FOUNDATION REPORT
(DRAFT)
SAND CREEK BRIDGE
ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA
(BRIDGE NO. 42C0697)

For

BKF ENGINEERS
4670 Willow Road, Suite 250
Pleasanton, CA 94588

PARIKH CONSULTANTS, INC.
2360 Qume Drive, Suite A
San Jose, CA 95131

November 2, 2016

Job No. 2015-115-FDN

TABLE OF CONTENTS **PAGE**

LIST OF PLATES

APPENDICES

FOUNDATION REPORT
(DRAFT)
SAND CREEK BRIDGE
ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA
(BRIDGE NO. 42C0697)

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed Sand Creek Bridge (New Bridge No. 42C0697) replacement project (Project) in Fresno County, California. The subject bridge is located on Ennis Road over Sand Creek, between Mistletoe Road to the north and Sand Creek Road to the south in the Squaw Valley area. The work was generally performed in accordance with our contract with BKF Engineers. The approximate project site is shown on the Project Location Map (Plate No. 1A) and Project Vicinity Map (Plate No. 1B).

2.0 SCOPE OF WORK

The purpose of this investigation was to evaluate the general subsurface soil/rock and groundwater conditions at the project site, to evaluate their engineering properties, and to provide foundation design recommendations for the proposed Project. The scope of work performed for this investigation included a review of the readily available geologic literature pertaining to the site, obtaining representative soil and rock samples and logging materials encountered in the exploratory borings, laboratory testing of the collected soil and rock samples, engineering analysis of the field and laboratory data, and preparation of this report.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used directly as specifications. These recommendations should not be used directly for bidding purposes.

3.0 PROJECT DESCRIPTION

The existing Sand Creek Bridge (Existing Bridge No. 42C0099) was built in 1975. It is a single-span timber structure supported at the ends by reinforced concrete abutments founded on large boulders and bedrock. The Caltrans Bridge Inspection Report (2012) has rated the bridge functionally obsolete. The County of Fresno plans to replace the existing bridge with a new bridge that will be designed to meet current Caltrans and AASHTO standards. Based on the information provided by Biggs Cardosa Associates, Inc. (Designer), two bridge options have been considered, 1) three-span cast-in-place (CIP) reinforced-concrete (RC) slab supported on substructure consisting of cast-in-drilled-hole (CIDH) concrete piles, and 2) single span cast-in-place (CIP), prestressed (PS) box girder supported on spread footings. The final selected bridge type is a single span CIP/PS box girder structure with spreading footings for the abutment foundations. The new bridge measures 101' in length, and 26'-10" in total width to carry two 10' wide traffic lanes and two 2' wide sidewalks.

4.0 EXCEPTIONS TO POLICY

Normal procedures were assumed for construction of the bridge structure throughout our analysis and represent one of the bases of recommendations presented herein. The investigation for the proposed foundations has generally followed Caltrans guidelines.

5.0 FIELD INVESTIGATION AND TESTING PROGRAM

Two borings were drilled and cored to depths of approximately 38 feet (R-15-001) and 17 feet (R-15-002) below the existing grade with a truck-mounted drill rig on May 21, 2015. Hollow stem auger drilling method was used at shallow depths in each boring, and then rotary coring method was used to complete the coring in bedrocks. Selected soil samples were obtained from either a 2.5-inch I.D. Modified California (MC) or 1.4-inch I.D. Standard Penetration Test (SPT) sampler at shallow depths. The samplers were driven into subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Log of Test Borings (LOTB) in Appendix A. The drilling subcontractor was Technicon Engineering Services, Inc. from Fresno, California. Based on the hammer energy calibration information provided, the hammer energy of the drill rig (CME 55) used is approximately 85%. Using a method suggested by Daniel, Howie and Sy (2003), when correlating standard penetration data, the blow counts for the Modified California Sampler may be converted to equivalent SPT blow counts by multiplying a conversion factor of 0.6. The bedrock cores were collected using a core barrel with a HQ diamond core drill bit (2.5-inch I.D.). The soil and rock samples were sealed and transported to our laboratory for further evaluation and testing. In addition, two bulk soil samples were collected from within the upper about 5 feet of subgrade for R-value tests for pavement design. The field investigation was conducted under the supervision of our field engineer who logged the test borings and prepared the samples for subsequent laboratory testing and evaluation. The approximate boring locations are shown on the Site Plan, Plate No. 2.

6.0 LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected samples to evaluate the physical and engineering properties of the subsoils. The tests performed for the study included the following: Moisture Content (ASTM D 2216), Grain Size (ASTM D 422), Unconfined Compressive Strength (ASTM C 42), Corrosion (California Test Methods 643, 417 and 422), and R-value (California Test Method 301). The corrosion tests were performed by Sunland Analytical in Rancho Cordova, California. The laboratory test results are included in Appendix B.

7.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

7.1 Site Geology

The project site is located at the western foothills of the Sierra Nevada mountain range of California, east of the San Joaquin Valley. General geologic features pertaining to the bridge site were evaluated by reference to Geologic Data Map No. 2 of the California Geological Survey (CGS 2010). Based on the publication, the project site and its vicinity is primarily underlain by following plutonic rocks:

gr^{Mz} - Mesozoic granite, quartz monzonite, granodiorite, and quartz diorite.

A portion of the published Geologic Map covering the project site is attached, Plate No. 3A.

7.2 Subsurface Conditions

The subsurface conditions are based on the field exploration. Based on the general plan and profile provided (2015), the existing ground surface elevations are estimated to be at approximately 955 feet (vertical datum NAVD 88) at both boring locations.

The two borings encountered predominantly decomposed granite bedrock or colluvium in the upper about 7 to 8 feet thick of soils underlain by granite bedrock to the maximum depths cored, approximately 38 feet in R-15-001 and 17 feet in R-15-002. The bedrock was generally varying from intensely to slightly weathered, intensely to moderately fractured, and moderate hard to very hard. Granite outcrops were observed at the site during the field exploration (Plate No. 3B). The pictures of bedrock core samples are attached in Appendix C.

Groundwater was not encountered in either boring during drilling. The historical groundwater readings in a monitoring well about 1,000 feet north of the bridge, published on the website of California Department of Water Resources (DWR, accessed 2015), reveal that the groundwater level fluctuates approximately from 4 to 28.5 feet below ground surface. Groundwater may vary with passage of time due to seasonal groundwater fluctuation, water level in the creek, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of investigation.

The boring logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs. The abrupt stratum changes shown on these logs may be gradual and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain a properly constructed project. Therefore, it is recommended that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

8.0 SCOUR EVALUATION

The existing creek is a natural earth channel flowing southerly. The bridge abutments should be set back adequate distances to protect from potential scour along the channel banks. Creek bank protection measures may be required along the upstream and downstream ends of the abutments. Ultimate design should be based on the findings of hydraulic study for the Project. Per the Designer, since the footings are set on relatively competent bedrock, no scour analysis is required.

9.0 CORROSION EVALUATION

The corrosion investigation was performed on selected soil samples in general accordance with the provisions of California Test Method 643, 417 and 422. Table 9.1 presents a summary of the corrosion test results.

TABLE 9.1 - CORROSION TEST RESULTS

Boring No.	Depth (ft)	pH	Minimum Resistivity (ohm-cm)	Chloride Content (ppm)	Sulfate Content (ppm)
R-15-001	2	7.04	7,500	14.5	7.8
R-15-002	5	6.46	10,450	12.5	3.5

The Caltrans Corrosion Guidelines (2015) considers a site to be corrosive to foundation elements if one or more of the following conditions exist:

- chloride concentration is greater than or equal to 500 ppm,
- sulfate concentration is greater than or equal to 2,000 ppm, or
- the pH is 5.5 or less.

