



Project No. S9962-05-23B
September 28, 2022

Ashok Jain
Department of Forestry and Fire Protection (CAL FIRE)
Technical Services Section
1132 S Street
Sacramento, California 95811

Subject: GEOTECHNICAL REPORT UPDATE – 2022
PROPOSED PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, MONTEREY COUNTY, CALIFORNIA

Reference: *Geotechnical Report Update, Proposed Parkfield Fire Station, 70331 Vineyard Canyon Road, Parkfield, California*, prepared by Geocon Consultants, Inc. (Project No. S9962-05-23B), June 12, 2020.

Mr. Jain:

In accordance with the request of Gregory Vincent with CalFIRE, we have prepared this geotechnical report update for the proposed Parkfield Fire Station located in Parkfield in Monterey County, California.

Geocon previously prepared the referenced geotechnical report update for the project (Geocon 2020). Our update report cited and provided geotechnical and seismic design recommendations per the 2019 California Building Code (CBC), the governing building code in effect at the time. We understand that the project will be designed per the 2022 CBC which will be in effect January 1, 2023.

Both the 2019 and 2022 CBC incorporate the American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) publication: *ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16); therefore, the site-specific ground motion hazard analysis (GMHA) and seismic parameters presented in our 2020 update remain applicable and are in general accordance with the 2022 CBC. However, if code-based (mapped parameters) are used in lieu of the GMHA site-specific parameters presented in our 2020 update, revised parameters are necessary in compliance with Supplement 3 of ASCE 7-16 issued November 5, 2021.

Table 1 summarizes the revised mapped acceleration parameters per Section 1613A of the 2022 CBC. The data was calculated using the *Structural Engineers Association of California* (SEAOC) and *Office of Statewide Health Planning and Development* (OSHPD) web application *Seismic Design Maps* (<https://seismicmaps.org/>).

TABLE 1
ASCE 7-16 (CODE-BASED) SEISMIC DESIGN PARAMETERS
SITE CLASS "D" – STIFF SOIL

Parameter	Value	ASCE 7-16 Reference
MCE _R Ground Motion Spectral Response Acceleration – Class B (short), S _S	1.913g	Figure 22-1
MCE _R Ground Motion Spectral Response Acceleration – Class B (1 sec), S ₁	0.797g	Figure 22-2
Site Coefficient, F _A	1.0	Table 11.4-1
Site Coefficient, F _V	1.7	Table 11.4-2
Site Class Modified MCE _R Spectral Response Acceleration (short), S _{MS}	1.913g	Eq. 11.4-1
Site Class Modified MCE _R Spectral Response Acceleration (1 sec), S _{M1}	2.03g*	Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S _{DS}	1.275g	Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S _{D1}	1.354g*	Eq. 11.4-4
* Per Supplement 3 of ASCE7-16 (effective November 5, 2021), the parameter S _{M1} is increased by 50% for all applications of S _{M1} . The values for parameters S _{M1} and S _{D1} presented above have been increased in accordance with Supplement 3 of ASCE 7-16.		

The remaining conclusions, recommendations, and design parameters presented in our 2020 geotechnical report update remain valid as presented. For convenience, a copy of our 2020 geotechnical report update is attached.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in this area at this time. We make no warranty, express or implied.

Please contact us if you have any questions concerning the contents of this report or if we may be of further service.

Respectfully Submitted,

GEOCON CONSULTANTS, INC.


Jeremy J. Zorne, PE, GE
Senior Engineer



Attachment: Geotechnical Report Update (Geocon 2020)



Project No. S9962-05-23B
June 12, 2020

Ashok Jain
Department of Forestry and Fire Protection (CAL FIRE)
Technical Services Section
1131 S Street
Sacramento, California 95811

Subject: GEOTECHNICAL REPORT UPDATE
PROPOSED PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, MONTEREY COUNTY, CALIFORNIA

Reference: *Geotechnical Investigation and Geologic Hazards Evaluation, Proposed Parkfield Fire Station, 70331 Vineyard Canyon Road, Parkfield, California*, prepared by Geocon Consultants, Inc. (Project No. S9962-05-23), July 27, 2018.

Mr. Jain:

In accordance with the request of Gregory Vincent with CalFIRE, we have prepared this geotechnical report update for the proposed Parkfield Fire Station located in Parkfield, San Luis Obispo County, California.

Geocon prepared the referenced geotechnical design report for the project (Geocon 2018). Our report cited and provided geotechnical and seismic design recommendations per the 2016 California Building Code (CBC), the governing building code in effect at the time. We understand that the project will be designed per the 2019 CBC which went into effect January 1, 2020. The 2019 CBC incorporates the American Society of Civil Engineers (ASCE)/Structural Engineering Institute (SEI) publication: *ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16). The 2019 CBC includes updates/changes to developing seismic design parameters and designing shallow foundation systems on sites with seismic-induced liquefaction potential. An update to our geotechnical report is required to incorporate the changes required by code. The following recommendations and design parameters are intended to supplement and/or supersede the recommendations contained in our 2018 report.

