

GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARDS EVALUATION

**PROPOSED PARKFIELD FIRE
STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA**



GEOCON
CONSULTANTS, INC.

**GEOTECHNICAL
ENVIRONMENTAL
MATERIALS**

PREPARED FOR

**DEPARTMENT OF FORESTRY AND FIRE
PROTECTION (CAL-FIRE)
SACRAMENTO, CALIFORNIA**

PROJECT NO. S9962-05-23

JULY 27, 2018



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Technical Services Section
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Subject: GEOTECHNICAL INVESTIGATION AND
GEOLOGIC HAZARDS EVALUATION
PROPOSED PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

Dear Mr. Jain:

In accordance with your authorization of Task Order No. 16 of Agreement No. 3178335, we have performed a geotechnical investigation for the proposed Parkfield Fire Station located at 70331 Vineyard Canyon Road in the City of Parkfield, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our geotechnical investigation, it is our opinion that the site can be developed as proposed, from a geotechnical standpoint, provided the recommendations of this report are followed and implemented during design and construction.

The site is located in close proximity to the main strand of the San Andreas Fault Zone and adjacent to the boundaries of a state-designated Alquist-Priolo Earthquake Fault Zone. A site-specific fault rupture hazard investigation should be conducted to identify if such hazards exist at the site prior to construction of the proposed project.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON CONSULTANTS, INC.

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GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARDS EVALUATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed Parkfield Fire Station located at 70331 Vineyard Canyon Road in the town of Parkfield, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on May 11, 2018, by excavating four 8-inch diameter borings (B1 through B4) to depths of approximately 50½ and 15½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. BACKGROUND REVIEW

As a part of the preparation of this report, we reviewed the prior letter performed by The California Geological Survey (CGS) provided to us by the Client:

Preliminary Geological Hazards Investigation of the Parkfield Fire Station prepared by the California Geological Survey dated September 14, 2017.

Based on our review the letter, the subject site was explored by CGS on June 20, 2017, to obtain preliminary shallow subsurface information. Three test pits (TP-1 through TP-3) were conducted to a maximum depth of 8 feet below the ground surface. Groundwater was not encountered during the test pit excavations. Additionally, shallow percolation testing was performed at six locations (PT-1 through PT-6) within the proposed leach field area on June 21, 2017. CGS concluded and recommended the evaluation of surface fault rupture associated faults (mapped and unmapped) that may extend across the site, evaluation of the liquefaction potential of the underlying soil below structures, and potential expansive soils. A copy of the letter is provided herein in Appendix C.

3. SITE AND PROJECT DESCRIPTION

The subject site is located at 70331 Vineyard Canyon Road in the town of Parkfield, California. The subject site is a 38-acre parcel of land that is bisected by Vineyard Canyon Road. The majority of the site lies south of Vineyard Canyon road and is relatively flat-lying vacant land used for agriculture. The area north of the road is gently sloping to the south and currently occupied with a single-family residential structure and appurtenant structures. Vegetation in the area of the site consists of grass, shrubs and localized trees.

Based on the information provided by the Client, it is our understanding that the proposed project will be sited on the south side of the parcel within the relatively flat vacant land. The project will consist of the construction of a fire station building consisting of apparatus and living quarters, a generator and pump building, a well house, a water storage tank (storage capacity of 50,000 gallons), and other support structures. The main building will be occupied more than 2,000 man-hours per year and is considered habitable structure per the California Administrative Code. The preliminary design layout presenting the proposed project, as well as the existing site conditions are depicted on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for proposed improvements will be up to 150 kips, and wall loads will be up to 1.5 kips per linear foot, which is typical for this type of structure.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure or improvement, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

4. GEOLOGIC SETTING

The site is situated in an alluvial valley in the unincorporated community of Parkfield. Regionally, the site is located within the Coast Ranges geomorphic province which consists of mountains and valleys located between the Great Valley of California and the Pacific Ocean. The geologic structure of the Coast Ranges is controlled by faults and folds, with the principal fault being the San Andreas Fault Zone. The majority of this province is underlain by the Jurassic age Franciscan Formation which is a mélange of folded and faulted blocks of volcanic and sedimentary rocks that were once part of an oceanic trench complex. Younger Tertiary and subsequently folded to form the Coast Ranges.

5. SOIL AND GEOLOGIC CONDITIONS

Locally, the site is situated in an alluvial valley formed from sediments derived from the adjacent Middle Mountain and the Cholame Hills. The site is underlain by interfingered alluvial and colluvial sediments from the Cholame Creek drainage and adjacent slopes to the north of the site. The site is shown with respect to local geologic conditions on Figure 3, Local Geologic Map. Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

5.1 Tilled Soil

The upper surface of the project site is capped by 1 to 2 feet of tilled or plowed soil generally consisting of dark brown silty sand, which can be characterized as slightly moist and firm. Deeper areas of tilled soil or disturbed soil may exist between excavations and in other portions of the site that were not directly explored.

5.2 Undifferentiated Alluvial and Colluvial Deposits

Undifferentiated Holocene age alluvial and colluvial sediments were encountered beneath the tilled soil, consisting of soft to firm clay and silt and medium dense sand and silty sand which gradually coarsen to very dense sand down to a depth of 41 feet. The sand is further underlain by hard clay and silt to the total depth explored of approximately 50½ feet.

6. GROUNDWATER

The site is elevated above the local groundwater producing areas that are generally located along Cholame Creek, approximately 2,180 feet to the southwest (Monterey County, 2010).

The USGS National Water Information System Website (2018) has cataloged various wells in the vicinity of the subject site. According to the USGS, the closest groundwater well with available data is USGS Well No. 355405120263301, located approximately 1,000 feet northeast of the project area. Review of the monitoring data for this well indicates monitoring data is available for the monitoring period between 1979 and 1994. During this time, the depth to groundwater has fluctuated between high and low measurements of 16.18 feet below the existing ground surface (measured on April 19, 1983) to 59.52 feet below the existing ground surface (measured on April 16, 1985), respectively. The most recent groundwater level measurement for Well No. 355405120263301 was measured in April, 1994 and groundwater was at a depth of approximately 34.38 feet below the existing ground surface. Groundwater data may not be representative of the subject site as geologic conditions may vary from the well location due to the positioning of the site within a valley deposits.

A domestic water well is located in the central portion of the subject site constructed in 1990. Although there are no continuous records of water levels for this well, according to the well construction records, the static water level at the time of construction was 70 feet below the ground surface. According to the property owner (personal communication), the current depth to groundwater in the well is 70 feet below the ground surface.

Based on these considerations, as well as the referenced CGS letter for the site, the historic high groundwater level of the site is considered to be approximately 16 feet below the ground surface.

Groundwater was not encountered in our field explorations, excavated to a maximum depth of 50½ feet below the existing ground surface. Based on available groundwater level data in the site vicinity and the depth of the proposed construction, groundwater is not anticipated to impact the proposed construction. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. Proper surface drainage of irrigation and precipitation will be critical to future performance of the project. Recommendations for drainage are provided in the *Surface Drainage* section of this report (see Section 8.16).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018a). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

As shown on Figure 4, Alquist Priolo Earthquake Fault Zone Map, the site is located in close proximity to the main strand of the San Andreas Fault Zone and adjacent to a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2018b; CGS, 2015). According to Manson (1985), the main trace of the San Andreas Fault Zone is approximately 780 feet to the northeast and the Southwest Fracture Zone is approximately 2,300 feet to the southwest of the site, respectively. The San Andreas fault zone, California's most prominent structural feature, trends in a general northwest direction for almost the entire length of the state. The San Andreas Fault consists of several en echelon or subparallel fault strands, most of the strands exhibit right-lateral strike-slip sense of displacement. Wallace (1968) estimated the recurrence interval for a Magnitude 8.0 earthquake along the total length of the fault to be between 50 and 200 years. More recent data by Sieh (1984) indicates an average earthquake recurrence interval of 140 to 200 years for the San Andreas Fault. Locally, the Parkfield-Cholame Segment of the San Andreas is thought to have a recurrence interval of approximately 30 years.

As previously noted, the proposed project site is situated between the main San Andreas Fault and the Southern Fracture Zone. These two fault zones coalesce approximately 3 miles to the northwest at Middle Mountain. According to the Fault Evaluation Report Figure 2A (Manson, 1985) an unnamed fault strand may project from the north towards the project site. This unnamed strand is interpreted to have Quaternary displacement; however, the fault could not be identified in aerial photographs south of Vineyard Canyon Road. Furthermore, a review of historical satellite imagery and as discussed by Mr. Don Lindsay (CGS, 2017), there appears to be tonal lineament across the northwest-trending drainage that not only parallels the fault but projects towards the project site. Although the lineation could be fault-related, it could potentially also represent the contact between alluvial (stream channel) deposits and the colluvial deposits derived from the adjacent hillside. Additional site-specific studies are warranted to evaluate if these features are fault related and if there is a potential for surface fault rupture at the proposed project site.

Other nearby active faults are the San Juan Fault Zone, the Nunez Fault, Rinconada Fault Zone, the Pond Fault, and the Pine Rock Fault Zone located approximately 11.4 miles southeast, 21.8 miles north, 23 miles southwest, 65 miles southwest, and 47 miles northwest of the site, respectively (Ziony and Jones, 1989). The site is located in the seismically active Central California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 5, Regional Fault Map.

7.2 Seismicity

Several moderate earthquakes have occurred along the Parkfield-Cholame segment of the San Andreas fault. Near Parkfield and the Cholame Valley, historic events near the project site in 1922, 1934, 1966, and 2004 have fault displacement up to about 2 inches from right-lateral strike-slip movement (Manson 1985). The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 6, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Central California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Cholame Valley	March 10, 1922	6.5	15	SE
SW of Lompoc	November 4, 1927	7.3	85	SSW
Parkfield, CA	June 8, 1934	6.0	10	SE
N of San Simeon	November 22, 1952	6.0	44	WSW
NW of Parkfield	June 27, 1966	5.5	4	WNW
Coalinga	May 2, 1983	6.7	24	NNE
Loma Prieta	October 18, 1989	7	112	NW
Parkfield, CA	September 28, 2004	6.0	7	SE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Central California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

7.3 Ground Motion Hazard Analysis

Ground motion hazard analyses were performed in accordance with ASCE 7-10 and Section 1613A of the 2016 CBC utilizing the computer program EZFRISK (version 7.65) in conjunction with data from the U.S. Seismic Design Maps web application provided by the USGS.

7.3.1 Probabilistic Seismic Hazard Analysis

The probabilistic Maximum Considered Earthquake (MCE_R) response spectrum consists of the spectral response accelerations which are expected to achieve a 1 percent probability of collapse within a 50-year period, evaluated at 5 percent damping. The procedure described in ASCE 7-10 Section 21.2.1.1 as Method 1 was used to evaluate the probabilistic response spectrum.

The spectral response accelerations having a 2 percent chance of exceedance in 50 years were evaluated at 5 percent damping. The probabilistic analysis was performed using the ground motion prediction equations (GMPEs) of Abrahamson and Silva (2008) NGA MRC (next generation attenuation, maximum rotated component), Boore and Atkinson (2008) NGA USGS 2008 MRC, Campbell-Bozorgnia (2008) NGA USGS 2008 MRC, and Chiou-Youngs (2007) NGA USGS 2008 MRC. Each GMPE was assigned an equal weight and the maximum rotated component of ground motion derived from the relationships was evaluated. It is our opinion that the use of these four GMPEs is appropriate for the subject site.

The probabilistic analysis was performed by evaluating the ground motions generated by active faults within a 120 mile (200 kilometer) radius of the site. The soil underlying the site was modeled as a Site Class “D” with a corresponding average shear wave velocity (V_{s30}) of 270 meters per second. Although there are liquefiable soils underlying the site, we assume that the proposed single-story structure will have a fundamental period of less than 0.5 seconds and therefore will not require a site-response analysis.

The GMPE of Campbell and Borzorgnia requires that the depth to where the shear wave velocity reaches 2.5 kilometers per second ($Z_{2.5}$) be defined. Additionally, the GMPE of Abrahamson and Silva requires that the depth to where the shear wave velocity reaches 1 kilometer per second ($Z_{1.0}$) be defined. The values of $Z_{2.5}$ and $Z_{1.0}$ were estimated using data from the USGS Bay Area Seismic Velocity Model. The values of $Z_{2.5}$ and $Z_{1.0}$ used in the analysis are 0.694 kilometers and 192 meters, respectively.

According to ASCE 7-10 Section 21.2.1.1 Method 1, the probabilistic MCE_R spectral response accelerations may be determined as the product of the spectral response accelerations having a 2 percent chance of exceedance in 50 years and the risk coefficient C_R . The value of C_R at 0.2 seconds (C_{RS}) and 1 second (C_{R1}) were determined from ASCE 7-10 Figures 22-17 and 22-18, respectively. At spectral response accelerations less than or equal to 0.2 seconds, the value of C_R was taken as C_{RS} and at spectral response accelerations greater than or equal to 1.0 seconds the value of C_R was taken as C_{R1} . Linear interpolation was used to evaluate the values of C_R between 0.2 and 1.0 seconds.

7.3.2 Deterministic Seismic Hazard Analysis

The deterministic analysis was performed using the same GMPEs as the probabilistic analysis, as well as the same active faults within a 120 mile (200 kilometer) radius of the site and the same values of $Z1.0$ and $Z2.5$. The 84th percentile of the maximum rotated component of ground motion derived from the GMPEs was evaluated.

Based on the results of the analysis, the fault source resulting in the highest spectral accelerations would be a magnitude 8.2 event on the Southern San Andreas fault.

The 84th percentile of the maximum rotated component of ground motion was compared to the Deterministic Lower Limit MCE_R response spectrum, and the maximum values taken as the deterministic MCE_R response spectrum.

7.3.3 Site-Specific Response Spectrum

The lesser of the probabilistic and deterministic MCE_R response spectrums is the Site-Specific MCE_R . Two-thirds of the Site-Specific MCE_R is the Design Earthquake (DE) Response Spectrum, provided the results are not less than 80 percent of the General Design Response Spectrum determined by ASCE 7-10 Section 11.4.5.

Graphical representations of the analyses are presented on Figures 7 and 8. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on Figure 9 and as Table 1.

7.3.4 Mapped Acceleration Parameters

The following table summarizes the mapped acceleration parameters obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16A Structural Design, Section 1613A Earthquake Loads. The data was calculated using the computer program U.S. Seismic Design Maps, provided by the USGS. The short spectral response uses a period of 0.2 second.

MAPPED SPECTRAL ACCELERATIONS

Parameter	Value	2016 CBC Reference
MCE _R Ground Motion Spectral Response Acceleration – Class B (short), S _S	2.675g	Figure 1613A.3.1(1)
MCE _R Ground Motion Spectral Response Acceleration – Class B (1 sec), S ₁	1.319g	Figure 1613A.3.1(2)

7.3.5 Site-Specific Seismic Design Criteria

Based the site-specific ground motion hazard analysis performed, and in accordance with the ASCE 7-10 Section 21.4, site-specific seismic design parameters shall be derived using the results of the site-specific ground motion hazard analysis.

The parameter S_{DS} shall be obtained from the site-specific spectra at a period of 0.2 second and not less than 90 percent of the peak spectra acceleration at any period larger than 0.2 second. The parameter S_{D1} shall be taken as the greater of the site-specific spectral acceleration at a period of 1.0 second or twice the spectral acceleration at a period of 2 seconds (whichever is greater). The values of S_{MS} and S_{M1} shall be taken as 1.5 times the site-specific values of S_{DS} and S_{D1}. The site-specific seismic design parameters shall not be less than 80 percent of the general seismic design values determined by ASCE 7-10 Section 11.4.

The following table presents the site-specific seismic design parameters based on the site-specific ground motion hazard analysis.

SITE-SPECIFIC SEISMIC DESIGN PARAMETERS

Parameter	Value
Site Class Modified MCE _R Spectral Response Acceleration (short), S _{MS}	2.157g
Site Class Modified MCE _R Spectral Response Acceleration – (1 sec), S _{M1}	3.144g
5% Damped Design Spectral Response Acceleration (short), S _{DS}	1.438g
5% Damped Design Spectral Response Acceleration (1 sec), S _{D1}	2.096g

7.3.6 Site-Specific Peak Ground Acceleration

The site-specific Maximum Considered Earthquake (MCE_G) geometric mean peak ground acceleration was evaluated in accordance with ASCE 7-10 Section 21.5.

The probabilistic geometric mean peak ground acceleration was evaluated using the computer program EZFrisk. The analysis was performed using the GMPE of Abrahamson and Silva (2008) NGA (next generation attenuation), Boore and Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008. Each GMPE was assigned an equal weight. The analysis used the same faults, Site Class, and values of Z2.5 and Z1.0 as described above.

The probabilistic geometric mean peak ground acceleration (probabilistic MCE_G) was evaluated at a 2 percent probability of exceedance with a 50 year period.

The deterministic geometric mean peak ground acceleration (deterministic MCE_G) was evaluated as the 84th percentile geometric mean peak ground acceleration. The deterministic MCE_G shall not be less than 0.5F_{PGA}, where F_{PGA} is determined from ASCE 7-10 Table 11.8-1 with the value of PGA taken as 0.5g.

The site-specific MCE_G peak ground acceleration is taken as the lesser of the probabilistic and deterministic MCE_G, provided the value is not less than 80 percent of the value of PGA_M as determined by ASCE 7-10 Equation 11.8.1.

ASCE 7-10 SITE-SPECIFIC PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Site-Specific MCE _G Peak Ground Acceleration, PGA _M	0.853g	Section 21.5

7.3.7 Deaggregated Seismic Source Parameters

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.64 magnitude event occurring at a hypocentral distance of 2.62 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.53 magnitude occurring at a hypocentral distance of 2.94 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

According to the Monterey County Safety Element (2010), the site is located within an area identified as having a potential for liquefaction. The current standard of practice, as outlined in the “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California” and “Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California” requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

Screening criteria presented by Bray and Sancio (2006) was used to evaluate the liquefaction susceptibility of the fine-grained soils encountered in the boring. Based on these screening criteria, fine-grained soils with a plasticity index of greater than 18 and a saturated water content of less than 85 percent of the liquid limit, or with a plasticity index of less than 18 but greater than 12 and a saturated water content of less than 80 percent of the liquid limit are considered not susceptible to liquefaction. Atterberg Limits laboratory test results used for the screening criteria are presented as Figure B7.

The liquefaction analysis was performed for a Design Earthquake (DE) level by using a historic high groundwater depth of 16 feet below the ground surface, a magnitude 6.53 earthquake, and a peak horizontal acceleration of 0.569g ($\frac{2}{3}PGA_M$). The enclosed liquefaction analysis, included herein for Boring B1, indicates that the alluvial soils below the historic high groundwater level could be susceptible to approximately 0.7 inch of liquefaction settlement, during a Design Earthquake ground motion (see enclosed calculation sheets, Figures 10 and 11).

It is our understanding that the intent of the Building Code is to maintain “Life Safety” during Maximum Considered Earthquake (MCE) level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for the MCE level by using a historic high groundwater depth of 16 feet below the ground surface, a magnitude 6.64 earthquake, and a peak horizontal acceleration of 0.853g (PGA_M). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level could be susceptible to approximately 1 inch of liquefaction settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 12 and 13).

7.5 Seismically-Induced Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. The seismically-induced settlement calculations were performed in accordance with the American Society of Civil Engineers, Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No. 9.

The calculations provided herein for Boring B1 indicate that the alluvial soils above the historic high groundwater could be prone to approximately 0.1 inch of settlement as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$).

The calculations provided herein for Boring B1 indicate that the alluvial soils above the historic high groundwater level of 16 feet could be prone to approximately 0.2 inch of settlement as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M).

Calculation of the anticipated seismically-induced settlements is provided as Figures 14 and 15.

7.6 Slope Stability

The topography at the site is relatively level (<10%) and the topography in the immediate site vicinity slopes gently to the southeast. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Additionally, the site is not located in a county-designated Landslide Hazard Area (Monterey County, 2010). It is our opinion that the potential for slope stability hazards to adversely affect the proposed development is considered low.

7.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the Monterey County Safety Element (Monterey County, 2010), the site is not located within a potential inundation area for an earthquake-induced dam failure. The probability of earthquake-induced flooding is considered very low.

7.8 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis, seismic sea waves, are not considered hazards at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

The site is not located within a designated flood zone (FEMA, 2018). It is our opinion that the potential for flooding to adversely affect the site is considered low.

7.9 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity (DOGGR, 2018). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

7.10 Naturally Occurring Asbestos

Naturally-occurring asbestos is a known carcinogen and inhalation of asbestos may result in the development of lung cancer or mesothelioma. Chrysotile and amphibole asbestos (such as tremolite) occur naturally in certain geologic settings in California, most commonly in association with ultramafic rocks and along associated fault lines. The CGS maintains records of known asbestos mines, prospects, reported occurrences and asbestos-bearing ultramafic rocks. According to the CGS, the site is not located near any known sources of naturally-occurring asbestos (CGS, 2011), therefore, the risk of exposure to naturally-occurring asbestos is considered to be low.

7.11 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence (Monterey County General Plan, 2010). No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7.12 Radon

Radon is a naturally occurring gas that occurs during the radioactive decay of uranium and thorium within certain shale and igneous rock. The site is located within an area identified as having a low potential for indoor radon. As recommended by the United States Environmental Protection Agency, less than 5 percent of indoor radon measurements within this area do not exceed 4 Picocuries per liter [pCi/L] (CGS, 2018c). It is our opinion that the potential for indoor radon to adversely affect the site is considered low.

7.13 Volcanic Eruption

Volcanic activity can cause many different hazards that can be harmful to life and property. These hazards include lava flows, lahars, ash fall, debris avalanches, and pyroclastic flows, among others. The USGS Volcano Hazards Program (VHP) monitors and studies active and potentially active volcanoes, assesses their hazards across the entire world. The closest volcanic activity to the site is the Coso volcanic field located approximately 146 miles to the east. The most recent eruption of this field was approximately 40,000 years ago, (USGS 2012). The California Volcanic Observatory division of the USGS VHP considers the threat potential from the Coso Volcanic Field to be moderate. Based on the distance from the site, we consider the volcanic hazard threat to the site to be very low.

7.14 Hydro Consolidation

The results of our laboratory testing indicate that the existing upper alluvial soils are subject to hydro consolidation upon saturation (see Figures B3 through B6). Hydro-consolidation is the tendency of a soil structure to collapse upon saturation, resulting in the overall settlement of the affected soils and any overlying soils, foundations, or improvements supported therein. The grading and foundation recommendations presented herein are intended to mitigate the potential for settlement as a result of hydroconsolidation.

7.15 Expansive Soil

The upper 5 feet of existing site soils encountered during this investigation are considered to have a “high” expansive potential (Expansion Index (EI) = 96); and the soils are classified as “expansive” in accordance with the 2016 California Building Code (CBC) Section 1803.5.3 (see Figure B9). Recommendations presented herein assume that foundations and slabs will derive support in these materials.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

- 8.1.1 The site is located in close proximity to the main strand of the San Andreas Fault Zone and adjacent to the boundaries of a state-designated Alquist-Priolo Earthquake Fault Zone. A site-specific fault rupture hazard investigation should be conducted to identify if such hazards exist at the site prior to construction of the proposed project. The geotechnical design parameters presented herein should be reviewed and updated, as necessary.
- 8.1.2 Up to 2 feet of existing tilled or plowed soil was encountered during the site investigation. The existing tilled soil encountered is believed to be the result of farming activities at the site. Additionally, burrow holes from ground animals were observed throughout the proposed project area and have likely disturbed the upper site soils. Deeper tilled soil and/or disturbed soil may exist in other areas of the site that were not directly explored. It is our opinion that the existing tilled/disturbed soil, in its present condition, is not suitable for direct support of proposed foundations or slabs. Therefore, remedial grading in the form of removal and re-compaction will be required as part of site grading. The site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 8.4).
- 8.1.3 Our liquefaction analyses indicate that the alluvial soils below the historic high groundwater depth of 16 feet could be prone to approximately 0.7 inch of settlement as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$). The resulting differential settlement at the ground surface is anticipated to be approximately two-thirds of the total anticipated liquefaction settlement, or less than $\frac{1}{2}$ inch of settlement over a distance of 30 feet.
- 8.1.4 The liquefaction analyses indicate that the alluvial soils below the historic high groundwater depth of 16 feet could be prone to approximately 1 inch of settlement as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M). The resulting differential settlement at the ground surface is anticipated to be approximately two-thirds of the total anticipated liquefaction settlement, or less than 0.7 inch of settlement over a distance of 30 feet.

- 8.1.5 Based on these considerations, we recommend that the upper 3 feet of soil be excavated and properly compacted within the proposed fire station building, support structures, and water storage tank footprint areas. Deeper excavations should be conducted as needed to remove any unsuitable or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances. The limits of existing disturbed and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 8.4).
- 8.1.6 Subsequent to the recommended remedial grading, the proposed fire station building and support structures (generator/pump building, well house, and storage building) may be supported on a conventional foundation system deriving support in newly placed engineered fill. As a minimum, proposed foundations should be underlain by at least 12 inches of newly placed engineered fill. Recommendations for the design of a conventional foundation system are provided in Section 8.7.
- 8.1.7 Subsequent to the recommended remedial grading, the proposed water storage tank may be supported on conventional foundation system or a mat foundation system bearing directly on newly placed engineered fill. As a minimum, proposed foundations should be underlain by at least 12 inches of newly placed engineered fill. Recommendations for the design of a conventional and mat foundation systems are provided in Sections 8.8 and 8.9, respectively.
- 8.1.8 Based on the results of the laboratory testing, the upper alluvial soils encountered are subject to hydroconsolidation upon saturation. The introduction of stormwater into these soils would likely induce settlement of the surrounding soils and would adversely affect the future performance of the site. Based on these considerations, a stormwater infiltration system is not recommended for this project. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.
- 8.1.9 The client should be aware that it is essential that proper drainage be maintained in order to reduce settlements of the compressible alluvial soils and any foundations or paving supported therein. These soils are highly sensitive to excessive moisture and in order to prevent settlements, measures should be taken to prevent saturation of the soils.
- 8.1.10 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

- 8.1.11 Any changes in the design, location or elevation of improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

8.2 Soil and Excavation Characteristics

- 8.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.
- 8.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.
- 8.2.3 All excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.15).

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

- 8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the upper site soils are considered “corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B10) and should be considered for design of underground structures.
- 8.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B10) and indicate that the on-site materials possess “negligible” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.
- 8.3.3 Geocon does not practice corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

8.4 Grading

- 8.4.1 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.
- 8.4.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon. The existing tilled/disturbed soil and alluvial soil encountered during exploration are suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.
- 8.4.3 Grading should commence with the removal of existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved in writing by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established, it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).
- 8.4.4 As a minimum, we recommend that the upper 3 feet of soil within the proposed fire station building and support structures footprint areas should be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove existing unsuitable or soft soils at the direction of the Geotechnical Engineer (a representative of Geocon). Proposed structure foundations should be underlain by a minimum of 12 inches of newly placed engineered fill. The excavation should extend laterally a minimum distance of 3 feet beyond the fire station building and support structure footprint areas.
- 8.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened conditioned to at least 2 percent above optimum moisture content, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon).
- 8.4.6 All fill and backfill soils should be placed in horizontal loose layers of approximately 6 to 8 inches thick, moisture conditioned to at least 2 percent above optimum moisture content, and properly compacted to a minimum of 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

- 8.4.7 We anticipate that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 8.15).
- 8.4.8 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill, which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.
- 8.4.9. Where new paving is to be placed, we recommend that all tilled, disturbed, and soft soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil subgrade should be scarified, moisture conditioned to at least 2 percent above optimum moisture content and compacted to at least 92 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.12).
- 8.4.10 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over pipes, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

- 8.4.11 If construction is performed during the rainy season and the excavation bottom becomes wet and unstable, stabilization measures may have to be implemented to prevent excessive disturbance the excavation bottom. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result.
- 8.4.12 Subgrade stabilization may be accomplished by placing a thin lift of 3- to 6-inch-diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon).
- 8.4.13 All imported fill shall be observed, tested, and approved by Geocon prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. Import soils used as structural fill should have an expansion index less than 50 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B10). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).
- 8.4.14 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding sands, fill, steel, gravel, or concrete.