Based on the test results, the on-site subsurface materials are considered non-corrosive. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructures. The guidelines presented in the California Amendments to the AASHTO LRFD Bridge Design Specifications, 6th Edition (BDS 2012), Article 5.12.3, for the minimum cement factor and cover thickness may be used for the bridge substructure.

10.0 SEISMIC RECOMMENDATIONS

10.1 Seismic Sources

The project site is located in a seismically active part of northern California, at the western foothills of the Sierra Nevada mountain range. The site is distant from major faults. However, many faults in the region are capable of producing earthquakes, which may cause moderate to strong ground shaking at the site. The proposed bridge is located at coordinates of approximately 36.6836 degrees north latitude and 119.2092 degrees west longitude (Google Earth, 2015). The Caltrans Fault Database (V2b, 2012) and Acceleration Response Spectrum (ARS) Online Report (V2, 2012) contain known active faults (if there is evidence of surface displacement in the past 700,000 years) in the State. The information of the active faults in the area, based on the Caltrans ARS Online Report (V2, 2012), is summarized below in Table 10.1. The maximum magnitudes (M_{max}) represent the largest earthquake that a fault is capable of generating and are related to the seismic moment. The attached Caltrans ARS Online Map, Plate No. 4, presents the location of the fault system relative to the project site.

TABLE 10.1 - CALTRANS ARS ONLINE INFORMATION

Fault	Fault ID	Maximum Magnitude, M_{max}	Fault Type	Approx. Distance R_{rup}/R_x (miles)
White Mountains	126	7.4	SS	62.19/58.07
Owens Valley	161	7.2	SS	58.46/58.46
Kern Canyon	189	7.5	N	44.29/43.96
Great Valley 13 (Coalinga)	205	7.0	R	62.67/61.88
Great Valley 14 (Kettleman Hills)	210	7.1	R	61.47/61.06
San Andreas (Creeping sec.) 2011 CFM	182	7.9	SS	87.72/86.59

R_{rup} = Closest distance to the fault rupture plane
R_x = Horizontal distance to the fault trace or surface projection of the top of rupture plane
SS = Strike-slip fault
N = Normal fault
R = Reverse fault

10.2 Seismic Design Criteria

The Caltrans ARS Online program (V2, 2012) was used for producing acceleration response spectra. Development of the design ARS curve is based on several input parameters, including site location (longitude/latitude), average shear wave velocity for the top 100 feet of soils (V_{s30}), and other site parameters, such as fault characteristics, site-to-fault distances. The design methods incorporate both deterministic and probabilistic seismic hazards to produce the Design Response Spectrum. The probabilistic response spectrum to be used for design of structures is based on the data from the USGS Interactive Deaggregations (Beta) program (2008) for a 5 percent in 50 years probability of exceedance (975-year return period) or the Caltrans ARS Online program (V2, 2012). In addition, to account for the potential for earthquakes to occur on previously unknown faults, a minimum deterministic spectrum is imposed statewide and generated on the Caltrans ARS Online report. This minimum deterministic spectrum is for a scenario M 6.5 vertical strike-slip event occurring at a distance of 12 km (7.5 miles) based on the average of the median predictions of Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) models. The controlling spectrum (upper envelope) is adopted for the design response spectrum.

The shear wave velocities for the top 100 feet of soils at the project site were estimated by using the established correlations and guidelines in Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (2012). An average shear wave velocity of 500 m/s was adopted. According to the Caltrans guidelines, the USGS Beta program should be checked and compared with the Caltrans ARS Online program for four spectral probabilistic values (at periods of 0, 0.3, 1 and 3 sec.). If the discrepancy between the USGS spectral acceleration values and the Caltrans Online results is less than 10 percent, then the probabilistic ARS curve generated by Caltrans ARS Online tool is acceptable for design. Otherwise, the probabilistic curve obtained from the USGS Beta program should be used. For this Project, the envelope of the probabilistic spectrum and the minimum deterministic spectrum generated by the Caltrans ARS online program governs. No adjustment is required for the near fault and the basin effect. The generated Acceleration Response Spectra Comparison Curves are presented on Plate No. 5A and the Recommended ARS Curve is presented on Plate No. 5B. The produced seismic parameters are summarized as follows:

- Approx. bridge location: 36.6836ºN / 119.2092ºW.
- Estimated V_{S30}: 500 m/s.
- Anticipated peak ground acceleration: 0.22g.
- The preliminary design spectrum is governed by the envelope of the Caltrans probabilistic ARS and the minimum deterministic ARS.
- No adjustment is required for the near fault and the basin effect.
- Estimated earthquake moment magnitude: 6.2.

10.3 Seismic Hazard

Faulting

The project site is located outside the designated State of California Alquist-Priolo Earthquake Fault Zones for active faulting and no mapped evidence of active or potentially active faulting was found for the site. The potential for fault rupture at the project site is considered to be low.

Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and low-plastic silts of low relative density are the type of soils that usually are susceptible to liquefaction. Clay is generally not susceptible to liquefaction. Since the subsurface profiles encountered in the borings are mainly composed of granite bedrocks, the liquefaction potential at the project site is considered to be very low.

Ground Subsidence

Ground subsidence can occur as a result of "shakedown" when dry, low cohesion soils are subjected to earthquake vibrations of high amplitude. In general, significant deposits of loose sandy soils do not exist at the site; therefore, seismic induced ground subsidence is not considered a geologic hazard on the site.

11.0 AS-BUILT FOUNDATION DATA

The Caltrans Bridge Inspection Report (2012) indicates that the existing bridge (Existing Bridge No. 42C0099) was built in 1975. The bridge measures approximately 30.84 feet (9.4 meters) in length and 23.95 feet (7.3 meters) in total width with two traffic lanes. The structure is described to be “Simply-supported single-span timber stringer (24), with CIP/RC deck and plywood subfloor, on timber sills on RC abutments founded on large boulders & bedrock.”

12.0 FOUNDATION RECOMMENDATIONS

12.1 General

This report was prepared specifically for the proposed Project as described earlier. Normal procedures were assumed for construction of the bridge structure throughout our analysis and represent one of the bases of recommendations presented herein. The design criteria have been based upon the materials encountered at the site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

12.2 Foundations

Based on the subsurface conditions and proposed structure, both CIDH concrete piles and shallow spread footings appear to be feasible for the abutment foundations of the proposed bridge. After discussions with the Designer, shallow spread footings are selected. The footing bottoms should be set in the relatively competent bedrock to reduce the impact of the potential scour.

Per Caltrans Memo to Designers (MTD) 4-1 (2014), design of spread footing should be performed using Load and Resistance Factor Design (LRFD) method in accordance with the California Amendments to the AASHTO LRFD BDS (2012). Pertinent foundation design information provided by the Designer, including Foundation Data and Scour Data, are tabulated in Tables 12.1 and 12.2.

TABLE 12.1 - FOUNDATION DATA

Support No.	Finished Grade Elev. (ft)	Bottom of Footing Elev. (ft)	Estimated Footing Dimensions (ft)		Permissible Settlement under Service-I Load (in)	Approx. Ratio of Permanent/Total Service-I Load
			B	L		
Abut. 1	963.14	947.50	11’-0”	39’-3”	2	0.74
Abut. 2	960.14	945.50	11’-0”	44’-11”	2	0.75

TABLE 12.2 - SCOUR DATA*

Support No.	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)
Abut 1	N/A	N/A
Abut 2	N/A	N/A

* Foundation is embedded into relatively competent bedrock.