Site-Specific Ground Motion Hazard Analysis

For this update, we performed a site-specific ground motion hazard analyses in accordance with the ASCE 7-16 Chapter 21 and Section 1613A of the 2019 CBC using the online applications developed by USGS.

Probabilistic Seismic Hazard Analysis

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic 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 mean spectral response accelerations having a 2 percent chance of exceedance in 50 years were evaluated at 5 percent damping using the USGS Unified Hazard Tool (UHT). The Dynamic U.S. 2014 (v4.2.0) edition was used within the analysis, which is based on the UCERF-3 fault model. The soil underlying the site was modeled as Site Class “D” with a corresponding average shear wave velocity (V_{s30}) of 270 meters per second. Although there are potentially 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 web application uses the ground motion prediction equations (GMPEs) from the NGA-West 2 project: Abrahamson-et al. (2014) NGA West 2, Boore et al. (2014) NGA West 2, Campbell-Bozorgnia (2014) NGA West 2, and Chiou-Youngs (2014) NGA West 2. Each GMPE was assigned an equal weight and the mean value of the four GMPEs was evaluated. The mean spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

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 GMPEs of Abrahamson-et al., Boore et al. and Chiou-Youngs require 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 $Z_{1.0}$ are internally calculated by the Uniform Hazard Tool.

The MCE uniform hazard response spectra was adjusted to risk-targeted spectral accelerations corresponding to a 1 percent chance of collapse in 50 years by using the USGS Risk-Targeted Ground Motion Calculator and following ASCE 7-16 Section 21.2.1.2 Method 2.

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum is provided on Figure 1.

Deterministic Seismic Hazard Analysis

In order to define the deterministic scenario events, deaggregation of the uniform hazard probabilistic response spectrum was performed using the USGS *Uniform Hazard Tool*. The inversion approach used by UCERF-3 allows for a large number of variations for each source scenario, including multi-fault ruptures. Therefore, deaggregation of UCERF-3 consists of the contributions from multi-fault ruptures rather than individual source contributions. To address this, the USGS *Unified Hazard Tool* aggregates the contributions on a per-fault-section basis, with rupture contributions only ever counted once. The *Unified Hazard Tool* deaggregation contributor list shows the fault sections which contribute most to hazard at a site and report a mean earthquake magnitude for each section identified by a 'parent' fault name and section index. Based on the deaggregation, we have considered scenario events with the greatest contribution to the deterministic ground motions.

The earthquake magnitude of the deterministic scenario event was based on the BSSC 2014 Scenario Event which includes the parent fault identified in the deaggregation and which has the largest earthquake magnitude. The closest distance from the site to the surface projection of the Southern San Andreas fault (Rx) was taken as 780 feet (see discussion in Section 7.1 of our 2018 geotechnical report). Other fault source parameters were defined by the values in the BSSC2014 Scenario Catalog. The values of $Z_{2.5}$ and $Z_{1.0}$ were estimated using data from the PEER NGA West-2 flat file for CGS Strong Motion Stations 36138 and 36531, which are located approximately 1.2 km and 1.3 km to the east.

The input values used to evaluate the deterministic scenario event are provided in the following table.

Parameter	Scenario 1	Reference
Parent Fault Name	Southern San Andreas	---
Scenario Name	S. San Andreas: PK+CH+CC+BB+ NM+SM+NSB+SSB+BG+CO	BSSC Online Scenario Catalog
Earthquake Magnitude	8.18	BSSC Online Scenario Catalog
Fault Mechanism	Strike Slip	
Fault Dip	180	BSSC 2014 ¹
Fault Width	13.1 km	BSSC 2014 ¹
Rake	180	BSSC 2014 ¹
Z_{TOR}	0.333 km	BSSC 2014 ¹
R _x	0.24 km	See Discussion in Section 7.1 of the 2018 Geotechnical Report
R _{rup}	0.42 km	Derived from R _x and Fault Type
R _{jb}	0.24 km	Derived from R _x and Fault Type
V _{s30}	260 m/s	Average Site Class D Value
$Z_{1.0}$	0.08 km	PEER NGA West-2 flat file
$Z_{2.5}$	7.34 km	PEER NGA West-2 flat file

1- BSSC 2014, aka. UCERF3_EventSet_All on GitHub

The deterministic median and standard deviation (sigma) for the scenario event was evaluated using the USGS NSHMP-HAZ-WS Response Spectra online application. The deterministic analysis used the same four GMPEs, equally weighted, to generate the median and standard deviation of the ground motion which were then used to calculate the 84th percentile at 5% damping. The geometric median spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014). The 84th percentile maximum rotated component deterministic response spectra is provided on Figure 2.