8.5 Shrinkage

- 8.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of 10 to 15 percent should be anticipated when excavating and compacting the upper 5 feet of soil on the site to an average relative compaction of 92 percent.
- 7.4.2 If import soils will be utilized in the building pad, the soils must be placed uniformly at the direction of the Geotechnical Engineer (a representative of Geocon). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

8.6 General Foundation Design

- 8.6.1 Our seismically induced settlement analysis indicate that the site soils could be prone to approximately 0.7 inch of total settlement as a result of the DE peak ground acceleration ($\frac{2}{3}PGAM$) and 1 inch of total settlement as a result of the MCE peak ground acceleration ($PGAM$). The resulting differential settlements at the foundation level for DE and MCE is anticipated to be less than $\frac{1}{2}$ inch and 0.7 inch over a distance of 30 feet, respectively. These settlements are in addition to the static settlements and must be considered in the structural design.
- 8.6.2 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.
- 8.6.3 Once the design and foundation loading configurations for the proposed improvements proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

8.7 Conventional Foundation Design – Fire Station and Support Structures

- 8.7.1 Continuous footings may be designed for an allowable bearing capacity of 2,000 pounds per square foot (psf), and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches into the newly placed engineered fill.
- 8.7.2 Isolated spread foundations may be designed for an allowable bearing capacity of 2,000 psf, and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches into newly placed engineered fill.
- 8.7.3 The allowable soil bearing pressure above may be increased by 150 psf and 300 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing value of 3,500 psf.
- 8.7.4 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

- 8.7.5 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the recommended bearing materials with a maximum allowable bearing value of up to 3,500 psf and deriving support in newly placed engineered fill is estimated to be approximately 1 inch and occur below the heaviest loaded structural element. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first 12 months. Differential settlement is expected to be approximately $\frac{3}{4}$ inch over a distance of 30 feet.
- 8.7.6 Based on seismic considerations, the proposed fire station and support structures should be designed for a combined static and seismically induced differential settlement of less than $1\frac{1}{4}$ inches over a distance of 30 feet.
- 8.7.7 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. The reinforcement for isolated spread footings should be designed by the project structural engineer.
- 8.7.8 If depth increases are utilized for the exterior wall footings, we should be provided a copy of the final foundation plans so that the excavation recommendations presented herein can be reviewed and revised if necessary. Additional grading should be conducted as-needed to maintain the required one-foot-thick blanket of engineered fill below new foundations.
- 8.7.9 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.
- 8.7.10 The upper 12 inches of subgrade should be moisture conditioned to at least 2 percent above optimum moisture content prior to placement of concrete. Slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

8.8 Conventional Foundation Design – Water Storage Tank

- 8.8.1 Continuous footings may be designed for an allowable bearing capacity of 2,000 pounds per square foot (psf), and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches newly placed engineered fill.
- 8.8.2 The allowable soil bearing pressure above may be increased by 150 psf and 300 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing value of 3,500 psf.

- 8.8.3 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.
- 8.8.4 The maximum expected static settlement for the water storage tank supported on a conventional foundation system deriving support in newly placed engineered fill and designed with a maximum bearing pressure of 3,500 psf is estimated to be approximately 1 inch and occur below the center of tank. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first 12 months. Differential settlement is expected to be approximately ½ inch over a distance of 30 feet.
- 8.8.5 Based on seismic considerations, the proposed water storage tank should be designed for a combined static and seismically induced differential settlement of less than 1¼ inches over a distance of 30 feet.
- 8.8.6 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. The reinforcement for isolated spread footings should be designed by the project structural engineer.
- 8.8.7 If depth increases are utilized, we should be provided a copy of the final foundation plans so that the excavation recommendations presented herein can be reviewed and revised if necessary. Additional grading should be conducted as-needed to maintain the required one-foot-thick blanket of engineered fill below new foundations.
- 8.8.8 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.
- 8.8.9 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

8.9 Mat Foundation Design – Water Storage Tank

- 8.9.1 As an alternative, a mat foundation system may be utilized for support of the proposed water storage tank provided foundations derive support in newly placed engineered fill. Foundations should be underlain by a minimum of 12 inches of newly placed engineered fill.
- 8.9.2 We anticipate that the mat foundation will impart an average pressure of less than 2,500 psf. The recommended maximum allowable bearing value is 3,500 pounds per square foot. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

- 8.9.3 The maximum expected total settlement for a structure supported on a mat foundation system designed with the maximum allowable bearing value of 3,500 psf and deriving support in the recommended bearing materials is estimated to be less than 1 inch and occur below the heaviest loaded structural element. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first 12 months. Differential settlement will be controlled by the rigidity of the mat.
- 8.9.4 Based on seismic considerations, the proposed water storage tank should be designed for a combined static and seismically induced differential settlement of less than 1 inch over a distance of 30 feet.
- 8.9.5 A unit modulus of subgrade reaction (k) of 150 pounds per cubic inch (pci) may be used in the design of mat foundations deriving support in newly placed engineered fill. This value is a unit value based on a 1-foot-square plate test or footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K \left[\frac{B+1}{2B} \right]^2$$

where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

- 8.9.6 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.
- 8.9.7 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slab and recommended bearing material without a moisture barrier, and 0.15 for slabs underlain by a vapor retarder or methane barrier.

8.10 Lateral Design

- 8.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in properly compacted engineered fill and undisturbed alluvial soils.
- 8.10.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils or properly compacted engineered fill may be computed as an equivalent fluid having a density of 300 pounds per cubic foot (pcf) with a maximum earth pressure of 3,000 pcf. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

8.11 Concrete Slabs-on-Grade

- 8.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Pavement Recommendations* section of this report (Section 8.12).
- 8.11.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 4 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.
- 6.8.3 If the near-surface soils of a building pad become dry prior to constructing the slab-on-grade, the building pad should be re-moistened by soaking or sprinkling such that the upper 12 inches of soil is at least 2% above optimum moisture content at least 24 hours before concrete placement. A Geocon representative should verify moisture conditions prior to slab-on-grade construction.
- 8.11.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.
- 8.11.4 For seismic design purposes (lateral resistance), a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

- 8.11.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to 2 percent above optimum moisture content and properly compacted to at least 90 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 8 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.
- 8.11.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.12 Preliminary Pavement Recommendations

- 8.12.1 Where new paving is to be placed, we recommend that all disturbed soil and soft alluvial soil be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all disturbed soil and soft alluvium in the area of new paving is not required; however, paving constructed over unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to 2 percent above optimum moisture content, and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).
- 8.12.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

- 8.12.3 The Traffic Indices listed below are estimates and based on the typical traffic loading expected. Geocon does not practice traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.5	3.0	3.0
Trash Truck & Fire Lanes	6.0	5.5	10.5

- 8.12.4 Asphalt concrete should conform to Section 203-6 of the “*Standard Specifications for Public Works Construction*” (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the “*Standard Specifications of the State of California, Department of Transportation*” (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the “*Standard Specifications for Public Works Construction*” (Green Book).
- 8.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, we recommend that the concrete be at least 6 inches thick reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 92 and 95 percent, respectively, relative compaction as determined by ASTM Test Method D 1557 (latest edition).
- 8.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

8.13 Retaining Wall Design

- 8.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 6 feet. In the event that walls significantly higher than 6 feet are planned, Geocon should be contacted for additional recommendations.
- 8.13.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Sections 8.6 through 8.9).
- 8.13.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than $0.001H$ (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 6	40	60

- 8.13.4 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.
- 8.13.5 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

- 8.13.6 We recommend that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$\text{For } x/H \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$\text{For } x/H > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where x is the distance from the face of the excavation or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth z .

- 8.13.7 We recommend that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$\text{For } x/H \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$\text{For } x/H > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H(z) = \sigma_H(z) \cos^2(1.1\theta)$$

where x is the distance from the face of the excavation/wall to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth z , θ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth z .

- 8.13.8 In addition to the recommended earth pressure, the upper 10 feet of the retaining wall adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the wall due to normal street traffic. If the traffic is kept back at least 10 feet from the wall, the traffic surcharge may be neglected.

8.14 Retaining Wall Drainage

- 8.14.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 16). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.
- 8.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 17). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.
- 8.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.
- 8.14.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.15 Temporary Excavations

- 8.15.1 Excavations up to 5 feet in height may be required during grading and construction operations. The excavations are expected to expose tilled, disturbed, and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

- 8.15.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum height of 10 feet. A uniform slope does not have a vertical portion.
- 8.15.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special temporary excavation measures can be provided under separate cover, if needed.
- 8.15.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

8.16 Surface Drainage

- 8.16.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.
- 8.16.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

- 8.16.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.
- 8.16.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

8.17 Plan Review

- 8.17.1 Grading, foundation plans, and if applicable, shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon.
2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.
3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.
4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

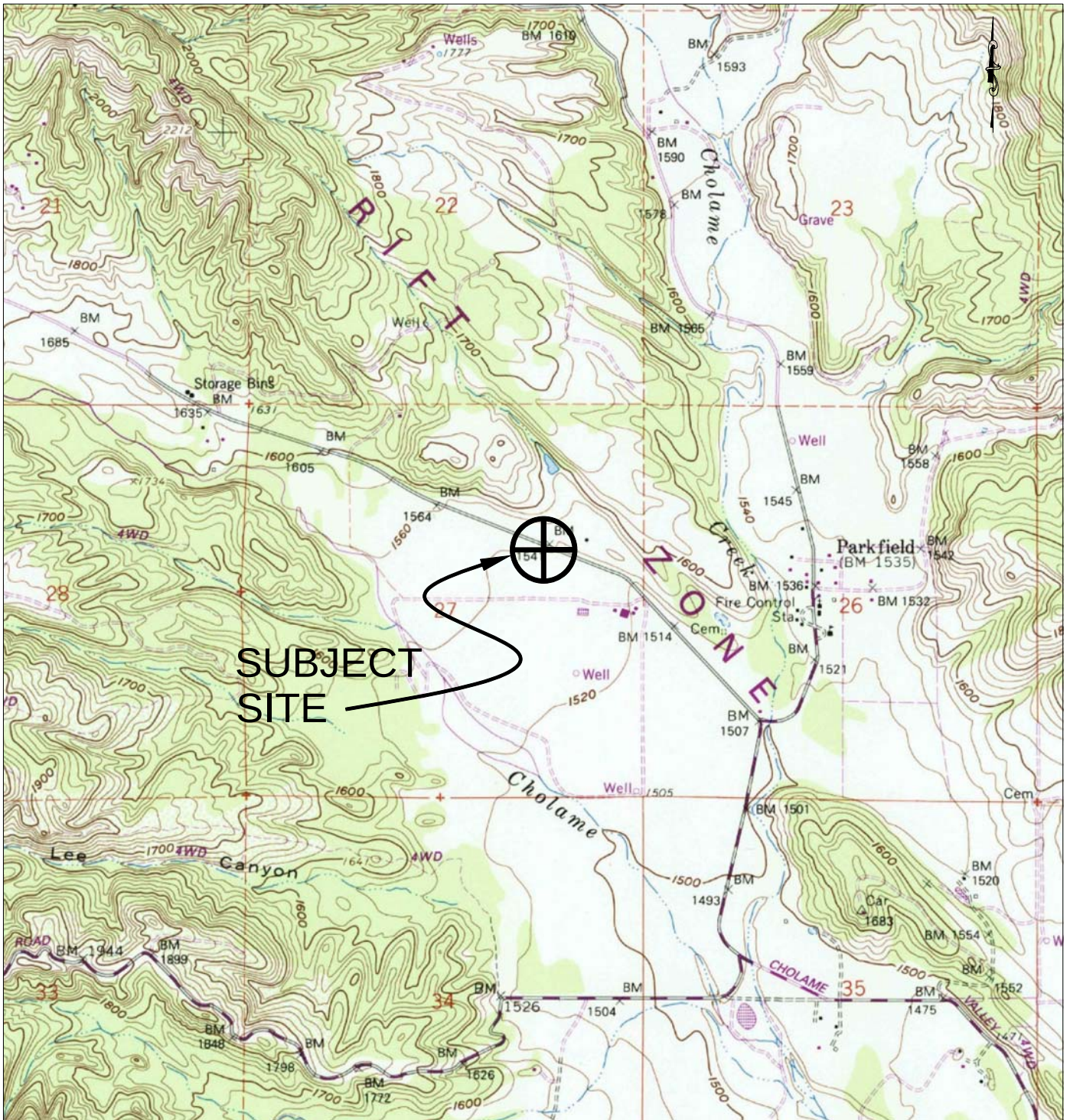
LIST OF REFERENCES

- Abrahamson, N., and Silva, W., Summary of the Abrahamson & Silva NGA Ground-Motion Relations, Earthquake Spectra, Volume 24, No. 1, pages 67–97, February 2008; © 2008, Earthquake Engineering Research Institute.
- Boore, D.M., and Atkinson G. M. (2008), Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods between 0.01 s and 10.0 s, Earthquake Spectra, Volume 24, No. 1, pages 99–138, February 2008; © 2008, Earthquake Engineering Research Institute.
- California Department of Water Resources, 2018, *Groundwater Level Data by Township, Range, and Section*, Web Site Address: http://www.water.ca.gov/waterdatalibrary/groundwater/hydrographs/index_trs.cfm.
- California Division of Oil, Gas and Geothermal Resources, 2018, Division of Oil, Gas, and Geothermal Resources Well Finder, <http://maps.conservation.ca.gov/doggr/index.html#close>. Accessed June 8, 2018.
- California Division of Mines and Geology, 1986, *Special Studies Zones, Parkfield Quadrangle, Revised Official Map*, Effective July 1, 1986.
- California Geological Survey, 2011, *Reported Historic Asbestos Mines, Asbestos Prospects, and Other Natural Occurrences of Asbestos in California*, California Geologic Survey Map Sheet 59.
- California Geological Survey, 2015, *California Geological Survey – Alquist-Priolo Earthquake Fault Zones*, www.conservation.ca.gov/cgs/rghm/ap/Pages/Index.aspx.
- California Geological Survey, 2017, letter report of Geologic Hazard investigation of the Proposed Parkfield Fire Station (FS), 70331 Vineyard Canyon Road, San Miguel, California, dated September 14, 2017.
- California Geological Survey, 2018a, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California*, Special Publication 42, Revised 2018.
- California Geological Survey, 2018b, CGS Information Warehouse, Regulatory Map Portal, <http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps>.
- California Geological Survey, 2018c, Interactive Radon Map of California, *Date Accessed: June 12, 2018*, <http://maps.conservation.ca.gov/cgs/radon/>
- Campbell, K.W., Bozorgnia, Y., NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s, Earthquake Spectra, Volume 24, No. 1, pages 139–171, February 2008; © 2008, Earthquake Engineering Research Institute.

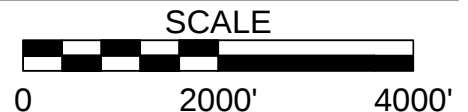
LIST OF REFERENCES (Continued)

- Chiou, B.S., and Youngs, R.R., *A NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, Earthquake Spectra, Volume 24, No. 1, pages 173–215, February 2008; © 2008, Earthquake Engineering Research Institute.
- FEMA, 2018, Online Flood Hazard Maps, *Flood Insurance Rate Map, Monterey County, California and Unincorporated Areas, Map Number 06053C1725G*, Date Accessed: June 8, 2018, <http://www.esri.com/hazards/index.html>.
- Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California*, California Geological Survey Geologic Data Map No. 6.
- Leinkaemper, J.J. and Brown, R.D., 1985, *Map of Faulting Accompanying the 1966 Parkfield, California, Earthquake*, U.S. Geological Survey, Open File Report 85-661
- Manson, M.W., 1985, *San Andreas fault (Middle Mountain-Chalome Valley segment) and San Juan fault (north end), Monterey and San Luis Obispo Counties, California: California Division of Mines and Geology Fault Evaluation Report FER-171*
- Risk Engineering, 2018, EZ-FRISK Version 7.65
- Sieh, K.E., 1984, "Lateral Offsets and Revised Dates of Large Prehistoric Earthquakes at Pallet Creek, California," *Journal of Geophysical Research*, Vol. 9, pp. 7641-7670.
- Sims, J.D., 1990, "Geologic Map of the San Andreas Fault in the Parkfield 7.5-minute Quadrangle, Monterey and Fresno Counties, California" *Miscellaneous Field Studies Map 2115*.
- U.S. Geological Survey, 1993, *Parkfield 7.5-Minute Topographic Map*.
- U.S. Geological Survey Seismic Design Maps Web Application, <http://earthquake.usgs.gov/designmaps/us/application.php>, accessed June 15, 2018.
- US Geologic Survey, *Bay Area Seismic Velocity Model*, Release 8.3.0, accessed June 15, 2018.
- U.S. Geological Survey, 2012, *Volcanics Hazard Program, Coso Volcanic Area*, Accessed June 2018, https://volcanoes.usgs.gov/volcanoes/coso_volcanic_field/
- U.S. Geological Survey, 2018 , *National Water Information System: Web Interface*, accessed June 11, 2018, and available at <https://nwis.waterdata.usgs.gov/nwis>
- Wallace, R.E., 1968, "Notes on Stream Channel Offset by San Andreas Fault, Southern Coast Ranges, California," in Dickerson, U.R., and Grantz, A., editors, *Proceedings of Conference of Geologic Problems on San Andreas Fault System*, Stanford University Publications Geological Sciences, Vol. IX, p. 5-21.

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, PARKFIELD, CA QUADRANGLE



LATITUDE: 35.901538
LONGITUDE: -120.445665



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ENVIRONMENTAL GEOTECHNICAL MATERIALS
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DRAFTED BY: RMA

CHECKED BY: GAK

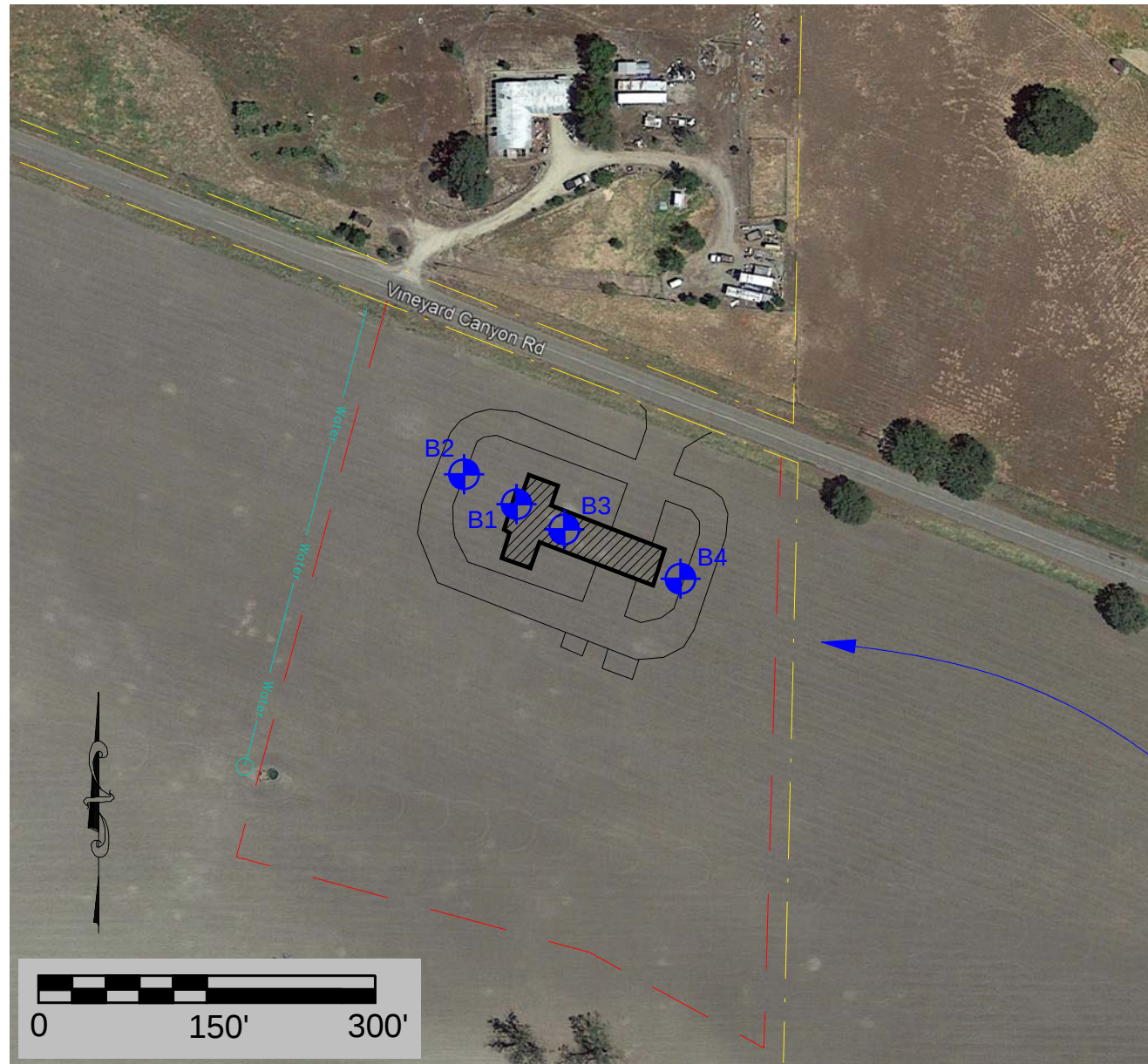
VICINITY MAP

PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. 1

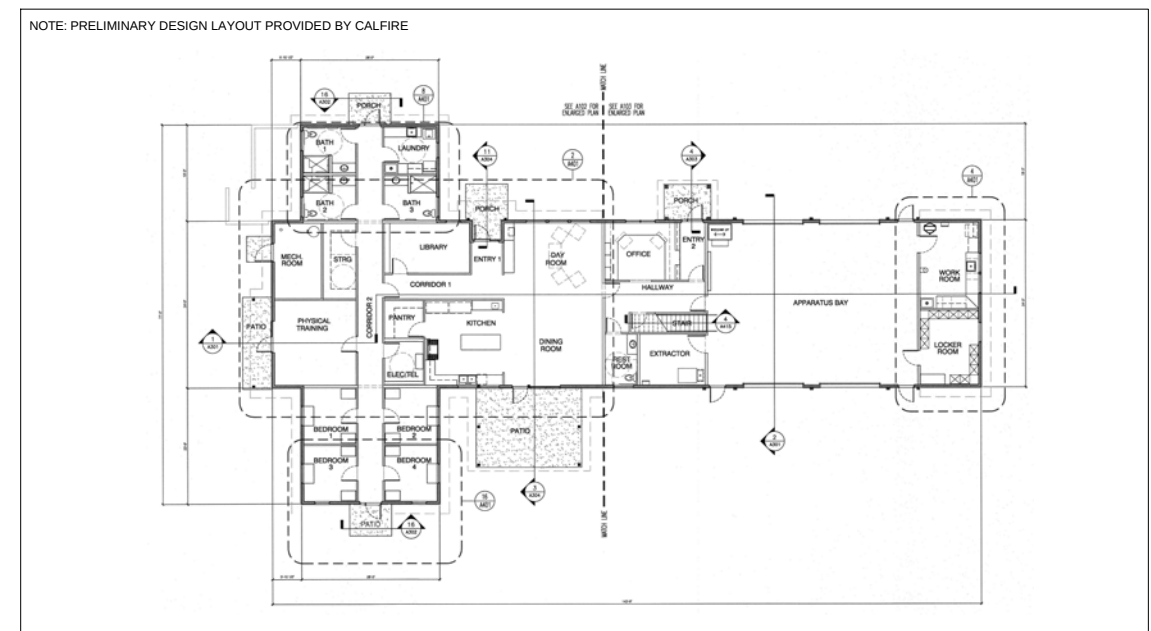


LEGEND

B4
Approximate Location of Boring

Approximate Project Boundary

Approximate Property Boundary



GEOCON CONSULTANTS, INC.		SITE PLAN	
ENVIRONMENTAL GEOTECHNICAL MATERIALS 3160 GOLD VALLEY DRIVE, SUITE 800 - RANCHO CORDOVA, CA 95742 PHONE (916) 852-9118 FAX (916) 852-9132		PARKFIELD FIRE STATION 70331 VINEYARD CANYON ROAD PARKFIELD, CALIFORNIA	
DRAFTED BY: RP	CHECKED BY: JTA/NDB	JULY 2018	PROJECT NO. S9962-05-23 FIG. 2

