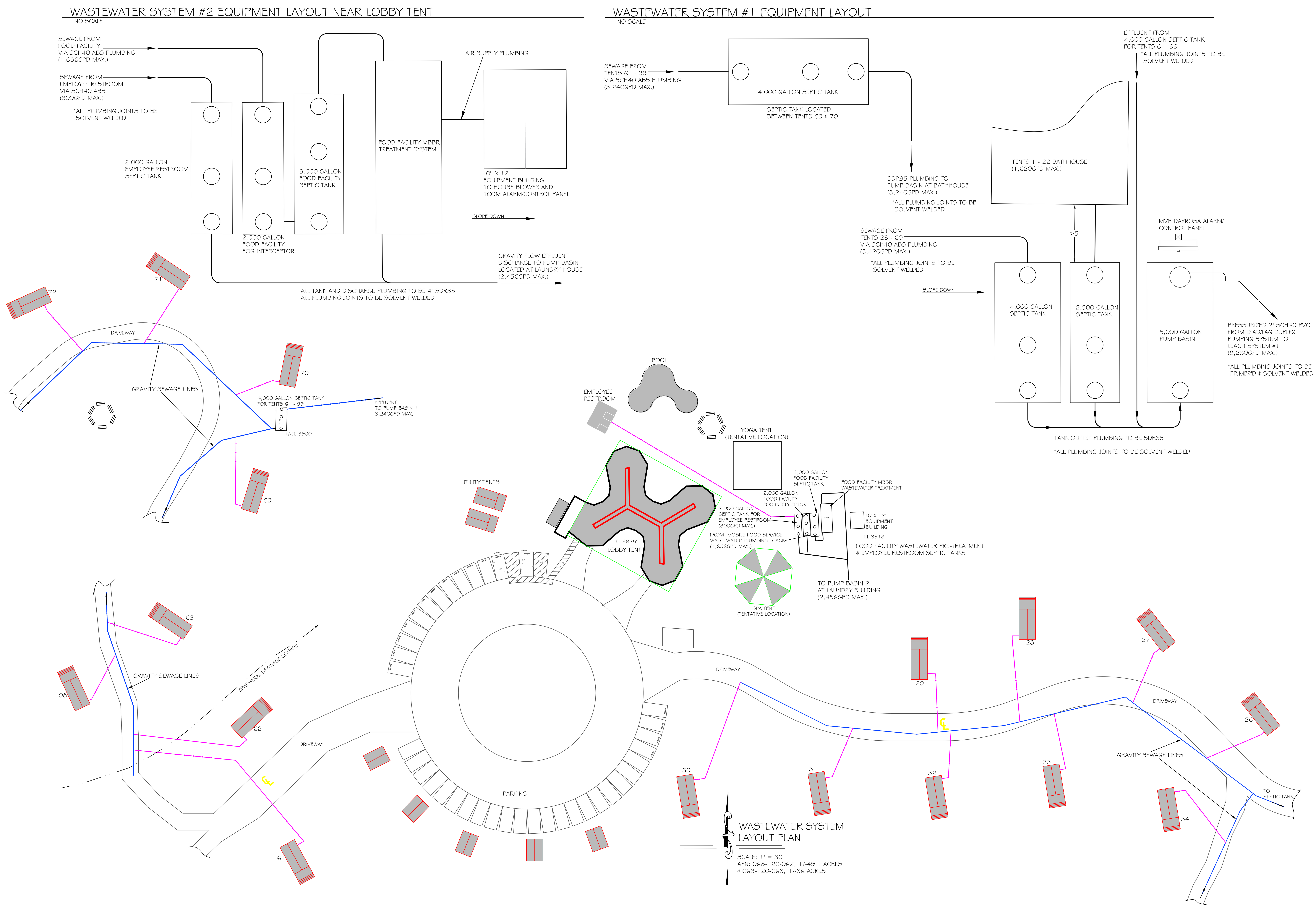

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JULY 16, 2019 SCALE: AS SHOWN PAGE 3 OF 7
UNDER CANVAS
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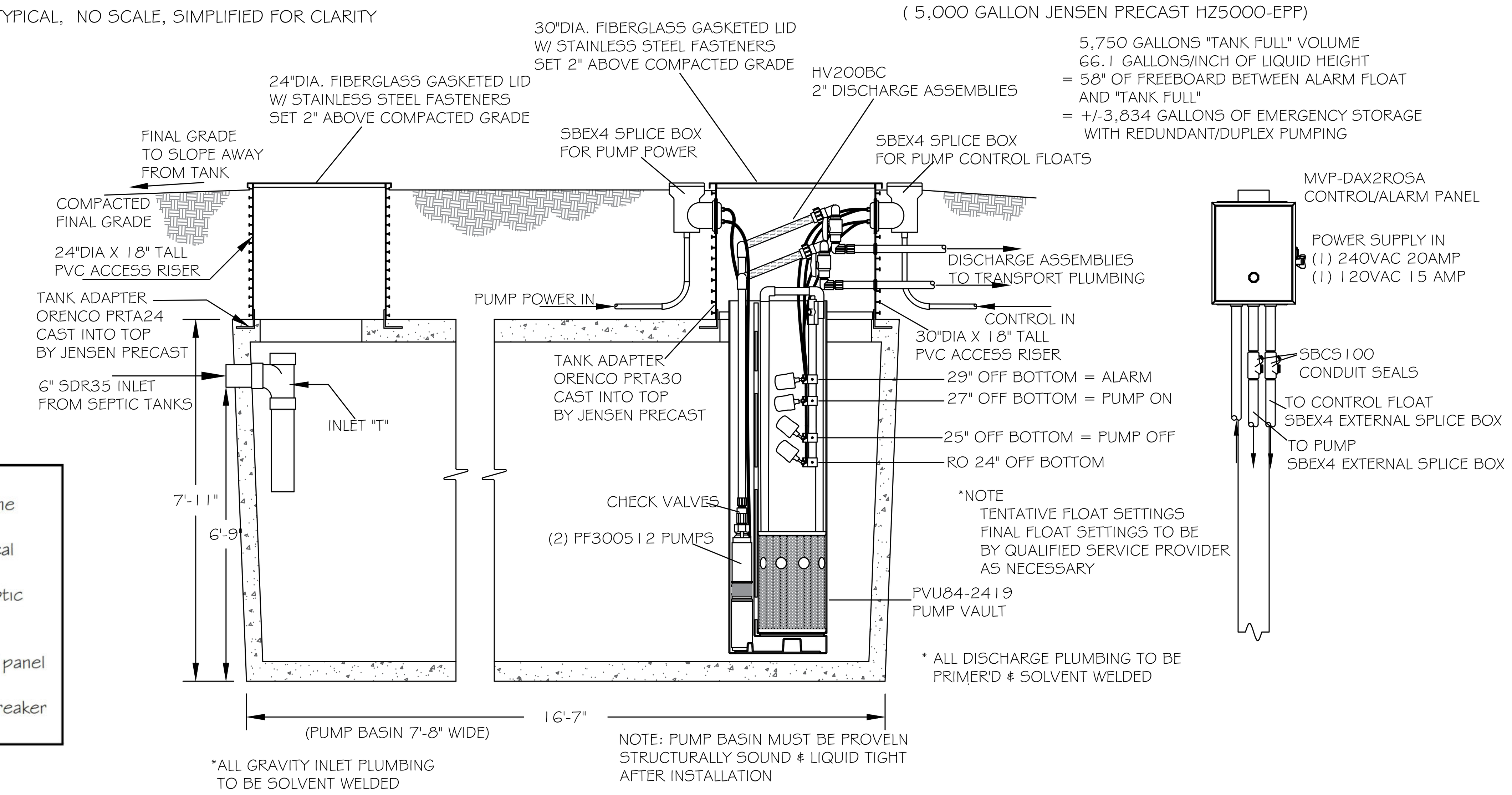
WASTEWATER SYSTEM #1
DETAIL SHEET

* ALL TANKS MUST BE KEPT FILLED WITH LIQUID, FILL TANKS WITH WATER AFTER PUMPING

WASTEWATER SYSTEM 1
MONOLITHIC CONCRETE PUMP BASIN WITH DUPLEX PUMPING SYSTEM

TYPICAL, NO SCALE, SIMPLIFIED FOR CLARITY (5,000 GALLON JENSEN PRECAST HZ5000-EPP)



Electrical Requirements

1) All electrical supply connections must be covered by an "Electrical Permit" issued by the local regulatory authority.
2) Any person performing the wiring work must be competent and familiar with the electrical requirements of this proposed septic system.
3) This proposed septic system requires (2) separate electrical supply circuits to the septic system control panel.
4) A dedicated 20amp, 240vac min. circuit is required for running the pumps.
5) A separate 15amp, 120vac min. circuit is to be supplied to the septic system control panel for alarm/ control functions.
6) The pump control circuit and the alarm/ control circuit must not use a common circuit breaker at the main electrical panel, or a household electrical sub-panel.

Pump Selection for a Pressurized System - Commerical Project

UNDER CANVAS PROJECT, HARDIN FLAT ROAD, GROVELAND / WASTEWATER SYSTEM #1, PDGT, 01/02/2020

Parameters

Discharge Assembly Size	2.00	inches
Transport Length Before Valve	460	feet
Transport Pipe Class	40	
Transport Line Size	2.00	inches
Distributing Valve Model	6404	
Transport Length After Valve	250	feet
Transport Pipe Class	40	
Transport Pipe Size	2.00	inches
Max Elevation Lift	-10	feet
Manifold Length	16	feet
Manifold Pipe Class	40	
Manifold Pipe Size	2.00	inches
Number of Laterals per Cell	8	
Lateral Length	287	feet
Lateral Pipe Class	40	
Lateral Pipe Size	2.00	inches
Orifice Size	1/8	inches
Orifice Spacing	8	feet
Residual Head	5	feet
Flow Meter	None	inches
'Add-on' Friction Losses	0	feet

Calculations

Minimum Flow Rate per Orifice	0.43	gpm
Number of Orifices per Zone	72	
Total Flow Rate per Zone	31.5	gpm
Number of Laterals per Zone	2	
% Flow Differential 1st/Last Orifice	4.4	%
Transport Velocity Before Valve	3.0	fps
Transport Velocity After Valve	3.0	fps