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 modified General Design Response Spectrum determined by ASCE 7-16 Section 11.4.6 with F_a and F_v determined as specified in Section 21.3.

Graphical representations of the analyses are presented on Figures 1 and 2. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on Figure 2 and in tabular form on Figure 3.

Code-Based (Mapped) Acceleration Parameters

The following table summarizes the mapped acceleration parameters per the Section 1613A of the 2019 CBC. The data was calculated using the *Structural Engineers Association of California* (SEAOC) and *Office of Statewide Health Planning and Development* (OSHPD) web application *Seismic Design Maps* (<https://seismicmaps.org/>).

ASCE 7-16 SEISMIC DESIGN PARAMETERS

Parameter	Value	ASCE 7-16 Reference
Site Class	D	Table 20.3-1
MCE _R Ground Motion Spectral Response Acceleration – Class B (short), S _S	1.913g	Figure 22-1
MCE _R Ground Motion Spectral Response Acceleration – Class B (1 sec), S ₁	0.797g	Figure 22-2
Site Coefficient, F _A	1.0	Table 11.4-1
Site Coefficient, F _V	1.7*	Table 11.4-2
Site Class Modified MCE _R Spectral Response Acceleration (short), S _{MS}	1.913g	Eq. 11.4-1
Site Class Modified MCE _R Spectral Response Acceleration (1 sec), S _{M1}	1.354g*	Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S _{DS}	1.275g	Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S _{D1}	0.903g*	Eq. 11.4-4
Long Period Transition Period, T _L	12 seconds	Figs. 22-14 through 22-17
T _S = S _{D1} /S _{DS}	0.71 seconds	Chapter 11
* Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis (GMHA) shall be performed for projects on Site Class “D” sites with long-period spectral acceleration (S₁) greater than or equal to 0.2g, which is true for this site. However, Section 11.4.8 also provides exceptions which indicate that the ground motion hazard analysis may be waived, provided the exceptions are followed. Using the code-based values presented in the table above, in lieu of a performing a ground motion hazard analysis requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed. Specifically for this site/project, Exception No. 2 would apply which states that a GMHA is not required for: <i>Structures on Site Class D sites with S₁ greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of T ≤ 1.5T_s and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for T_L ≥ T > 1.5T_s or Eq. (12.8-4) for T > T_L.</i>		

Site-Specific Seismic Design Criteria

Based the site-specific ground motion hazard analysis performed, and in accordance with the ASCE 7-16 Section 21.4, site-specific design acceleration parameters shall be derived using the results of the site-specific ground motion hazard analysis.

The parameter S_{DS} shall be taken as equal to 90 percent of the maximum spectral acceleration obtained from the site-specific analysis at any period within the range from 0.2 to 5 seconds, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product of the spectral acceleration and period for periods from 1 to 5 seconds, inclusive. 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 design acceleration parameters shall not be less than 80 percent of the general seismic design values determined by ASCE 7-16 Section 11.4.

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

SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value
Site Class Modified MCE _R Spectral Response Acceleration (short), S _{MS}	2.818g
Site Class Modified MCE _R Spectral Response Acceleration – (1 sec), S _{M1}	2.856g
5% Damped Design Spectral Response Acceleration (short), S _{DS}	1.879g
5% Damped Design Spectral Response Acceleration (1 sec), S _{D1}	1.904g

Site-Specific Peak Ground Acceleration

The site-specific Maximum Considered Earthquake (MCE_G) geometric mean peak ground acceleration (PGA_M) was evaluated in accordance with ASCE 7-16 Section 21.5. The PGA_M is used for liquefaction analysis.

The probabilistic geometric mean peak ground acceleration and the deterministic 84th percentile geometric mean peak ground acceleration were analyzed using the same approaches as described above. The analysis used the same Site Class and scenario earthquake.

The deterministic MCE_G shall not be less than 0.5F_{PGA}, where F_{PGA} is determined from ASCE 7-16 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 7-16 Equation 11.8.1.

ASCE 7-16 SITE-SPECIFIC PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site-Specific MCE _G Peak Ground Acceleration, PGA _M	0.927g	Section 21.5

Liquefaction

With the change in site-specific PGA_M, we reevaluated liquefaction potential at the site. Previously, we estimated up to 1 inch of total liquefaction settlement under the MCE ground motion (0.853g) and 0.7 inches under the Design Earthquake (DE) ground motion (0.569g or 2/3PGA_M).

Section 12.13.9 of ASCE 7-16 provides requirements for foundations of structures that are located on sites that have been determined to have the potential to liquefy when subjected to the PGA_M ground motion. This section specifies that the MCE ground motion be used for liquefaction analysis rather than the DE ground motion. For Risk Category IV Essential Facilities, the provisions seek to limit damage attributable to liquefaction to levels that would permit post-earthquake use. The limits for Risk Category IV are intended to maintain seismic differential settlements less than the distortion that will cause doors to jam due to the MCE-level earthquake.