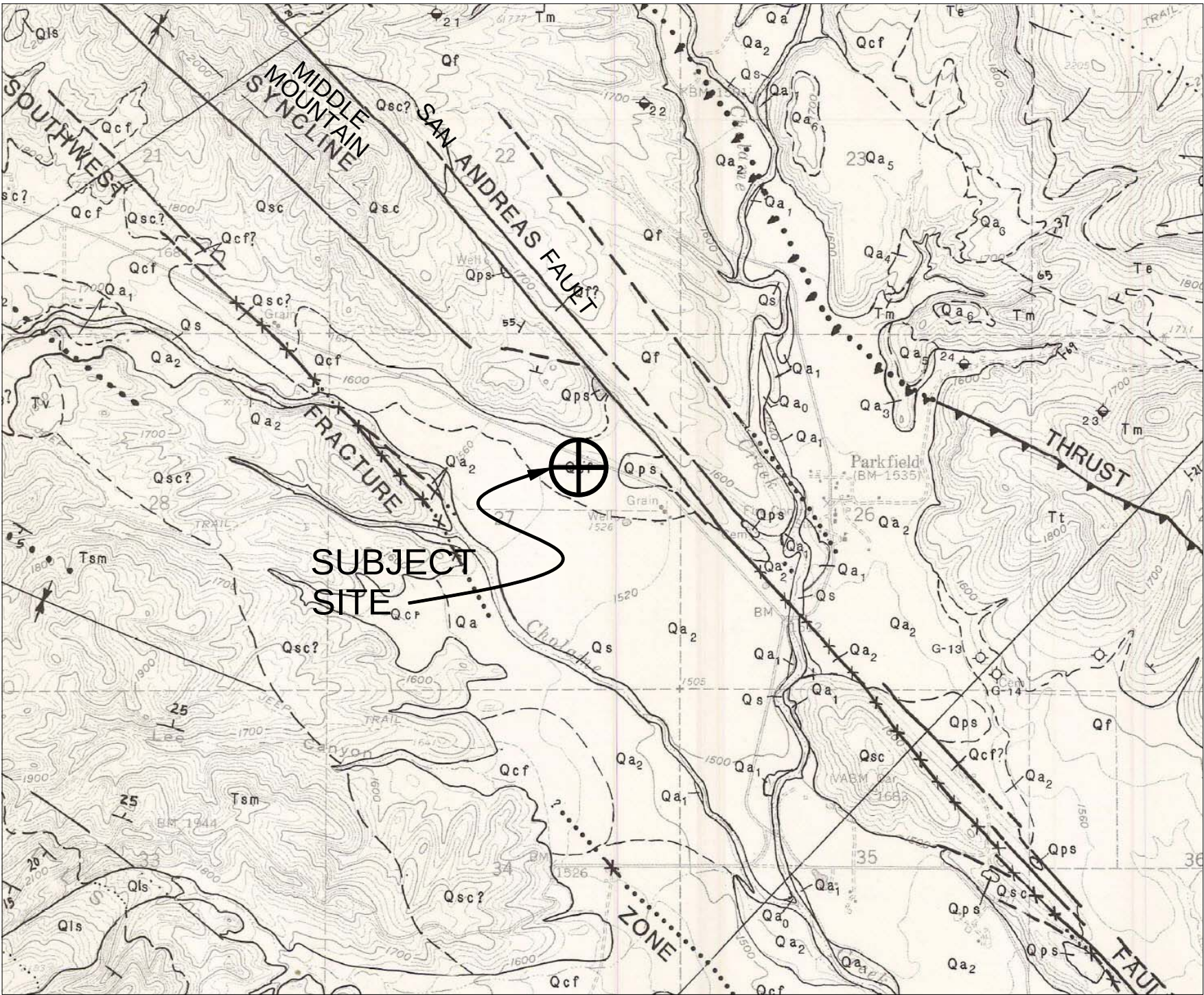
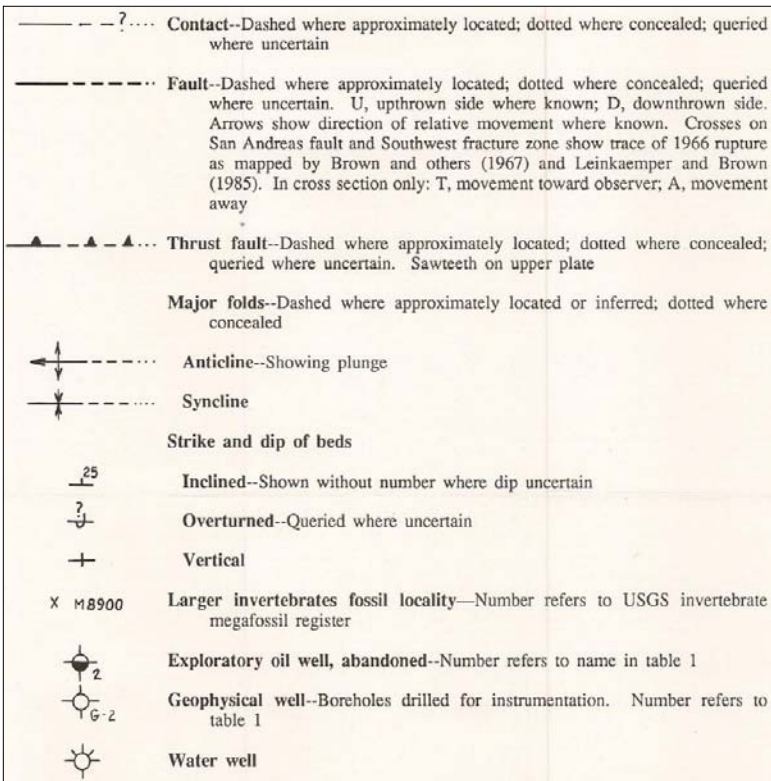
LEGEND

ROCKS SOUTHWEST OF THE SAN ANDREAS FAULT

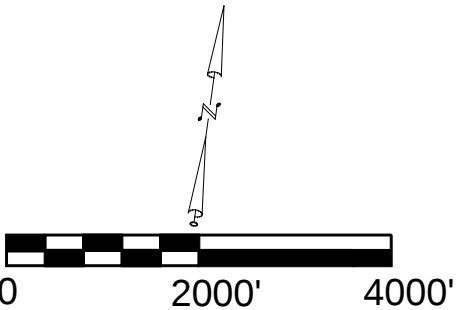
- Qs - Sand and gravel (Holocene)
- Qa₀₋₄ - Alluvial terrace deposits (Holocene)
- Qcf - Colluvial fan deposits (Holocene)
- Qps - Pond sediments (Holocene)
- Qls - Landslide deposits (Holocene and Pleistocene?)
- Qsc - Sandstone and conglomerate (Pleistocene?)
- Tsm - Santa Margarita Formation (upper Miocene)
- Tv - Volcanic rocks of Lang Canyon (lower Miocene)

ROCKS NORTHEAST OF THE SAN ANDREAS FAULT

- Qs - Sand and gravel (Holocene)
- Qa₀₋₄ - Alluvial terrace deposits (Holocene)
- Qcf - Colluvial fan deposits (Holocene)
- Qps - Pond sediments (Holocene)
- Qls - Landslide deposits (Holocene and Pleistocene?)
- Qf - Alluvial fan deposits (Pleistocene)
- Tm - Monterey Formation (middle Miocene)
- Tt - Temblor Formation (lower Miocene)



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LOCAL GEOLOGIC MAP

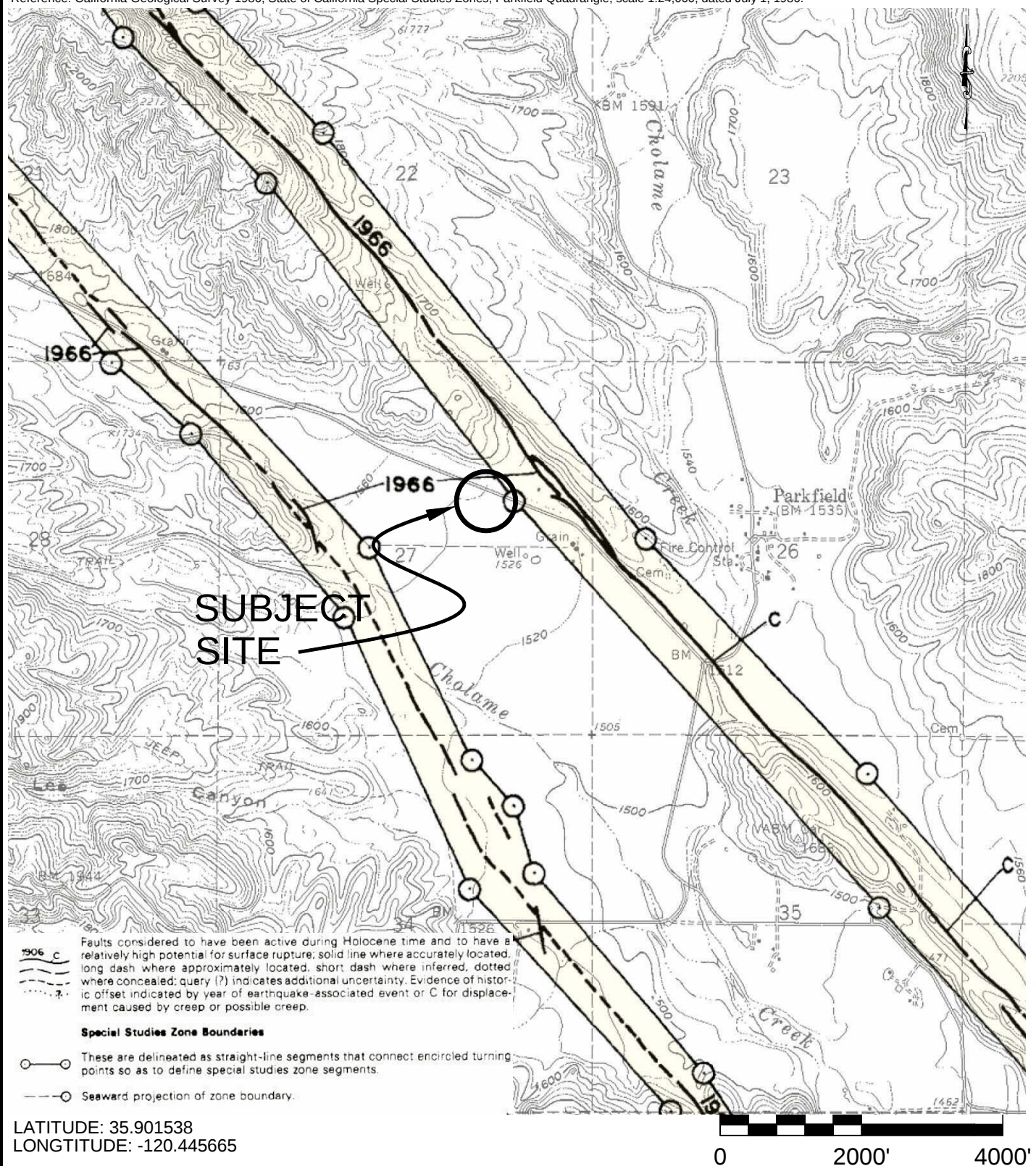
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FIG. 3

Reference: California Geological Survey 1986, State of California Special Studies Zones, Parkfield Quadrangle, scale 1:24,000, dated July 1, 1986.



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ALQUIST-PRIOLO EARTHQUAKE FAULT ZONE MAP

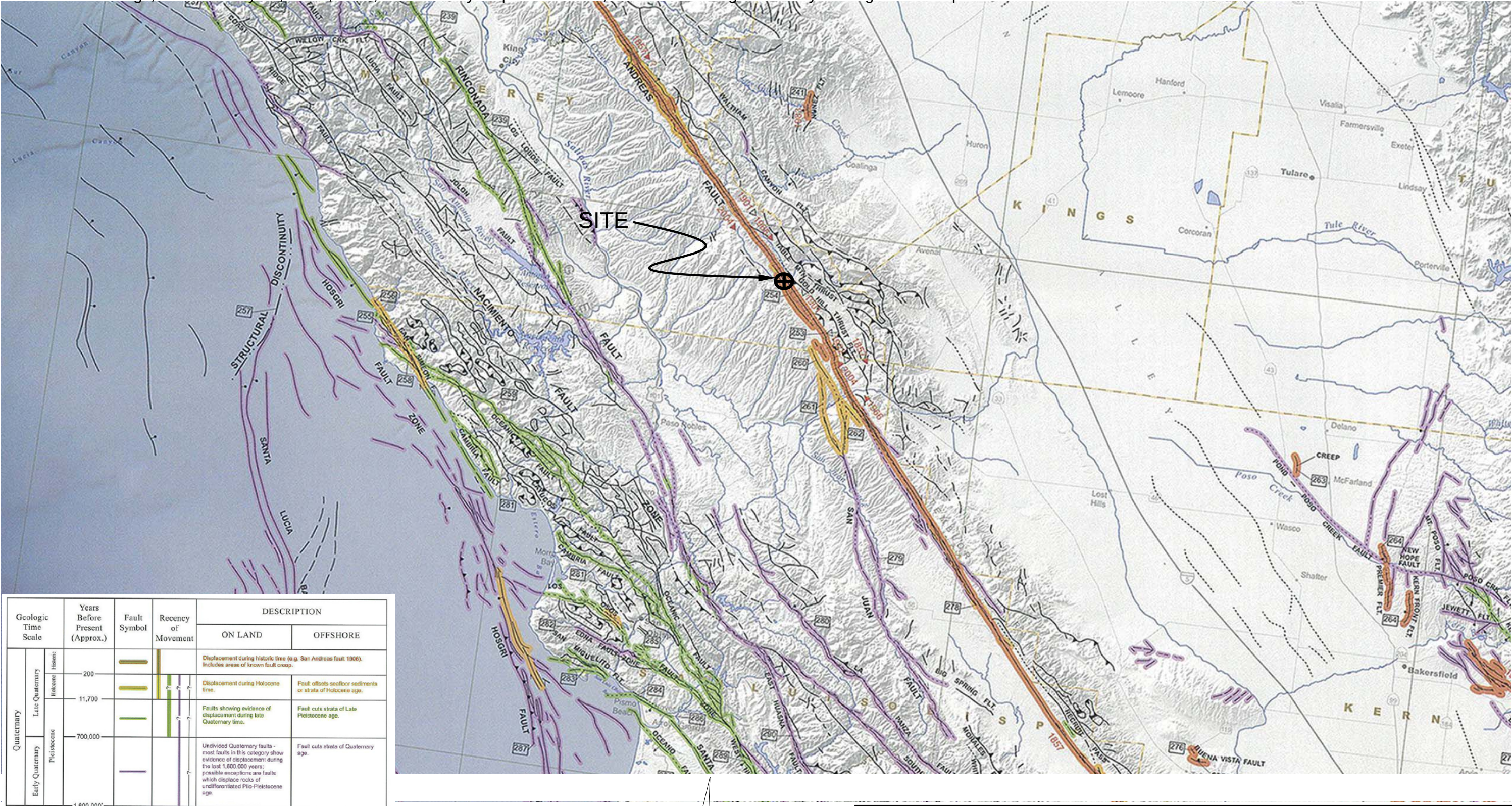
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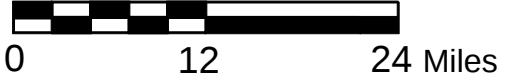
FIG. 3

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
					ON LAND	OFFSHORE
Quaternary	Late Quaternary	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		11,700			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
Pre-Quaternary	Early Quaternary	1,600,000			Undiscovered Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
		4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.



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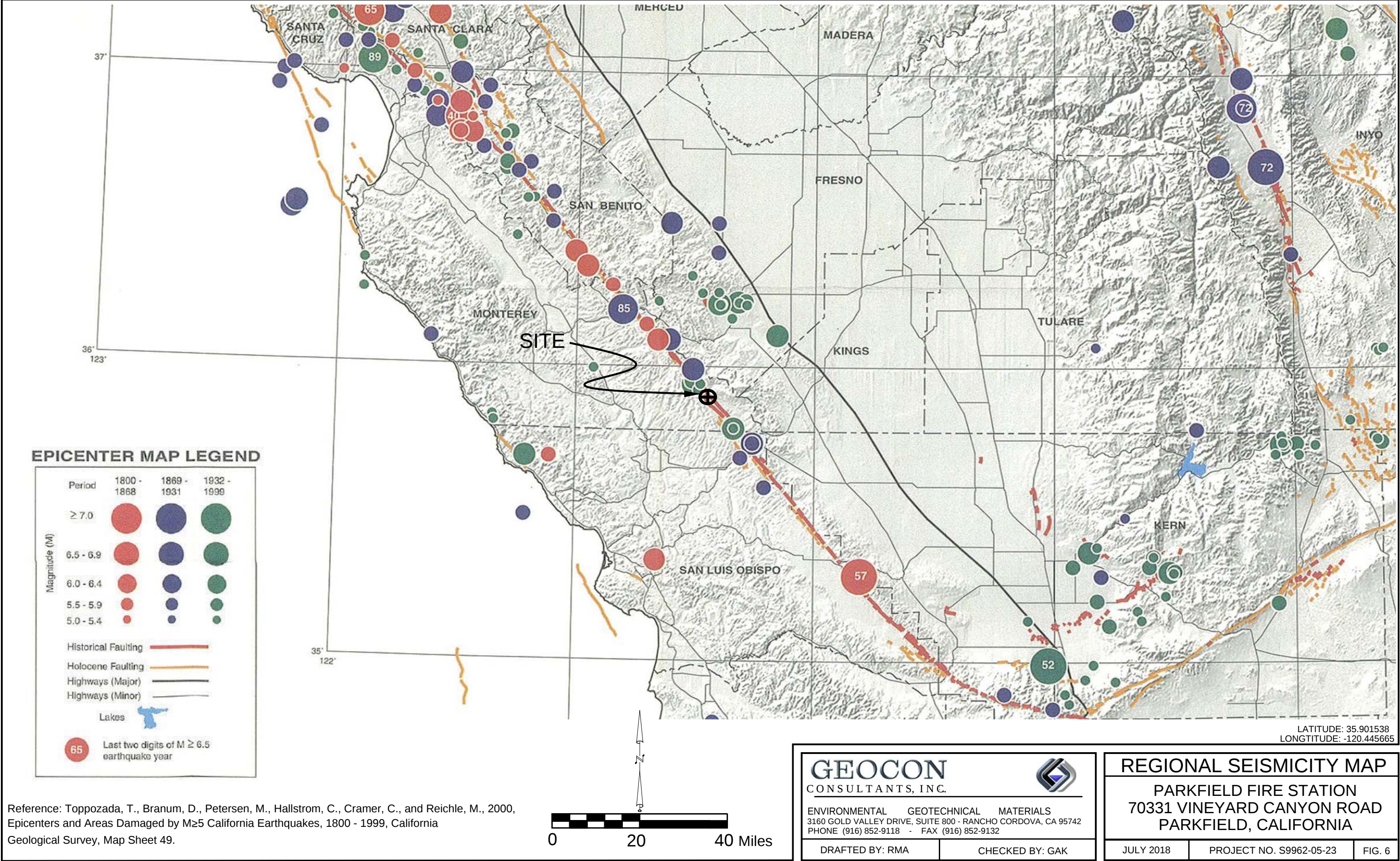
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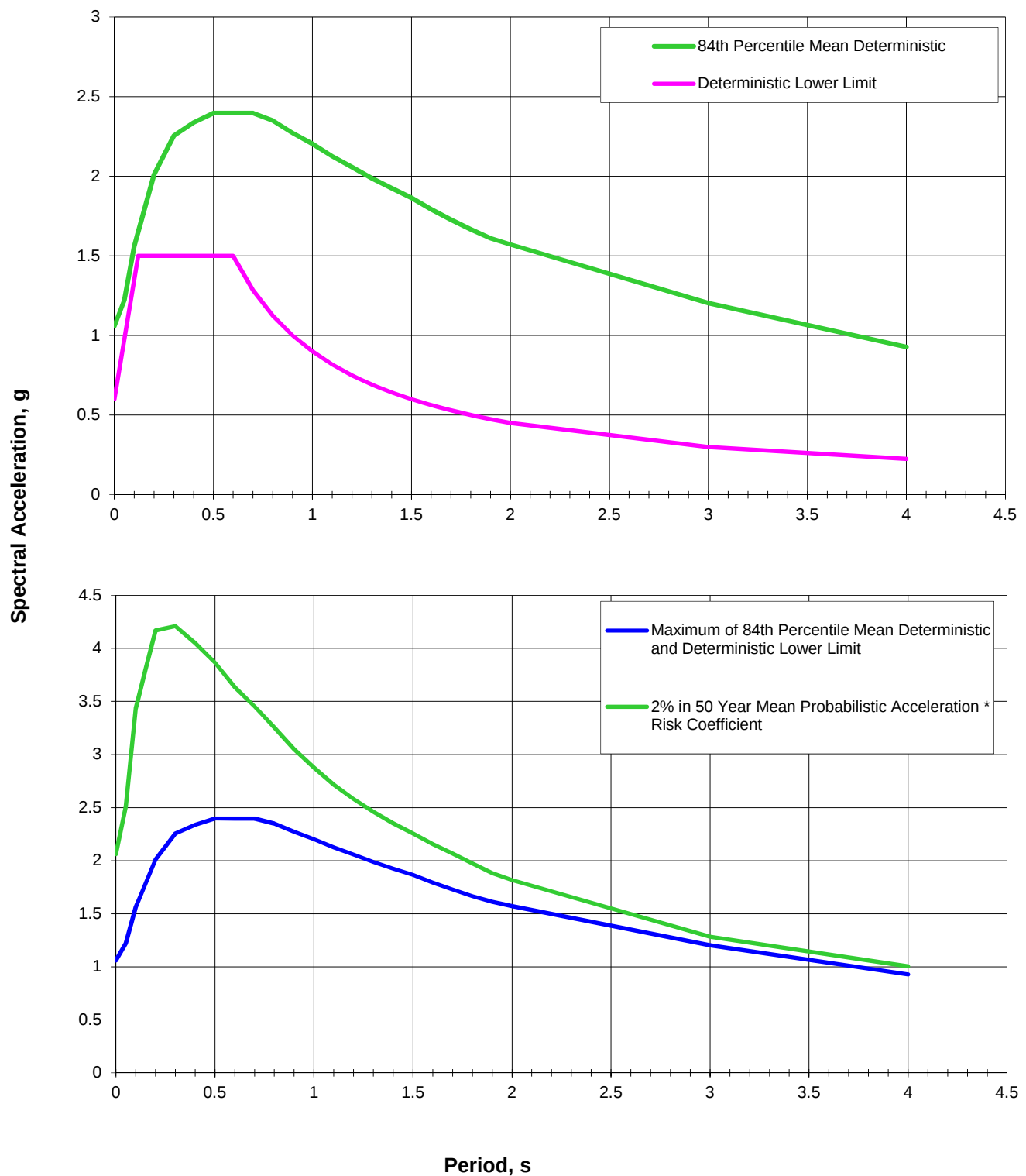
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REGIONAL FAULT MAP

PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018 PROJECT NO. S9962-05-23 FIG. 5





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JTA

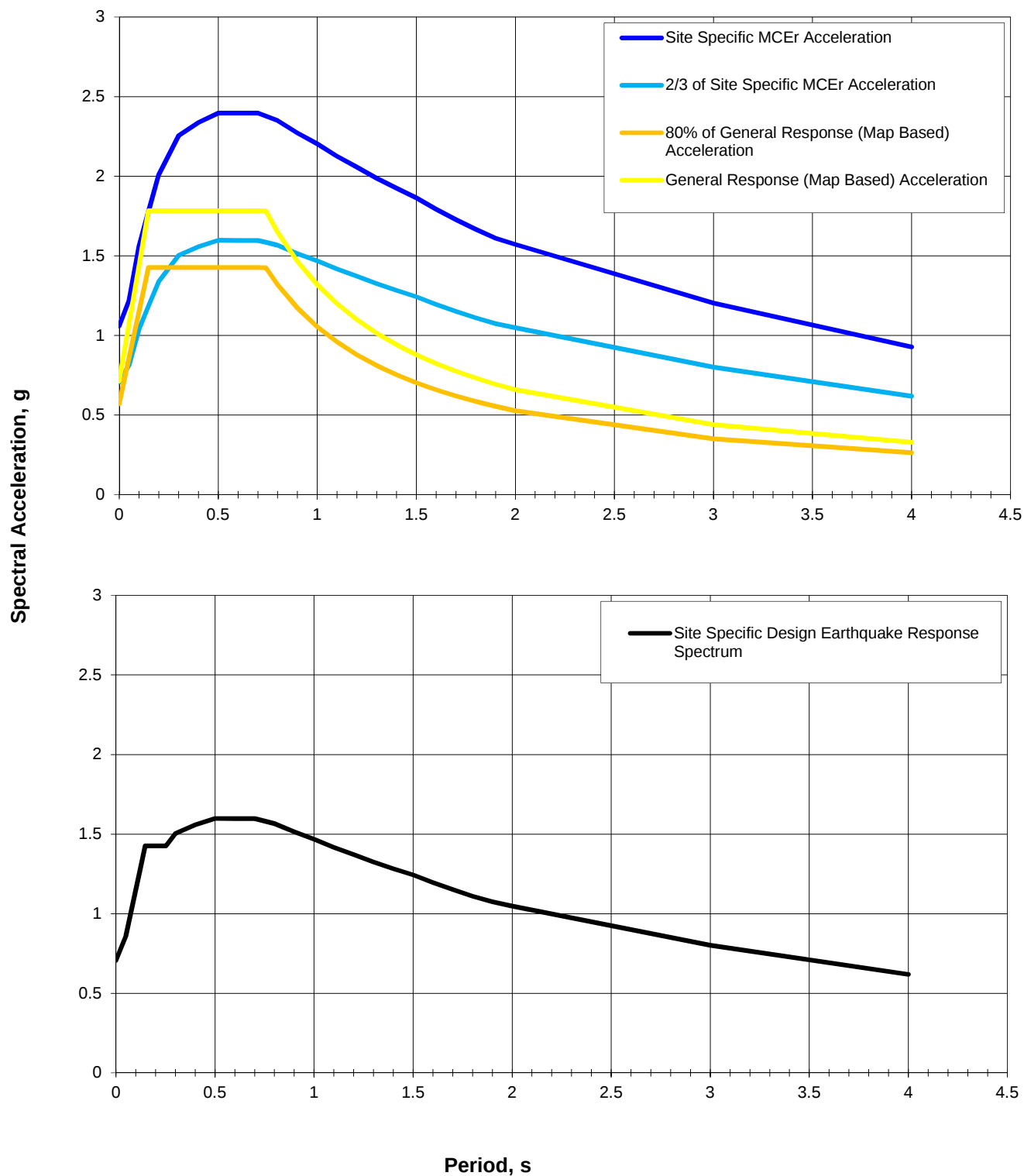
DESIGN RESPONSE SPECTRUM

PARKFIELD FIRE STATION
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JULY 2018

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FIG. 7



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3160 GOLD VALLEY DRIVE - RANCHO CORDOVA, CALIFORNIA 95742
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JTA

DESIGN RESPONSE SPECTRUM

PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. 8

Ground Motion Spectral Accelerations, (g)

Spectral Period (seconds)	2% in 50 Year Mean Prob.	Risk Coeff. C_R	Prob. MCE_R	84th Percentile Mean Det.	Det. Lower Limit	Site Specific MCE_R	80% of General Response	Site Specific Design Response Spectrum
PGA	1.852	1.114	2.063	1.060	0.600	1.060	0.571	0.707
0.050	2.253	1.114	2.510	1.222	0.975	1.222	0.860	0.860
0.100	3.083	1.114	3.434	1.561	1.350	1.561	1.149	1.149
0.120	3.215	1.114	3.581	1.651	1.500	1.651	1.264	1.264
0.148	3.399	1.114	3.787	1.777	1.500	1.777	1.426	1.426
0.200	3.742	1.114	4.169	2.010	1.500	2.010	1.426	1.426
0.253	3.784	1.107	4.190	2.140	1.500	2.140	1.426	1.426
0.300	3.822	1.102	4.210	2.255	1.500	2.255	1.426	1.503
0.400	3.720	1.089	4.051	2.337	1.500	2.337	1.426	1.558
0.500	3.590	1.077	3.865	2.397	1.500	2.397	1.426	1.598
0.600	3.415	1.064	3.634	2.396	1.500	2.396	1.426	1.597
0.700	3.284	1.052	3.453	2.396	1.286	2.396	1.426	1.597
0.740	3.224	1.047	3.374	2.378	1.221	2.378	1.426	1.585
0.800	3.134	1.039	3.256	2.350	1.125	2.350	1.319	1.567
0.900	2.972	1.027	3.051	2.272	1.000	2.272	1.172	1.515
1.000	2.837	1.014	2.877	2.203	0.900	2.203	1.055	1.469
1.100	2.679	1.014	2.717	2.126	0.818	2.126	0.959	1.417
1.200	2.546	1.014	2.582	2.058	0.750	2.058	0.879	1.372
1.300	2.425	1.014	2.459	1.988	0.692	1.988	0.812	1.325
1.400	2.319	1.014	2.351	1.925	0.643	1.925	0.754	1.283
1.500	2.224	1.014	2.255	1.865	0.600	1.865	0.703	1.243
1.600	2.125	1.014	2.155	1.793	0.563	1.793	0.660	1.195
1.700	2.038	1.014	2.067	1.727	0.529	1.727	0.621	1.151
1.800	1.947	1.014	1.974	1.666	0.500	1.666	0.586	1.111
1.900	1.856	1.014	1.882	1.611	0.474	1.611	0.555	1.074
2.000	1.791	1.014	1.816	1.572	0.450	1.572	0.528	1.048
3.000	1.263	1.014	1.281	1.202	0.300	1.202	0.352	0.801
4.000	0.990	1.014	1.004	0.928	0.225	0.928	0.264	0.619

GEOCON
CONSULTANTS, INC.