Frictional Head Losses

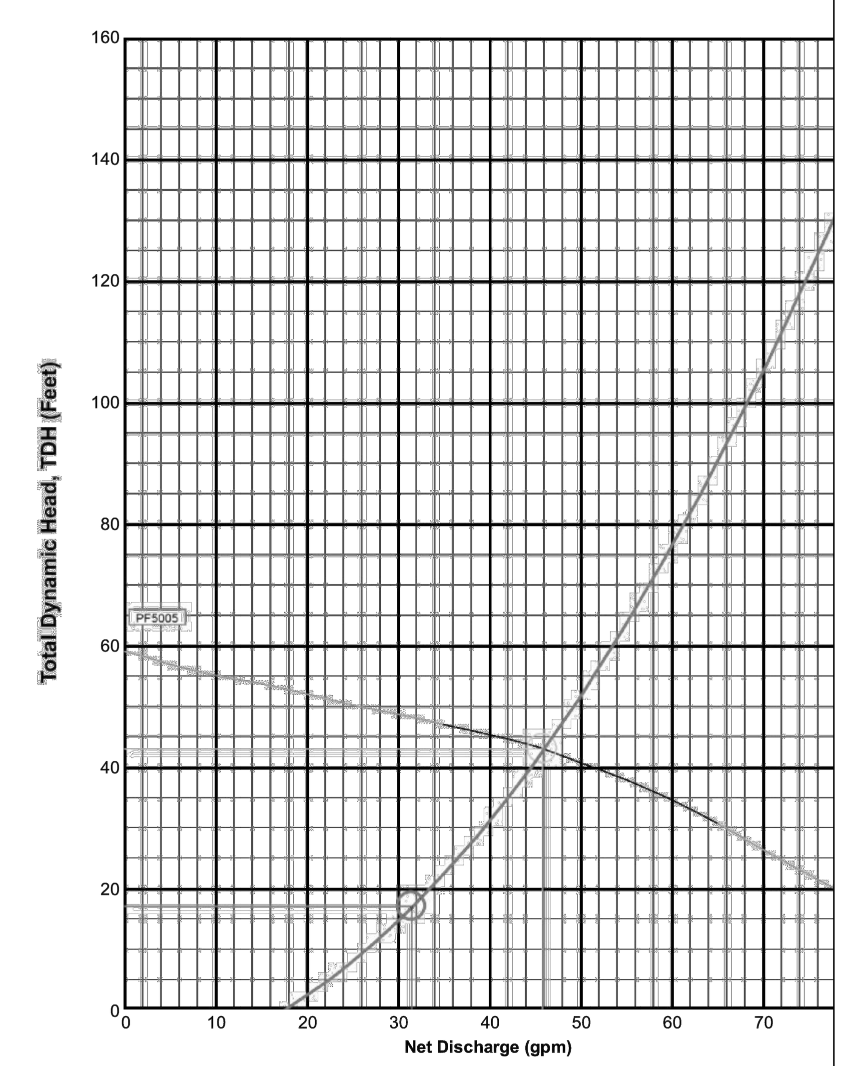
Loss through Discharge	1.9	feet
Loss in Transport Before Valve	7.8	feet
Loss through Valve	7.4	feet
Loss in Transport after Valve	4.3	feet
Loss in Manifold	0.1	feet
Loss in Laterals	0.5	feet
Loss through Flowmeter	0.0	feet
'Add-on' Friction Losses	0.0	feet

Pipe Volumes

Vol of Transport Line Before Valve	80.2	gals
Vol of Transport Line After Valve	43.6	gals
Vol of Manifold	2.8	gals
Vol of Laterals per Zone	100.1	gals
Total Vol Before Valve	80.2	gals
Total Vol After Valve	146.4	gals

Minimum Pump Requirements

Design Flow Rate	31.5	gpm
Total Dynamic Head	17.1	feet



PumpData

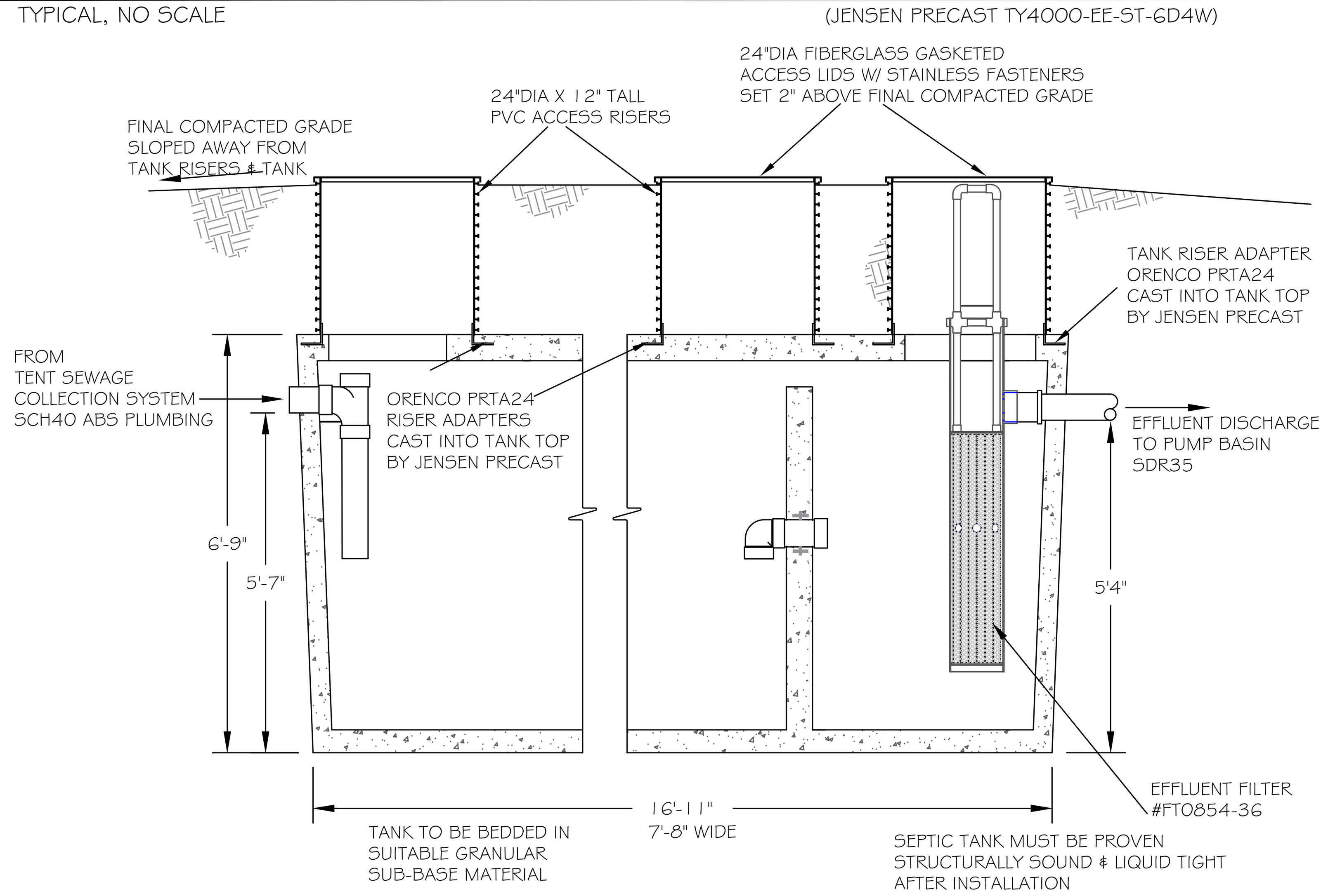
PF5005 High Head Effluent Pump
50 GPM, 1/2HP
115/230V 1Ø 60Hz, 200/230V 3Ø 60Hz

Legend

System Curve:
Pump Curve:
Pump Optimal Range:
Operating Point:
Design Point:

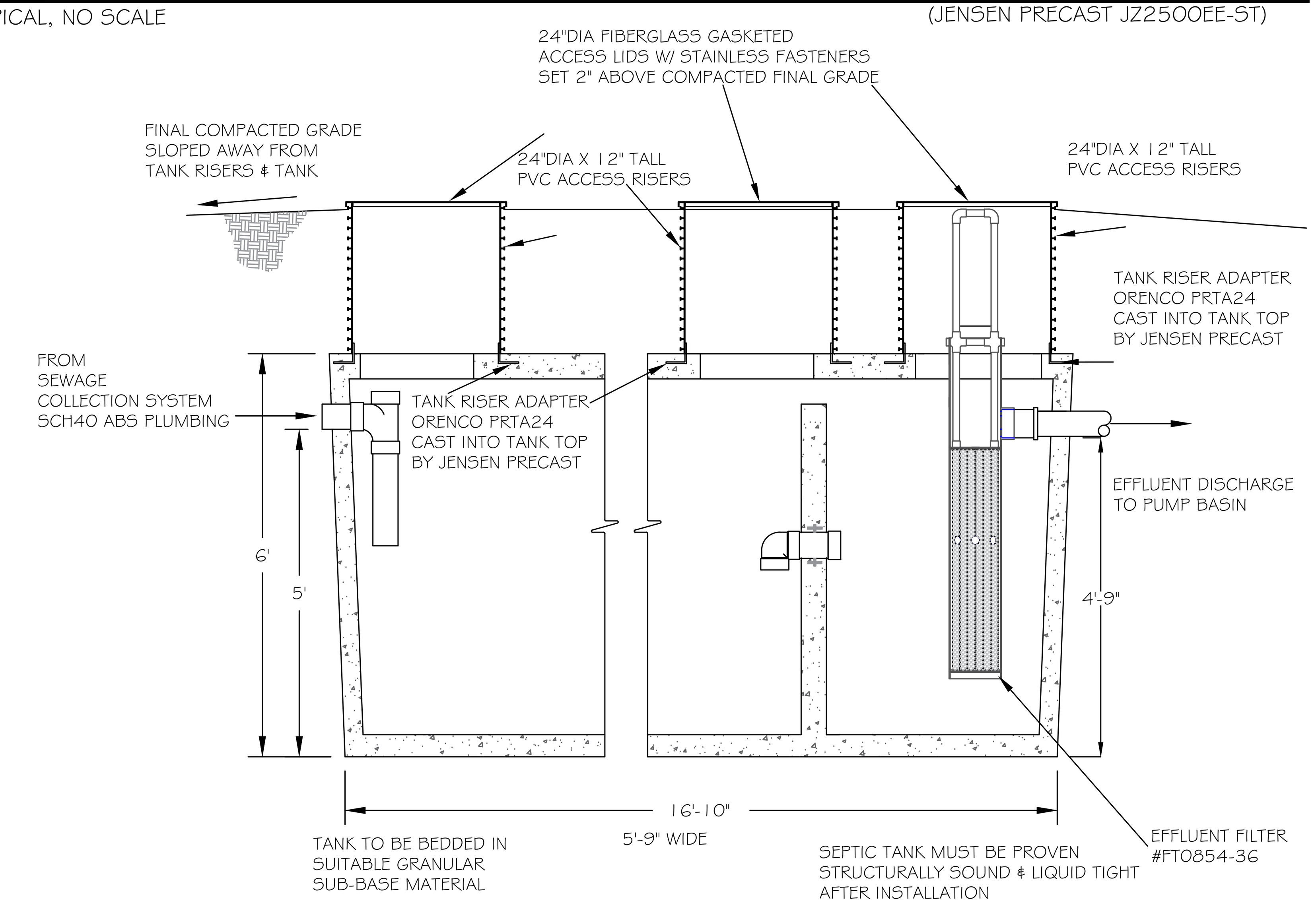
WASTEWATER SYSTEM #1
(2) 4,000 GALLONS (EACH)
MONOLITHIC CONCRETE SEPTIC TANKS (TENTS 61 - 99 & TENTS 23 - 60)