Based on our re-analysis of the conditions encountered in Boring B1, under the updated MCE-level PGA_M of 0.927g, we estimate total seismic settlement (liquefaction and unsaturated seismic settlement) of up to 1.4 inches. Estimated differential liquefaction settlement would be approximately one-half of the total seismic settlement and is assumed, conservatively, to occur over a distance of 30 feet (typical column spacing in most buildings). This estimated value of seismic differential settlement is

approximately 0.7 inches over a distance of 30 feet (or $0.0019L$). Per Section 12.13.9 of ASCE 7-16, this magnitude of differential settlement is less than the maximum allowable threshold for Risk Category IV buildings; therefore, shallow foundations may be used. However, special detailing per ACSE 7-16 Section 12.13.9.2.1 is required. Detailing includes interconnecting individual spread footings with foundation tie beams and connecting individual spread footings to a reinforced concrete slab-on-grade at least 5 inches thick with a minimum reinforcing ratio of 0.0025. Alternatively, a post-tensioned (PT) concrete slab foundation may be used.

Shallow Foundations with Tie-Beams / Interior Slab-on-Grade

Provided the building pad is graded in accordance with the recommendations of Section 8.4 of our 2018 report, the proposed building may be supported on shallow foundations consisting of interconnected continuous footings or individual spread footings interconnected with tie beams detailed in accordance with ASCE 7-16 Section 12.13.9.2.1 bearing on undisturbed native soil or engineered fill.

Continuous footings should be at least 12 inches wide and spread footings should be at least 18 inches square. All footings should be embedded at least 24 inches below pad grade. Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom of the footing.

Continuous footings should be reinforced with at least four No. 4 reinforcement bars, two each placed near the top and bottom of the footing to allow footings to span isolated soil irregularities. The reinforcement recommended here is for soil characteristics only and is not intended to replace reinforcement required for structural considerations. The project structural engineer should evaluate the need for additional reinforcement.

Shallow foundations may be designed for an allowable bearing capacity of 2,000 pounds per square foot (psf) for dead plus live load conditions with a one-third increase for short-term transient loading such as wind and seismic. 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.

Allowable passive pressure used to resist lateral movement of the footings may be assumed to be equal to a fluid weighing 300 pounds per cubic foot (pcf). The coefficient of friction to resist sliding is 0.35 for concrete against soil. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

Foundations designed in accordance with the recommendations above should experience total post-construction static settlement due to building loads of less than one inch and differential settlement of ½ inch or less over a distance of 30 feet. The majority of settlement will be immediate and occur as the building is constructed.

A representative of the Geotechnical Engineer should observe foundation excavations prior to placing reinforcing steel or concrete to observe that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

Per ACSE 7-16 Section 12.13.9.2.1, the interior slab-on-grade used in conjunction with spread footings and tie beams should be at least 5 inches thick with a minimum reinforcing ratio of 0.0025. Control joints should be provided at periodic intervals in accordance with American Concrete Institute (ACI) or Portland Cement Association (PCA) recommendations, as appropriate.

The top 12 inches of the building pad should consist of low-expansive fill (LEF). LEF should be primarily granular with a “low” expansion potential (Expansion Index less than 50), a Plasticity Index less than 15, be free of organic material and construction debris, not contain rock larger than 6 inches in greatest dimension, and contain sufficient fines to act as a binder to reduce caving potential when excavated. LEF may also consist of lime-treated native clay soils provided the treated materials meet the expansion criteria above. Based on our experience with similar soil, a lime application rate of approximately 5% by dry weight would likely be required to stabilize the soil. If lime treatment is selected, additional laboratory testing will be required to determine the appropriate lime spread rate.

If building pad soils become dry, they should be re-moistened prior to concrete slab-on-grade construction. Building pads should be moistened to at least optimum moisture content, at least 48 hours before placing the vapor barrier. Moisture content should be verified by a representative of the Geotechnical Engineer prior to placing the vapor barrier.

PT Slab Foundation

PT slab foundations should be designed by a structural engineer experienced in PT slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or WRI/CRSI *Design of Slab-on-Ground Foundations*, Third Edition. Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to liquefaction settlement. The post-tensioned design should incorporate the geotechnical parameters presented in the following table:

POST-TENSIONED SLAB DESIGN PARAMETERS

Design Parameter	Recommended Value
1. Thornthwaite Index	-20
2. Equilibrium Suction	3.9 pF
3. Edge Lift Moisture Variation Distance, e_M	4.3 feet
4. Edge Lift, y_M	1.1 inches
5. Center Lift Moisture Variation Distance, e_M	9.0 feet
6. Center Lift, y_M	0.5 inch
7. Total Liquefaction Settlement	1.4 inches
8. Liquefaction-induced Differential Settlement	0.7 inches over 30 feet (0.0019L)

If a uniform-thickness PT mat foundation system is planned, the slab should include thickened edges with a minimum width of 12 inches that extend below the clean sand or crushed rock layer.