ENVIRONMENTAL GEOTECHNICAL MATERIALS
3160 GOLD VALLEY DRIVE - RANCHO CORDOVA, CALIFORNIA 95742
PHONE 916.852.9118 - FAX 916.852.9132

JTA

DESIGN RESPONSE SPECTRUM

PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. 9



Client : Parkfield
File No. : S9962-05-23
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL DESIGN EARTHQUAKE

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.35
Peak Horiz. Acceleration PGA_M (g):	0.853
2/3 PGA_M (g):	0.569
Calculated Mag.Wtg.Factor:	0.656
Historic High Groundwater:	16.0
Groundwater Depth During Exploration:	70.0

By Thomas F. Blake (1994-1996)

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):		62.4													
Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.	
1.0	116.0	0	8.0	2.0	1		67	2.000	20.7	116.0	0.226	0.998	0.242	--	
2.0	116.0	0	8.0	2.0	1		67	2.000	20.7	116.0	0.226	0.993	0.241	--	
3.0	116.0	0	8.0	2.0	1		67	2.000	20.7	116.0	0.226	0.989	0.240	--	
4.0	116.0	0	8.0	2.0	1		67	2.000	20.7	116.0	0.226	0.984	0.239	--	
5.0	116.0	0	10.0	7.0	1		66	2.000	25.9	116.0	0.301	0.979	0.238	--	
6.0	116.0	0	10.0	7.0	1		66	1.809	23.4	116.0	0.261	0.975	0.237	--	
7.0	116.0	0	10.0	7.0	1		66	1.664	21.5	116.0	0.236	0.970	0.235	--	
8.0	116.0	0	10.0	7.0	1		66	1.549	20.0	116.0	0.218	0.966	0.234	--	
9.0	116.0	0	15.0	12.0	1		74	1.455	28.2	116.0	0.357	0.961	0.233	--	
10.0	123.0	0	15.0	12.0	1		74	1.374	26.7	123.0	0.317	0.957	0.232	--	
11.5	123.0	0	15.0	12.0	1		74	1.288	25.0	123.0	0.285	0.951	0.231	--	
12.0	123.0	0	15.0	12.0	1		74	1.257	24.4	123.0	0.276	0.946	0.230	--	
13.0	123.0	0	15.0	12.0	1		74	1.190	23.1	123.0	0.257	0.943	0.229	--	
14.0	123.0	0	15.0	12.0	1		74	1.143	22.2	123.0	0.244	0.938	0.228	--	
15.5	103.0	0	15.0	12.0	1		74	1.097	21.3	103.0	0.233	0.933	0.226	--	
16.0	103.0	1	12.0	17.0	0			1.081	18.9	40.6	~	0.928	0.227	~	
17.5	103.0	1	12.0	17.0	0			1.037	18.2	40.6	~	0.923	0.237	~	
18.0	103.0	1	12.0	17.0	0			1.023	17.9	40.6	~	0.919	0.240	~	
19.0	103.0	1	12.0	17.0	0			0.992	17.4	40.6	~	0.915	0.248	~	
20.0	121.0	1	12.0	17.0	0			0.967	16.9	58.6	~	0.911	0.253	~	
21.5	121.0	1	12.0	17.0	0			0.936	16.4	58.6	~	0.905	0.259	~	
22.0	113.0	1	14.0	22.0	1	40	65	0.924	27.6	50.6	0.338	0.901	0.260	1.30	
23.0	113.0	1	14.0	22.0	1	40	65	0.899	27.0	50.6	0.324	0.897	0.266	1.22	
24.0	113.0	1	14.0	22.0	1	40	65	0.880	26.6	50.6	0.315	0.893	0.270	1.17	
25.0	113.0	1	15.0	27.0	1	40	65	0.862	28.7	50.6	0.369	0.888	0.273	1.35	
26.0	113.0	1	15.0	27.0	1	40	65	0.845	28.3	50.6	0.354	0.883	0.276	1.28	
27.0	113.0	1	15.0	27.0	1	43	65	0.829	27.9	50.6	0.342	0.879	0.279	1.22	
28.0	113.0	1	15.0	27.0	1	43	65	0.814	27.5	50.6	0.332	0.874	0.282	1.18	
29.0	113.0	1	15.0	27.0	1	43	65	0.800	27.2	50.6	0.323	0.870	0.285	1.14	
30.0	121.0	1	20.0	32.0	1	43	73	0.786	34.1	58.6	Inf.	0.865	0.287	Non-Liq.	
31.5	121.0	1	20.0	32.0	1	43	73	0.769	33.5	58.6	Inf.	0.859	0.289	Non-Liq.	
32.0	121.0	1	20.0	32.0	1	23	73	0.762	30.5	58.6	Inf.	0.855	0.289	Non-Liq.	
33.0	121.0	1	20.0	32.0	1	23	73	0.747	30.0	58.6	0.475	0.851	0.292	1.63	
34.0	121.0	1	20.0	32.0	1	23	73	0.735	29.6	58.6	0.411	0.847	0.293	1.40	
35.0	120.0	1	78.0	37.0	1	7	139	0.724	97.9	57.6	Inf.	0.842	0.294	Non-Liq.	
36.0	120.0	1	78.0	37.0	1	7	139	0.713	96.4	57.6	Inf.	0.838	0.295	Non-Liq.	
37.0	120.0	1	78.0	37.0	1	7	139	0.703	95.1	57.6	Inf.	0.833	0.296	Non-Liq.	
38.0	120.0	1	78.0	37.0	1	7	139	0.693	93.7	57.6	Inf.	0.829	0.297	Non-Liq.	
39.0	120.0	1	78.0	37.0	1			139	0.684	92.0	57.6	Inf.	0.824	0.298	Non-Liq.
40.0	120.0	1	78.0	37.0	1			139	0.675	90.8	57.6	Inf.	0.819	0.298	Non-Liq.
41.5	120.0	1	78.0	37.0	1			139	0.664	89.4	57.6	Inf.	0.814	0.299	Non-Liq.
42.0	120.0	1	36.0	42.0	1			92	0.660	41.0	57.6	Inf.	0.809	0.298	Non-Liq.
43.0	120.0	1	36.0	42.0	1			92	0.650	40.4	57.6	Inf.	0.806	0.299	Non-Liq.
44.0	120.0	1	36.0	42.0	1			92	0.642	39.9	57.6	Inf.	0.801	0.300	Non-Liq.
45.0	120.0	1	50.0	47.0	1			105	0.635	54.7	57.6	Inf.	0.797	0.300	Non-Liq.
46.0	120.0	1	50.0	47.0	1			105	0.627	54.1	57.6	Inf.	0.792	0.300	Non-Liq.
47.0	120.0	1	50.0	47.0	1			105	0.620	53.5	57.6	Inf.	0.787	0.300	Non-Liq.
48.0	120.0	1	50.0	47.0	1			105	0.614	52.9	57.6	Inf.	0.783	0.300	Non-Liq.
49.0	120.0	1	50.0	47.0	1			105	0.607	52.4	57.6	Inf.	0.778	0.299	Non-Liq.
50.5	120.0	1	50.0	47.0	1			105	0.599	51.7	57.6	Inf.	0.773	0.299	Non-Liq.

Figure 10



Client : Parkfield
File No. : S9962-05-23
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.35
PGAM (g):	0.853
2/3 PGAM (g):	0.57
Calculated Mag.Wtg.Factor:	0.656
Historic High Groundwater:	16.0
Groundwater @ Exploration:	70.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	T_{av}/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}] (%)	EQ. SETTLE. Pe (in.)
1	8	116	0.029	0.029	67	21	0.370	--	0.00	0.00
2	8	116	0.087	0.087	67	21	0.370	--	0.00	0.00
3	8	116	0.145	0.145	67	21	0.370	--	0.00	0.00
4	8	116	0.203	0.203	67	21	0.370	--	0.00	0.00
5	10	116	0.261	0.261	66	26	0.370	--	0.00	0.00
6	10	116	0.319	0.319	66	23	0.370	--	0.00	0.00
7	10	116	0.377	0.377	66	22	0.370	--	0.00	0.00
8	10	116	0.435	0.435	66	20	0.370	--	0.00	0.00
9	15	116	0.493	0.493	74	28	0.370	--	0.00	0.00
10	15	123	0.553	0.553	74	27	0.370	--	0.00	0.00
11.5	15	123	0.630	0.630	74	25	0.370	--	0.00	0.00
12	15	123	0.660	0.660	74	24	0.370	--	0.00	0.00
13	15	123	0.737	0.737	74	23	0.370	--	0.00	0.00
14	15	123	0.799	0.799	74	22	0.370	--	0.00	0.00
15.5	15	103	0.868	0.868	74	21	0.370	--	0.00	0.00
16	12	103	0.894	0.886		19	0.373	~	0.00	0.00
17.5	12	103	0.971	0.916		18	0.392	~	0.00	0.00
18	12	103	0.997	0.927		18	0.398	~	0.00	0.00
19	12	103	1.061	0.952		17	0.412	~	0.00	0.00
20	12	121	1.117	0.977		17	0.423	~	0.00	0.00
21.5	12	121	1.193	1.013		16	0.435	~	0.00	0.00
22	14	113	1.222	1.027	65	28	0.440	1.30	0.65	0.08
23	14	113	1.293	1.059	65	27	0.452	1.22	1.10	0.13
24	14	113	1.349	1.084	65	27	0.460	1.17	1.10	0.13
25	15	113	1.406	1.109	65	29	0.469	1.35	0.00	0.00
26	15	113	1.462	1.135	65	28	0.477	1.28	0.75	0.09
27	15	113	1.519	1.160	65	28	0.484	1.22	0.75	0.09
28	15	113	1.575	1.185	65	28	0.492	1.18	0.75	0.09
29	15	113	1.632	1.210	65	27	0.499	1.14	1.10	0.13
30	20	121	1.690	1.238	73	34	0.505	Non-Liq.	0.00	0.00
31.5	20	121	1.766	1.274	73	34	0.512	Non-Liq.	0.00	0.00
32	20	121	1.796	1.289	73	31	0.515	Non-Liq.	0.00	0.00
33	20	121	1.872	1.326	73	30	0.522	1.63	0.00	0.00
34	20	121	1.932	1.355	73	30	0.527	1.40	0.00	0.00
35	78	120	1.992	1.384	139	98	0.532	Non-Liq.	0.00	0.00
36	78	120	2.052	1.413	139	96	0.537	Non-Liq.	0.00	0.00
37	78	120	2.112	1.442	139	95	0.542	Non-Liq.	0.00	0.00
38	78	120	2.172	1.470	139	94	0.546	Non-Liq.	0.00	0.00
39	78	120	2.232	1.499	139	92	0.551	Non-Liq.	0.00	0.00
40	78	120	2.292	1.528	139	91	0.555	Non-Liq.	0.00	0.00
41.5	78	120	2.367	1.564	139	89	0.560	Non-Liq.	0.00	0.00
42	36	120	2.397	1.578	92	41	0.562	Non-Liq.	0.00	0.00
43	36	120	2.472	1.614	92	40	0.566	Non-Liq.	0.00	0.00
44	36	120	2.532	1.643	92	40	0.570	Non-Liq.	0.00	0.00
45	50	120	2.592	1.672	105	55	0.573	Non-Liq.	0.00	0.00
46	50	120	2.652	1.701	105	54	0.577	Non-Liq.	0.00	0.00
47	50	120	2.712	1.730	105	54	0.580	Non-Liq.	0.00	0.00
48	50	120	2.772	1.758	105	53	0.583	Non-Liq.	0.00	0.00
49	50	120	2.832	1.787	105	52	0.586	Non-Liq.	0.00	0.00
50.5	50	120	2.907	1.823	105	52	0.590	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.7 INCHES

Figure 11



Client : Parkfield
File No. : S9962-05-23
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.64
Peak Horiz. Acceleration PGA_M (g):	0.853
Calculated Mag.Wtg.Factor:	0.736
Historic High Groundwater:	16.0
Groundwater Depth During Exploration:	70.0

By Thomas F. Blake (1994-1996)

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):			62.4												
Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.	
1.0	116.0	0	8.0	2.0	1		67	2.000	20.7	116.0	0.226	0.998	0.407	--	
2.0	116.0	0	8.0	2.0	1		67	2.000	20.7	116.0	0.226	0.993	0.405	--	
3.0	116.0	0	8.0	2.0	1		67	2.000	20.7	116.0	0.226	0.989	0.403	--	
4.0	116.0	0	8.0	2.0	1		67	2.000	20.7	116.0	0.226	0.984	0.401	--	
5.0	116.0	0	10.0	7.0	1		66	2.000	25.9	116.0	0.301	0.979	0.400	--	
6.0	116.0	0	10.0	7.0	1		66	1.809	23.4	116.0	0.261	0.975	0.398	--	
7.0	116.0	0	10.0	7.0	1		66	1.664	21.5	116.0	0.236	0.970	0.396	--	
8.0	116.0	0	10.0	7.0	1		66	1.549	20.0	116.0	0.218	0.966	0.394	--	
9.0	116.0	0	15.0	12.0	1		74	1.455	28.2	116.0	0.357	0.961	0.392	--	
10.0	123.0	0	15.0	12.0	1		74	1.374	26.7	123.0	0.317	0.957	0.390	--	
11.5	123.0	0	15.0	12.0	1		74	1.288	25.0	123.0	0.285	0.951	0.388	--	
12.0	123.0	0	15.0	12.0	1		74	1.257	24.4	123.0	0.276	0.946	0.386	--	
13.0	123.0	0	15.0	12.0	1		74	1.190	23.1	123.0	0.257	0.943	0.385	--	
14.0	123.0	0	15.0	12.0	1		74	1.143	22.2	123.0	0.244	0.938	0.383	--	
15.5	103.0	0	15.0	12.0	1		74	1.097	21.3	103.0	0.233	0.933	0.380	--	
16.0	103.0	1	12.0	17.0	0			1.081	18.9	40.6	~	0.928	0.382	~	
17.5	103.0	1	12.0	17.0	0			1.037	18.2	40.6	~	0.923	0.399	~	
18.0	103.0	1	12.0	17.0	0			1.023	17.9	40.6	~	0.919	0.403	~	
19.0	103.0	1	12.0	17.0	0			0.992	17.4	40.6	~	0.915	0.416	~	
20.0	121.0	1	12.0	17.0	0			0.967	16.9	58.6	~	0.911	0.425	~	
21.5	121.0	1	12.0	17.0	0			0.936	16.4	58.6	~	0.905	0.435	~	
22.0	113.0	1	14.0	22.0	1	40	65	0.924	27.6	50.6	0.338	0.901	0.437	0.77	
23.0	113.0	1	14.0	22.0	1	40	65	0.899	27.0	50.6	0.324	0.897	0.447	0.73	
24.0	113.0	1	14.0	22.0	1	40	65	0.880	26.6	50.6	0.315	0.893	0.453	0.70	
25.0	113.0	1	15.0	27.0	1	40	65	0.862	28.7	50.6	0.369	0.888	0.459	0.80	
26.0	113.0	1	15.0	27.0	1	40	65	0.845	28.3	50.6	0.354	0.883	0.464	0.76	
27.0	113.0	1	15.0	27.0	1	43	65	0.829	27.9	50.6	0.342	0.879	0.469	0.73	
28.0	113.0	1	15.0	27.0	1	43	65	0.814	27.5	50.6	0.332	0.874	0.474	0.70	
29.0	113.0	1	15.0	27.0	1	43	65	0.800	27.2	50.6	0.323	0.870	0.478	0.68	
30.0	121.0	1	20.0	32.0	1	43	73	0.786	34.1	58.6	Inf.	0.865	0.482	Non-Liq.	
31.5	121.0	1	20.0	32.0	1	43	73	0.769	33.5	58.6	Inf.	0.859	0.486	Non-Liq.	
32.0	121.0	1	20.0	32.0	1	23	73	0.762	30.5	58.6	Inf.	0.855	0.486	Non-Liq.	
33.0	121.0	1	20.0	32.0	1	23	73	0.747	30.0	58.6	0.475	0.851	0.490	0.97	
34.0	121.0	1	20.0	32.0	1	23	73	0.735	29.6	58.6	0.411	0.847	0.493	0.84	
35.0	120.0	1	78.0	37.0	1	7	139	0.724	97.9	57.6	Inf.	0.842	0.495	Non-Liq.	
36.0	120.0	1	78.0	37.0	1	7	139	0.713	96.4	57.6	Inf.	0.838	0.496	Non-Liq.	
37.0	120.0	1	78.0	37.0	1	7	139	0.703	95.1	57.6	Inf.	0.833	0.498	Non-Liq.	
38.0	120.0	1	78.0	37.0	1	7	139	0.693	93.7	57.6	Inf.	0.829	0.499	Non-Liq.	
39.0	120.0	1	78.0	37.0	1		139	0.684	92.0	57.6	Inf.	0.824	0.500	Non-Liq.	
40.0	120.0	1	78.0	37.0	1		139	0.675	90.8	57.6	Inf.	0.819	0.501	Non-Liq.	
41.5	120.0	1	78.0	37.0	1		139	0.664	89.4	57.6	Inf.	0.814	0.502	Non-Liq.	
42.0	120.0	1	36.0	42.0	1		92	0.660	41.0	57.6	Inf.	0.809	0.501	Non-Liq.	
43.0	120.0	1	36.0	42.0	1		92	0.650	40.4	57.6	Inf.	0.806	0.503	Non-Liq.	
44.0	120.0	1	36.0	42.0	1		92	0.642	39.9	57.6	Inf.	0.801	0.504	Non-Liq.	
45.0	120.0	1	50.0	47.0	1		105	0.635	54.7	57.6	Inf.	0.797	0.504	Non-Liq.	
46.0	120.0	1	50.0	47.0	1		105	0.627	54.1	57.6	Inf.	0.792	0.504	Non-Liq.	
47.0	120.0	1	50.0	47.0	1		105	0.620	53.5	57.6	Inf.	0.787	0.504	Non-Liq.	
48.0	120.0	1	50.0	47.0	1		105	0.614	52.9	57.6	Inf.	0.783	0.503	Non-Liq.	
49.0	120.0	1	50.0	47.0	1		105	0.607	52.4	57.6	Inf.	0.778	0.503	Non-Liq.	
50.5	120.0	1	50.0	47.0	1		105	0.599	51.7	57.6	Inf.	0.773	0.503	Non-Liq.	

Figure 12



Client : Parkfield
File No. : S9962-05-23
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.64
PGA _M (g):	0.853
Calculated Mag.Wtg.Factor:	0.736
Historic High Groundwater:	16.0
Groundwater @ Exploration:	70.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60		LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e ₁₅] (%)	EQ. SETTLE. Pe (in.)
							Tav/σ' _o			
1	8	116	0.029	0.029	67	21	0.554	--	0.00	0.00
2	8	116	0.087	0.087	67	21	0.554	--	0.00	0.00
3	8	116	0.145	0.145	67	21	0.554	--	0.00	0.00
4	8	116	0.203	0.203	67	21	0.554	--	0.00	0.00
5	10	116	0.261	0.261	66	26	0.554	--	0.00	0.00
6	10	116	0.319	0.319	66	23	0.554	--	0.00	0.00
7	10	116	0.377	0.377	66	22	0.554	--	0.00	0.00
8	10	116	0.435	0.435	66	20	0.554	--	0.00	0.00
9	15	116	0.493	0.493	74	28	0.554	--	0.00	0.00
10	15	123	0.553	0.553	74	27	0.554	--	0.00	0.00
11.5	15	123	0.630	0.630	74	25	0.554	--	0.00	0.00
12	15	123	0.660	0.660	74	24	0.554	--	0.00	0.00
13	15	123	0.737	0.737	74	23	0.554	--	0.00	0.00
14	15	123	0.799	0.799	74	22	0.554	--	0.00	0.00
15.5	15	103	0.868	0.868	74	21	0.554	--	0.00	0.00
16	12	103	0.894	0.886		19	0.559	~	0.00	0.00
17.5	12	103	0.971	0.916		18	0.587	~	0.00	0.00
18	12	103	0.997	0.927		18	0.596	~	0.00	0.00
19	12	103	1.061	0.952		17	0.618	~	0.00	0.00
20	12	121	1.117	0.977		17	0.634	~	0.00	0.00
21.5	12	121	1.193	1.013		16	0.653	~	0.00	0.00
22	14	113	1.222	1.027	65	28	0.660	0.77	0.75	0.09
23	14	113	1.293	1.059	65	27	0.677	0.73	1.10	0.13
24	14	113	1.349	1.084	65	27	0.690	0.70	1.10	0.13
25	15	113	1.406	1.109	65	29	0.703	0.80	0.75	0.09
26	15	113	1.462	1.135	65	28	0.715	0.76	0.75	0.09
27	15	113	1.519	1.160	65	28	0.726	0.73	0.75	0.09
28	15	113	1.575	1.185	65	28	0.737	0.70	0.75	0.09
29	15	113	1.632	1.210	65	27	0.747	0.68	1.10	0.13
30	20	121	1.690	1.238	73	34	0.757	Non-Liq.	0.00	0.00
31.5	20	121	1.766	1.274	73	34	0.768	Non-Liq.	0.00	0.00
32	20	121	1.796	1.289	73	31	0.773	Non-Liq.	0.00	0.00
33	20	121	1.872	1.326	73	30	0.783	0.97	0.75	0.09
34	20	121	1.932	1.355	73	30	0.791	0.84	0.75	0.09
35	78	120	1.992	1.384	139	98	0.798	Non-Liq.	0.00	0.00
36	78	120	2.052	1.413	139	96	0.805	Non-Liq.	0.00	0.00
37	78	120	2.112	1.442	139	95	0.812	Non-Liq.	0.00	0.00
38	78	120	2.172	1.470	139	94	0.819	Non-Liq.	0.00	0.00
39	78	120	2.232	1.499	139	92	0.826	Non-Liq.	0.00	0.00
40	78	120	2.292	1.528	139	91	0.832	Non-Liq.	0.00	0.00
41.5	78	120	2.367	1.564	139	89	0.839	Non-Liq.	0.00	0.00
42	36	120	2.397	1.578	92	41	0.842	Non-Liq.	0.00	0.00
43	36	120	2.472	1.614	92	40	0.849	Non-Liq.	0.00	0.00
44	36	120	2.532	1.643	92	40	0.854	Non-Liq.	0.00	0.00
45	50	120	2.592	1.672	105	55	0.860	Non-Liq.	0.00	0.00
46	50	120	2.652	1.701	105	54	0.865	Non-Liq.	0.00	0.00
47	50	120	2.712	1.730	105	54	0.870	Non-Liq.	0.00	0.00
48	50	120	2.772	1.758	105	53	0.874	Non-Liq.	0.00	0.00
49	50	120	2.832	1.787	105	52	0.879	Non-Liq.	0.00	0.00
50.5	50	120	2.907	1.823	105	52	0.884	Non-Liq.	0.00	0.00
TOTAL SETTLEMENT =									1.0 INCHES	

Figure 13

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9 **EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS** **DESIGN EARTHQUAKE**

DE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.35
Peak Horiz. Acceleration (g):	0.569