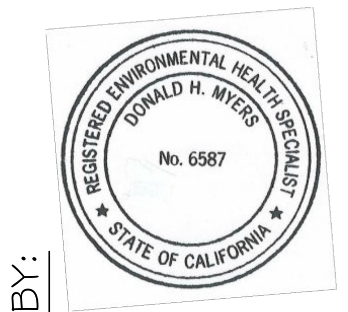
TYPICAL, NO SCALE (JENSEN PRECAST TY4000-EE-ST-6D4W)



WASTEWATER SYSTEM #1
2,500 GALLON MONOLITHIC CONCRETE SEPTIC TANK FOR THE BATHHOUSE (TENTS 1 - 22)

TYPICAL, NO SCALE (JENSEN PRECAST JZ2500EE-ST)





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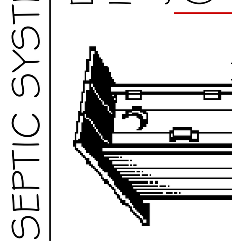

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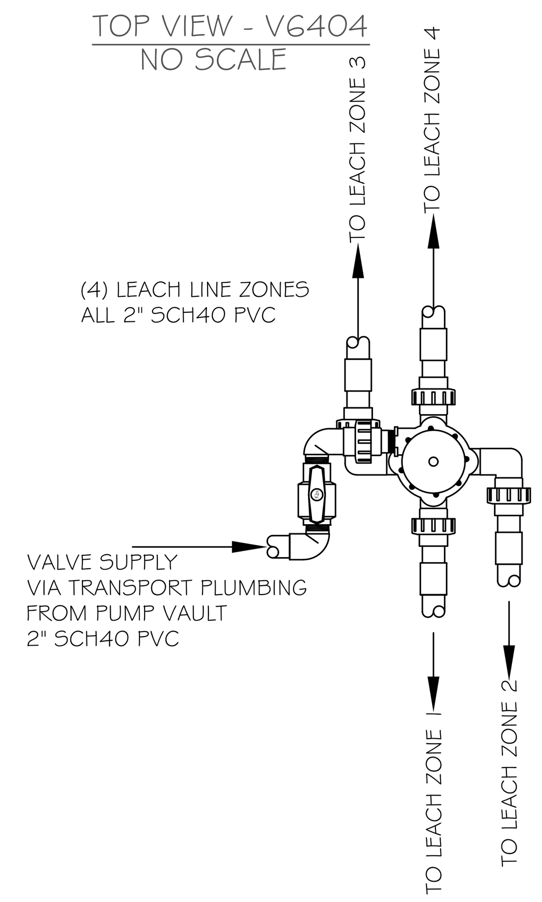
JULY 16, 2019 SCALE: AS SHOWN PAGE 4 OF 7
UNDER CANVAS
COMMERCIAL WASTEWATER SYSTEM
Note: Plan set For Wastewater System Approval Only

Wastewater System #1 Specific Leach Trench Construction Requirements
Modified Pressure Dosed Triple Depth Leach Trenches
Call for All Required Inspections

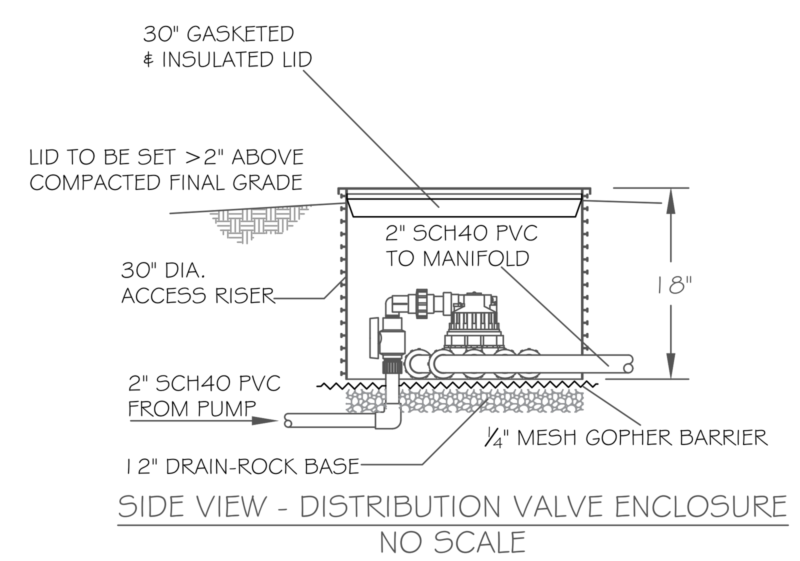
1) No construction is to take place with wet soil conditions.
2) Open trench inspections must be performed by Tuolumne County Environmental Health and the Design Consultant.
3) Trench side walls must be mechanically scarified so as to remove surface "glaze", "greasy", or "smeared" appearance.
4) Leach trenches are to be excavated on contour with terrain to a total depth of **84"** (down-hill side), with level trench bottoms of **24"** in width.
5) Trenches are to be constructed at a minimum of **14'** side wall to side wall apart (or 16' on-center).
6) Inspection risers are to be constructed in the trenches and inspected prior to covering with drain rock, AKA Open Trench Inspection (see detail drawings).
7) Drain rock is to be installed level with a minimum depth of **72"**.
8) Pressurized distribution laterals are to be installed level on top of the drain rock. 2" SCH40 PVC pressurized distribution laterals are to be prepared for installation by drilling 1/8" orifices **96"** on center and **48"** off trench ends (no more than **36** orifices per lateral). Orifices are to be orientated so that they face up and then be covered by an orifice shield after a lateral squirt test (AKA Open Install Inspection). Distribution laterals to be connected to manifold via 2" gate valves (Do not use ball valves). A septic tank liquid tightness test should also be performed at this time.
9) Place drain rock over the pressure laterals to a depth of **6"**.
10) Filter fabric is to be used as a soil barrier over drain rock.
11) Native organic topsoil is to be used as the final capping fill over constructed leach trenches to a total depth of **12"** over filter fabric.
12) Final grading is to allow for a gently sloped toe on the down-slope, and be contoured into the terrain on the up-slope of the leach trench system (the final grading should not have a speed bump appearance).
13) A "V" ditch may be required in the final grading up-slope from the leach trenches to re-direct surface run-off around and away from the sand trenches.
14) Erosion prevention measures are to be completed after final grading on all disturbed ground (see details).
15) A Final inspection is required before the job is considered complete. Inspection includes: erosion protection, lateral end cap squirt test (generator may be used), alarm function, and others as necessary.

AUTOMATIC DISTRIBUTION VALVE ASSEMBLY FOR LEACH SYSTEM #1
TYPICAL, NO SCALE, SIMPLIFIED FOR CLARITY



NOTE: DISTRIBUTION VALVE MUST BE INSTALLED AT THE HIGHEST ELEVATION IN THE SEPTIC SYSTEM (ABOVE SEPTIC TANK, PUMP BASIN, AND LEACH SYSTEM)

THE DISTRIBUTION VALVE MUST HAVE FREEZE PROTECTION



*ALL PLUMBING JOINTS TO BE PRIMER'D & SOLVENT WELDED

Under Canvas Project Wastewater System
Specified Parts List
(No substitutions without Design Consultant approval)

Septic Tanks (Custom Made By Jensen Precast):
- (2) 4,000 gallon monolithic concrete septic tanks (Jensen Precast TY4000-EE-ST-6D4W)
- (6) Tank riser adapters (Orenco #PRTA24) cast into tank tops by Jensen Precast
- (1) 3,000 gallon monolithic concrete septic tank (Jensen Precast JZ3000EE-ST)
- (3) Tank riser adapters (Orenco #PRTA24) cast into tank tops by Jensen Precast
- (1) 2,500 gallon monolithic concrete septic tank (Jensen Precast JZ2500EE-ST)
- (3) Tank riser adapters (Orenco #PRTA24) cast into tank tops by Jensen Precast
- (2) 2,000 gallon monolithic concrete septic tanks (Jensen Precast JP2000EE-ST)
- (6) Tank riser adapters (Orenco #PRTA24) cast into tank tops by Jensen Precast

FOG Interceptor (Custom Made By Jensen Precast)
- (1) 2,000 gallon monolithic concrete FOG Interceptor (Jensen Precast JP2000EPE-G)
- (3) Tank riser adapters (Orenco #PRTA24) cast into tank tops by Jensen Precast

Pump Basins (Custom Made By Jensen Precast):
- (1) 5,000 gallon monolithic concrete pump basin (Jensen Precast HZ5000-EPP)
- (1) Tank riser adapter (Orenco #PRTA24) cast into inlet side of tank top
- (1) Tank riser adapter (Orenco #PRTA30) cast into outlet side of tank top
- (1) 3,000 gallon monolithic concrete pump basin (Jensen Precast HS3000-EPP)
- (1) Tank riser adapter (Orenco #PRTA24) cast into inlet side of tank top
- (1) Tank riser adapter (Orenco #PRTA30) cast into outlet side of tank top

Primary/MBBR/Clarifier Aerobic Treatment System:
- (1) Complete System by Orenco Systems

Details with support components, (Compressors, TCOM Control/Alarm, Hold-Downs, Etc.) to be provided by manufacturer

Pumping Systems:
- (2) Pump controller/alarm panel (Orenco #MVP-DAX2ROSA)
- (4) Conduit Seals (Orenco #SBCS100)
- (2) Float Switch Assembly (Orenco #MF4P)
- (4) Electrical Splice Boxes (Orenco #SBEX4)
- (4) Tank Riser Outlet Grommets (Orenco G200L)
- (2) Discharge assemblies with check valve, ball valve (Orenco HV200BC)
- (2) Discharge assemblies with anti-siphon valve, ball valve (Orenco HV200BAS)
- (1) Pump Vault w/ Screen Effluent Filter (Orenco PVU72-2419)
- (1) Pump Vault w/ Screen Effluent Filter (Orenco PVU84-2419)
- (4) Pumps (Orenco PF500512)

Piping
- SCH40 PVC to be used for all leach system plumbing
- 4" SDR-35 pipe for tank to tank, and tank connections
- 4" SCH40 ABS pipe between building plumbing stacks and septic tanks or treatment unit
- (All Pipe Joints are to be solvent welded)

Miscellaneous:
- (2) 18"tall x 24"dia Tank access risers (Orenco #RR2418)
- (2) 18"tall x 30"dia Tank access risers (Orenco #RR3018)
- (21) 12"tall X 24"dia Tank access risers (Orenco #RR2412)
- (2) 30"dia Gasketed security lid (Orenco #FLD30G)
- (23) 24"dia Gasketed lids (Orenco #FLD24G)
- (12) PVC Gate Valves (Orenco #VG2000S)
- (396) Orifice Shield (Orenco #OS200)
- (25) Sealant needed (Orenco #ADH200)
- (1) Automatic Distribution Valve (Orenco #V6404)
- (1) Distribution Valve Vault (Orenco #RR3018)
- (1) Insulated Vault Lid (Orenco #FLD30GI2)

ABS & PVC Piping, and Misc. ABS & PVC Fittings as Required, 3/8" Pea Rock as Needed, ¼" Mesh Gopher Barrier, Valve Boxes, Etc.