12.3 Bearing Capacity

The geotechnical engineering design of the spread footing foundation was performed in accordance with the procedures outlined in Section 10.6 “Spread Footings” of the AASHTO (2012). Groundwater was established at the footing level for design purposes. The gross nominal bearing capacity was estimated to be about 28 ksf assuming a footing width of 10 feet. If the footing width is less than 10 feet, the bearing capacity should be recalculated. According to the AASHTO (2012), a resistance factor of 0.45 should be applied for bearing capacity at the strength or construction limit state. The calculation of footing bearing capacity is presented in Appendix D. The settlement of the footings is anticipated to be less than 1 inch if the design service load does not exceed the permissible net bearing capacity. The recommended foundation design data for spread footing are presented in Table 12.3.

TABLE 12.3 - FOUNDATION DESIGN RECOMMENDATIONS

Support No.	Footing Size (ft)		Bottom of Footing Elev. (ft)	Minimum Footing Embedment Depth (ft)	Total Permissible Support Settlement (in)	Service Limit State	Strength or Construction Limit State Φ_b=0.45	Extreme Limit State Φ_b=1.0
	B	L				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Abut 1	11’-0”	39’-3”	947.50	5	2	9	28*	N/A
Abut 2	11’-0”	44’-1	945.50	5	2	9	28*	N/A

* This is the gross nominal bearing capacity. The resistance factor of 0.45 has not been applied.

To provide uniform subgrade support for the footings, the subgrade of the abutment footing foundations should be over-excavated a minimum of 2 feet and replaced with Caltrans Lean Concrete (LC). If soft and loose, saturated native soil deposits are encountered, deeper excavation will be required to expose firm native soils. The over-excavation should extend to a minimum of 1 foot beyond the footprint of the footings in all directions. It is possible that the excavation will be into hard bedrocks. Special excavation equipment may be necessary during foundation construction. The contractor should be prepared for such conditions.

The recommended nominal passive pressure against the side of the footing is 350 pcf (saturated) equivalent fluid pressure (EFP) using the log spiral method. A base friction coefficient of 0.45 can be used to estimate the friction resistance at the bottom of footing. Only dead loads should be used for estimation of base friction resistance. To utilize the side passive resistance, the minimum horizontal distance between the top near face of the spread footing and the face of the finished slope should be minimum 4 feet according to Section 4 “Foundations” of the Caltrans Bridge Design Specifications (2003). When combining the base friction resistance with side passive resistance, not more than 50% of the recommended nominal passive pressure shall be considered in the determining the factor of safety against sliding.

12.4 Lateral Earth Pressures at Abutments

Abutment and wing walls should be designed to resist the following Applied Lateral Earth Pressures. These values assume no hydrostatic pore pressure buildup behind the walls. The walls should be provided with permanent drains to prevent the buildup of hydrostatic pressures. The backfill materials should conform to the structure backfill requirements contained in Section 19 of the Caltrans Standard Specifications (2015).

Active Condition	36 pcf Equivalent Fluid Pressure.
At-Rest Condition	55 pcf Equivalent Fluid Pressure.
Passive Resistance	5 ksf (ultimate) for seismic design of the abutment backwall (5.5 feet high or greater); for activated height less than 5.5 feet, modify proportionally, i.e. 5×(H/5.5) ksf per Caltrans Seismic Design Criteria V1.7 (2013). A

minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive pressure is required.

Cantilever walls which are free to rotate at least 0.004 radian may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharges (dead or live loads) should be added to the preceding lateral earth pressures. The traffic loads on walls can be determined by referring to Section 3.11.6.4 "Live Load Surcharge" of the AASHTO (2012) using a soil unit weight of 120 pcf. A coefficient of 0.3 and 0.5 may be used to determine the additional horizontal earth pressure resulting from the surcharge for active and at-rest conditions, respectively. The horizontal earth pressure in front of the abutment walls should be ignored.

13.0 PAVEMENT SECTIONS

Pavement design for flexible pavement sections using hot mix asphalt (HMA) was based on the Caltrans Highway Design Manual (HDM 2014). The test on an existing subgrade sample produced an R-value of 74. Since the pavement design will be controlled by import fill, it is recommended to use an R-value of 15 for design of pavement. A Traffic Index (TI) of 5 was provided by the Designer for a design life of 20 years. Table 13.1 presents the recommendations of structural pavement sections. The Caltrans Standard Specifications (2015) should be referred to for pavement materials, and their placement and compaction.

TABLE 13.1 - STRUCTURAL PAVEMENT SECTIONS

TI	R-value	Structural Pavement Section (ft)					
		Option 1	Option 2		Option 3		
		Full-Depth HMA	HMA	AB	HMA	AB	AS
5	15	0.60	0.25	0.70	0.25	0.35	0.35

HMA: Hot Mix Asphalt (Type A)
AB: Aggregate Base (Class 2) with R-value equal to 78
AS: Aggregate Sub-base (Class 2) with R-value equal to 50

14.0 GRADING

All grading and compaction operations should be performed in accordance with the project specifications and Section 19 "Earthwork" of the Caltrans Standard Specifications (2015). A representative from this office or regulating agency should observe all excavated areas during grading and perform moisture and density tests on prepared subgrade and compacted fill material.

Areas to receive embankment fill should be clean of vegetation, shrubs, trees, and their roots greater than 1 inch in diameter. If any soft or saturated soils are encountered during site grading, deeper excavation may be required to expose firm soils.

Any fill materials imported to the project site should be non-expansive, relatively granular material having a Plasticity Index (PI) of less than 15 and a minimum Sand Equivalent (SE) of 10. The maximum particle size of fill material should not be greater than 4 inches in largest dimension. It should also be non-corrosive, free of deleterious material and should be reviewed by the Geotechnical Engineer. In addition, it is recommended that the material within 4 feet of the proposed pavement subgrade have a minimum R-value of 15.

The gradient of both cut and fill slopes should not be steeper than 2H:1V (horizontal to vertical). At locations where it needs to cut into existing slopes, the top portion maybe highly weathered and decomposed. The top and ends of the cut line should be rounded so that the wedge of excavation is laid back with lesser gradient. A Slope Rounding Diagram from Caltrans Storm Water Quality Handbooks (2007) is attached as Plate No. 6. It should be noted that local irregularities such as loose layers and pockets and seepage might require flatter slopes. This office should review the final grading plans prior to grading to see that the intent of our recommendations is included in the plans.

15.0 CONSTRUCTION CONSIDERATIONS

15.1 General

To a degree, the performance of any structure is dependent upon construction procedures and quality. Hence, observation of foundation construction and grading operations should be carried out by the geotechnical engineer. If the encountered subsurface conditions differ from those forming the basis of our recommendations, this office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

15.2 Waiting Period

About 10 feet of fill for new approach embankments and roadway widening is expected, which needs to be confirmed once final grading plan is made available. Since no saturated soft materials were encountered in our borings and the site is mostly underlain by granite bedrock, the settlement due to fill is insignificant and can be ignored. The waiting period is not required. However, it is recommended that the bridge foundation construction start after earth work operation is complete.