Allowable bearing capacity for PT slabs should not exceed 2,000 psf for dead plus live load conditions. This value may be increased by one-third to evaluate all transient loads, including wind or seismic forces. The structural engineer should determine slab thickness and reinforcing based on anticipated use and loading of the slab.

The allowable coefficient of friction to resist sliding is 0.35 for concrete against crushed aggregate. Since PT slab foundations are typically not embedded into the building pad, resistance to sliding from passive soil resistance does not apply. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended. Where this condition cannot be avoided, the isolated footings should be embedded at least 24 inches below pad grade and be connected to the building foundation with tie beams.

Prior to placing the vapor retarder, pad subgrade soil should be moisture-conditioned at or above optimum moisture content to a depth of at least 12 inches. Geocon should confirm the moisture content of the subgrade soils at least 24 hours prior to placing the moisture retarder.

Our experience indicates PT slab foundations are potentially susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings/thickened edges and the interior stiffener beams may reduce this potential. Current PTI design procedures primarily address the potential center lift of slabs but, because of the placement of the reinforcing tendons near the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to reduce edge lift.

During the construction of the PT foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints be allowed to form.

REFERENCES

1. Abrahamson, N.A, Silva, W.J, and Kamai, R., 2014, *Summary of the ASK14 Ground Motion Relation for Active Crustal Regions*, Earthquake Spectra, Volume 30, No. 3, pages 1025-1055, August 2014.
2. Boore, D.M., Stewart, J.P., Seyhan, E., and Atkinson, G.M., 2014, *NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes*, Earthquake Spectra, Volume 30, No. 3, pages 1057-1085, August 2014.
3. Campbell, K.W. and Bozorgnia, Y., 2014, *NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra*, Earthquake Spectra, Volume 30, No. 3, pages 1087-1115, August 2014.
4. Chiou, B. S.-J., and Youngs, R.R., 2014, *Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, Earthquake Spectra, Volume 30, No. 3, pages 1117-1153, August 2014.
5. OSHPD Seismic Design Maps Web Application, <https://seismicmaps.org/>, accessed June 2020.
6. Shahi, S.K., Baker, J.W., 2013, *NGA-West2 Models for Ground-Motion Directionality*, Pacific Earthquake Engineering Research Center, PEER 2013/10.
7. Shahi, S.K., Baker, J.W., 2014, *NGA-West2 Models for Ground-Motion Directionality*, Earthquake Spectra, Volume 30, No. 3, pages 1285-1300, August 2014.
8. USGS, BSSC2014 (Scenario Catalog), <https://earthquake.usgs.gov/scenarios/catalog/bssc2014/>, accessed June 2020.
9. USGS, NSHMP_HAZ Response Spectral Application, <https://earthquake.usgs.gov/nshmp-haz-aws/apps/spectra-plot.html/>, accessed June 2020.
10. USGS, Unified Hazard Tool, <https://earthquake.usgs.gov/hazards/interactive/>, accessed June 2020.

CLOSURE

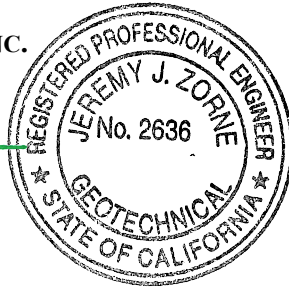
The remaining conclusions, recommendations, and design parameters presented in our 2018 report (Reference 1) remain valid as presented. Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in this area at this time. We make no warranty, express or implied.

Please contact us if you have any questions concerning the contents of this report or if we may be of further service.

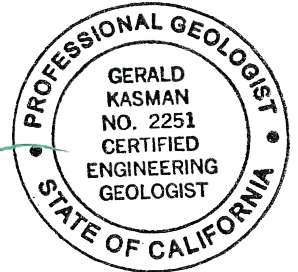
Respectfully Submitted,

GEOCON CONSULTANTS, INC.