Fig 4.1 Fig 4.2

Fig 4.4

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav] (tsf)	Field SPT [N]	Correction Factor [C _{er}]	Relative Density [D _r] (%)	Correction Factor [C _n]	Corrected [N] ₆₀	rd Factor	Maximum Shear Mod. [G _{max}] (tsf)	[y _{eff}]*[G _{eff}] [G _{max}]	y _{eff} Shear Strain	[y _{eff}]*100%	Volumetric Strain M7.5 [E15] (%)	Number of Strain Cycles [N _c]	Corrected Vol. Strains [E _c]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	116.0	0.03	0.02	0.011	8	1.25	66.7	2.0	20.7	1.0	171.081	6.21E-05	1.00E-04	0.010	9.60E-03	6.3858	6.53E-03	0.00
2.0	1.0	1.5	116.0	0.09	0.06	0.032	8	1.25	66.7	2.0	20.7	1.0	296.320	1.05E-04	2.30E-04	0.023	2.21E-02	6.3858	1.50E-02	0.00
3.0	1.0	2.5	116.0	0.15	0.10	0.054	8	1.25	66.7	2.0	20.7	1.0	382.548	1.33E-04	2.30E-04	0.023	2.21E-02	6.3858	1.50E-02	0.00
4.0	1.0	3.5	116.0	0.20	0.14	0.075	8	1.25	66.7	2.0	20.7	1.0	452.637	1.55E-04	1.70E-04	0.017	1.63E-02	6.3858	1.11E-02	0.00
5.0	1.0	4.5	116.0	0.26	0.17	0.096	10	1.25	66.2	2.0	25.9	1.0	552.873	1.60E-04	1.70E-04	0.017	1.25E-02	6.3858	8.50E-03	0.00
6.0	1.0	5.5	116.0	0.32	0.21	0.118	10	1.25	66.2	1.8	23.4	1.0	591.120	1.79E-04	1.50E-04	0.015	1.24E-02	6.3858	8.46E-03	0.00
7.0	1.0	6.5	116.0	0.38	0.25	0.139	10	1.25	66.2	1.7	21.5	1.0	624.970	1.97E-04	1.50E-04	0.015	1.37E-02	6.3858	9.35E-03	0.00
8.0	1.0	7.5	116.0	0.44	0.29	0.160	10	1.25	66.2	1.5	20.0	1.0	655.503	2.12E-04	4.50E-04	0.045	4.49E-02	6.3858	3.06E-02	0.01
9.0	1.0	8.5	116.0	0.49	0.33	0.181	15	1.25	73.9	1.5	28.2	1.0	782.332	1.98E-04	1.50E-04	0.015	9.91E-03	6.3858	6.75E-03	0.00
10.0	1.0	9.5	123.0	0.55	0.37	0.203	15	1.25	73.9	1.4	26.7	1.0	812.740	2.10E-04	4.50E-04	0.045	3.19E-02	6.3858	2.17E-02	0.01
11.5	1.5	10.8	123.0	0.63	0.42	0.230	15	1.25	73.9	1.3	25.0	1.0	848.795	2.24E-04	4.50E-04	0.045	3.44E-02	6.3858	2.35E-02	0.01
12.0	0.5	11.8	123.0	0.69	0.46	0.252	15	1.25	73.9	1.3	24.4	0.9	882.244	2.32E-04	4.50E-04	0.045	3.54E-02	6.3858	2.41E-02	0.00
13.0	1.0	12.5	123.0	0.74	0.49	0.268	15	1.25	73.9	1.2	23.1	0.9	894.638	2.41E-04	4.50E-04	0.045	3.79E-02	6.3858	2.58E-02	0.01
14.0	1.0	13.5	123.0	0.80	0.54	0.290	15	1.25	73.9	1.1	22.2	0.9	918.853	2.50E-04	3.70E-04	0.037	3.27E-02	6.3858	2.22E-02	0.01
15.5	1.5	14.8	103.0	0.87	0.58	0.314	15	1.25	73.9	1.1	21.3	0.9	944.714	2.58E-04	3.70E-04	0.037	3.43E-02	6.3858	2.34E-02	0.01
16.0	0.5	15.8	103.0	0.92	0.62	0.332	12	1.25	61.5	1.1	18.9	0.9	935.248	2.71E-04	3.70E-04	0.037	3.95E-02	6.3858	2.69E-02	0.00
17.5	1.5	16.8	103.0	0.97	0.65	0.349	12	1.25	61.5	1.0	18.2	0.9	947.889	2.78E-04	3.70E-04	0.037	4.15E-02	6.3858	2.83E-02	HISTORIC HIGH GROUNDWATER
18.0	0.5	17.8	103.0	1.02	0.69	0.367	12	1.25	61.5	1.0	17.9	0.9	968.458	2.82E-04	3.70E-04	0.037	4.22E-02	6.3858	2.87E-02	
19.0	1.0	18.5	103.0	1.06	0.71	0.379	12	1.25	61.5	1.0	17.4	0.9	976.339	2.86E-04	3.70E-04	0.037	4.38E-02	6.3858	2.98E-02	
20.0	1.0	19.5	121.0	1.12	0.75	0.398	12	1.25	61.5	1.0	16.9	0.9	993.214	2.91E-04	3.70E-04	0.037	4.52E-02	6.3858	3.07E-02	
21.5	1.5	20.8	121.0	1.19	0.80	0.423	12	1.25	61.5	0.9	16.4	0.9	1015.132	2.98E-04	3.70E-04	0.037	4.70E-02	6.3858	3.20E-02	
22.0	0.5	21.8	113.0	1.25	0.84	0.442	14	1.25	64.6	0.9	27.6	0.9	1236.911	2.53E-04	3.70E-04	0.037	2.52E-02	6.3858	1.71E-02	
23.0	1.0	22.5	113.0	1.29	0.87	0.456	14	1.25	64.6	0.9	27.0	0.9	1248.939	2.55E-04	3.70E-04	0.037	2.58E-02	6.3858	1.76E-02	
24.0	1.0	23.5	113.0	1.35	0.91	0.474	14	1.25	64.6	0.9	26.6	0.9	1269.199	2.58E-04	3.70E-04	0.037	2.63E-02	6.3858	1.79E-02	
25.0	1.0	24.5	113.0	1.41	0.94	0.491	15	1.25	64.8	0.9	28.7	0.9	1329.579	2.53E-04	3.70E-04	0.037	2.40E-02	6.3858	1.63E-02	
26.0	1.0	25.5	113.0	1.46	0.98	0.508	15	1.25	64.8	0.8	28.3	0.9	1349.296	2.55E-04	3.70E-04	0.037	2.44E-02	6.3858	1.66E-02	
27.0	1.0	26.5	113.0	1.52	1.02	0.525	15	1.25	64.8	0.8	27.9	0.9	1368.575	2.57E-04	3.00E-04	0.030	2.01E-02	6.3858	1.37E-02	
28.0	1.0	27.5	113.0	1.58	1.06	0.542	15	1.25	64.8	0.8	27.5	0.9	1387.440	2.59E-04	3.00E-04	0.030	2.04E-02	6.3858	1.39E-02	
29.0	1.0	28.5	113.0	1.63	1.09	0.559	15	1.25	64.8	0.8	27.2	0.9	1405.917	2.60E-04	3.00E-04	0.030	2.08E-02	6.3858	1.41E-02	
30.0	1.0	29.5	121.0	1.69	1.13	0.576	20	1.25	72.7	0.8	34.1	0.9	1543.620	2.42E-04	3.00E-04	0.030	1.58E-02	6.3858	1.08E-02	
31.5	1.5	30.8	121.0	1.77	1.18	0.597	20	1.25	72.7	0.8	33.5	0.9	1568.635	2.44E-04	3.00E-04	0.030	1.61E-02	6.3858	1.10E-02	
32.0	0.5	31.8	121.0	1.83	1.22	0.614	20	1.25	72.7	0.8	30.5	0.9	1545.768	2.52E-04	3.00E-04	0.030	1.81E-02	6.3858	1.23E-02	
33.0	1.0	32.5	121.0	1.87	1.26	0.627	20	1.25	72.7	0.7	30.0	0.9	1555.597	2.54E-04	3.00E-04	0.030	1.85E-02	6.3858	1.26E-02	
34.0	1.0	33.5	121.0	1.93	1.30	0.643	20	1.25	72.7	0.7	29.6	0.8	1573.332	2.55E-04	3.00E-04	0.030	1.88E-02	6.3858	1.28E-02	
35.0	1.0	34.5	120.0	1.99	1.34	0.659	78	1.25	138.9	0.7	97.9	0.8	2381.155	1.71E-04	1.30E-04	0.013	1.93E-03	6.3858	1.32E-03	
36.0	1.0	35.5	120.0	2.05	1.38	0.675	78	1.25	138.9	0.7	96.4	0.8	2404.843	1.72E-04	1.30E-04	0.013	1.97E-03	6.3858	1.34E-03	
37.0	1.0	36.5	120.0	2.11	1.42	0.690	78	1.25	138.9	0.7	95.1	0.8	2428.077	1.73E-04	1.30E-04	0.013	2.00E-03	6.3858	1.36E-03	
38.0	1.0	37.5	120.0	2.17	1.46	0.705	78	1.25	138.9	0.7	93.7	0.8	2450.877	1.74E-04	1.30E-04	0.013	2.04E-03	6.3858	1.39E-03	
39.0	1.0	38.5	120.0	2.23	1.50	0.720	78	1.25	138.9	0.7	92.0	0.8	2469.095	1.74E-04	1.30E-04	0.013	2.08E-03	6.3858	1.42E-03	
40.0	1.0	39.5	120.0	2.29	1.54	0.734	78	1.25	138.9	0.7	90.8	0.8	2490.992	1.75E-04	1.30E-04	0.013	2.12E-03	6.3858	1.44E-03	
41.5	1.5	40.8	120.0	2.37	1.59	0.751	78	1.25	138.9	0.7	89.4	0.8	2517.832	1.76E-04	1.30E-04	0.013	2.16E-03	6.3858	1.47E-03	
42.0	0.5	41.8	120.0	2.43	1.63	0.765	36	1.25	91.8	0.7	41.0	0.8	1966.143	2.27E-04	3.00E-04	0.030	1.27E-02	6.3858	8.64E-03	
43.0	1.0	42.5	120.0	2.47	1.66	0.775	36	1.25	91.8	0.6	40.4	0.8	1974.108	2.28E-04	3.00E-04	0.030	1.29E-02	6.3858	8.80E-03	
44.0	1.0	43.5	120.0	2.53	1.70	0.788	36	1.25	91.8	0.6	39.9	0.8	1989.931	2.29E-04	3.00E-04	0.030	1.31E-02	6.3858	8.93E-03	
45.0	1.0	44.5	120.0	2.59	1.74	0.801	50	1.25	105.2	0.6	54.7	0.8	2237.586	2.05E-04	3.00E-04	0.030	8.96E-03	6.3858	6.10E-03	
46.0	1.0	45.5	120.0	2.65	1.78	0.813	50	1.25	105.2	0.6	54.1	0.8	2254.698	2.06E-04	3.00E-04	0.030	9.09E-03	6.3858	6.19E-03	
47.0	1.0	46.5	120.0	2.71	1.82	0.825	50	1.25	105.2	0.6	53.5	0.8	2271.554	2.06E-04	3.00E-04	0.030	9.21E-03	6.3858	6.27E-03	
48.0	1.0	47.5	120.0	2.77	1.86	0.837	50	1.25	105.2	0.6	52.9	0.8	2288.164	2.06E-04	3.00E-04	0.030	9.33E-03	6.3858	6.35E-03	
49.0	1.0	48.5	120.0	2.83	1.90	0.849	50	1.25	105.2	0.6	52.4	0.8	2304.535	2.06E-04	3.00E-04	0.030	9.45E-03	6.3858	6.44E-03	
50.5	1.5	49.8	120.0	2.91	1.95	0.862	50	1.25	105.2	0.6	51.7	0.8	2324.678	2.06E-04	3.00E-04	0.030	9.60E-03	6.3858	6.54E-03	
TOTAL SETTLEMENT =																			0.07	

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9 EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS MAXIMUM CONSIDERED EARTHQUAKE

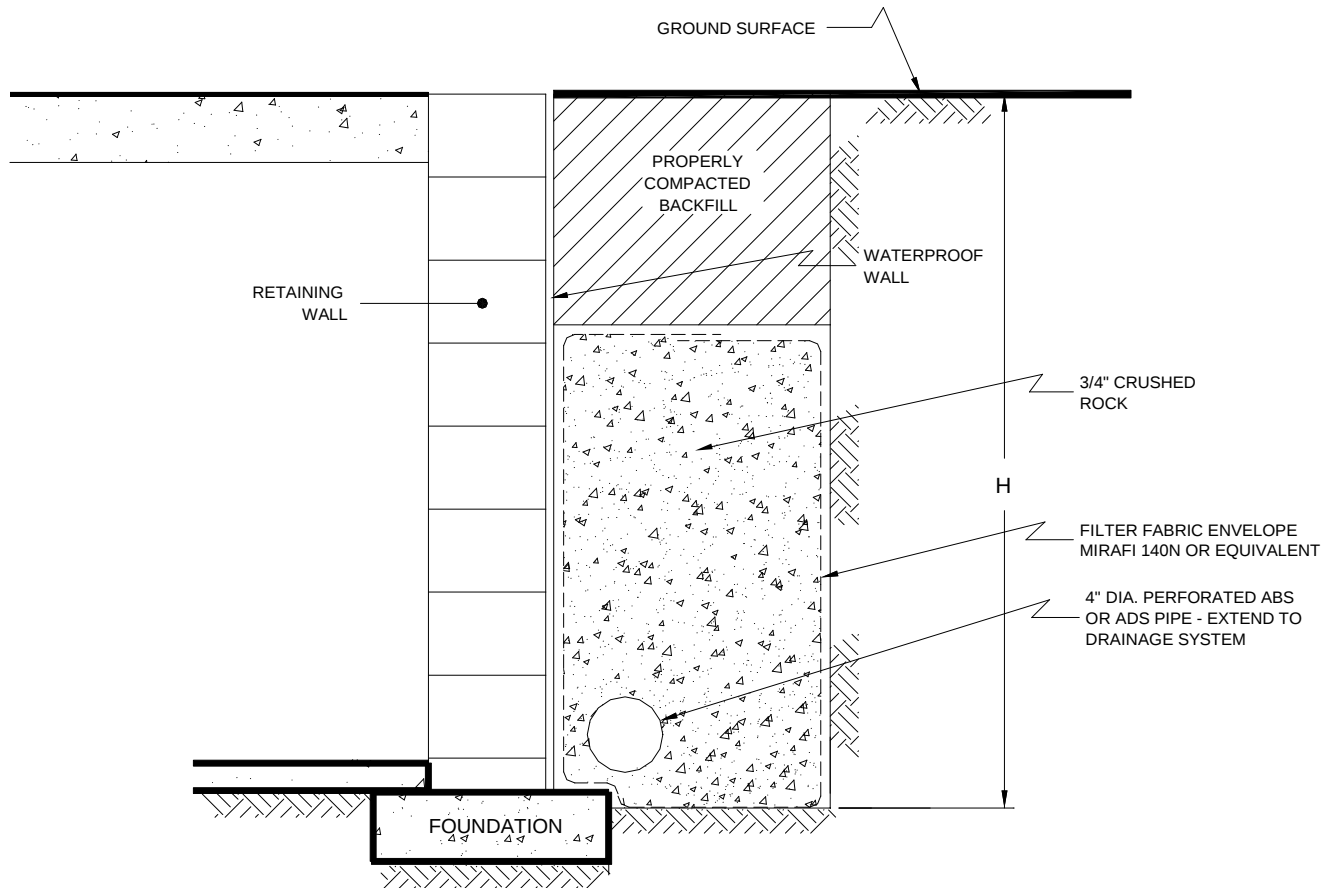
MCE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.64
Peak Horiz. Acceleration (g):	0.853

Fig 4.1 Fig 4.2

Fig 4.4

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav] (tsf)	Field SPT [N]	Correction Factor [C _{er}]	Relative Density [D _r] (%)	Correction Factor [C _n]	Corrected [N] ₆₀	rd Factor	Maximum Shear Mod. [G _{max}] (tsf)	[y _{eff}]*[G _{eff}] [G _{max}]	y _{eff} Shear Strain	[y _{eff}]*100%	Volumetric Strain M7.5 [E15] (%)	Number of Strain Cycles [N _c]	Corrected Vol. Strains [E _c]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	116.0	0.03	0.02	0.016	8	1.25	66.7	2.0	20.7	1.0	171.081	9.31E-05	1.90E-04	0.019	1.82E-02	8.2202	1.39E-02	0.00
2.0	1.0	1.5	116.0	0.09	0.06	0.048	8	1.25	66.7	2.0	20.7	1.0	296.320	1.58E-04	2.30E-04	0.023	2.21E-02	8.2202	1.68E-02	0.00
3.0	1.0	2.5	116.0	0.15	0.10	0.080	8	1.25	66.7	2.0	20.7	1.0	382.548	2.00E-04	3.00E-03	0.300	2.88E-01	8.2202	2.20E-01	0.05
4.0	1.0	3.5	116.0	0.20	0.14	0.112	8	1.25	66.7	2.0	20.7	1.0	452.637	2.32E-04	8.10E-04	0.081	7.77E-02	8.2202	5.93E-02	0.01
5.0	1.0	4.5	116.0	0.26	0.17	0.144	10	1.25	66.2	2.0	25.9	1.0	552.873	2.40E-04	8.10E-04	0.081	5.95E-02	8.2202	4.54E-02	0.01
6.0	1.0	5.5	116.0	0.32	0.21	0.176	10	1.25	66.2	1.8	23.4	1.0	591.120	2.69E-04	4.50E-04	0.045	3.73E-02	8.2202	2.84E-02	0.01
7.0	1.0	6.5	116.0	0.38	0.25	0.208	10	1.25	66.2	1.7	21.5	1.0	624.970	2.95E-04	4.50E-04	0.045	4.12E-02	8.2202	3.14E-02	0.01
8.0	1.0	7.5	116.0	0.44	0.29	0.240	10	1.25	66.2	1.5	20.0	1.0	655.503	3.18E-04	1.00E-03	0.100	9.97E-02	8.2202	7.61E-02	0.02
9.0	1.0	8.5	116.0	0.49	0.33	0.271	15	1.25	73.9	1.5	28.2	1.0	782.332	2.97E-04	4.50E-04	0.045	2.97E-02	8.2202	2.27E-02	0.01
10.0	1.0	9.5	123.0	0.55	0.37	0.304	15	1.25	73.9	1.4	26.7	1.0	812.740	3.14E-04	1.00E-03	0.100	7.08E-02	8.2202	5.40E-02	0.01
11.5	1.5	10.8	123.0	0.63	0.42	0.345	15	1.25	73.9	1.3	25.0	1.0	848.795	3.35E-04	1.00E-03	0.100	7.65E-02	8.2202	5.84E-02	0.02
12.0	0.5	11.8	123.0	0.69	0.46	0.378	15	1.25	73.9	1.3	24.4	0.9	882.244	3.48E-04	1.00E-03	0.100	7.88E-02	8.2202	6.01E-02	0.01
13.0	1.0	12.5	123.0	0.74	0.49	0.402	15	1.25	73.9	1.2	23.1	0.9	894.638	3.61E-04	1.00E-03	0.100	8.42E-02	8.2202	6.42E-02	0.02
14.0	1.0	13.5	123.0	0.80	0.54	0.435	15	1.25	73.9	1.1	22.2	0.9	918.853	3.74E-04	7.10E-04	0.071	6.27E-02	8.2202	4.78E-02	0.01
15.5	1.5	14.8	103.0	0.87	0.58	0.471	15	1.25	73.9	1.1	21.3	0.9	944.714	3.87E-04	7.10E-04	0.071	6.59E-02	8.2202	5.03E-02	0.02
16.0	0.5	15.8	103.0	0.92	0.62	0.498	12	1.25	61.5	1.1	18.9	0.9	935.248	4.07E-04	1.20E-03	0.120	1.28E-01	8.2202	9.77E-02	0.01
17.5	1.5	16.8	103.0	0.97	0.65	0.524	12	1.25	61.5	1.0	18.2	0.9	947.889	4.17E-04	1.20E-03	0.120	1.35E-01	8.2202	1.03E-01	HISTORIC HIGH GROUNDWATER
18.0	0.5	17.8	103.0	1.02	0.69	0.550	12	1.25	61.5	1.0	17.9	0.9	968.458	4.22E-04	1.20E-03	0.120	1.37E-01	8.2202	1.04E-01	
19.0	1.0	18.5	103.0	1.06	0.71	0.569	12	1.25	61.5	1.0	17.4	0.9	976.339	4.29E-04	1.20E-03	0.120	1.42E-01	8.2202	1.08E-01	
20.0	1.0	19.5	121.0	1.12	0.75	0.597	12	1.25	61.5	1.0	16.9	0.9	993.214	4.37E-04	1.20E-03	0.120	1.46E-01	8.2202	1.12E-01	
21.5	1.5	20.8	121.0	1.19	0.80	0.634	12	1.25	61.5	0.9	16.4	0.9	1015.132	4.47E-04	1.20E-03	0.120	1.52E-01	8.2202	1.16E-01	
22.0	0.5	21.8	113.0	1.25	0.84	0.663	14	1.25	64.6	0.9	27.6	0.9	1236.911	3.79E-04	7.10E-04	0.071	4.83E-02	8.2202	3.68E-02	
23.0	1.0	22.5	113.0	1.29	0.87	0.683	14	1.25	64.6	0.9	27.0	0.9	1248.939	3.83E-04	7.10E-04	0.071	4.95E-02	8.2202	3.78E-02	
24.0	1.0	23.5	113.0	1.35	0.91	0.710	14	1.25	64.6	0.9	26.6	0.9	1269.199	3.87E-04	7.10E-04	0.071	5.05E-02	8.2202	3.85E-02	
25.0	1.0	24.5	113.0	1.41	0.94	0.736	15	1.25	64.8	0.9	28.7	0.9	1329.579	3.79E-04	7.10E-04	0.071	4.60E-02	8.2202	3.51E-02	
26.0	1.0	25.5	113.0	1.46	0.98	0.762	15	1.25	64.8	0.8	28.3	0.9	1349.296	3.82E-04	7.10E-04	0.071	4.68E-02	8.2202	3.57E-02	
27.0	1.0	26.5	113.0	1.52	1.02	0.788	15	1.25	64.8	0.8	27.9	0.9	1368.575	3.85E-04	5.20E-04	0.052	3.49E-02	8.2202	2.66E-02	
28.0	1.0	27.5	113.0	1.58	1.06	0.813	15	1.25	64.8	0.8	27.5	0.9	1387.440	3.88E-04	5.20E-04	0.052	3.54E-02	8.2202	2.70E-02	
29.0	1.0	28.5	113.0	1.63	1.09	0.838	15	1.25	64.8	0.8	27.2	0.9	1405.917	3.90E-04	5.20E-04	0.052	3.60E-02	8.2202	2.75E-02	
30.0	1.0	29.5	121.0	1.69	1.13	0.863	20	1.25	72.7	0.8	34.1	0.9	1543.620	3.62E-04	5.20E-04	0.052	2.74E-02	8.2202	2.09E-02	
31.5	1.5	30.8	121.0	1.77	1.18	0.895	20	1.25	72.7	0.8	33.5	0.9	1568.635	3.65E-04	5.20E-04	0.052	2.80E-02	8.2202	2.13E-02	
32.0	0.5	31.8	121.0	1.83	1.22	0.921	20	1.25	72.7	0.8	30.5	0.9	1545.768	3.78E-04	5.20E-04	0.052	3.13E-02	8.2202	2.39E-02	
33.0	1.0	32.5	121.0	1.87	1.26	0.940	20	1.25	72.7	0.7	30.0	0.9	1555.597	3.80E-04	5.20E-04	0.052	3.20E-02	8.2202	2.44E-02	
34.0	1.0	33.5	121.0	1.93	1.30	0.964	20	1.25	72.7	0.7	29.6	0.8	1573.332	3.82E-04	5.20E-04	0.052	3.25E-02	8.2202	2.48E-02	
35.0	1.0	34.5	120.0	1.99	1.34	0.988	78	1.25	138.9	0.7	97.9	0.8	2381.155	2.57E-04	3.00E-04	0.030	4.46E-03	8.2202	3.40E-03	
36.0	1.0	35.5	120.0	2.05	1.38	1.012	78	1.25	138.9	0.7	96.4	0.8	2404.843	2.58E-04	3.00E-04	0.030	4.54E-03	8.2202	3.47E-03	
37.0	1.0	36.5	120.0	2.11	1.42	1.034	78	1.25	138.9	0.7	95.1	0.8	2428.077	2.59E-04	3.00E-04	0.030	4.62E-03	8.2202	3.53E-03	
38.0	1.0	37.5	120.0	2.17	1.46	1.057	78	1.25	138.9	0.7	93.7	0.8	2450.877	2.60E-04	3.00E-04	0.030	4.70E-03	8.2202	3.59E-03	
39.0	1.0	38.5	120.0	2.23	1.50	1.079	78	1.25	138.9	0.7	92.0	0.8	2469.095	2.62E-04	3.00E-04	0.030	4.81E-03	8.2202	3.67E-03	
40.0	1.0	39.5	120.0	2.29	1.54	1.100	78	1.25	138.9	0.7	90.8	0.8	2490.992	2.62E-04	3.00E-04	0.030	4.88E-03	8.2202	3.72E-03	
41.5	1.5	40.8	120.0	2.37	1.59	1.127	78	1.25	138.9	0.7	89.4	0.8	2517.832	2.63E-04	3.00E-04	0.030	4.98E-03	8.2202	3.80E-03	
42.0	0.5	41.8	120.0	2.43	1.63	1.147	36	1.25	91.8	0.7	41.0	0.8	1966.143	3.41E-04	5.20E-04	0.052	2.20E-02	8.2202	1.68E-02	
43.0	1.0	42.5	120.0	2.47	1.66	1.162	36	1.25	91.8	0.6	40.4	0.8	1974.108	3.42E-04	5.20E-04	0.052	2.24E-02	8.2202	1.71E-02	
44.0	1.0	43.5	120.0	2.53	1.70	1.182	36	1.25	91.8	0.6	39.9	0.8	1989.931	3.43E-04	5.20E-04	0.052	2.27E-02	8.2202	1.73E-02	
45.0	1.0	44.5	120.0	2.59	1.74	1.201	50	1.25	105.2	0.6	54.7	0.8	2237.586	3.08E-04	5.20E-04	0.052	1.55E-02	8.2202	1.19E-02	
46.0	1.0	45.5	120.0	2.65	1.78	1.219	50	1.25	105.2	0.6	54.1	0.8	2254.698	3.08E-04	5.20E-04	0.052	1.58E-02	8.2202	1.20E-02	
47.0	1.0	46.5	120.0	2.71	1.82	1.237	50	1.25	105.2	0.6	53.5	0.8	2271.554	3.09E-04	5.20E-04	0.052	1.60E-02	8.2202	1.22E-02	
48.0	1.0	47.5	120.0	2.77	1.86	1.255	50	1.25	105.2	0.6	52.9	0.8	2288.164	3.09E-04	5.20E-04	0.052	1.62E-02	8.2202	1.23E-02	
49.0	1.0	48.5	120.0	2.83	1.90	1.272	50	1.25	105.2	0.6	52.4	0.8	2304.535	3.09E-04	5.20E-04	0.052	1.64E-02	8.2202	1.25E-02	
50.5	1.5	49.8	120.0	2.91	1.95	1.293	50	1.25	105.2	0.6	51.7	0.8	2324.678	3.09E-04	5.20E-04	0.052	1.66E-02	8.2202	1.27E-02	
TOTAL SETTLEMENT =																			0.22	



NO SCALE

GEOCON
CONSULTANTS, INC.



ENVIRONMENTAL GEOTECHNICAL MATERIALS
3160 GOLD VALLEY DRIVE, SUITE 800 - RANCHO CORDOVA, CA 95742
PHONE (916) 852-9118 - FAX (916) 852-9132

DRAFTED BY: RP

CHECKED BY: JMT/NDB

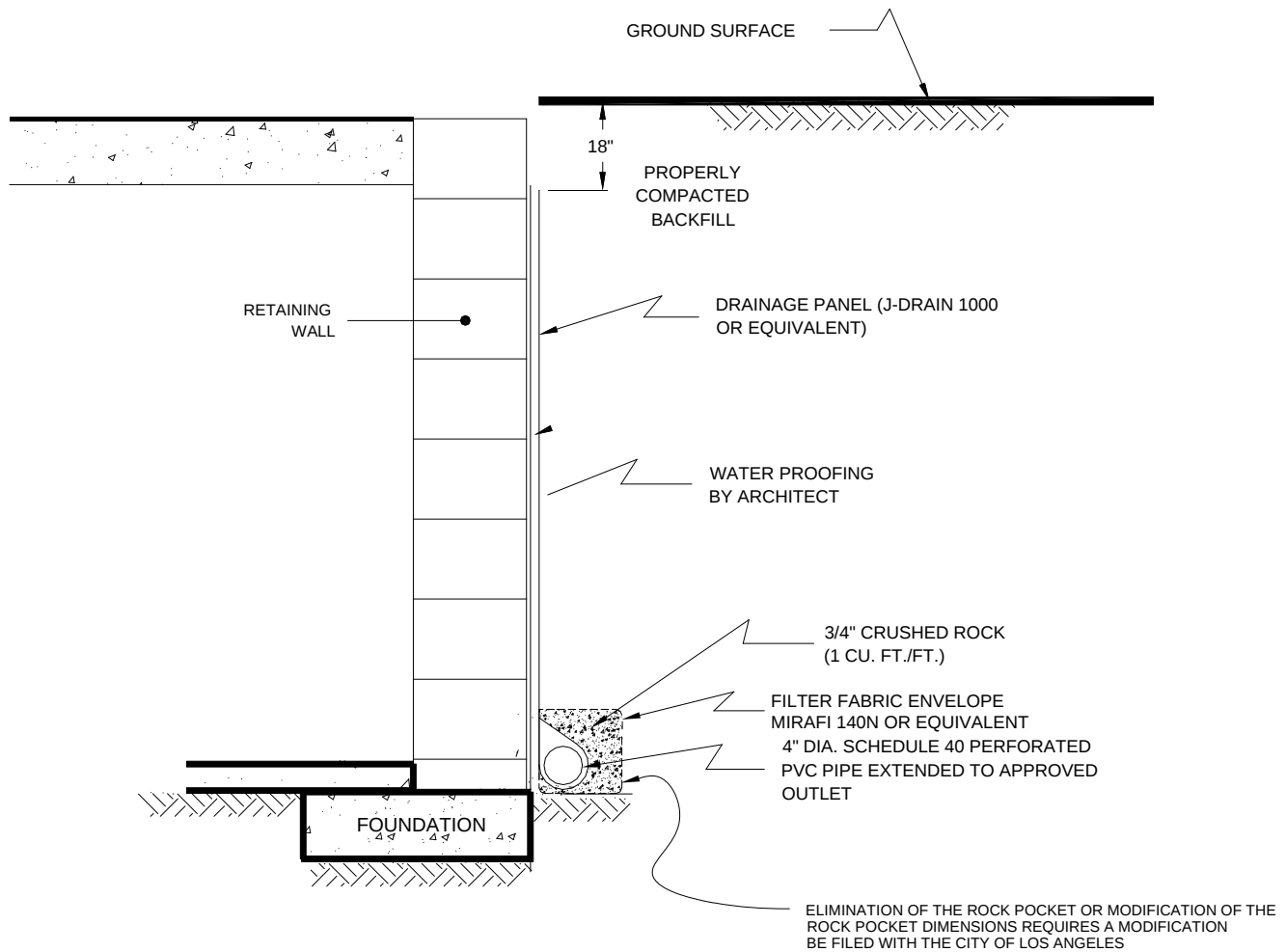
RETAINING WALL DRAIN DETAIL

PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. 16



NO SCALE

GEOCON
CONSULTANTS, INC.