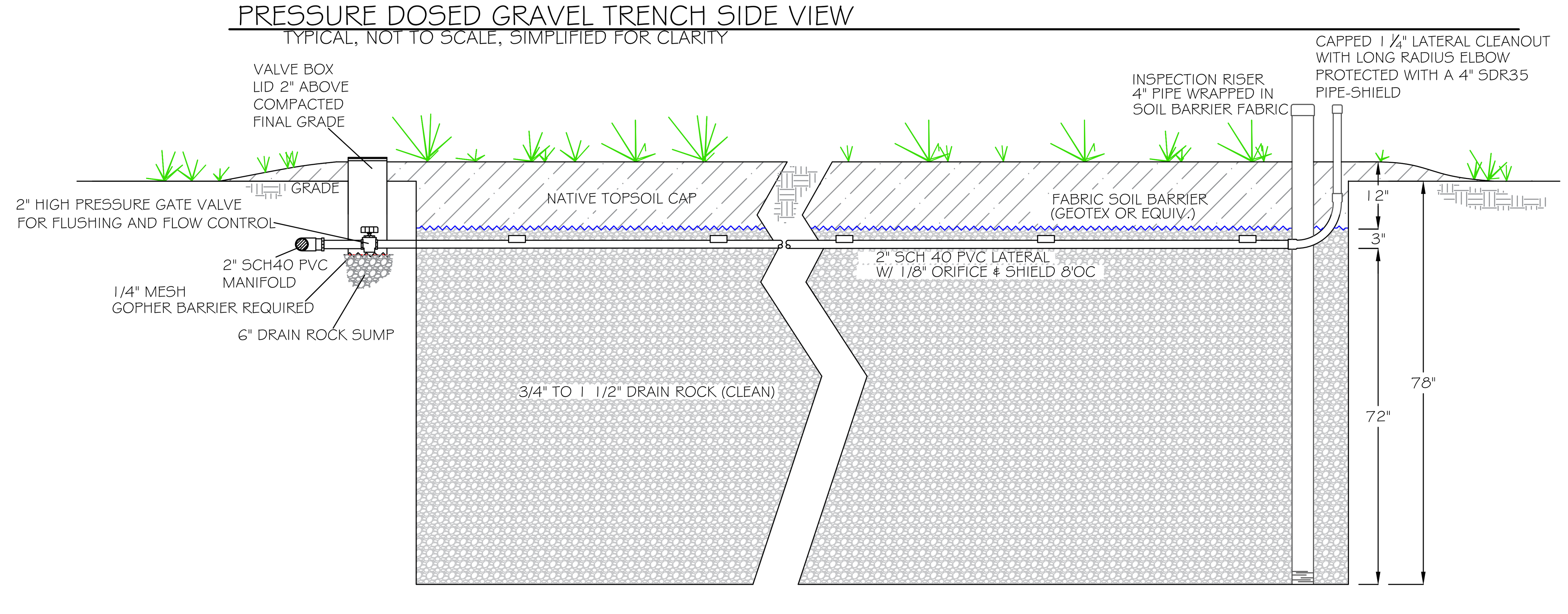
Plumbing
- SCH40 PVC to be used for all leach system plumbing
- 4" SDR-35 pipe for all tank to tank connections
- SCH40 ABS pipe between building plumbing stacks and septic tanks or treatment unit
- As specified by civil engineering plan
- (All Pipe Joints are to be solvent welded)

Media:
- Clean (dust free) ¾" to 1 ½" drain rock

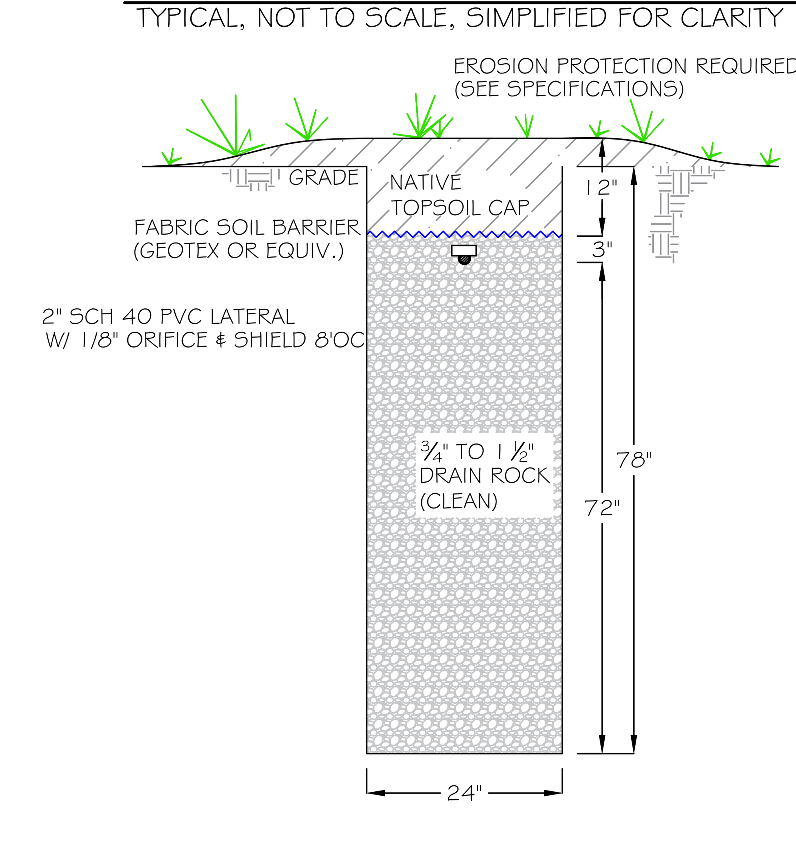
WASTEWATER SYSTEM #1
DETAIL SHEET

Erosion Protection
The erosion protection specifications are required to be in place by October 15th and through May 15th, in addition to any other time there is a risk of soil erosion. The resulting disturbed terrain from the wastewater system installation process shall be protected using the following techniques:
1) The area is to be protected by a straw or "pine straw" layer of 1" to 3" deep.
2) Erosion protection measures may have to be reapplied if they are compromised by wind, animals, or by other means prior to the rain season.
3) A silt barrier may be required down-slope from all disturbed ground.

PRESSURE DOSED GRAVEL TRENCH SIDE VIEW
TYPICAL, NOT TO SCALE, SIMPLIFIED FOR CLARITY



PRESSURE DOSED GRAVEL TRENCH END VIEW
TYPICAL, NOT TO SCALE, SIMPLIFIED FOR CLARITY



SEPTIC SYSTEM CONSULTING & DESIGN BY:

REGISTERED ENVIRONMENTAL HEALTH SPECIALIST
DONALD H. MYERS
No. 6587
STATE OF CALIFORNIA

DON MYERS, REHS
14801 TWIST ROAD
JAMESTOWN, CA 95327
(209) 743-9493

"#1 Consultant In The #2 Business"

PROJECT SITE:
UNDER CANVAS
HARDIN FLAT ROAD
GROVELAND, CA

APN: 068-120-062
& 068-120-063
+/-85.1 ACRES

CURRENT PROPERTY OWNER(S):
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P.O. BOX 130
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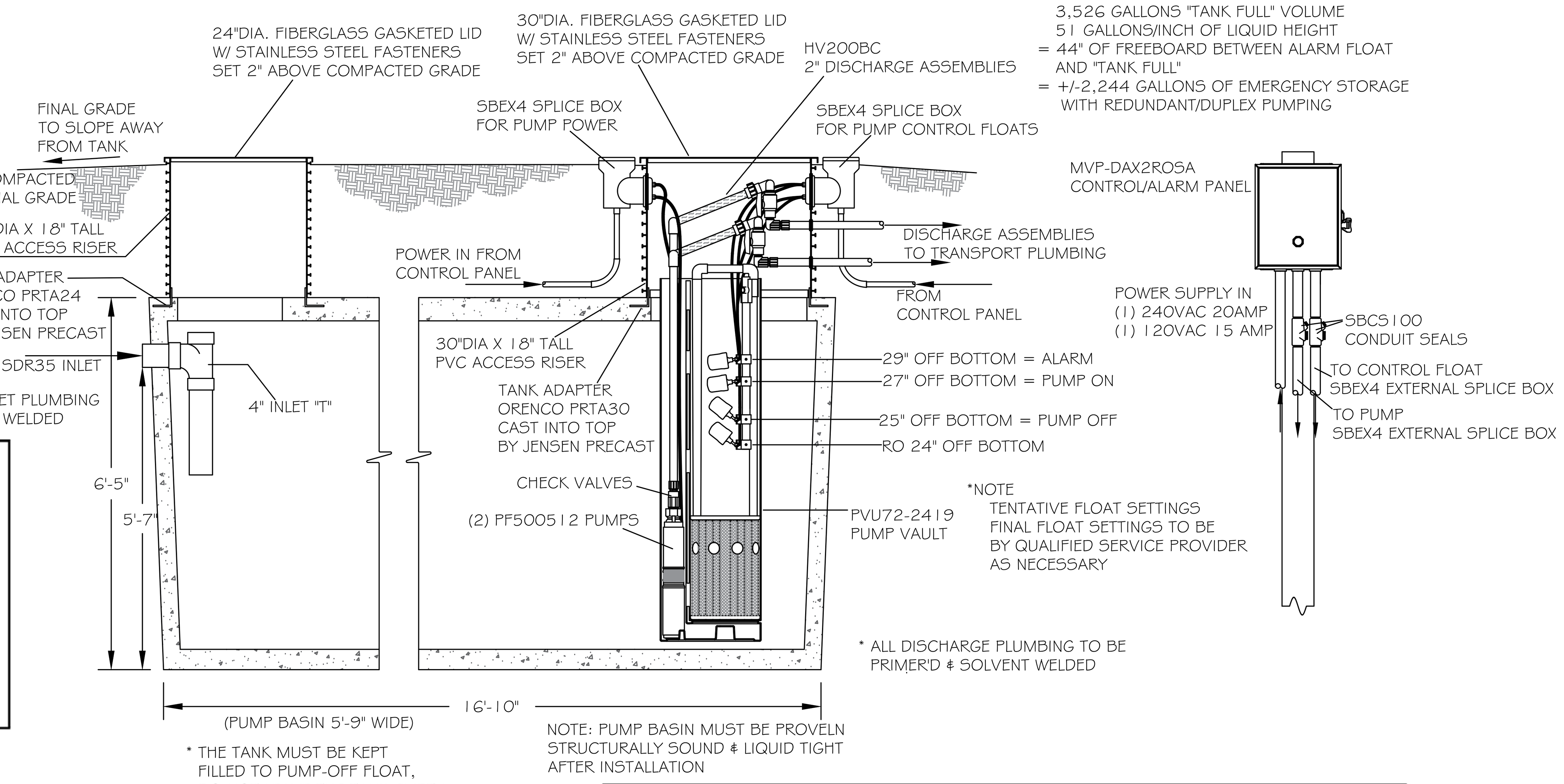
JULY 16, 2019 SCALE: AS SHOWN PAGE 5 OF 7
UNDER CANVAS
COMMERCIAL WASTEWATER SYSTEM