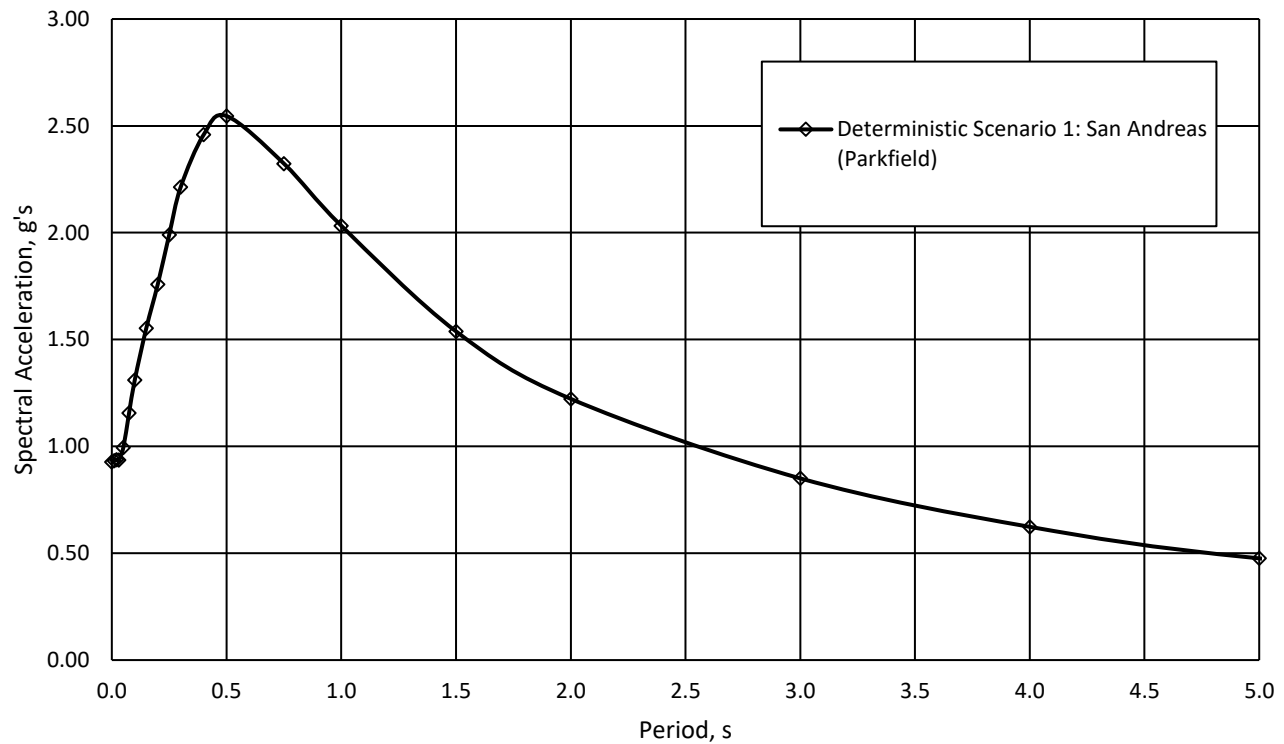
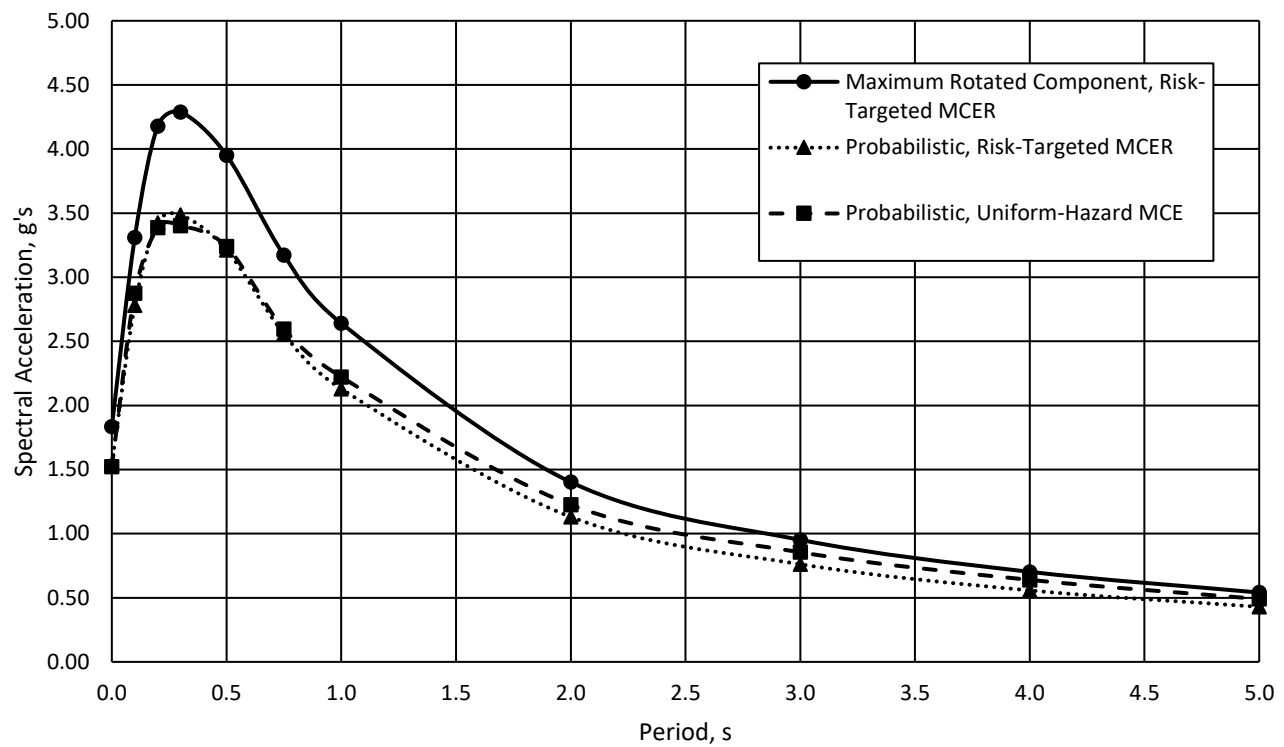