15.3 Construction Dewatering

Groundwater may cause instability of excavation walls and bottom (piping, erosion, blow-outs, etc.) and difficult working conditions. For excavation below the groundwater table, construction dewatering will be required. The contractor should evaluate the subsurface conditions before selecting a dewatering method, which may include shoring, sumps or tremie slabs. Groundwater should be lowered to at least 2 feet below the bottom of excavation to provide workable condition. Designing dewatering system should be the contractor’s responsibility. The Caltrans Standard Specifications (2015), Section 19, provides guidelines for water control and foundation treatment.

All dewatering systems should be properly designed to prevent pumping soil fines with the discharge water. The contractor should sample and test the groundwater for soil fines content from the discharge, as needed. If soil fines are pumped, the contractor should revise his dewatering operations. Otherwise, failure of shoring, partial instability of trench bottom resulting in intolerable ground settlement/ movement of existing utilities and unsafe working conditions may occur. The contractor should provide discharge sampling locations for each pump. The contractor is encouraged to perform their own investigation, test program, etc. prior to construction in order to satisfy their design requirements for an effective dewatering program. Contractor should confirm the design groundwater level (for shoring) prior to actual construction.

15.4 Temporary Excavation and Shoring

Excavation will be required for installation of bridge foundations. It is possible that unknown old buried utilities are located at the site. It might require special equipment and additional efforts to remove these buried objects. Foundation excavation may go into hard bedrock formation. Special excavation equipment/tools should be prepared for foundation construction.

According to OSHA Safety Standards, temporary excavations with personnel working within the excavations should be sloped or shored if the excavations are deeper than 5 feet. All excavations for the Project should be made and supported in accordance with OSHA standards. For excavations up to 20 feet deep in homogenous soils, OSHA guidelines state that the maximum allowable slope should be 3/4H:1V, 1H:1V and 1-1/2H:1V for Types A, B and C soil, respectively (In general, Type A soils are stronger; Type B soils are intermediate, and Type C soils are weaker). The boring data suggest that most on-site soils approximately above elevation 947 feet should be considered as OSHA Type C materials. It should be noted that the slope ratio recommended by OSHA is for temporary, unsurcharged slopes and properly dewatered conditions. Traffic and surcharge loads should be set back at least 15 feet from the top of the excavations unless they are accounted for in the design.

The excavation should be closely monitored during construction to detect any evidence of instability, soil creep, settlement, etc. Appropriate mitigation measures should be implemented to correct such situations that may cause or lead to future damage to facilities, utilities and other improvements.

15.5 Working Platform

Groundwater should be expected during excavation. Soft and loose, saturated native soil deposits may be encountered at the bottom of excavation. In such case, working conditions at the bottom of excavation may become difficult; equipment used at the bottom of the excavation may lose mobility, etc. The contractor should take adequate measures to minimize the disturbance of the sensitive deposits at the excavation subgrade. The contractor may minimize the disturbance of sensitive deposits or mitigate existing soft ground conditions by constructing a working platform at the bottom of the excavation. The working platform may be installed by 1) over excavating about 2 feet below the planned subgrade; 2) placing a stabilizing subgrade enhancement geotextile at the bottom of the resulting excavation; and 3) backfilling with 2-inch crushed rock, compacted AB or other such approved bridging material. The contractor may use other methods of subgrade stabilization. The contractor's proposed method should be reviewed by the geotechnical engineer.

16.0 NOTES TO DESIGNER

Should there be any alterations of the proposed construction that will affect the stated bases of our recommendations, we should be informed so that we can review such changes and amend or submit additional recommendations.

17.0 PLAN REVIEW

This report is prepared for the proposed Sand Creek Bridge replacement project. It is recommended that the final foundation plans for this Project be reviewed by this office prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

18.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed Project as described earlier, to assist the engineer in the design of this Project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the Designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the Project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Respectfully submitted,
PARIKH CONSULTANTS, INC.

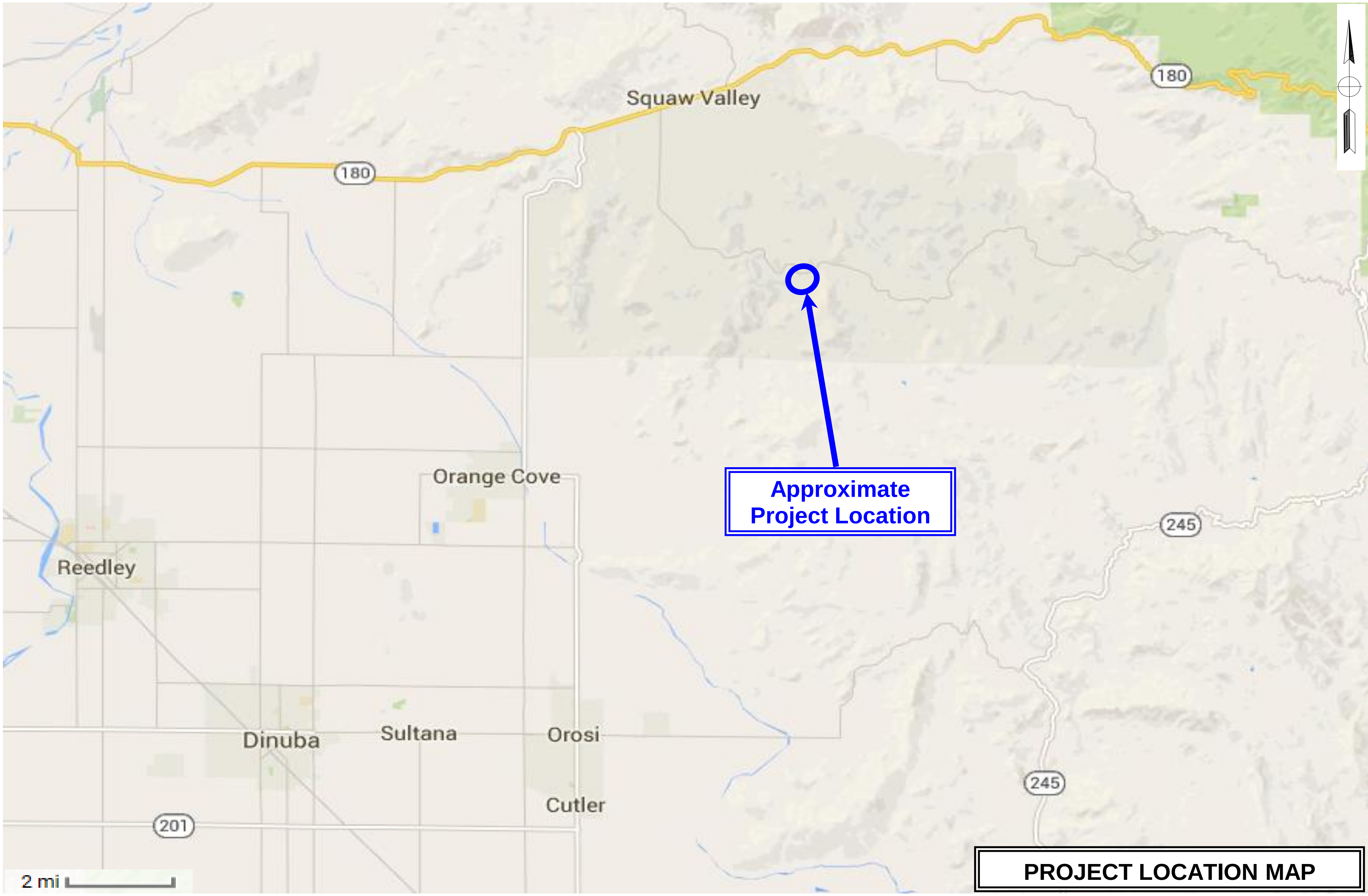
Peter Wei, PE, GE 2922
Sr. Project Engineer