Note: Plan set For Wastewater System Approval Only

WASTEWATER SYSTEM #2
MONOLITHIC CONCRETE PUMP BASIN WITH DUPLEX PUMPING SYSTEM

TYPICAL NO SCALE, SIMPLIFIED FOR CLARITY

(3,000 GALLON JENSEN PRECAST HS-3000)



*ALL GRAVITY INLET PLUMBING TO BE SOLVENT WELDED

Electrical Requirements

1) All electrical supply connections must be covered by an "Electrical Permit" issued by the local regulatory authority.
2) Any person performing the wiring work must be competent and familiar with the electrical requirements of this proposed septic system.
3) This proposed septic system requires (2) separate electrical supply circuits to the septic system control panel.
4) A dedicated 20amp, 240vac min. circuit is required for running the pumps.
5) A separate 15amp, 120vac min. circuit is to be supplied to the septic system control panel for alarm/ control functions.
6) The pump control circuit and the alarm/ control circuit <u>must not</u> use a common circuit breaker at the main electrical panel, or a household electrical sub-panel.

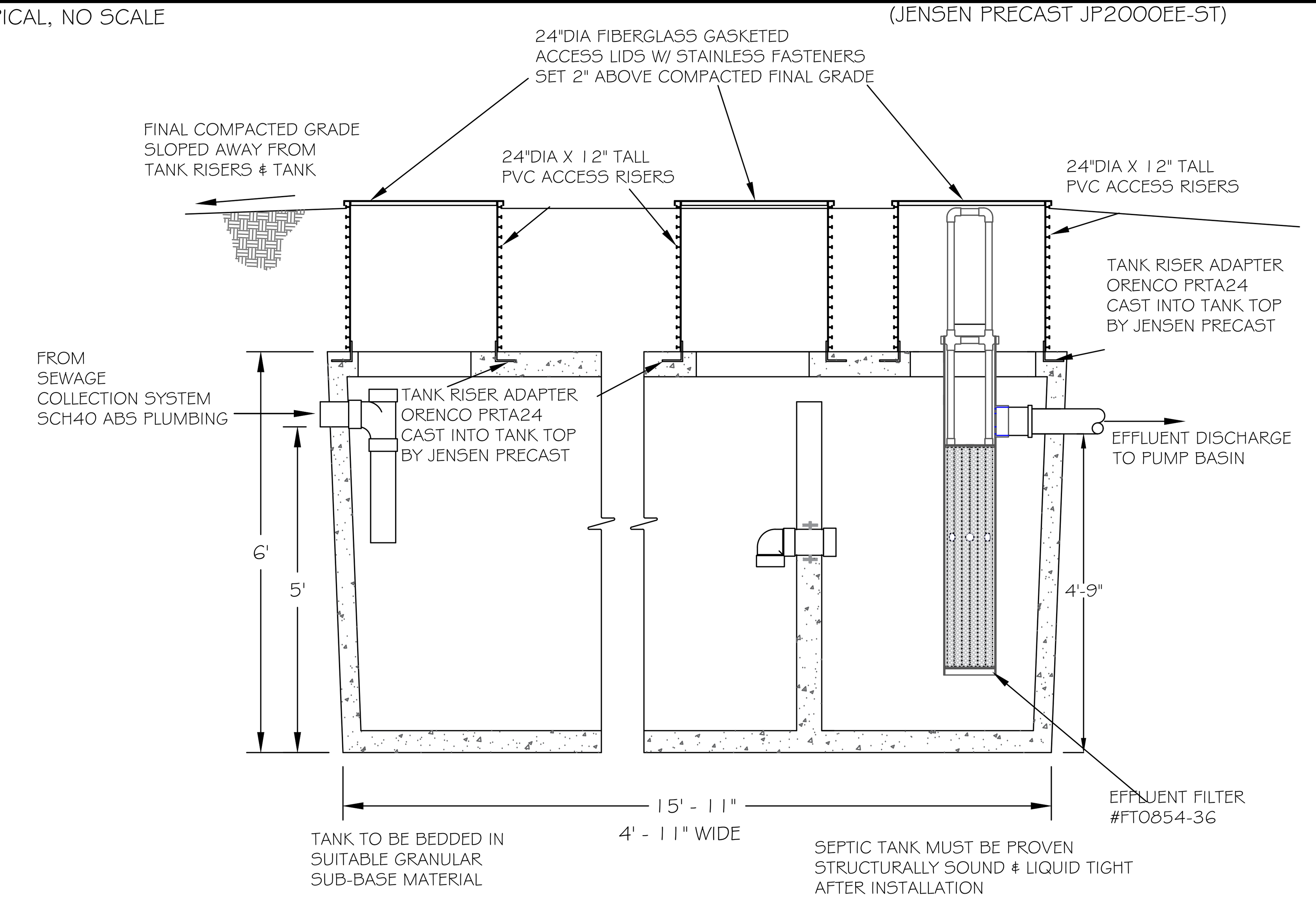
* THE TANK MUST BE KEPT FILLED TO PUMP-OFF FLOAT, DO NOT LEAVE THE TANK EMPTY AFTER PUMPING IN MONTHS OF FALL THROUGH SPRING

NOTE: PUMP BASIN MUST BE PROVEN STRUCTURALLY SOUND & LIQUID TIGHT AFTER INSTALLATION

WASTEWATER SYSTEM #2
(2) 2,000 GALLONS MONOLITHIC CONCRETE SEPTIC TANKS FOR EMPLOYEE RESTROOM & LAUNDRY

TYPICAL, NO SCALE

(JENSEN PRECAST JP2000EE-ST)



WASTEWATER SYSTEM #2 SEPTIC TANK, PUMPING SYSTEM, & LEACH SYSTEM DETAILS

* ALL TANKS MUST BE KEPT FILLED WITH LIQUID, FILL TANKS WITH WATER AFTER PUMPING

Pump Selection for a Pressurized System - Single Family Residence Project

Under Canvas Project; Hardin Flat Road, Groveland / Septic System #2, Pressure Dosed Gravel Trenches, 06/21/2019

Parameters

Discharge Assembly Size	2.00	inches
Transport Length	260	feet
Transport Pipe Class	40	
Transport Line Size	2.00	inches
Distributing Valve Model	None	
Max Elevation Lift	10	feet
Manifold Length	75	feet
Manifold Pipe Class	40	
Manifold Pipe Size	2.00	inches
Number of Laterals per Cell	4	
Lateral Length	210	feet
Lateral Pipe Class	40	
Lateral Pipe Size	2.00	inches
Orifice Size	1/8	inches
Orifice Spacing	8	feet
Residual Head	5	feet
Flow Meter	None	inches
'Add-on' Friction Losses	0	feet

Calculations

Minimum Flow Rate per Orifice	0.43	gpm
Number of Orifices per Zone	108	
Total Flow Rate per Zone	46.9	gpm
Number of Laterals per Zone	4	
% Flow Differential 1st/Last Orifice	1.9	%
Transport Velocity	4.5	fps

Frictional Head Losses

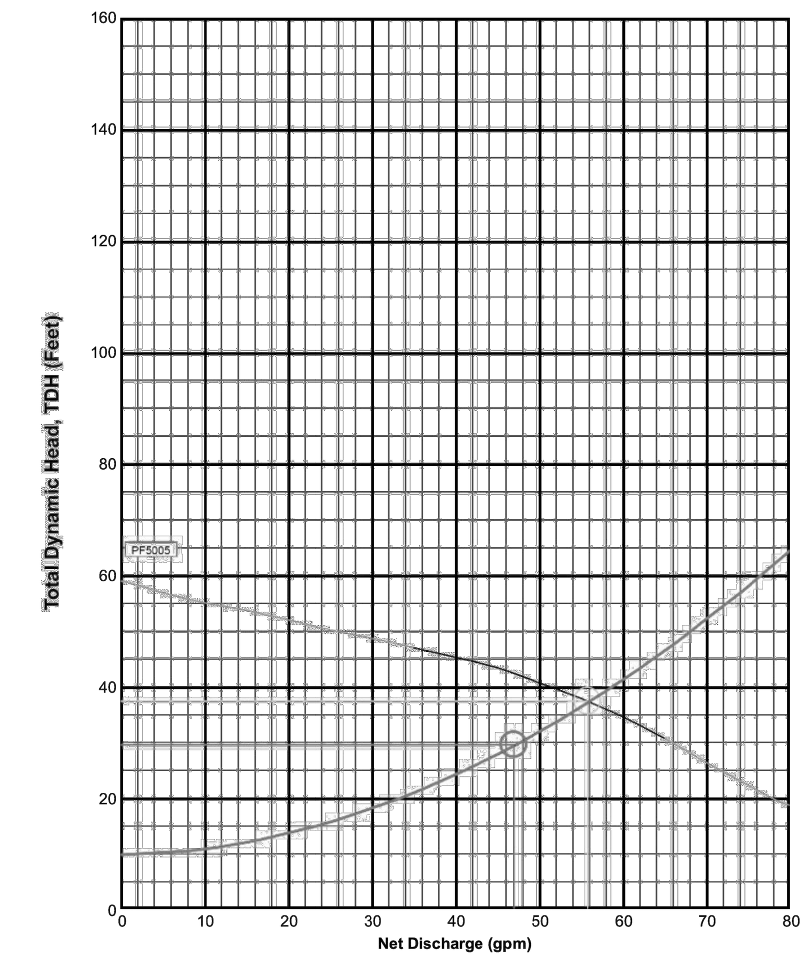
Loss through Discharge	4.4	feet
Loss in Transport	9.2	feet
Loss through Valve	0.0	feet
Loss in Manifold	0.7	feet
Loss in Laterals	0.2	feet
Loss through Flowmeter	0.0	feet
'Add-on' Friction Losses	0.0	feet