Jeremy J. Zorne, PE, GE
Senior Engineer




Gerald Kasman, CEG
Senior Geologist



Attachments: Figures 1 and 2, Design Response Spectrum (Graphical)
Figure 3, Design Response Spectrum (Tabular)



DESIGN RESPONSE SPECTRUM

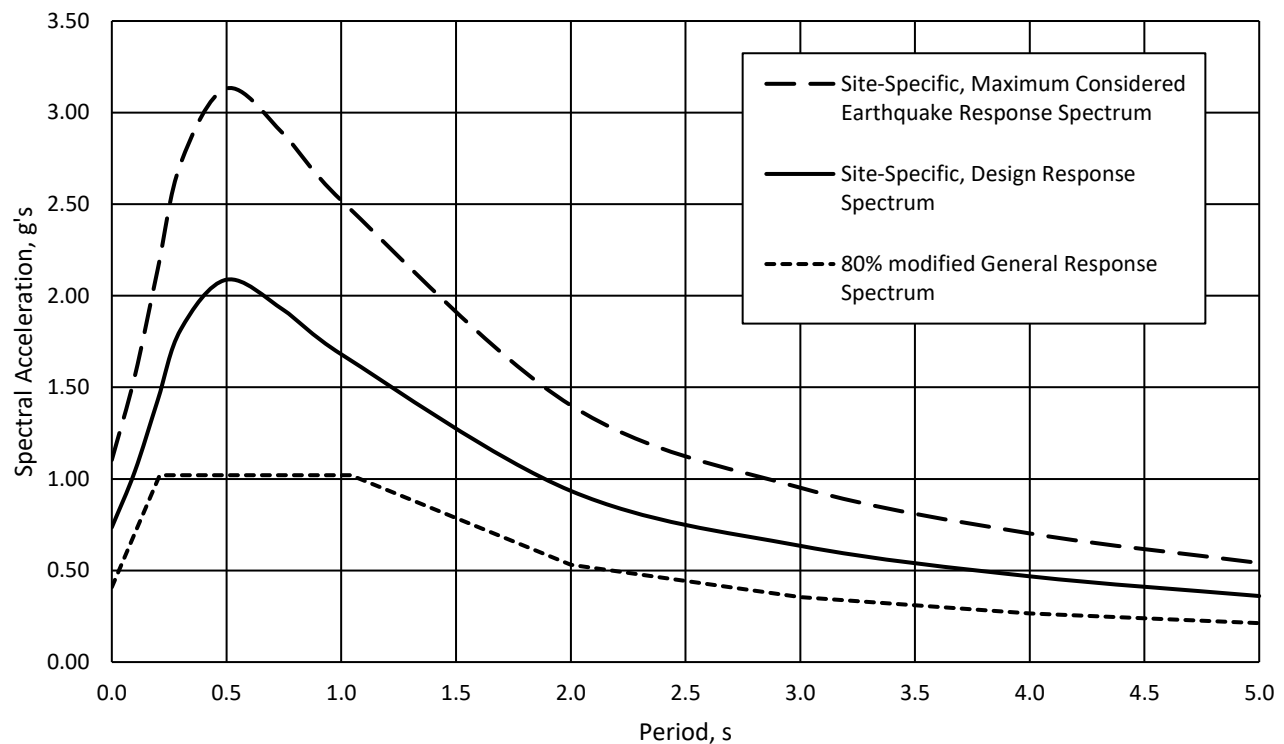
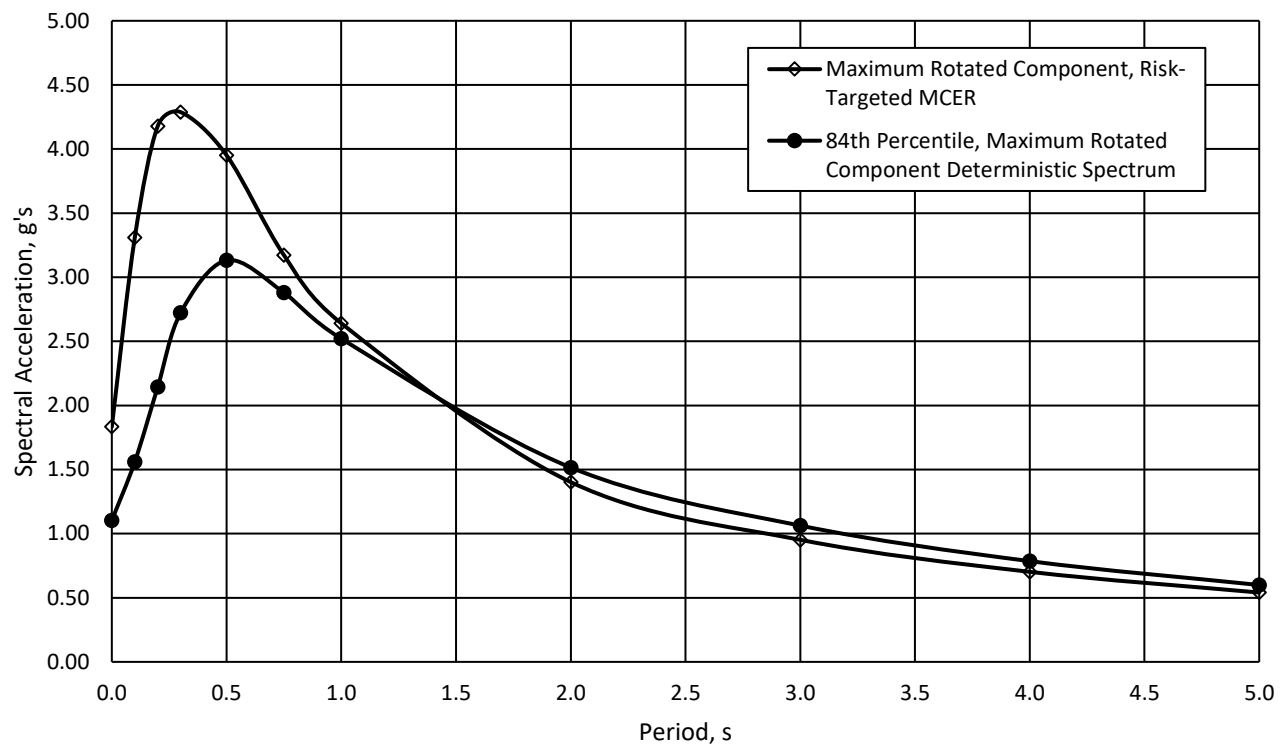
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PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