Y. David Wang, PhD. PE 52911
Project Manager

REGISTERED PROFESSIONAL ENGINEER
Y. DAVID WANG
PE 52911
EXP 12/31/16
CIVIL
STATE OF CALIFORNIA

REFERENCES

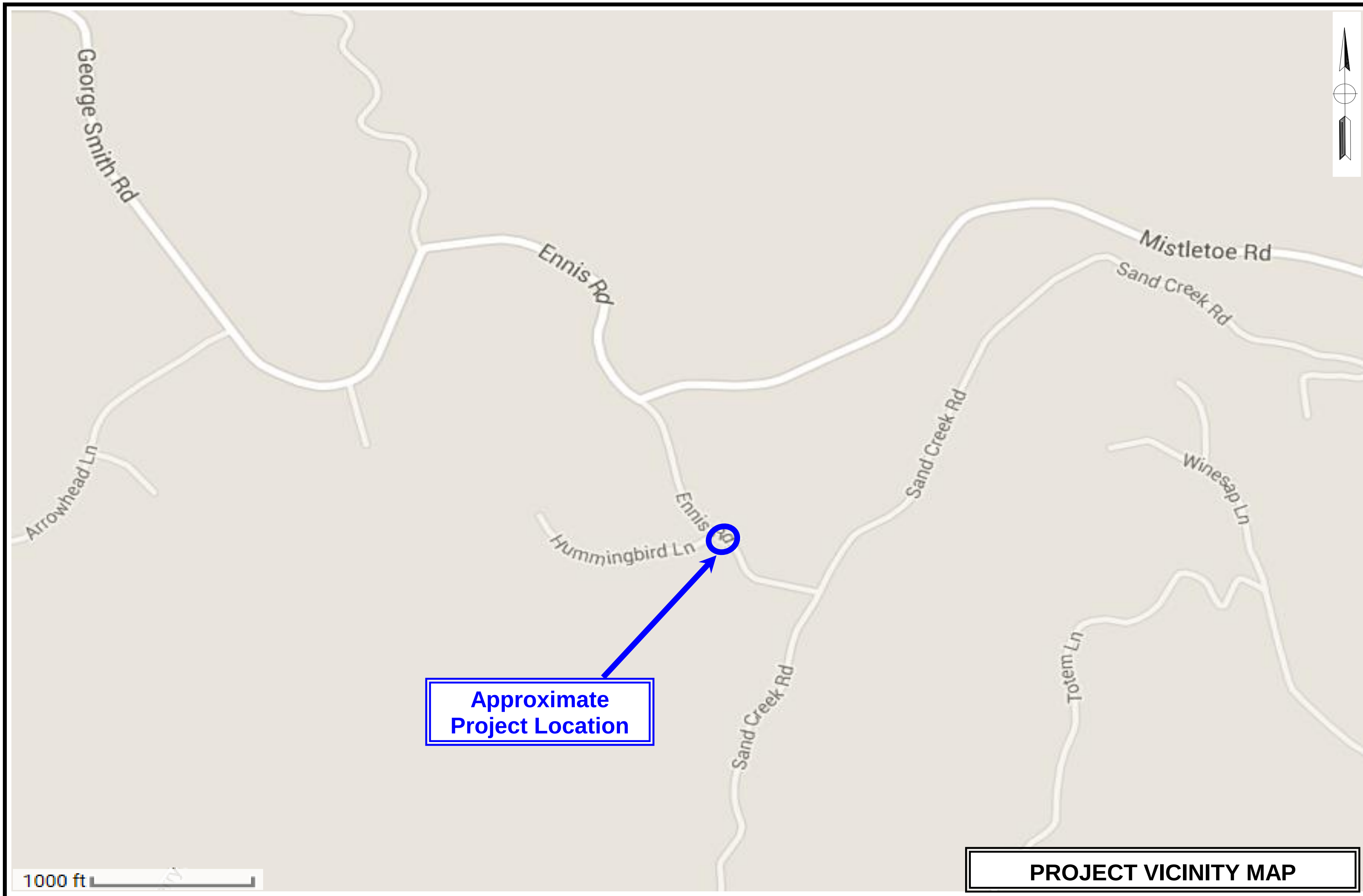
1. Caltrans, 2009, Guidelines for Structure Foundation Reports, V2.0.
2. Caltrans, 2010, Soil & Rock Logging, Classification, and Presentation Manual, Office of Structural Foundations California Department of Transportation.
3. Caltrans, 2012, ARS Online, V2, (http://dap3.dot.ca.gov/ARS_Online/index.php).
4. Caltrans, 2012, Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations.
5. Caltrans, 2013, Seismic Design Criteria, V1.7.
6. Caltrans, 2014, Memo to Designers 4-1.
7. Caltrans, 2014, California Amendments to the AASHTO LRFD Bridge Design Specifications, 6th Edition (2012).
8. Caltrans, 2014, Highway Design Manual.
9. Caltrans, 2015, Corrosion Guidelines, V2.1.
10. Caltrans, 2015, Standard Plans.
11. Caltrans, 2015, Standard Specifications.
12. California Geological Survey, 2010, Fault Activity Map of California, Geologic Data Map No. 6, Compilation and Interpretation by Jennings, C. W. and Bryant, W. A.
13. California Geological Survey, 2010, Geologic Map of California, Geologic Data Map No. 2, Compilation and Interpretation by Jennings, C. W. (1977).
14. California Department of Water Resources, 2015, (http://www.water.ca.gov/waterdatalibrary/).
15. Coduto, D. P., 1999, *Geotechnical Engineering, Principles and Practices*, Prentice-Hall, Inc., Upper Saddle River, NJ, p 511-512.
16. Daniel, C. R., Howie, J. A. and Sy, A., 2003, ‘A Method for Correlating Large Penetration Test (LPT) to Standard Penetration Test (SPT) Blow Counts,’ *Canadian Geotechnical Journal*, 40:66-77, 2003.
17. USGS, 2008, Online Interactive Deaggregation Program (Beta), (https://geohazards.usgs.gov/deaggint/2008/).
18. Youd, T. L. and Idriss, I. M., Co-Chairs, 2001, ‘Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,’ ASCE, *Journal of Geotechnical and Geoenvironmental Engineering*, V. 127, No. 4, p 297-313.



PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN | PLATE NO.: 1 A



PARIKH
Practicing in the Geosciences

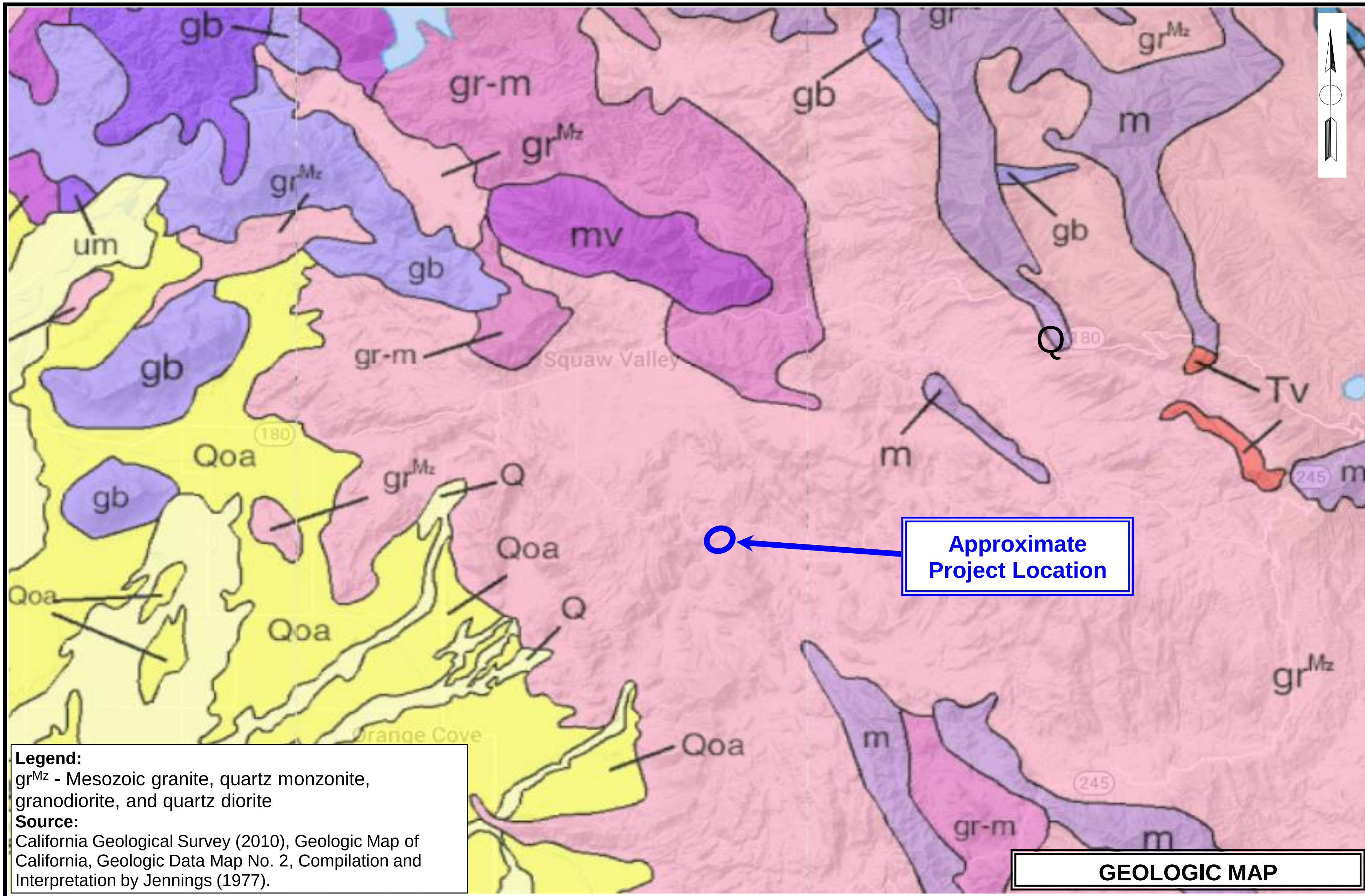
SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN	PLATE NO.: 1 B

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN	**PLATE NO.: 2**



Legend:
gr^{Mz} - Mesozoic granite, quartz monzonite, granodiorite, and quartz diorite
Source:
California Geological Survey (2010), Geologic Map of California, Geologic Data Map No. 2, Compilation and Interpretation by Jennings (1977).