Pipe Volumes

Vol of Transport Line	45.3	gals
Vol of Manifold	13.1	gals
Vol of Laterals per Zone	146.4	gals
Total Volume	204.8	gals

Minimum Pump Requirements

Design Flow Rate	46.9	gpm
Total Dynamic Head	29.6	feet



PumpData

PF5005 High Head Effluent Pump
50 GPM, 1/2HP
115/230V 1Ø 60Hz, 200/230V 3Ø 60Hz

Legend

System Curve:
Pump Curve:
Pump Optimal Range:
Operating Point:
Design Point:

Wastewater System #2 Specific Leach Trench Construction Requirements
Modified Pressure Dosed Triple Depth Leach Trenches
Call for All Required Inspections

1) No construction is to take place with wet soil conditions.
2) Open trench inspections must be performed by Tuolumne County Environmental Health and the Design Consultant.
3) Trench side walls must be mechanically scarified so as to remove surface "glaze", "greasy", or "smeared" appearance.
4) Leach trenches are to be excavated on contour with terrain to a total depth of **<u>84"</u>** (down-hill side), with level trench bottoms of **<u>24"</u>** in width.
5) Trenches are to be constructed at a minimum of **<u>14'</u>** side wall to side wall apart (or 16' on-center).
6) <u>Inspection risers</u> are to be constructed in the trenches and inspected prior to covering with drain rock, <u>AKA Open Trench Inspection</u> (see detail drawings).
7) Drain rock is to be installed level with a minimum depth of **<u>72"</u>**.
8) Pressurized distribution laterals are to be installed level on top of the drain rock. 2" SCH40 PVC pressurized distribution laterals are to be prepared for installation by drilling 1/8" orifices **<u>96"</u>** on center and **<u>24"</u>** off trench ends (no more than **<u>108</u>** orifices in leach system). Orifices are to be orientated so that they face up and then be covered by an orifice shield after a lateral squirt test **<u>(AKA Open Install Inspection)</u>**. Distribution laterals to be connected to manifold via 1 ¼" <u>gate valves</u> (Do not use ball valves). A septic tank liquid tightness test should also be performed at this time.
9) Place drain rock over the pressure laterals to a depth of **<u>6"</u>**.
10) Filter fabric is to be used as a soil barrier over drain rock.
11) Native organic topsoil is to be used as the final capping fill over constructed leach trenches to a total depth of **<u>12"</u>** over filter fabric.
12) Final grading is to allow for a gently sloped toe on the down-slope, and be contoured into the terrain on the up-slope of the leach trench system (the final grading should not have a speed bump appearance).
13) A "V" ditch may be required in the final grading up-slope from the leach trenches to re-direct surface run-off around and away from the sand trenches.
14) Erosion prevention measures are to be completed after final grading on all disturbed ground (see details).
15) <u>A Final inspection is required before the job is considered complete</u>. Inspection includes: erosion protection, lateral end cap squirt test (generator may be used), alarm function, and others as necessary.

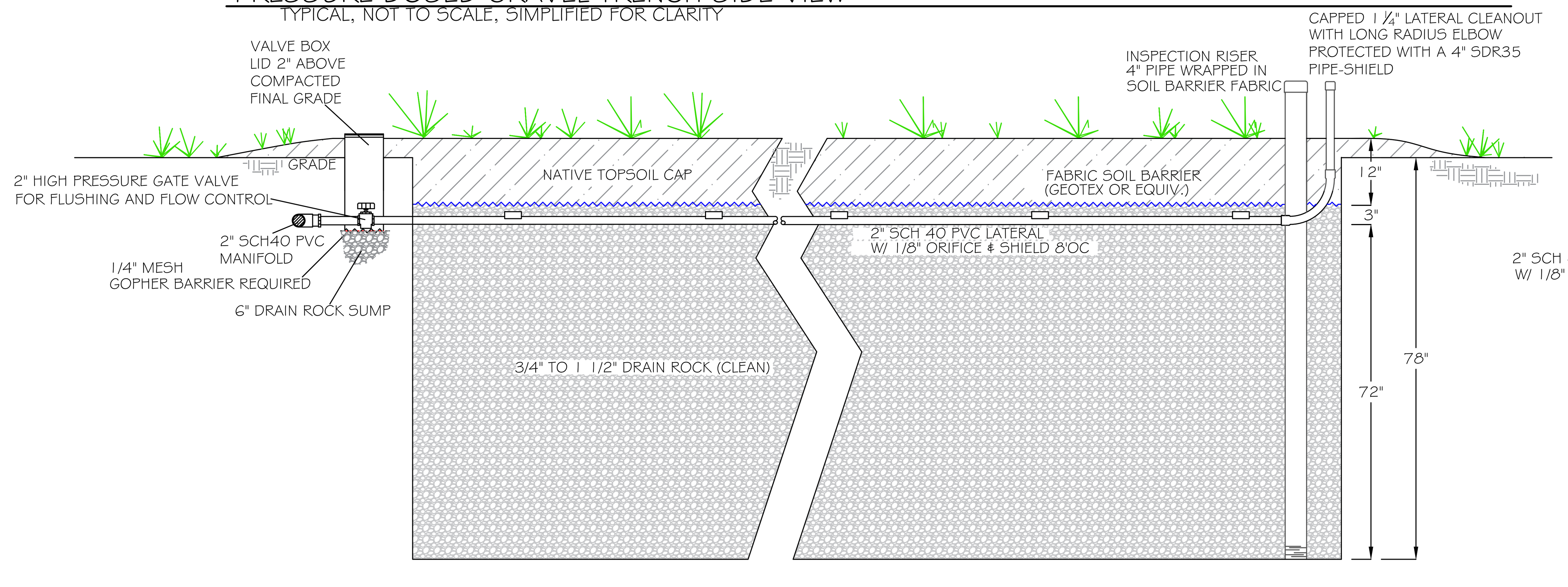
Erosion Protection

The erosion protection specifications are required to be in place <u>by October 15th and through May 15th</u>, in addition to any other time there is a risk of soil erosion. The resulting disturbed terrain from the wastewater system installation process shall be protected using the following techniques:

1) The area is to be protected by a straw or "pine straw" layer of 1" to 3" deep.
2) Erosion protection measures may have to be reapplied if they are compromised by wind, animals, or by other means prior to the rain season.
3) A silt barrier may be required down-slope from all disturbed ground.

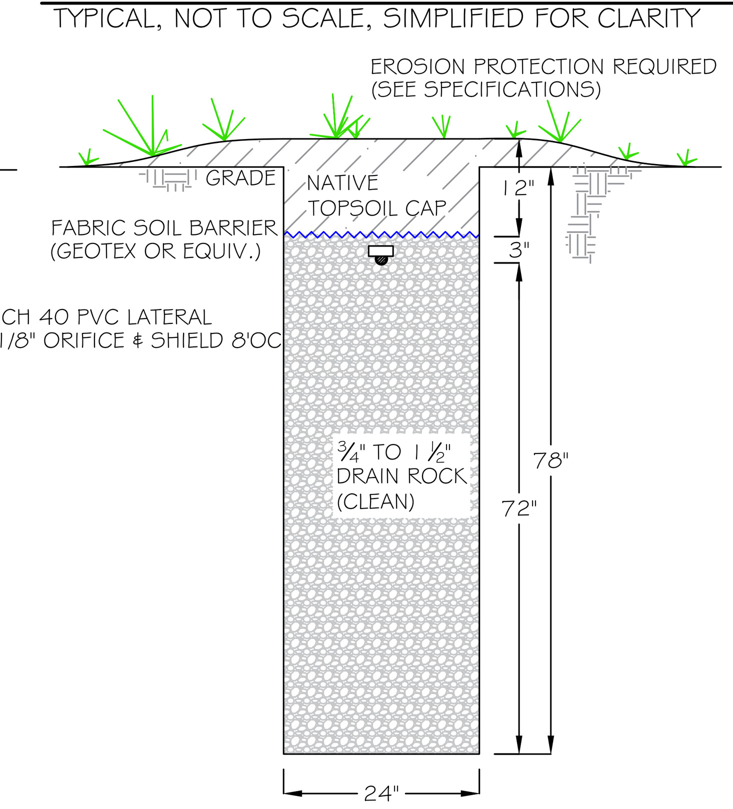
PRESSURE DOSED GRAVEL TRENCH SIDE VIEW

TYPICAL, NOT TO SCALE, SIMPLIFIED FOR CLARITY



PRESSURE DOSED GRAVEL TRENCH END VIEW

TYPICAL, NOT TO SCALE, SIMPLIFIED FOR CLARITY



REGISTERED ENVIRONMENTAL HEALTH SPECIALIST DONALD H. MYERS No. 6587 STATE OF CALIFORNIA

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(209) 743-9493
"#1 Consultant In The #2 Business"

PROJECT SITE:
UNDER CANVAS
HARDIN FLAT ROAD
GROVELAND, CA
APN: 068-120-062 & 068-120-063
+/-85.1 ACRES

CURRENT PROPERTY OWNER(S):
HARDIN FLAT LLC
P.O. BOX 130
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JULY 16, 2019 SCALE: AS SHOWN PAGE 6 OF 7
UNDER CANVAS
COMMERCIAL WASTEWATER SYSTEM
Note: Plan set For Wastewater System Approval <u>Only</u>