ENVIRONMENTAL GEOTECHNICAL MATERIALS
3160 GOLD VALLEY DRIVE, SUITE 800 - RANCHO CORDOVA, CA 95742
PHONE (916) 852-9118 - FAX (916) 852-9132

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RETAINING WALL DRAIN DETAIL

PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. 17

APPENDIX

A

APPENDIX A

FIELD INVESTIGATION

The site was explored on May 11, 2018 by excavating four 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 50½ and 15½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 and A4. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1 ELEV. (MSL.) - - DATE COMPLETED <u>5/11/18</u> EQUIPMENT <u>HOLLOW STEM AUGER</u> BY: <u>RP</u>	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
					TILLED SOIL Sandy Silt, firm, slightly moist, dark brown, fine-grained, trace coarse-grained, trace roots.			
2	B1@2'			ML	ALLUVIUM Silt, soft, slightly moist, olive brown, fine-grained.	8	---	---
4					Clay, soft, slightly moist, olive brown, slight plasticity.			
6	B1@5'					11	99.1	17.3
8	B1@7'			CL	- firm, olive brown with dark brown mottles	10	---	---
10	B1@10'				- stiff, some fine-grained sand and silt	28	103.5	18.7
12	B1@12'			CL	Sandy Clay, firm, slightly moist, olive brown to light brown, fine-grained.	15	---	---
14								
16	B1@15'			SP	Sand, poorly graded, loose, slightly moist, pale brown, fine-grained, trace silt.	18	99.2	4.0
18	B1@17'				Sandy Clay, firm, slightly moist, pale brown with olive brown mottles, fine-grained.	12	---	---
20	B1@20'			CL		21	107.9	12.8
22	B1@22'				Silty Sand, medium dense, slightly moist, light olive brown, fine-grained.	14	---	---
24								
26	B1@25'			SM		19	107.2	6.5
28	B1@27'				- olive brown	15	---	---

Figure A1,
Log of Boring 1, Page 1 of 2

S9962-05-23 BORING LOGS.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1 ELEV. (MSL.) - - DATE COMPLETED <u>5/11/18</u> EQUIPMENT <u>HOLLOW STEM AUGER</u> BY: <u>RP</u>	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B1@30'			SM	- trace fine gravel	35	114.5	6.3
32	B1@32'				- medium- to coarse-grained - fine- to medium-grained	20	---	---
34	B1@35'				Sand with Silt, poorly graded, very dense, slightly moist, olive brown, fine- to medium-grained.	50 (3")	117.2	2.8
36	B1@37'			SP-SM	- medium- to coarse-grained - fine- to medium-grained	78		
38	B1@40'				- no recovery	50 (4")	---	---
40	B1@42'				Clay, hard, slightly moist, dark olive brown, fine-grained, plastic, trace silt, trace oxidation staining.	36	---	---
42	B1@45'			ML	Silt with Gravel, hard, slightly moist, olive brown with dark brown mottles, some oxidation staining.	50 (4")	---	---
44	B1@47'				- no recovery	50 (5")	---	---
46	B1@50'					50 (3")	---	---
48					Total depth of boring: 50.5 feet Fill to 1.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

Figure A1,
Log of Boring 1, Page 2 of 2

S9962-05-23 BORING LOGS.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2 ELEV. (MSL.) - - DATE COMPLETED 5/11/18 EQUIPMENT HOLLOW STEM AUGER BY: RP	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				MATERIAL DESCRIPTION			
2					TILLED SOIL Sandy Silt, firm, dry to slightly moist, brown to dark brown, fine-grained.			
4	B2@4'			ML	ALLUVIUM Silt, firm, slightly moist, olive brown with light brown mottles, some clay, trace rootlets and calcium deposits.	14	97.1	18.8
6					- stiff, increase in clay content			
8	B2@8'			CL	Clay, stiff, slightly moist, olive brown.	25	94.9	17.2
10								
12	B2@12'			ML	Sandy Silt, stiff, slightly moist, olive brown, fine-grained, trace medium- to coarse-grained.	27	95.7	8.5
14								
	B2@15'			CL	Clay, firm, slightly moist, olive brown, plastic.	20	99.9	12.2
					Total depth of boring: 15.5 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A2,
Log of Boring 2, Page 1 of 1

S9962-05-23 BORING LOGS.GPJ

SAMPLE SYMBOLS	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3 ELEV. (MSL.) - - DATE COMPLETED <u>5/11/18</u> EQUIPMENT <u>HOLLOW STEM AUGER</u> BY: <u>RP</u>	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
					TILLED SOIL Sandy Silt, soft to firm, dry, brown, fine-grained.			
2					ALLUVIUM Sandy Silt, firm, slightly moist, olive brown, fine-grained.			
4	B3@4'			ML		12	97.9	11.9
6								
8	B3@8'					21	93.0	14.5
10								
12	B3@12'				- no recovery	18	---	---
14								
	B3@15'				- stiff, light olive brown	17	95.3	20.6
					Total depth of boring: 15.5 feet Fill to 1 foot. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

Figure A3,
Log of Boring 3, Page 1 of 1

S9962-05-23 BORING LOGS.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 4 ELEV. (MSL.) - - DATE COMPLETED <u>5/11/18</u> EQUIPMENT <u>HOLLOW STEM AUGER</u> BY: <u>RP</u>	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				MATERIAL DESCRIPTION			
2					TILLED SOIL Sandy Silt, soft, slightly moist, dark brown, fine-grained.			
4	B4@4'			ML	ALLUVIUM Sandy Silt, soft, slightly moist, olive brown, fine-grained, porous.	9	104.9	18.1
6								
8	B4@8'			CL	Clay, firm, slightly moist, olive brown to brown, fine-grained, plastic.	18	97.3	17.2
10								
12	B4@12'				- stiff	22	98.1	19.6
14				SP	Sand, poorly graded, medium dense, slightly moist, pale brown, fine-grained, trace silt.			
	B4@15'			CL	Clay, firm, slightly moist, olive brown, plastic.	27	90.6	27.3
Total depth of boring: 15.5 feet Fill to 1 foot. No groundwater encountered. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.								

Figure A4,
Log of Boring 4, Page 1 of 1

S9962-05-23 BORING LOGS.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

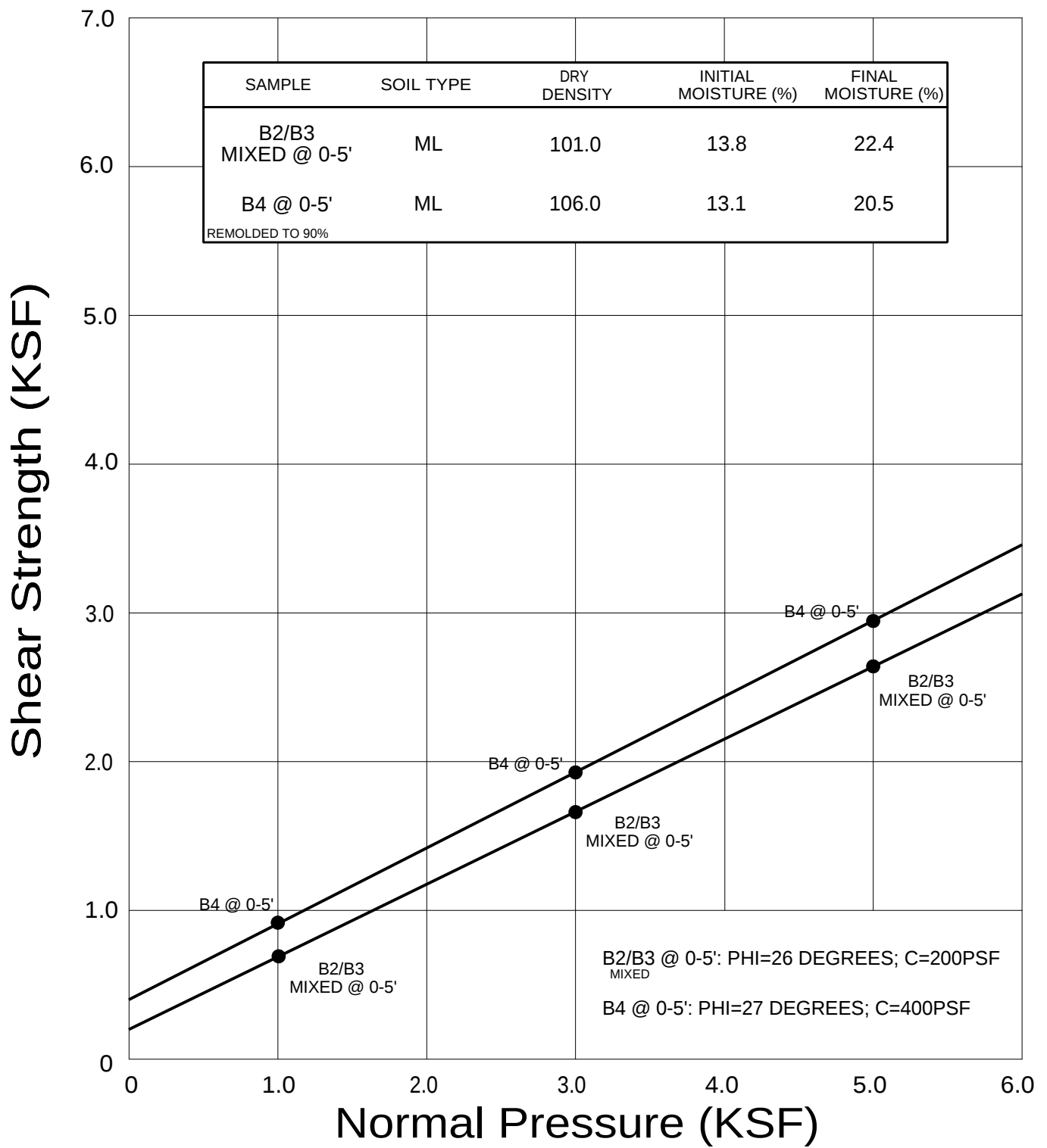
APPENDIX

B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for grain size, plasticity indices, direct shear strength, consolidation characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B10. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



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ENVIRONMENTAL GEOTECHNICAL MATERIALS
3160 GOLD VALLEY DRIVE, SUITE 800 - RANCHO CORDOVA, CA 95742
PHONE (916) 852-9118 - FAX (916) 852-9132

DRAFTED BY: RP

CHECKED BY: JMT/NDB

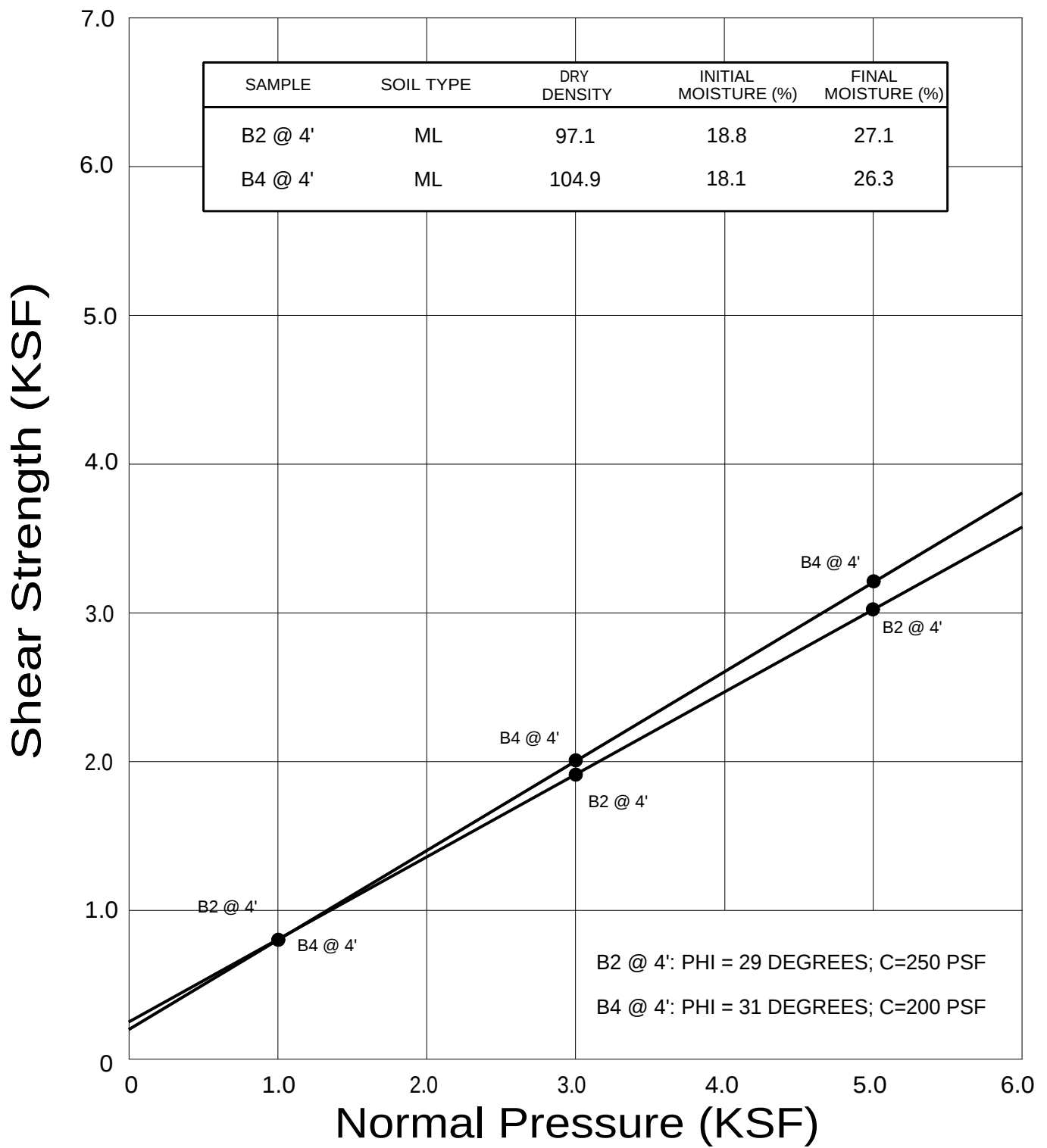
DIRECT SHEAR TEST RESULTS

PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. B1



● Direct Shear, Saturated

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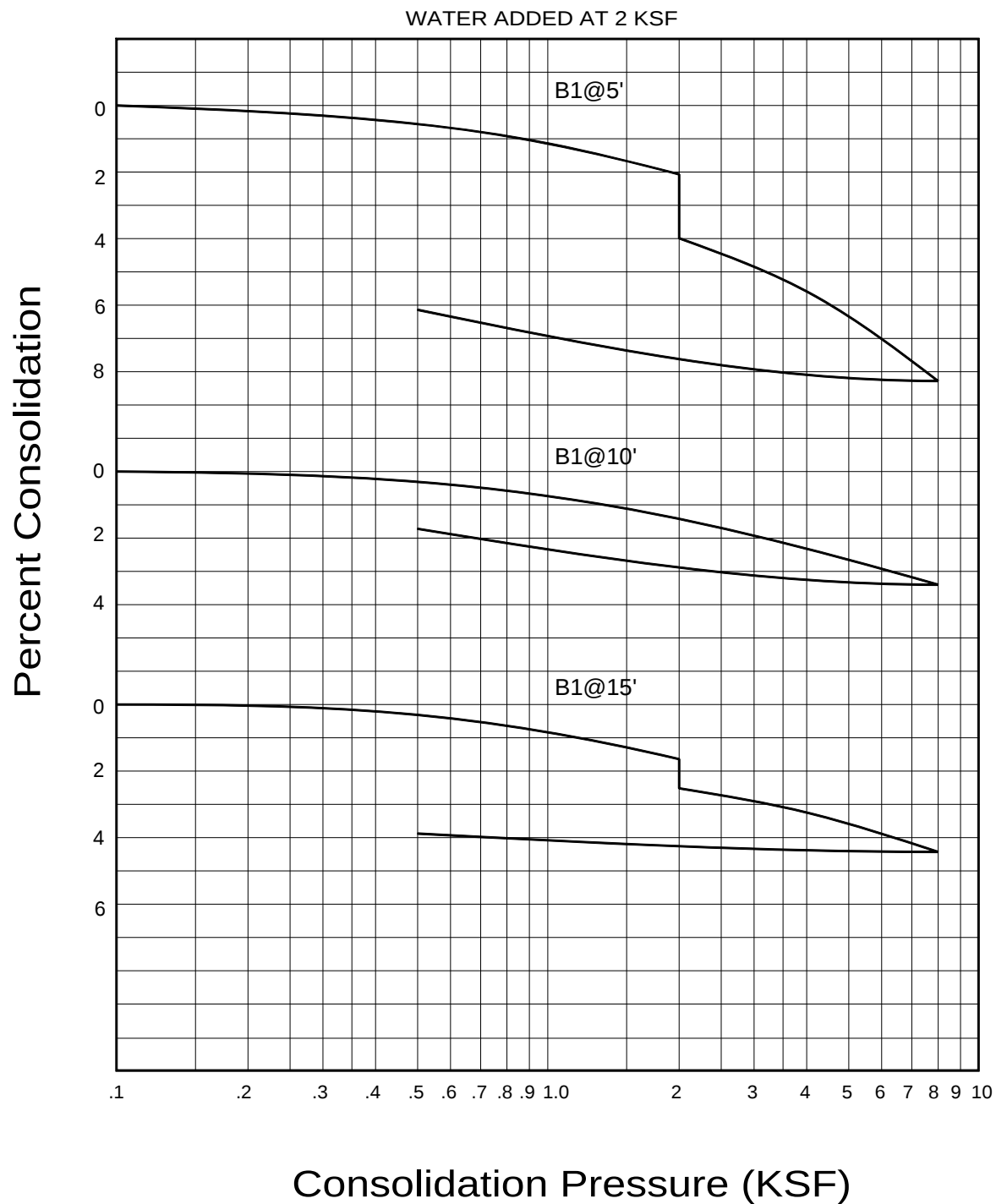
DIRECT SHEAR TEST RESULTS

PARKFIELD FIRE STATION
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FIG. B2



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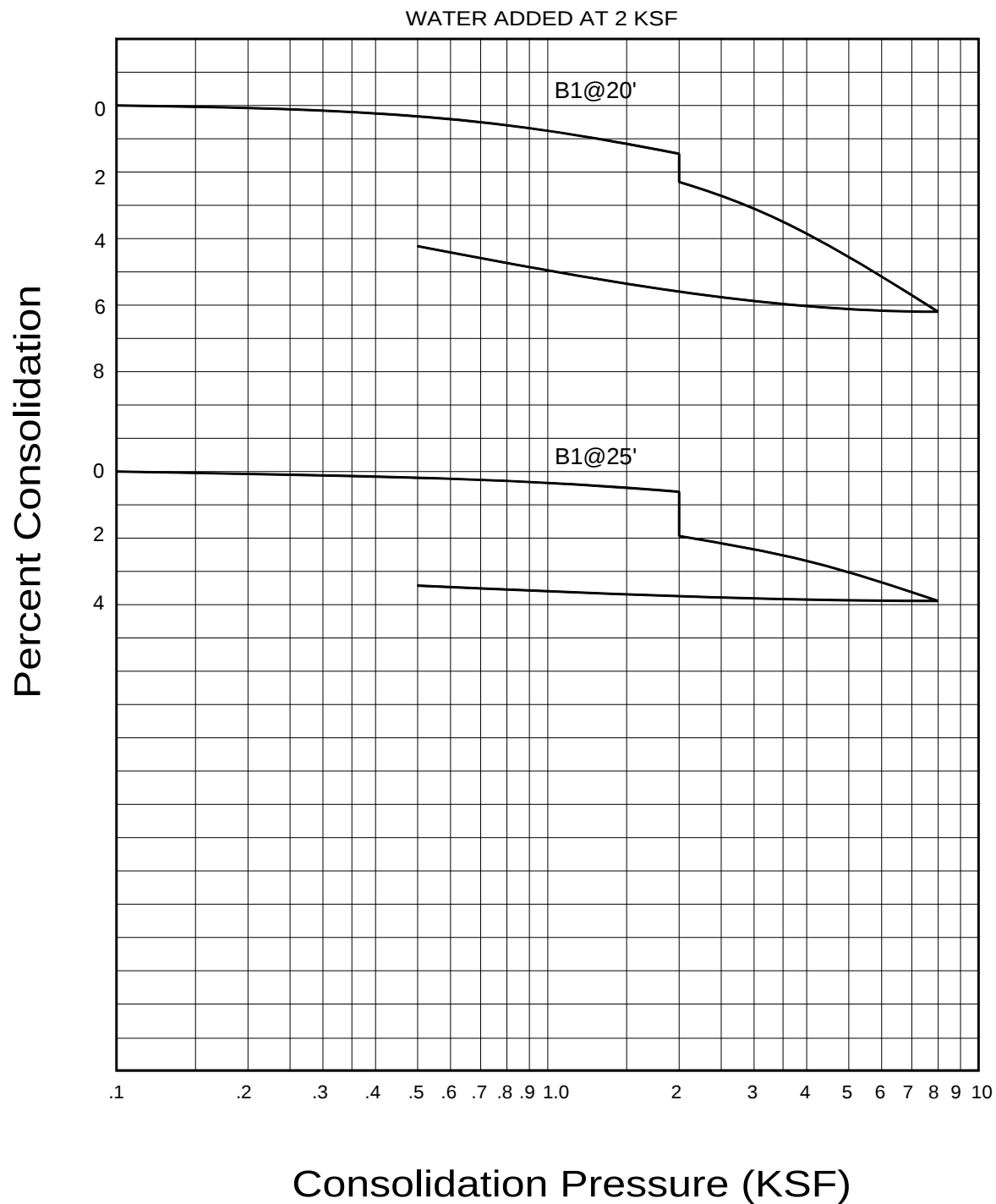
CONSOLIDATION TEST RESULTS

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FIG. B3



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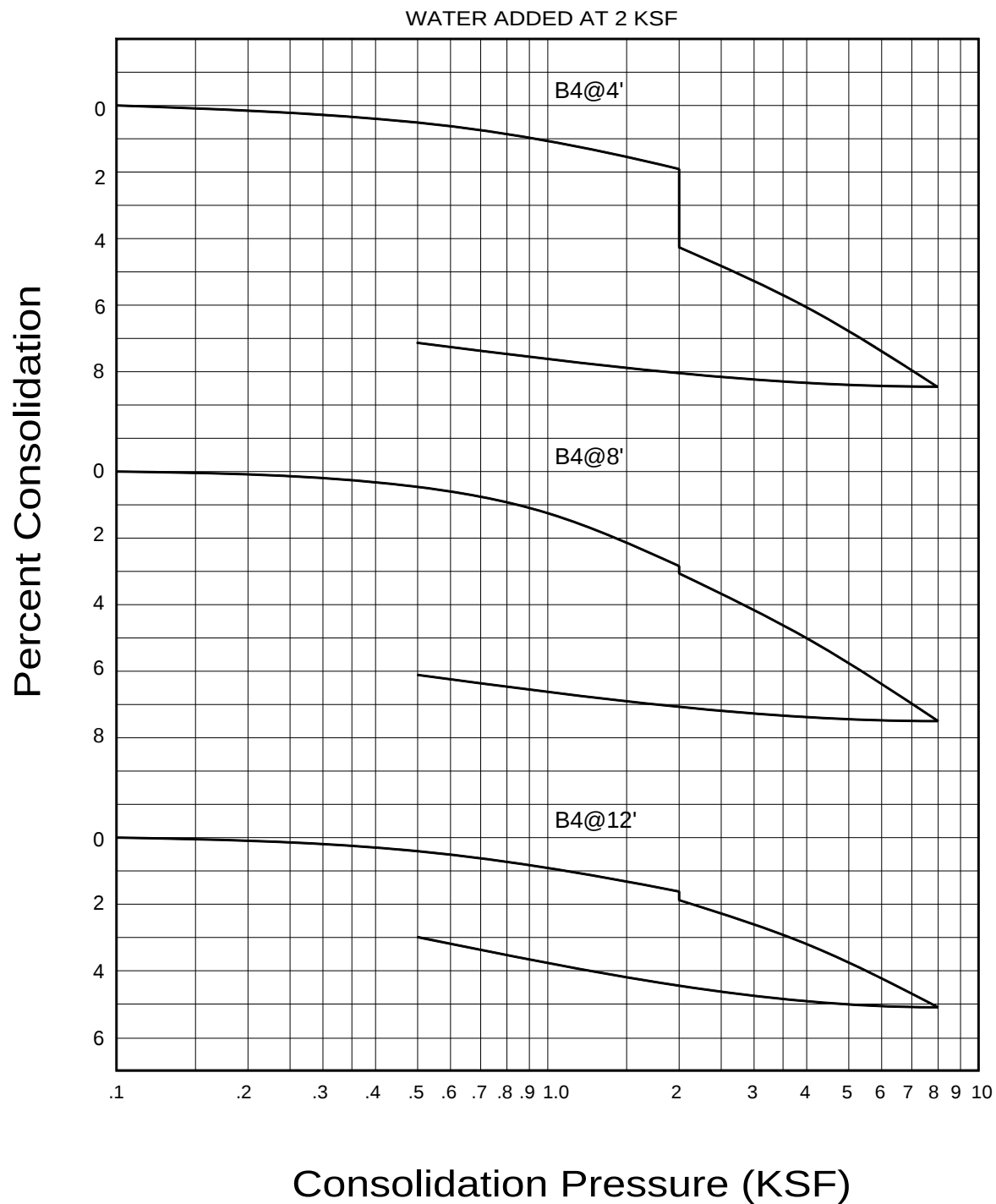
CONSOLIDATION TEST RESULTS

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FIG. B4



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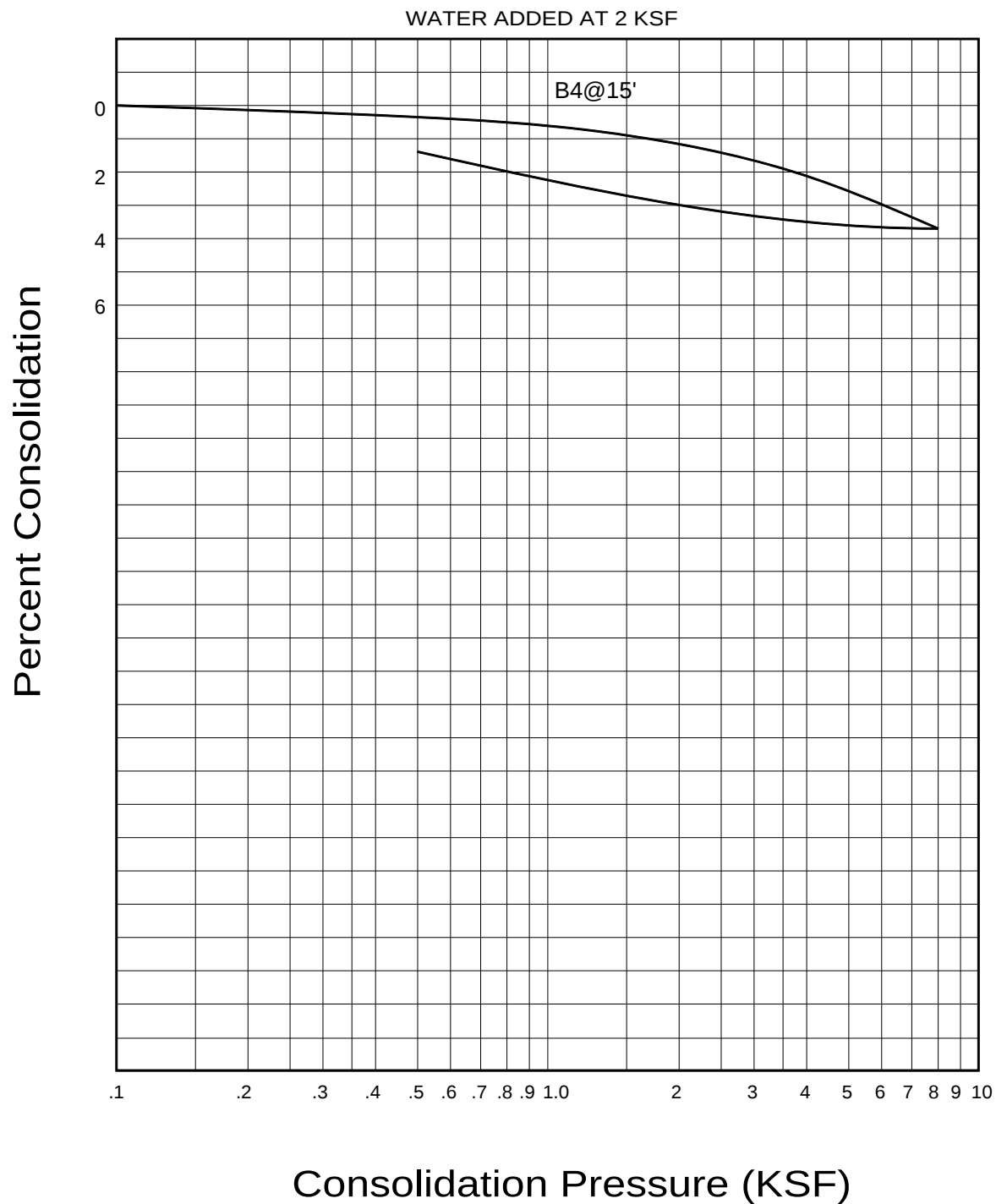
CONSOLIDATION TEST RESULTS