Jun 20

Figure 1



DESIGN RESPONSE SPECTRUM

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PARKFIELD FIRE STATION
70331 VINEYARD CANYON ROAD
PARKFIELD, CALIFORNIA

Jun 20

Figure 2

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Component Scale Factor	MRC, Risk-Targeted Probabilistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	Design Earthquake Floor	Site-Specific Maximum Considered Earthquake
0	1.521	1.541	1.013	1.190	1.834	1.103	0.736	0.408	1.103
0.1	2.874	2.780	0.967	1.190	3.309	1.560	1.040	0.702	1.560
0.2	3.386	3.425	1.011	1.220	4.178	2.144	1.429	0.996	2.144
0.3	3.402	3.486	1.025	1.230	4.288	2.722	1.815	1.020	2.722
0.5	3.239	3.213	0.992	1.230	3.952	3.132	2.088	1.020	3.132
0.75	2.597	2.558	0.985	1.240	3.172	2.881	1.921	1.020	2.881
1	2.222	2.130	0.958	1.240	2.641	2.520	1.680	1.020	2.520
2	1.225	1.131	0.923	1.240	1.402	1.516	0.935	0.531	1.402
3	0.855	0.762	0.891	1.250	0.952	1.063	0.635	0.354	0.952
4	0.640	0.557	0.871	1.260	0.702	0.786	0.468	0.266	0.702
5	0.490	0.429	0.875	1.260	0.541	0.600	0.361	0.213	0.541

$$SM_5 = \frac{2.818}{g}$$

$$SM_1 = \frac{2.856}{g}$$

$$SD_5 = \frac{1.879}{g}$$

$$SD_1 = \frac{1.904}{g}$$

Reference: ASCE 7-16 21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a , obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a , for periods from 1 to 2 s for sites with $V_{s,30} > 1,200$ ft/s ($V_{s,30} > 365.76$ m/s) and for periods from 1 to 5 s for sites with $V_{s,30} \leq 1,200$ ft/s ($V_{s,30} \leq 365.76$ m/s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1} , respectively. The values so obtained shall not be less than 80% of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1} .

	DESIGN RESPONSE SPECTRUM				Project No.:	S9962-05-23B
					PARKFIELD FIRE STATION 70331 VINEYARD CANYON ROAD PARKFIELD, CALIFORNIA	
	Checked by: JTA				Jun 20	Figure 3