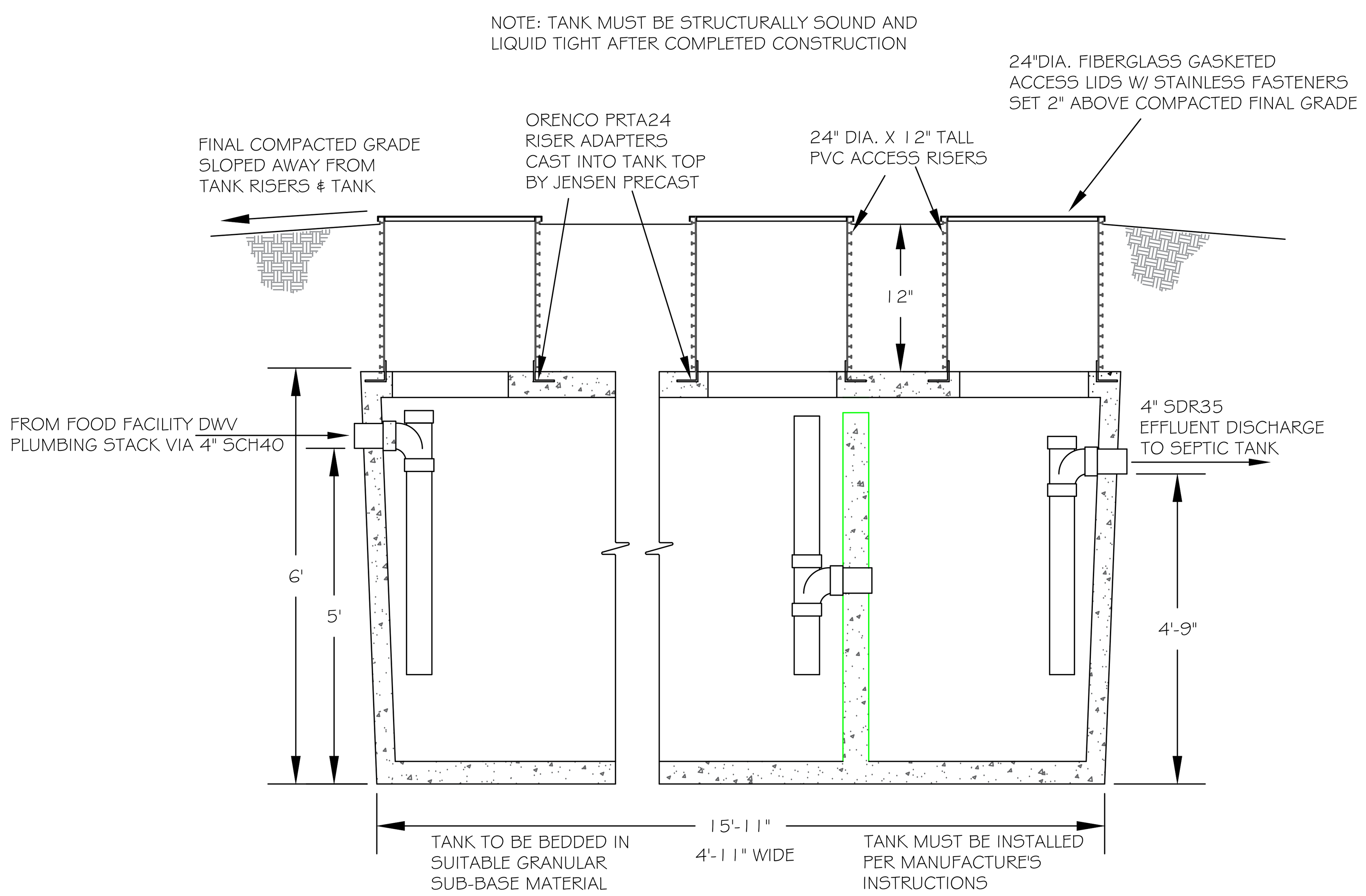
WASTEWATER SYSTEM #2 FOOD FACILITY
WASTEWATER TREATMENT SYSTEM DETAILS

* ALL TANKS MUST BE KEPT FILLED WITH LIQUID, FILL TANKS WITH WATER AFTER PUMPING

FOOD FACILITY GREASE (FOG) INTERCEPTOR

(JENSEN PRECAST MODEL #JP2000EPE-G)

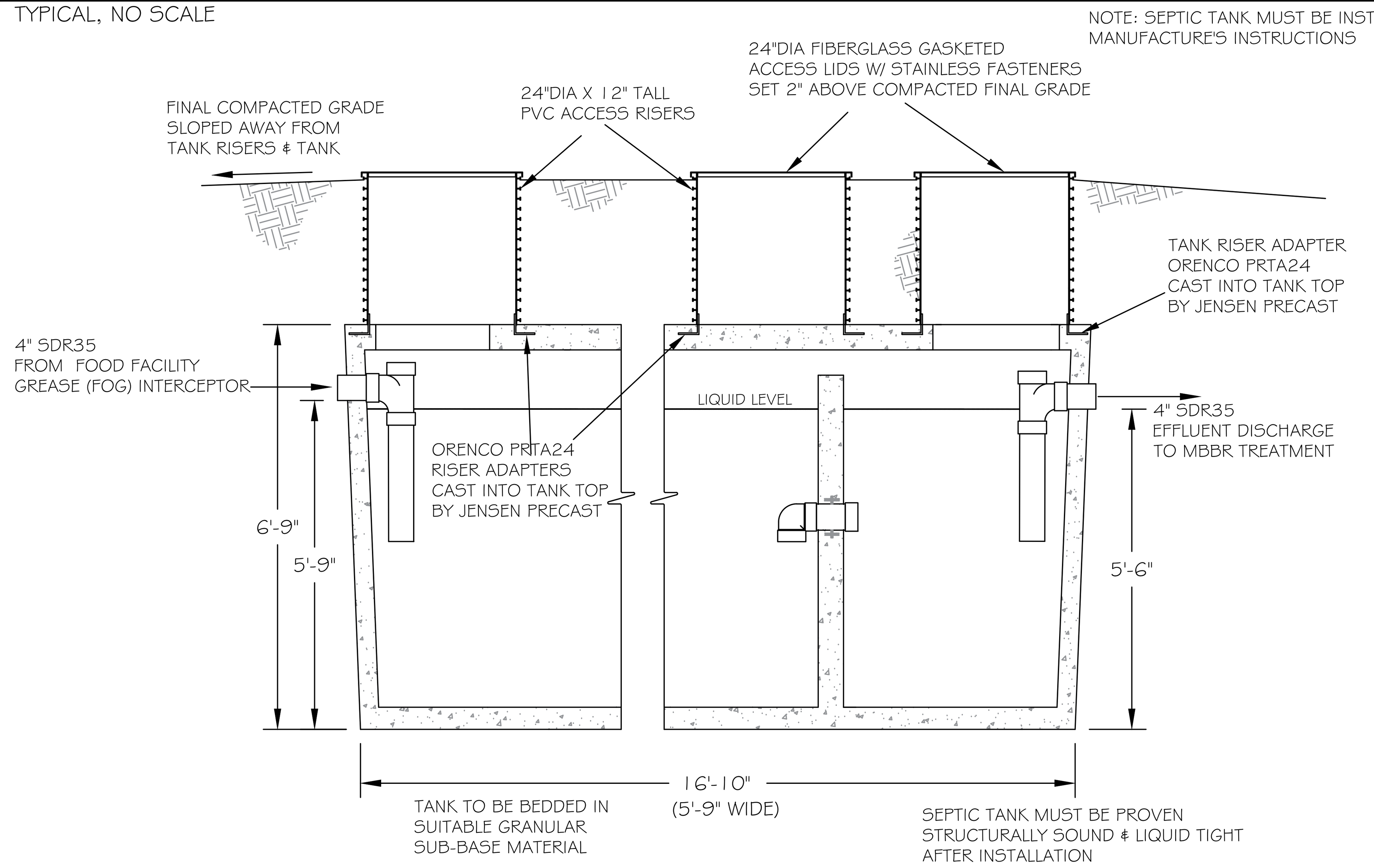
TYPICAL, NO SCALE 2,000 GALLON MONOLITHIC TANK



FOOD FACILITY 3,000 GALLON SEPTIC TANK

(JENSEN PRECAST JZ3000EE-ST)