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FIG. B5



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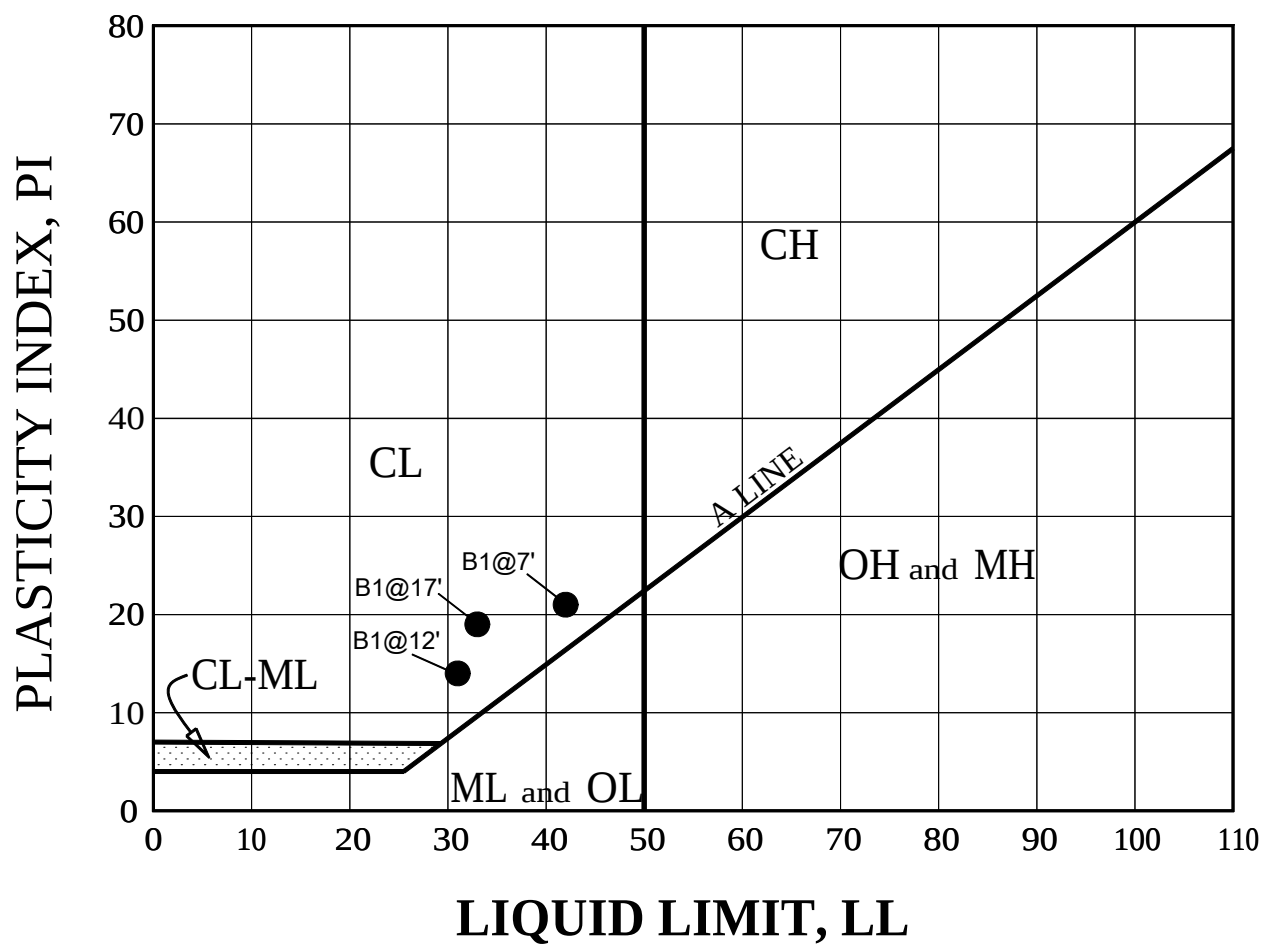
CONSOLIDATION TEST RESULTS

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FIG. B6



N/P: NON-PLASTIC

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DRAFTED BY: RP

CHECKED BY: JMT/NDB

ATTERBERG LIMITS

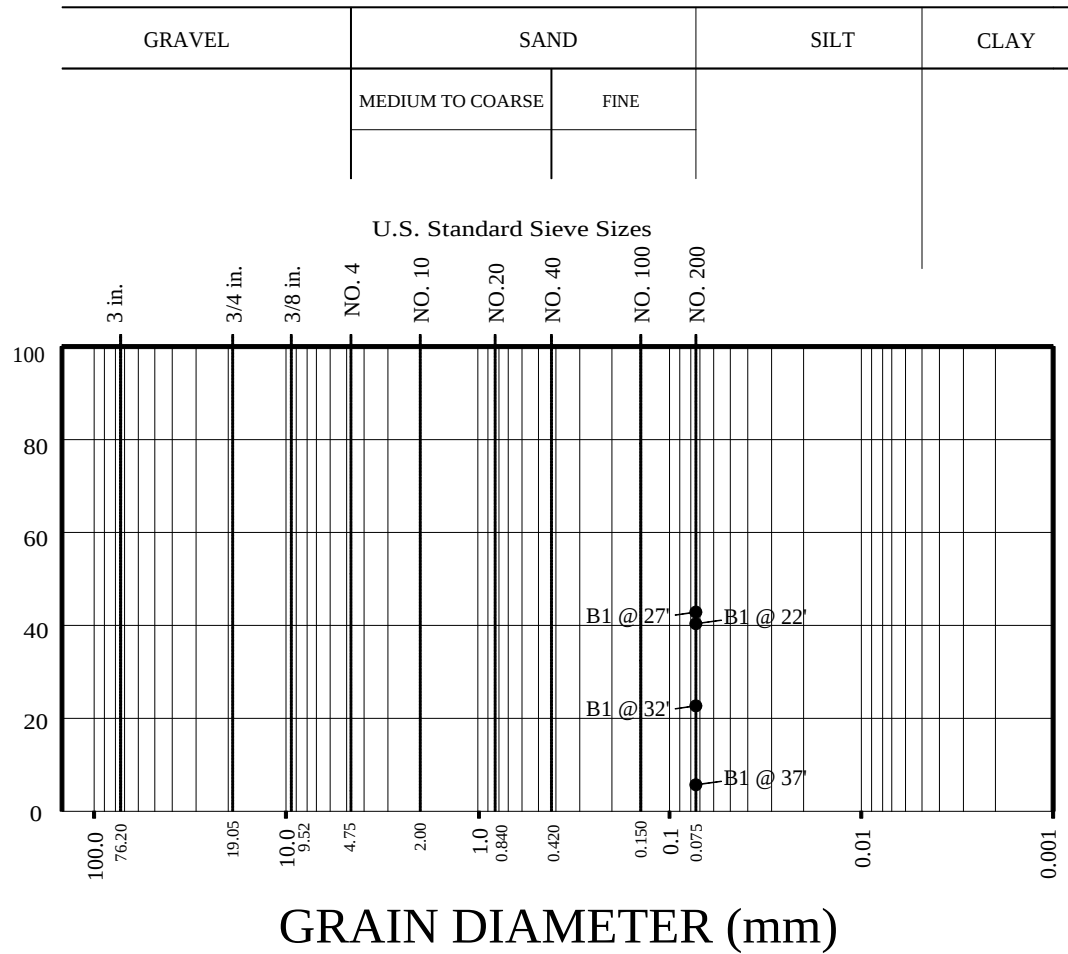
PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. B7

PERCENT PASSING NO. 200 SIEVE



SAMPLE	PERCENT PASSING NO. 200 SIEVE
B1 @ 22'	40.4
B1 @ 27'	42.9
B1 @ 32'	22.6
B1 @ 37'	6.5

B1 @ 22'	40.4
B1 @ 27'	42.9
B1 @ 32'	22.6
B1 @ 37'	6.5

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DRAFTED BY: RP

CHECKED BY: JMT/NDB

GRAIN SIZE ANALYSIS

PARKFIELD FIRE STATION

70331 VINEYARD CANYON ROAD

PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. B8

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B2/B3 @ 0-5'	13.4	31.6	96.7	91	High	Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2016 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B2/B3 @ 0-5'	Brown Sandy Silt	112.5	13.5
B4 @ 0-5'	Olive Brown Sandy Silt	118.0	12.5

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LABORATORY TEST RESULTS

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PARKFIELD, CALIFORNIA

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PROJECT NO. S9962-05-23

FIG. B9

**SUMMARY OF LABORATORY POTENTIAL OF
HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643**

Sample No.	pH	Resistivity (ohm centimeters)
B2/B3 @ 0-5'	8.09	1000 (Corrosive)
B4 @ 0-5'	7.80	1200 (Corrosive)

**SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3**

Sample No.	Chloride Ion Content (%)
B2/B3 @ 0-5'	0.009
B4 @ 0-5'	0.003

**SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417**

Sample No.	Water Soluble Sulfate (% SO ₄)	Sulfate Exposure*
B2/B3 @ 0-5'	0.001	Negligible
B4 @ 0-5'	0.000	Negligible

* Reference: 2016 California Building Code, Section 1904 and ACI 318-11 Section 4.3.

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CORROSIVITY TEST RESULTS

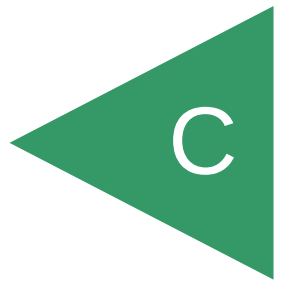
PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

JULY 2018

PROJECT NO. S9962-05-23

FIG. B10

APPENDIX





State of California • Natural Resources Agency
Department of Conservation
California Geological Survey
6105 Airport Road
Redding, CA 96002-9422

Edmund G. Brown Jr., Governor
John G. Parrish, Ph.D., State Geologist

September 14, 2017

Mr. Don Clark, P.E.
Supervising Civil Engineer
CAL FIRE
PO BOX 944246
1300 U STREET, Sacramento, CA 94244

Dear Mr. Clark,

The California Geological Survey (CGS) is pleased to present this letter providing the results of this preliminary Geological Hazards investigation of the proposed Parkfield Fire Station (FS), located at 70331 Vineyard Canyon Road, San Miguel, California (Figure 1). The proposed Parkfield FS (site) is to replace an existing, but outdated, FS located at 70578 Parkfield-Coalinga Road, Parkfield, California.

The following presents CGS's: 1) understanding of the project, 2) scope of work, 3) results of field work performed, and 4) conclusions and recommendations pertaining to the project development.

PROJECT UNDERSTANDING

Based on discussions with CAL FIRE personnel familiar with the project and on a review of the existing preliminary site plan, it is CGS's understanding the proposed project consists of the construction of a new Parkfield FS facility within the eastern portion of the site that includes: 1) an apparatus building, 2) a single-story barracks building, 3) a generator and pump building, 4) a well house with water treatment and storage tanks, and 5) ancillary improvements including miscellaneous outbuildings, fencing, underground utilities, concrete sidewalks, and asphaltic concrete paving.

PROJECT SCOPE

At the request of CAL FIRE, CGS performed the following work to assess the site for the presence of potential geologic hazards that may significantly impact the property and the planned improvements:

- Reconnoitered the site to evaluate the local geologic and topographic conditions;
- Conduct preliminary percolation testing for proposed on-site leach field;
- Review of published documents pertaining to potential geologic hazards and flooding;
- Review of historical aerial photographs;
- Excavated and logged three test pits in the approximate location of the new FS to evaluate the shallow subsurface conditions; and
- Prepared this letter presenting the study findings.

SITE DESCRIPTION

The approximate 40 acre, square shaped site is bounded by vacant land to the north and east, agricultural land to the south, and a single-family dwelling to the west. The northern portion of the site is bisected by Vineyard Canyon road which separates moderately-sloping ground to the north (approximately 10 acres), from gently sloping ground to the south. Structures onsite consist of a private residence and a well located in the northeast and southern portion of the site respectively. The well feeds a water tank that is located along the northern site boundary. The vegetation consists of low level grasses with a few scattered trees. Surface flow within most of the site is by sheet flow that drains towards an ephemeral swale within the southern portion of the site.

HISTORICAL PROPERTY USE

As part of this report, CGS reviewed published aerial photographs of the project site (see Historical Photos, June 2017). Based on this review, the site was generally vacant land as far back as 1994. By 2003, the residence, well and water tank had been built. Significant changes to the site were not observed within more recent photographs.

SITE CONDITIONS

Geologic Setting

Published geologic maps (Diblee and Minch., 2005) indicate that the project site is located within the southern portion of the Coast Range geologic and geomorphic province. The topography within the Coast Range is structurally controlled by numerous active faults including the San Andreas Fault and consists of a series of long, northwest trending mountain ranges separated by river valleys. The most prominent mountain ranges in the southern portion of the Coast Range are the Santa Lucia Range and Gabilan Range that flank the Salinas Valley west of the project site. The province is drained to the Pacific Ocean by several rivers including the Eel and Salinas Rivers, the latter of which has a present day course approximately 22 miles southwest of the project site (Harden, 2004).

A review of published geologic maps indicates that the project site is located within the northern extent of the Cholame Valley and is underlain primarily by Quaternary-age surficial sediments (Qa) consisting of mixtures of sands, gravels and clays. A minor portion of the site is underlain by Pleistocene-age Paso Robles Formation (Qtp) that is described as gravel conglomerate, with white siliceous shale (Diblee and Minch, 2005). The geology of the project site and vicinity is shown in Figure 2, *Regional Geologic Map*. According to the USDA, (2016) the local soils consists of well drained mixtures of silty clay, clayey, and sandy loams.

This area of California generally experiences a Mediterranean Climate, with warm summers and cool winters. Over the past 100 years, the average climatic conditions at the site include an average annual precipitation of about 13 inches and mean annual temperatures between 55° to 60° F (USDA, 2014 and U.S. Climate Data).

Surface Conditions

The project site generally consists of vacant agricultural land that slopes gently (average of about 5%) towards the south. A minor portion of the site north of Vineyard Canyon has been graded in order to construct the residence and associated structures.

Subsurface Conditions

A total of three (3) test pits (TP-1 through TP-3) were excavated on June 20, 2017, to obtain preliminary shallow subsurface information. The test pits were excavated utilizing a rubber tire backhoe (Caterpillar Model 420F) equipped with a 24-inch wide bucket. The test pits were logged by a California Certified Engineering Geologist in accordance with the Unified Soil Classification System (USCS) [ASTM 2488]. The approximate test pit locations are shown in Figure 3 – *Test Pit and Percolation Test Location Map*. In addition, the approximate latitude and longitude in decimal degrees for each test pit is presented in the following table.

Table No. 1, Approximate Latitude and Longitude of Test Pits

Test Pit Number	Latitude	Longitude	Depth of Excavation (feet bgs)
TP-1	35.90142	-120.44600	8.0
TP-2	35.90130	-120.44567	8.0
TP-3	35.90106	-120.44607	8.0

The test pits were excavated to 8 feet below ground surface (bgs). In general, the soils encountered within the test pits are assumed to be representative across the site and consist of Clayey Sands (SC), Silty Sands (SM) and low-plastic Silts (ML). Fill soils were not encountered within our test pits. Detailed descriptions of the subsurface soils encountered in each of the test pits are presented in Appendix A, Table No.2 - *Summary of Test Pit Logs*.

Percolation Testing

A total of six (6) percolation tests (PT-1 through PT-6) were conducted in accordance with the *Monterrey County Code, Chapter 15.20* which references the *U.S. Public Health Service Manual No. 526, pages 3 through 8 (Robert A. Taft Engineering Center Percolation Test Procedure)*. The results indicate that the in-situ percolation rates range from about 4.6 minutes per inch to about 15 minutes per inch (see *Appendix B, for the Percolation Test Worksheets*). The approximate percolation test locations are shown in Figure 3 – *Test Pit and Percolation Test Location Map*.

The test pits were backfilled loosely with the excavated spoils and surface tracked. As a result, the test pits will need to be re-located, over excavated and properly backfilled in thin lifts with compacted effort.

FAULTING

The area is within a seismically active area and is adjacent to several mapped active or potentially active faults, including the Middle Mountain Fault and the San Andreas Fault located approximately 0.30 miles southwest and 0.20 miles northeast of the project site respectively (Jennings and Bryant, 2010) and (Diblee and Minch, 2005). Based on regional geologic mapping, there does appear to be a potential active fault extending across the project site, north of Vineyard Canyon (Diblee and Minch, 2005). Moreover, although not shown on published geologic maps, there appears to be a topographic lineation formed by a northwest-trending drainage and swale that parallels the San Andreas Fault to the west and projects through the site. This topographic lineation appears to be structurally controlled and may be associated with the trace of an unmapped fault as shown in Figure 4 – *Possible Fault Trace Location Map*.

A review of Earthquake Fault Zone maps (referred to as Special Studies Zone maps prior to January 1, 1994) shows the site to be positioned between two mapped Earthquake Fault Zones, and may partially overlap a zone in the far northeast corner of the site Figure 5 – *AP Zone Map* (CGS, 1986).

Due to the proximity of the site to mapped active and potentially active faults, it is likely the site would be subjected to strong to very strong levels of ground shaking during its life time (Cao et al., 2003). Furthermore, to mitigate against potential fault rupture impacts, a detailed investigation may be required to adequately assess the location of mapped and unmapped active fault traces that may cross the site.

GROUNDWATER

Groundwater was not encountered within any of the test pits excavated to a maximum explored depth of 8.0 feet bgs. Moreover, depth to groundwater within the existing onsite well could not be measured due to poor access.

Historical groundwater levels were evaluated for wells within 1 mile of the project site. Based on our review of the National Water Information System (USGS, 2017), the nearest “documented” well to the site is USGS Well No. 355405120263301, which is approximately 0.15 miles northeast of the project site at an elevation of 1533 feet amsl. A review of records from 1979 through 1994 indicate that the historical high groundwater level is approximately 16.1 feet bgs, or approximately 1517 feet amsl, measured in April, 1983. This well was abandoned in 1994.

The potential presence and depth to groundwater within the project site should be verified in a site specific geotechnical investigation.

FLOODING

According to the Federal Emergency Management Agency (FEMA), the site is designated as Zone X on the Flood Insurance Rate Map (FIRM), which is defined as “areas determined to be outside the 0.2 percent annual chance floodplain” (FEMA, 2009). Consequently, the site has a low risk of being impacted by flooding.

DISCUSSION OF GEOLOGIC HAZARDS

Landsliding

The site topography is gentle (<10%) and there are no mapped landslides within the site vicinity (Diblee and Minch, 2005). For these reasons, it is unlikely the site would be significantly adversely impacted by landsliding.

Ground Shaking

The anticipated peak ground acceleration at the site with a 10 percent probability of being exceeded in 50 years (475-year return period) is about 0.949 times the acceleration of gravity (CGS, 2008). Thus, the project site could be subject to strong to very strong ground shaking in the event of an earthquake.

Liquefaction, Lateral Spreading, and Seismic-Induced Settlement

Liquefaction is described as the sudden loss of soil shear strength due to a rapid increase of soil pore water pressures caused by cyclic loading from a seismic event. In simple terms, it means that a liquefied soil acts more like a fluid than a solid when shaken during an earthquake. In order for liquefaction to occur during a seismic event, the following are needed:

- Granular soils (sand, silty sand, sandy silt, and some gravels);
- A high groundwater table; and
- A low density in the granular soils underlying the site.

If the above are present and strong ground motion occurs, then the soils could liquefy, depending upon the intensity and duration of the strong ground motion. Liquefaction that produces surface effects generally occurs in the upper 50 feet of the soil column, thus, the potential for liquefaction to have an adverse effect would generally require the criteria above to persist within 50 feet below the surface.

The project site is underlain by loose to moderately-dense deposits consisting primarily of Clayey Sands (SC), Silty Sands (SM) and Silts (ML). In addition, according to the Monterey County General Plan, the project site is located within an area considered to have a high potential for liquefaction (Monterey County, 2007).

In consideration of these site conditions, we conclude the potential for liquefaction and its associated adverse effects (e.g. settlement, lurch cracking, etc.) are high and should be verified in a site specific geotechnical investigation.

Tsunamis

Tsunamis are large waves generated in open bodies of water by fault displacement or major ground movement. Based on the inland location and elevation of the site, tsunamis do not pose a hazard to the site.

Seiches

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Due to the absence of significant bodies of water near the site, the potential for flooding due to seiching is considered to be low.

Earthquake-Induced Flooding

Dams or other water-retaining structures may fail as a result of large earthquakes. The project site is not located within a mapped flood inundation zone associated within any nearby dams or other water retaining structures (Monterey County, 2007). It should be noted that rupture of onsite water tanks associated with a seismic event could cause localized flooding.

Expansive Soils

The potential for clay-rich soils to swell and shrink with variations in soil moisture content is correlated to the plasticity index of the soil, with expansive soils generally having a high plasticity index. Because the on-site soils encountered in our investigation were found to consist of clayey sands (SC), silty sands (SM) and silts (ML) with low plasticity, we conclude the risk of adverse consequences related to expansive soils is low to moderate. However, the potential for expansive soils should be confirmed in a site specific geotechnical investigation.

Naturally Occurring Asbestos

Naturally Occurring Asbestos (NOA) is the term applied to the natural geologic occurrence of asbestos that is related to the chemistry of rocks in an area and the different geologic processes that have acted on those rocks through time. Serpentinized ultramafic rocks can commonly contain concentrations of NOA.

A review of the General Location Guide for Ultramafic Rocks in California - Areas More Likely to Contain Naturally Occurring Asbestos (Churchill and Hill, 2000), indicates that the project site is not located within an area known to contain ultramafic rocks.

Based on this, it is our opinion the risk to human health from NOA exposure through the development of the site is low due to the absence of ultramafic serpentinized rock. If serpentinized rock, or other ultramafic rock commonly associated with NOA is encountered on-site, additional sampling and laboratory analyses may be warranted.

Radon

Radon is a radioactive gas that is produced from radioactive decay of uranium. Uranium is naturally present in the earth's crust with higher concentrations in certain types of rock, such as granite and shale. A review of the Radon Potential Zone Map for Eastern Monterey County (Churchill, 2007), shows the site in an area of low potential for indoor Radon levels above 4 PicoCuries per liter [pCi/L]. As a result, adverse effects from Radon gas at the site are considered to be low.

Volcanic Eruption

As discussed above, the site is located within the southern portion of the Coastal Range, which has not been subject to volcanic activity in the Geologic past. The closest Volcanic Center is the Coso Volcanic Area, approximately 150 miles east of the project site. Coso Volcanic Area last erupted approximately 40,000 years ago (USGS, 2012). Therefore, the potential for adverse impacts to the project site due to an eruption within the Coso Volcanic area are considered to be low.

CONCLUSIONS AND RECOMMENDATIONS

Based on the findings from our literature review and limited field exploration, we identified several areas of potential concern that should be evaluated and addressed as part of a site-specific geotechnical evaluation, including: surface fault rupture associated with mapped and unmapped faults that may extend across the site, liquefaction and its associative impacts, and potentially expansive soils. It is CGS's understanding that a site-specific geotechnical investigation will be completed at a future date.

Original signed by:

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Concur:

September 14, 2017 Original signed by:

Date Donald N. Lindsay, CEG 2323, PE 76899
Senior Engineering Geologist/Geotechnical Engineer
Redding, California



Attachments:

Figures 1-5
Appendix A – Summary of Test Pit Logs

REFERENCES

ASTM (American Society for Testing Materials), Latest Edition.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, *The Revised 2002 California Probabilistic Seismic Hazard Maps* June 2003: California Geological Survey Web Page
http://www.conserv.ca.gov/cgs/rghm/psha/fault_parameters/pdf/2002_CA_Hazard_Map.pdf

CGS, 1986, *State of California Special Studies Zones, Parkfield Quadrangle*, scale 1:24,000, dated July 1, 1986.

CGS, 2002, *California Geomorphic Provinces*, California Geological Survey, Note 36, 4 pages, available at
http://www.conserv.ca.gov/cgs/information/publications/cgs_notes/note_36/Documents/note_36.pdf.

CGS, 2008, *Earthquake Shaking Potential for California Ground Motion Interpolator*, accessed in June 2016, and available at http://www.quake.ca.gov/gmaps/PSHA/psha_interpolator.html

Churchill R.K., 2007, *Radon Potential Map for Eastern Monterey County, California*, California Department of Conservation, Special Report 201, Plate 2 of 2, scale 1:100,000.

Churchill R.K., Hill R.L., 2000, *A General Location Guide for Ultramafic Rocks in California-Areas More Likely to Contain Naturally Occurring Asbestos*, California Department of Conservation, Open-File Report 2000-19, scale 1:100,000.

Diblee, T.W., and Minch, J.A., (compilers), 2005, *Geologic map of the Parkfield Quadrangle, Fresno and Monterey Counties, California*: Diblee Geological Foundation, Diblee Foundation Map DF-139, scale 1:24,000, downloaded from NationalMap.gov.

Federal Emergency Rate Management (FEMA), 2009, *Flood Insurance Rate Map (FIRM), Monterey County, California and Unincorporated Areas, Panel 1725, Map No. 06053C1725G*, dated April 2, 2009.

Harden, D.R., 2004, *California Geology*, Second Edition, Chapter 12.

Historical Aerials, by NETR online, accessed June 2017, and available at
<http://www.historicaerials.com/aerials.php?op=home>

Jennings, C.W., and Bryant, W.A., 2010, *Fault activity map of California*: California Geological Survey Geologic Data map No. 6, map scale 1:750,000.

MONTEREY COUNTY CODE, 2004, *A Codification of the General Ordinances of Monterey County, California*, Chapter 15.20, accessed June 2017, and available at
https://library.municode.com/ca/monterey_county/codes/code_of_ordinances?nodeId=TIT15PUSE_CH15.20SEDI_15.20STSP

MONTEREY COUNTY, 2010, *Monterey County General Plan*, October 26, 2010, accessed June 2017, and available at <http://www.co.monterey.ca.us/government/departments-i-z/resource-management-agency-rma/-planning/resources-documents/2010-general-plan>.

MONTEREY COUNTY, 2007, *Monterey County General Plan, Draft Environmental Impact Report*, Volumes 1 and 2, accessed June 2017, and available at
<http://www.co.monterey.ca.us/government/departments-i-z/resource-management-agency-rma/-planning/resources-documents/2010-general-plan/draft-environmental-impact-report-deir>.