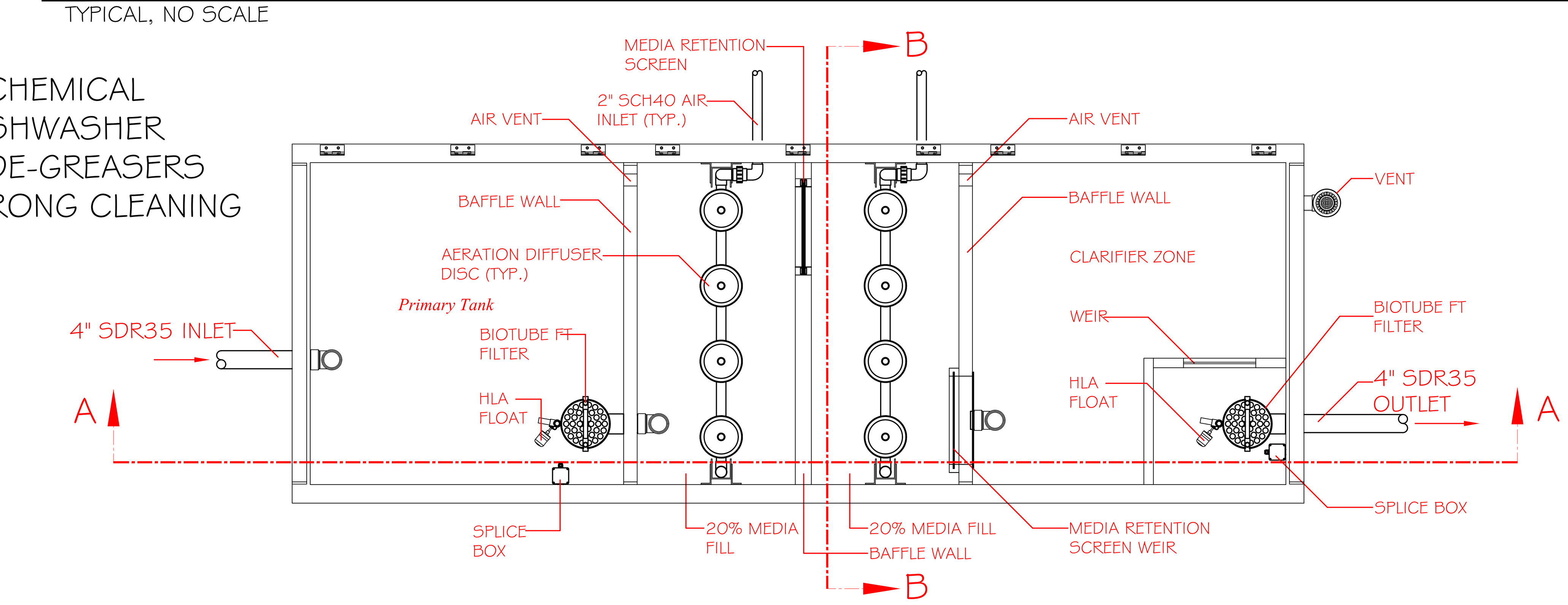
TYPICAL, NO SCALE



NOTE:
- *DO NOT USE CHEMICAL SANITIZING DISHWASHER
- *DO NOT USE DE-GREASERS OR HARSH/STRONG CLEANING CHEMICALS

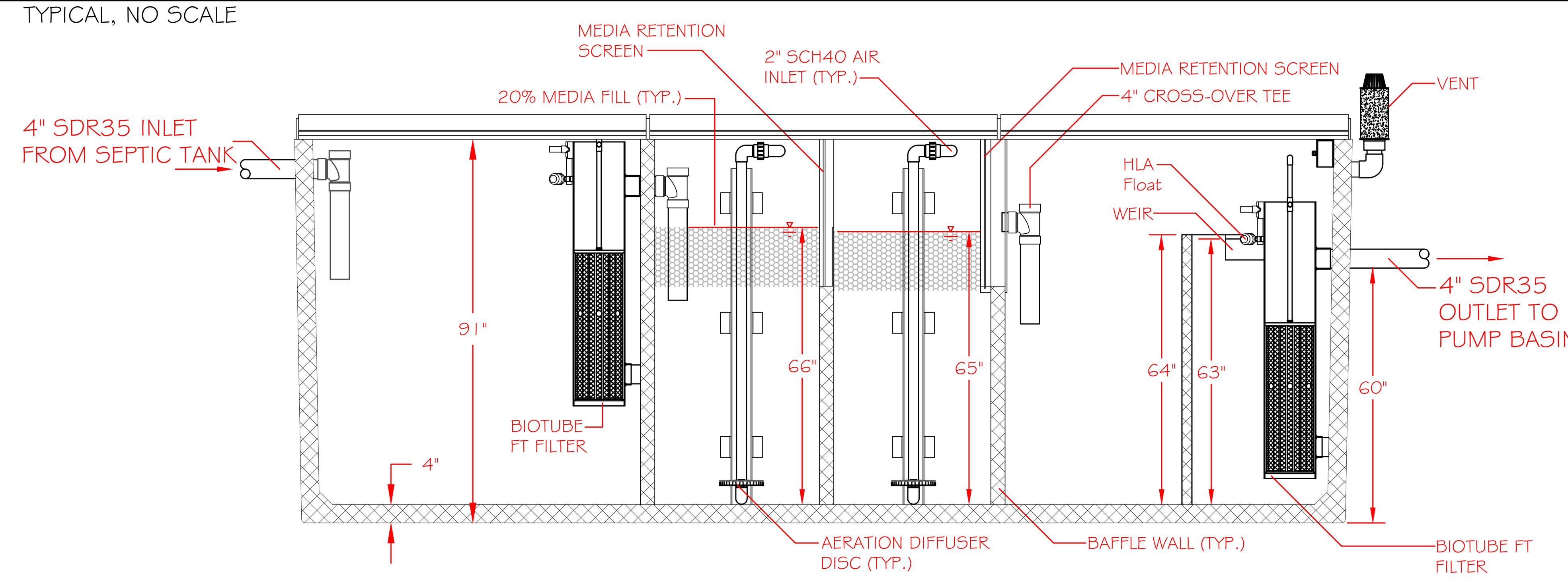
ORENCO 21' MOVING BED BIOFILM REACTOR (MBBR) WITH CLARIFIER BASIN TOP VIEW

TYPICAL, NO SCALE



ORENCO 21' MOVING BED BIOFILM REACTOR (MBBR) WITH CLARIFIER BASIN SIDE CROSSECTION VIEW

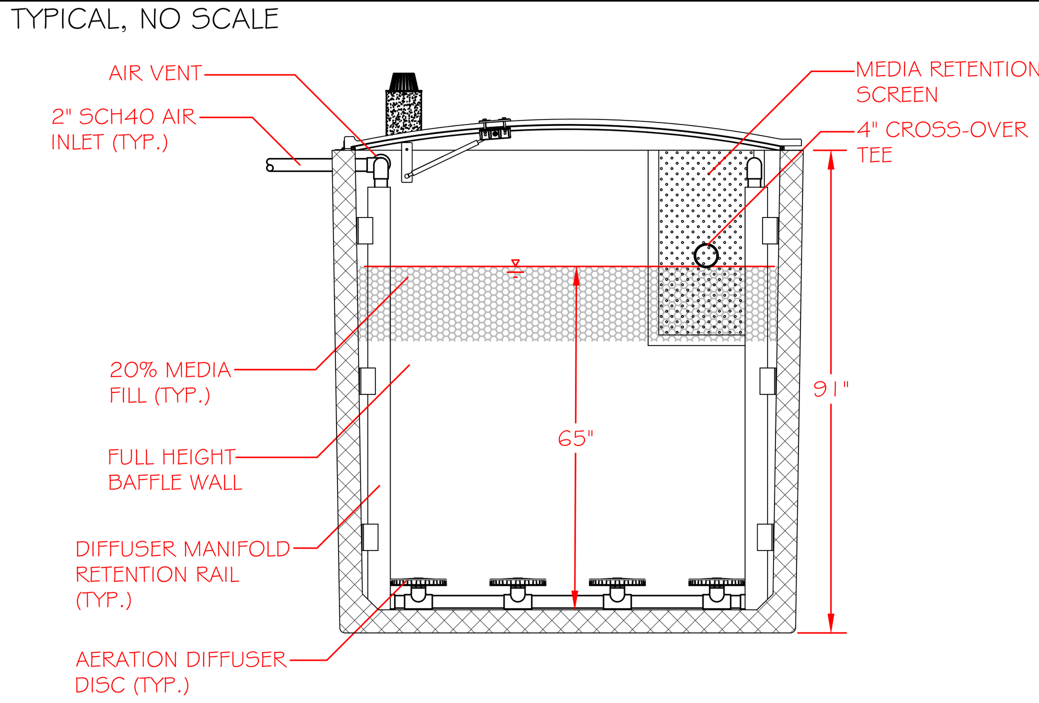
TYPICAL, NO SCALE



*THE MBBR, CONTROL/ALARM, AND AIR SUPPLY MUST BE INSTALLED PER MANUFACTURER'S INSTRUCTIONS
*ELECTRICAL REQUIREMENTS PER MANUFACTURER'S INSTRUCTIONS
*HOUSING REQUIREMENTS FOR CONTROL/ALARM, MISCELLANEOUS SUPPORT EQUIPMENT, AND AIR COMPRESSORS PER MANUFACTURER'S INSTRUCTIONS

ORENCO 21' MOVING BED BIOFILM REACTOR (MBBR) WITH CLARIFIER BASIN END CROSSECTION VIEW

TYPICAL, NO SCALE



REGISTERED ENVIRONMENTAL HEALTH SPECIALIST
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JULY 16, 2019 SCALE: AS SHOWN PAGE 7 OF 7
UNDER CANVAS
COMMERCIAL WASTEWATER SYSTEM
Note: Plan set For Wastewater System Approval Only

Don Myers, REHS

Registered Environmental Health Specialist

Qualified Wastewater System Design and Consultation
"#1 Consultant in the #2 Business"

14801 Twist Road
Jamestown, CA 95327
(209) 743-9493

January 2, 2019

Property Owner(s): Hardin Flat LLC
Project Location: Hardin Flat Road, Groveland
APN: 068-120-062 & 068-120-063

Under Canvas Inc.

Commercial Wastewater System

Public Sewer <u>is not</u> available for this parcel
Served By an Onsite Public Water System

Under Canvas Inc. is in the process of developing a pre-erected tent campground. There are 99 tents planned for throughout the 85.1 acre property. Most of the tents include a private restroom. There are some tents without restrooms that utilize a centralized bathhouse. Meals are available to guests via an on-site mobile food facility. There will also be an on-site laundry facility.

The soil profile examination revealed soils suitable for a leach system to at least 13' of depth. Excavation refusal was not encountered at any of the soil profile pits. The average percolation test result in leach area #1 is 111 mpi. A leach system application rate of .25gpd/ft^2 is used for the leach area #1 system size calculations. The average percolation test result in leach area #2 after two weeks of heavy rain is 84mpi. A leach system application rate of .375gpd/ft^2 is used for the leach area #2 system size calculations.

Proposed Wastewater Systems Overview:
For all domestic strength wastewater (BOD < 250mg/l) there will be just primary treatment via a code compliant septic tank. After primary treatment a pump package with duplex pumping (with lead/lag configuration) will pressure dose the gravel loaded leach system.

Wastewater resulting from food handling and preparation produces high strength wastewater. The food facility wastewater will be treated with a grease interceptor, post grease interceptor septic tank, followed by a moving bed biofilm reactor (MBBR). The treatment process will reduce the BOD to 200mg/l to 250mg/l prior to dispersal. Effluent dispersal will be via a duplex pumping system (with lead/lag configuration) to a pressure dosed gravel loaded leach system.

To prevent conflicts with wastewater treatment and the wastewater system, chemical sanitizing dishwashers are not allowed. If a mechanical dishwasher is utilized it should be a "high heat" type unit. All efforts necessary should be utilized to prevent "grease digesting" cleaning chemicals, and "harsh" or "high strength" chemicals from entering the wastewater system.

The wastewater system plans do not include the proposed sewage collection system. The sewage collection system will be designed by the project civil engineer, Dax Consulting, Inc. The wastewater daily flow will be divided between (2) <10,000gpd wastewater systems.

Estimated Maximum Daily Wastewater Flow Rates:
All flow rate calculations and tank sizing specifications come from Appendix H of the 2016 California Plumbing Code. Specifically 2016 California Plumbing Code, Estimated Waste/Sewage Flow Rates, Table H 201.1 (2), 9. Hotels (No kitchen); 30gpd/person. Also Chart H 901.7 Design Criteria for commercial kitchen/food preparation wastewater treatment and dispersal using disposable utensils. Per Tuolumne County Environmental Health policy, the maximum daily volumes used for wastewater system design will be maximum daily volumes at maximum occupancy. The maximum occupancy and employee/staff information was supplied by Under Canvas®.

Wastewater System #1 will be a domestic strength wastewater system which will receive primary treatment from code compliant septic tanks, and will be delivered to gravel filled leach trenches via pressure dosing. Wastewater System #2 will be a hybrid system between the high strength food facility wastewater, and the domestic strength wastewater from employees and the laundry service. The high strength food facility waste will have primary treatment via a code compliant grease interceptor and septic tank. High strength food facility wastewater will then receive secondary treatment from a properly sized moving bed bio-film reactor (MBBR) to reduce the high strength wastewater to domestic strength wastewater. Both employee generated wastewater and laundry service wastewater are treated as domestic strength wastewater, and receive primary treatment from code compliant sized septic tanks. The treated food facility wastewater, employee generated wastewater, and the laundry service wastewater are combined and delivered to a gravel filled leach system via pressure dosing.

Proposed Use	**Design GPD (Maximum)**	**Unit Per**	**Number of Units**	**GPD**
Wastewater System #1				
Tents (1-99) At Maximum Occupancy	30	Person	276	8,280
Total Wastewater System #1				**8,280**
Wastewater System #2				
Food Service Wastewater (276 Guests X 3 Meals)	2	Meal	828	1,656
Employee Generated Wastewater	20	Employee	40	800
Laundry Service	42.5	Laundry Load	26	1,105
Total Wastewater System #2				**3,561**

Provided by Don Myers, February 3, 2020