Mr. Don Clark, P.E.
September 14, 2017
Page 9

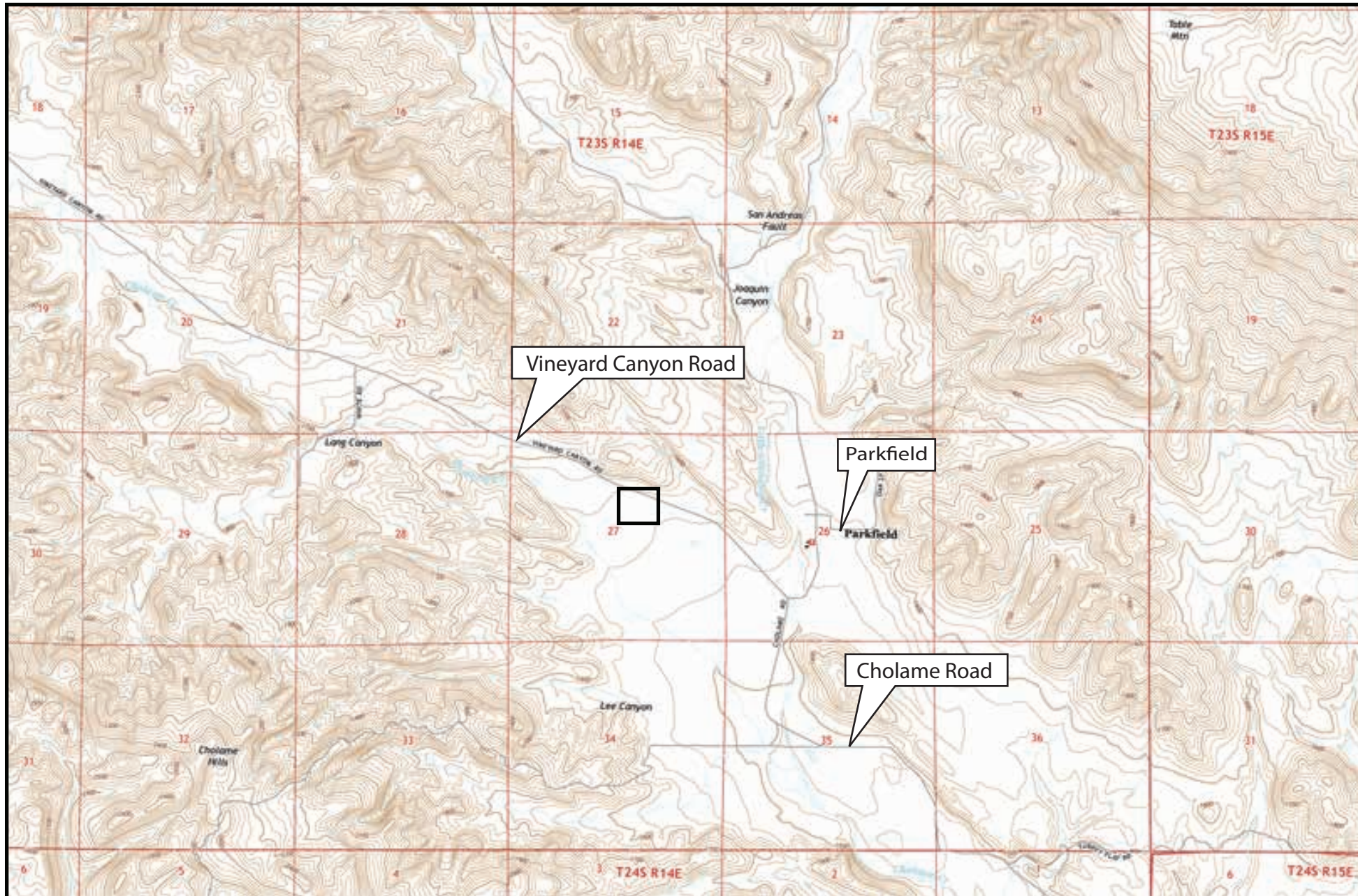
Proposed Parkfield FS

USDA, 2016, Natural Resources Conservation Service, *Web Soil Survey*, national Cooperative Soil Survey, accessed June 2017, and available at <http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx>.

USGS, 2012, *Volcanics Hazard Program, Coso Volcanic Area*, accessed June 2017, and available at <http://volcanoes.usgs.gov/observatories/calvo/>.

U.S. Climate Data, 2017, Version 2.2, accessed June 2017, and available at <http://www.usclimatedata.com/climate/california/united-states/3174>

U.S. Department of Health, Education, and Welfare, 1969, Public Health Service Manual, Publication No. 526, Robert A. Taft Engineering Center Percolation Test Procedure, pages 3-8.



Date: 7/7/20110
Scale: 1" = 3,406'

Approved By:
Chris Gryszan, CGS

Vicinity Map
To Accompany
Engineering Geologic Review of
Proposed Parkfield FS

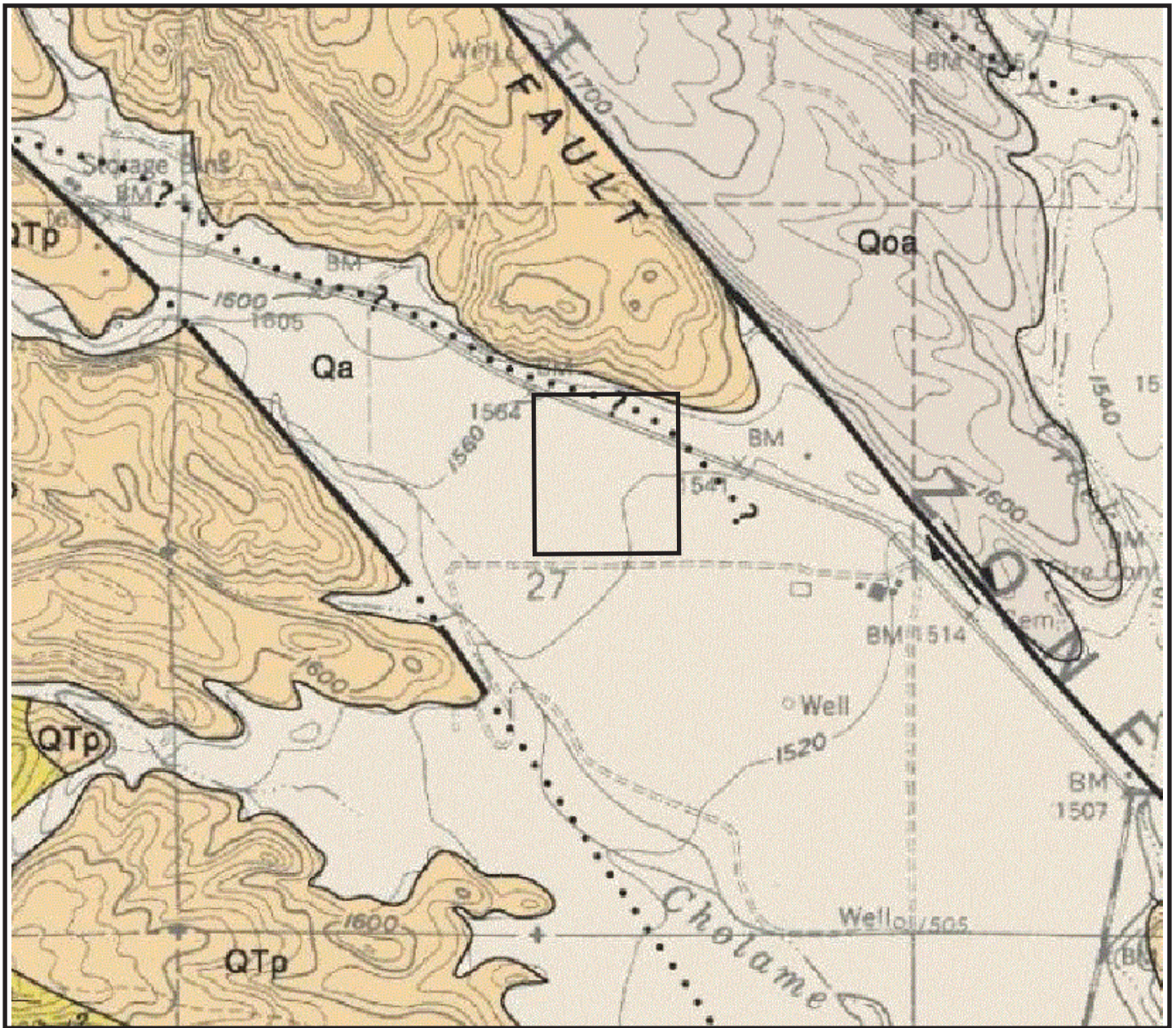
Figure:
1

Explanation

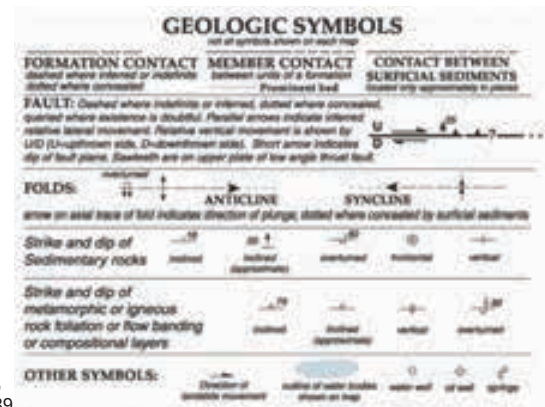
 Approximate Site Location

Base Map: Modified from USGS, 2015, Topographic Map of the Parkfield
7.5' Quadrangle, 1:24,000, downloaded from
www.NationalMap.gov.





- Qa Surficial Sediments (Quaternary)
- Qtp Paso Robles Formation (Pleistocene)



Base Map: Modified from Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the Parkfield quadrangle, Fresno and Monterey Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-139 scale 1:24,000 downloaded from www.NationalMap.gov.

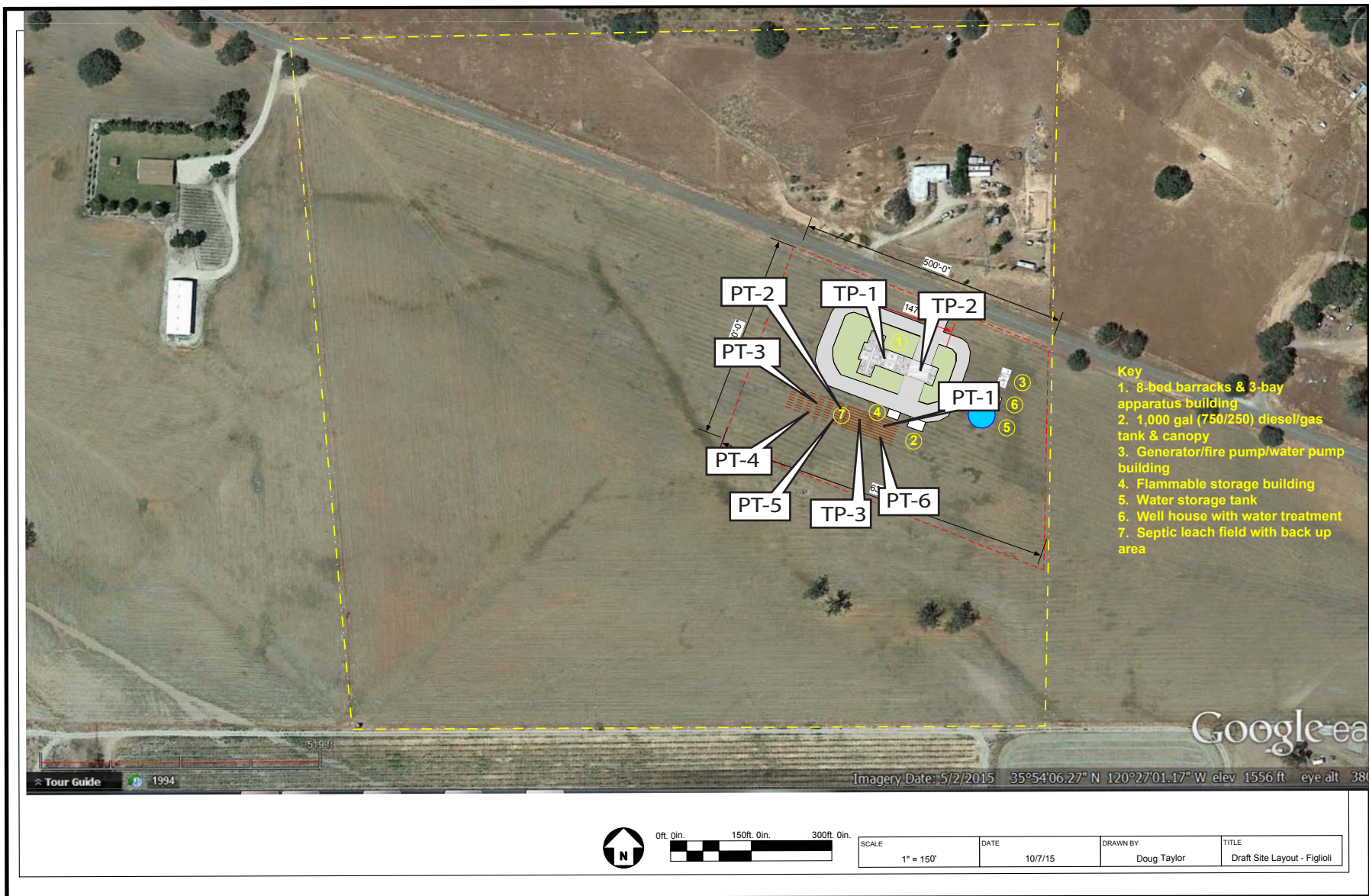
Date: 7/10/17

Scale: 1" = 1,123'

Approved By:
C. Grysza, CGS

Regional Geologic Map
To Accompany
Engineering Geologic Review of the
Proposed Parkfield FS

Figure:
2



Explanation

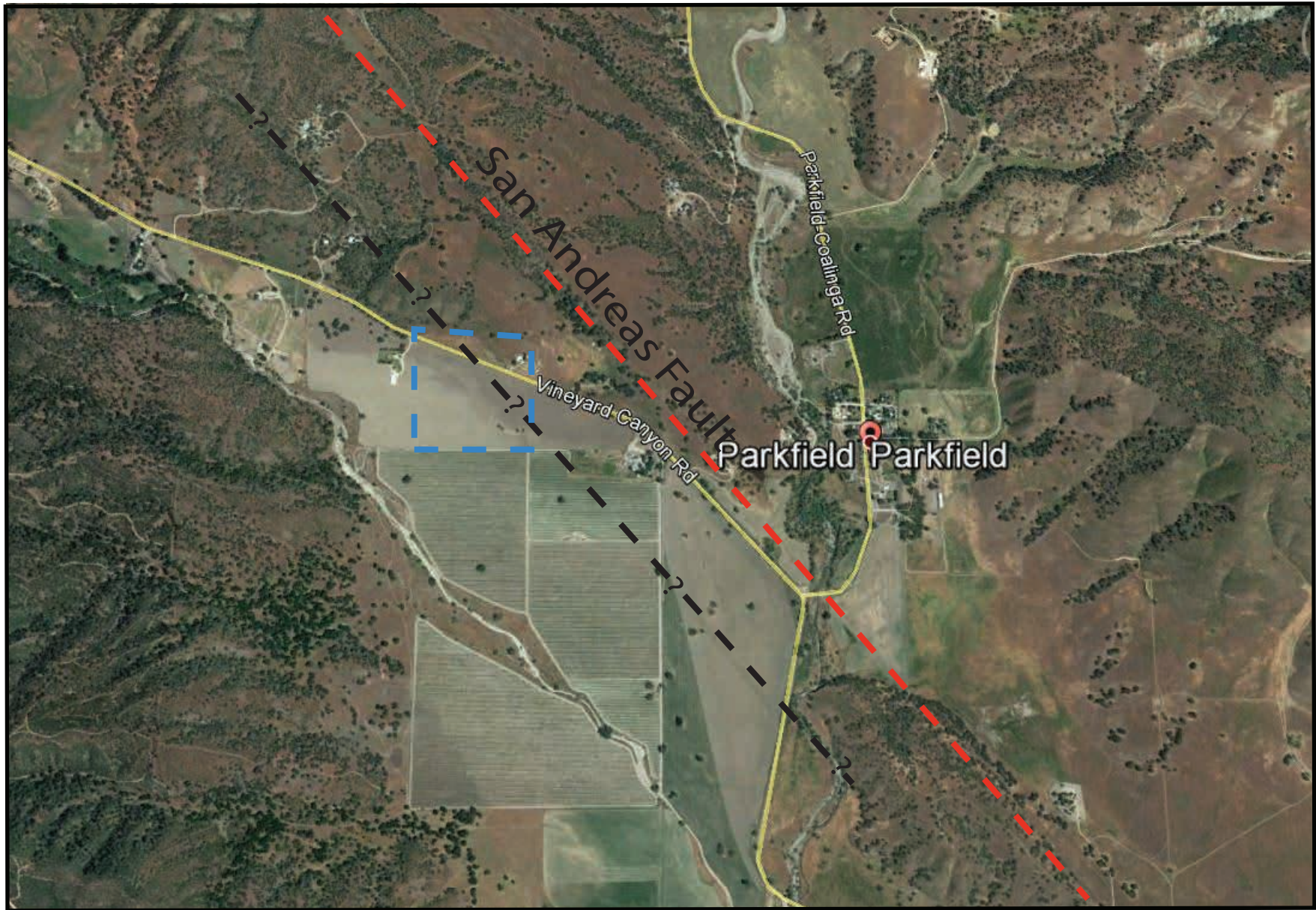
Date: 7/10/2017	Test Pit and Percolation Test Location Map To Accompany Engineering Geologic Review of Proposed Parkfield FS	Figure: 3
Scale: 1" = 255'		
Approved By: Chris Grysza, CGS		



Approximate Percolation Test Location



Approximate Test Pit Location



Base map modified from: Google Earth, 2016

Explanation

Date: 7/10/2017

Scale: 1" = 1,460'

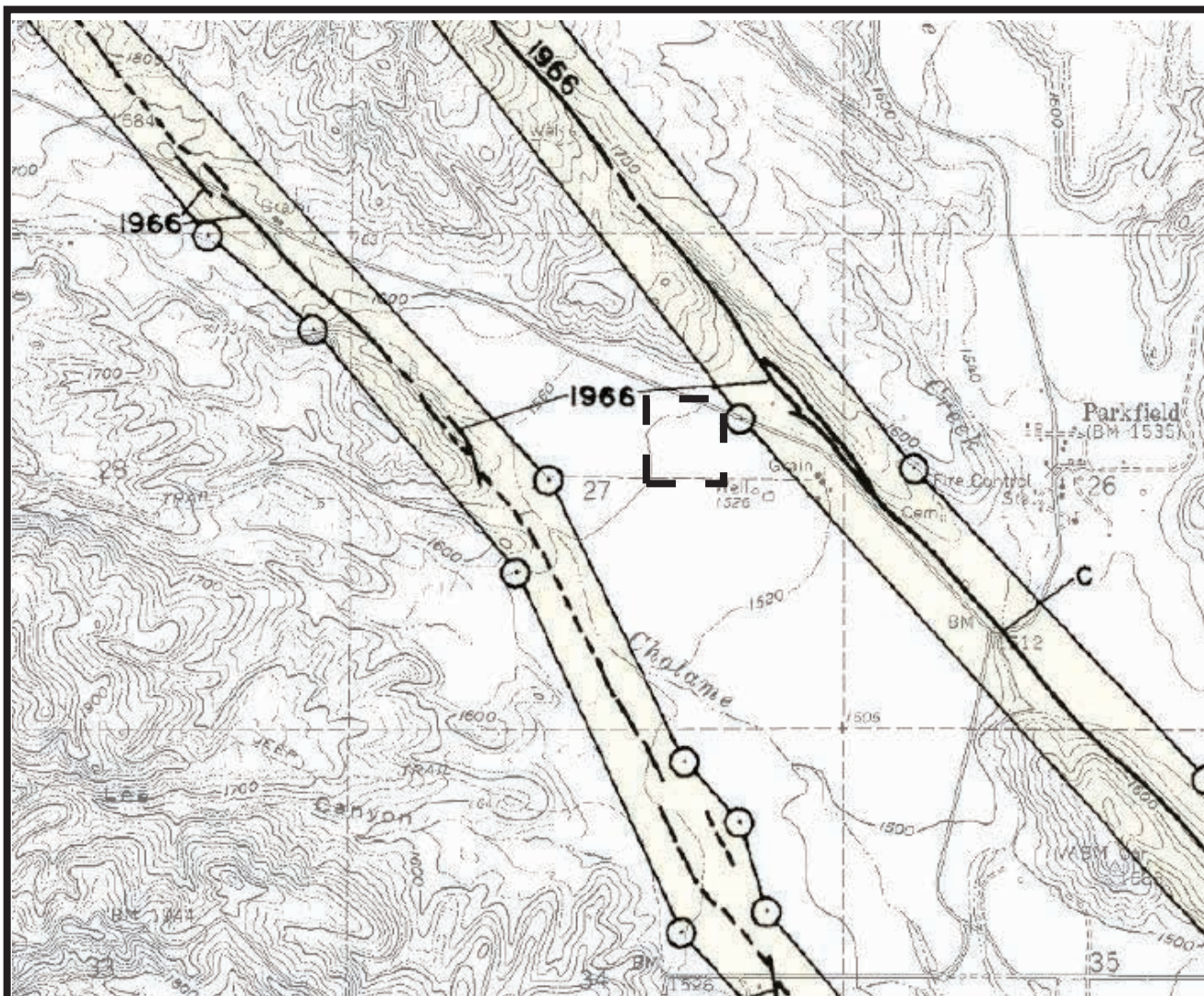
Approved By:
Chris Grysza, CGS

Possible Fault Trace Location Map
To Accompany
Engineering Geologic Review of
Proposed Parkfield FS

Figure:

4

— — Possible fault trace □ □ Approximate Site Location



MAP EXPLANATION

Potentially Active Faults



Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Special Studies Zone Boundaries



These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.



Seaward projection of zone boundary.

STATE OF CALIFORNIA SPECIAL STUDIES ZONES

Delineated in compliance with
Chapter 35, Division 2 of the California Public Resources Code
(Alquist-Priolo Special Studies Zones Act)

PARKFIELD QUADRANGLE

REVISED OFFICIAL MAP

Effective: July 1, 1986

State Geologist

Explanation



Approximate Site Location

Date: 7/10/17

Scale: 1" = 1,885'

Approved By:
C. Gryszan, CGS

AP Zone Map
To Accompany
Engineering Geologic Review of
Proposed Parkfield FS

Figure:

5

Appendix A

Table 2 – Summary of Test Pit Logs			
Test Pit No.	Depth below ground surface	USCS Classification ¹	Soils Description
TP-1	0-to 1-foot	SC	<u>TOPSOIL:</u> Clayey Sand (SC): fine-to coarse-grained, hard, dry, very dark brown (10YR/2/2).
	1 to 3 feet	SM	<u>ALLUVIUM (Qa):</u> Silty Sand (SM) trace clay: fine-to coarse-grained, medium stiff, dry to slightly moist, dark brown (7.5YR/3/2).
	3 to 8 feet	SM-ML	Silty Sand to Sandy Silt (SM-ML): fine-to coarse-grained, loose to medium stiff, light brown (7.5YR/6/3).
-End of test pit at 8 feet bgs. -No groundwater encountered. -Test pit backfilled loose with spoils and surfaced rolled on June 20, 2017.			
TP-2	0-to 1-foot	SC	<u>TOPSOIL:</u> Clayey Sand (SC): fine-to coarse-grained, hard, dry, very dark brown (10YR/2/2).
	1 to 2 feet	SM	<u>ALLUVIUM (Qa):</u> Silty Sand (SM) trace clay, fine-to coarse-grained, medium stiff, dry to slightly moist, dark brown (7.5YR/3/2).
	3 to 8 feet	SM-ML	Silty Sand to Sandy Silt (SM-ML): fine-to coarse-grained, loose to medium stiff, light brown (7.5YR/6/3).
-End of test pit at 8 feet bgs. -No groundwater encountered. -Test pit backfilled loose with spoils and surfaced rolled on June 20, 2017.			
¹ See ASTM D2488-09 for description (ASTM)			

Summary of Test Pit Logs			
Test Pit No.	Depth below ground surface	USCS Classification ¹	Soils Description
TP-3	0-to 1-foot	SC	<u>TOPSOIL:</u> Clayey Sand (SC): fine-to coarse-grained, hard, dry, very dark brown (10YR/2/2).
	1 to 8 feet	SM-ML	<u>ALLUVIUM (Qa):</u> Silty Sand to Sandy Silt (SM-ML): fine-to coarse-grained, loose to medium stiff, light brown (7.5YR/6/3).
-End of test pit at 8 feet bgs. -No groundwater encountered. -Test pit backfilled loose with spoils and surfaced rolled on June 21, 2017.			
¹ See ASTM D2488-09 for description (ASTM)			

Appendix B

A total of six (6) percolation tests (PT-1 through PT-6) were conducted within the proposed leach field area on June 21, 2017. Incremental soil percolation rates, as a function of time, were calculated by measuring the vertical head decrease within the percolation holes over time intervals and calculating the volume needed to restore the water level to a set reference point at the top of each percolation hole. The test results in min/inch are presented in the following table. Pertinent field conditions and observations made while conducting the percolation tests were recorded in the field and are summarized on the attached field sheets. A typical percolation test hole is shown in Photograph No.1.

Table No. 3, Percolation Test Results

Percolation Test Number	Percolation Rate min/inch
PT-1	6.0
PT-2	15.0*
PT-3	4.6
PT-4	6.0
PT-5	6.6
PT-6	5.0

*Significant caving was observed during testing.



Photograph 1: Percolation Test (PT-1).

Percolation Hole PT-1

Reading	Minutes	Start Time	End Time	Duration of Test Minutes	Starting Water Elev. Inches	Ending Water Elev. Inches	Change in Elev. Inches
1	30	7:20	7:50	30	0.00	9.500	9.500
2	60	7:52	8:22	30	0.00	8.000	8.000
3	90	8:22	8:52	30	0.00	7.000	7.000
4	120	8:52	9:22	30	0.00	6.000	6.000
5	150	9:22	9:51	29	0.00	5.000	5.000
6	180	9:51	10:21	30	0.00	5.000	5.000
7	210	10:21	10:51	30	0.00	5.000	5.000
8	240	10:51	11:21	30	0.00	5.000	5.000

Performed By: Chris Gyszan

Date: June 21, 2017

Performed after a minimum 24 hour pre-soak

Perc holes were 6" diameter.

Perc holes were 14 inches deep with 2" of pea gravel placed at the bottom of the hole

Perc holes were excavated from 3-4.1 feet bgs

*The drop that occurs during the final 30 minutes is used to determine the percolation rate.

Percolation Rate : 6 min/inch*

Percolation Hole PT-2

[illegible]

Performed By: Chris Gryszan

Percolation Rate : 15 min/inch

Date: June 21, 2017

Significant caving of the test hole was observed during testing and therefore the test results may not be representative of the site conditions.

Percolation Hole PT-3

[illegible]

Performed By: Chris Gryszan
Date: June 21, 2017

Percolation Rate : 4.6 min/inch

Percolation Hole PT-4

[illegible]

Performed By: Chris Gryszan
Date: June 21, 2017

Percolation Rate : 6 min/inch

Percolation Hole PT-5

[illegible]

Performed By: Chris Gryszan
Date: June 21, 2017

Percolation Rate : 6.6 min/inch

Percolation Hole PT-6

[illegible]

Performed By: Chris Gryszan
Date: June 21, 2017

Percolation Rate : 5 min/inch