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

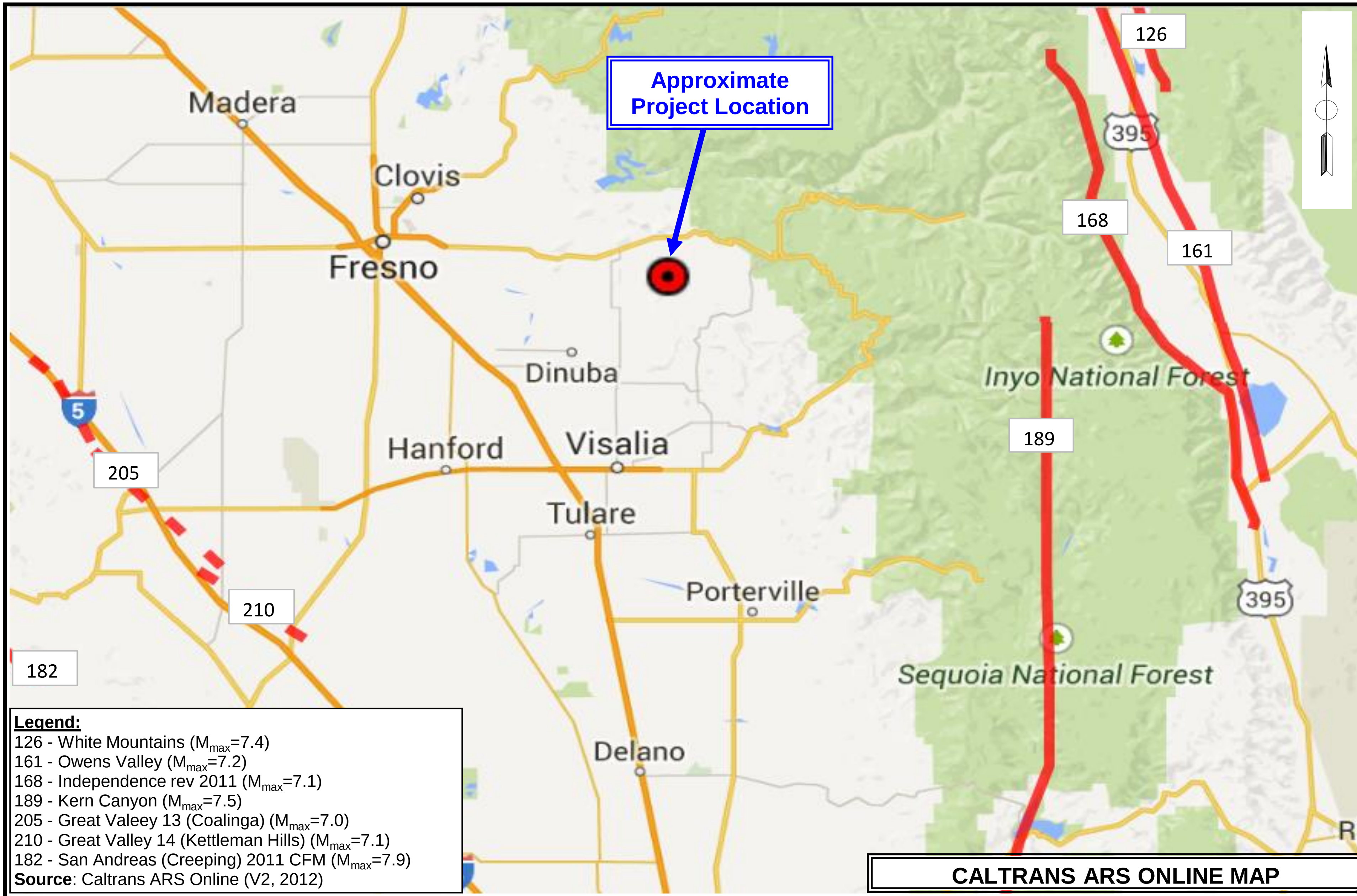
JOB NO.: 2015-115-FDN	PLATE NO.: 3A


BEDROCK OUTCROPS

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

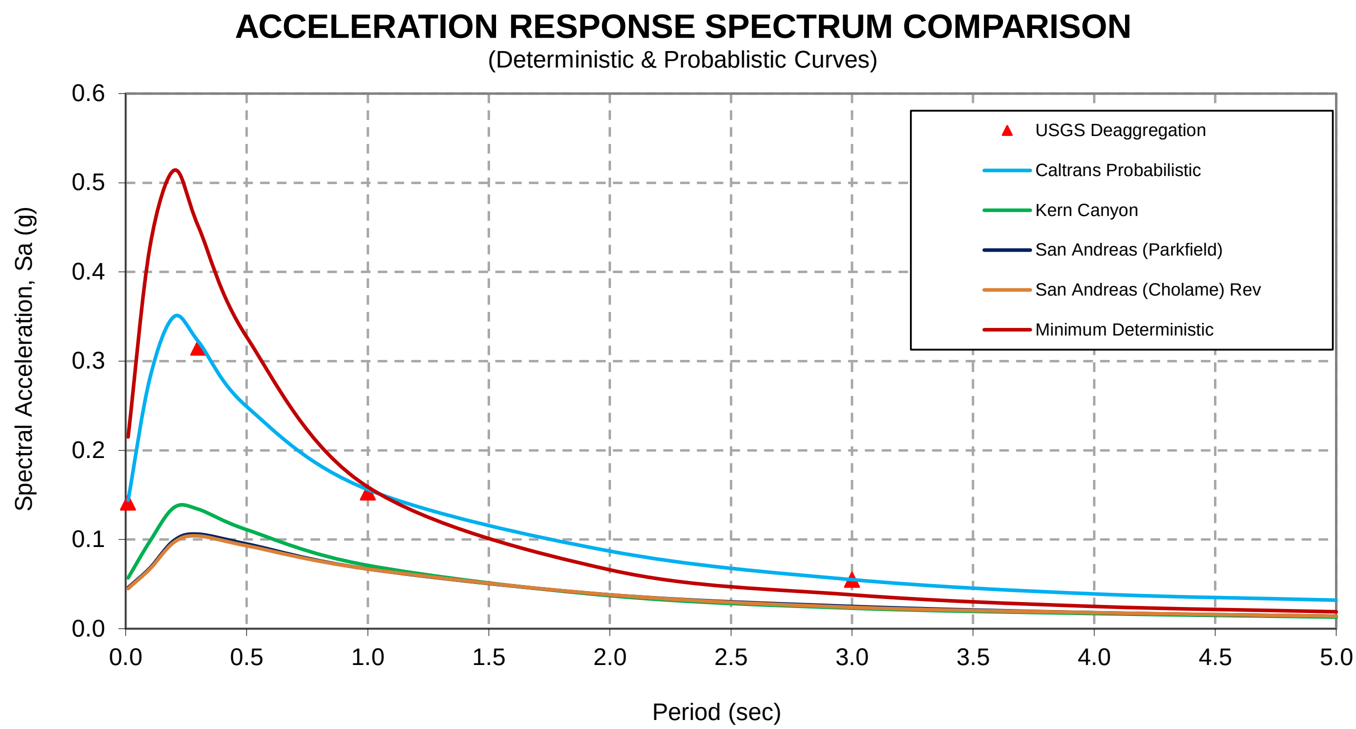
JOB NO.: 2015-115-FDN	PLATE NO.: 3B



PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN	**PLATE NO.: 4**



Site Information

Latitude:	36.6836
Longitude	-119.2092
V_{S30} (m/s) =	500
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	8.5

Final Adjusted Spectral Accelerations (g)						
Period (sec)	Kern Canyon	San Andreas (Parkfield)	San Andreas (Cholame) Rev	Minimum Deterministic	Caltrans Probabilistic	USGS Deaggregation
0.0	0.057	0.046	0.045	0.215	0.144	0.142
0.1	0.098	0.068	0.067	0.428	0.281	
0.2	0.136	0.099	0.097	0.514	0.350	
0.3	0.134	0.106	0.104	0.451	0.322	0.315
0.5	0.111	0.095	0.093	0.327	0.249	
1.0	0.071	0.067	0.067	0.159	0.156	0.153
2.0	0.037	0.038	0.038	0.066	0.087	
3.0	0.023	0.025	0.024	0.038	0.055	0.055
4.0	0.017	0.018	0.018	0.025	0.039	
5.0	0.013	0.014	0.014	0.019	0.032	

Source:

1. Caltrans ARS Online tool (V2, http://dap3.dot.ca.gov/shake_stable/v2/index.php)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

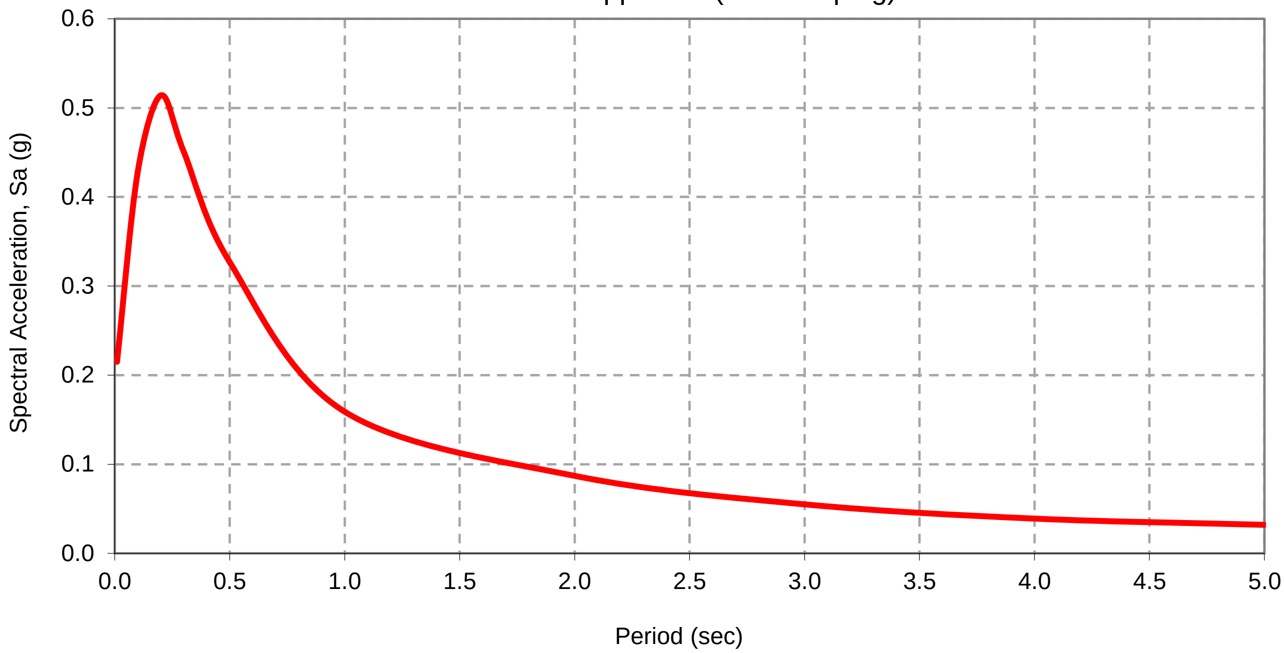
PARIKH
Practicing in the Geosciences

Sand Creek Bridge on Ennis Road (Replace)
Fresno County, California

Project No.: 2015-115-FDN | **Plate No.: 5A**

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

Probabilistic Approach (5% Damping)



Site Information

Latitude:	36.6836
Longitude	-119.2092
V_{S30} (m/s) =	500
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	8.5

Governing Curve:

Envelope of Caltrans Online Probabilistic & Deterministic ARS

Recommended Response Spectrum

Period (sec)	Envelope Spectral Acceleration(g)	Adjusted for Near Fault Effect	Adjusted For Basin Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.215	N/A	N/A	0.215
0.1	0.428	N/A	N/A	0.428
0.2	0.514	N/A	N/A	0.514
0.3	0.451	N/A	N/A	0.451
0.5	0.327	N/A	N/A	0.327
1.0	0.159	N/A	N/A	0.159
2.0	0.087	1.0	1.0	0.087
3.0	0.055	1.0	1.0	0.055
4.0	0.039	1.0	1.0	0.039
5.0	0.032	1.0	1.0	0.032

Note:

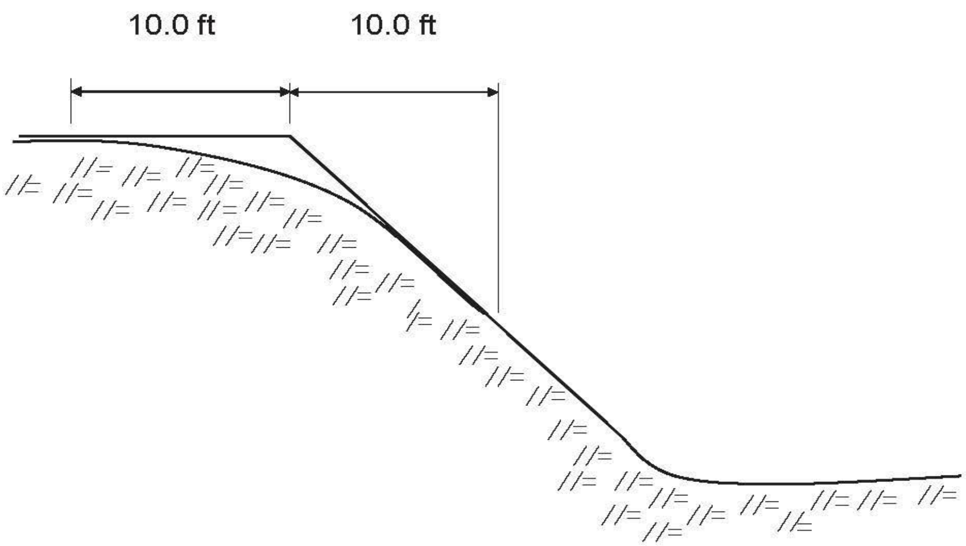
Source:

1. Caltrans ARS Online tool (V2, http://dap3.dot.ca.gov/shake_stable/v2/index.php)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

PARIKH
Practicing in the Geosciences

Sand Creek Bridge on Ennis Road (Replace)
Fresno County, California

Project No.: 2015-115-FDN	**Plate No.: 5B**



Slope Rounding

N.T.S.

Source:
Caltrans Storm Water Quality Handbooks, Project Planning and Design Guide, May 2007, Appendix A, Figure A-9.

SLOPE ROUNDING DIAGRAM

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN	**PLATE NO.: 6**

APPENDIX A

Notes:
Standard Penetration Test Sampler: I.D. = 1.4"; O.D. = 2"
Modified California Sampler: I.D. = 2.5"; O.D. = 3"
Hammer Assembly: A 140 lb hammer with a 30" drop (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (2010)

See Caltrans 2015 Standard Plans A10F, A10G and A10H for Soil and Rock Legends.

All dimensions are in feet unless otherwise shown.

Base map is provided by Biggs Cardosa Associates Inc 2016.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
6	Fresno	LOCAL			

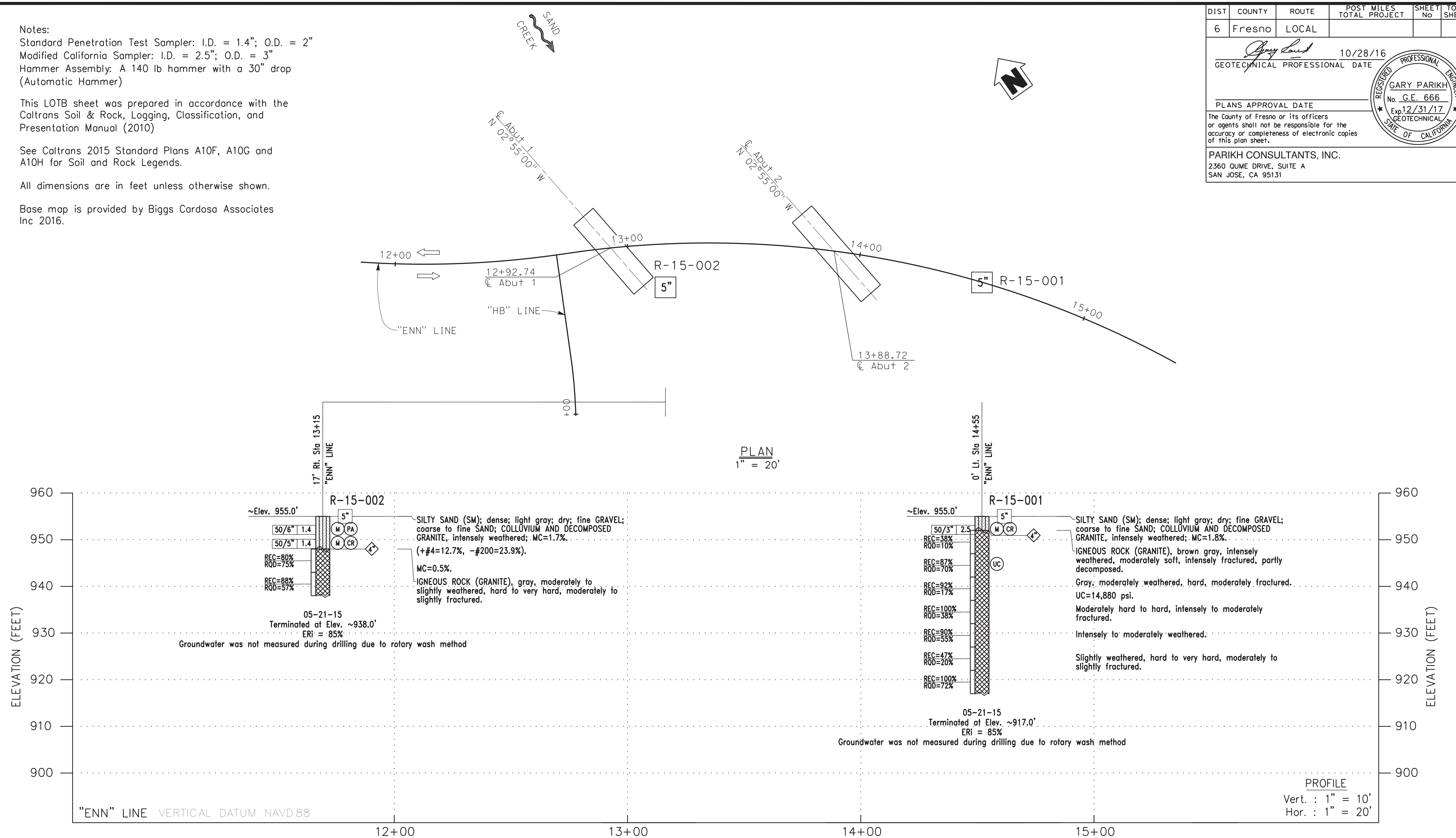
GEOTECHNICAL PROFESSIONAL DATE 10/28/16

PLANS APPROVAL DATE

The County of Fresno or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/17 GEOTECHNICAL STATE OF CALIFORNIA

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131



DRAWN BY	KIM OUYANG	V. SANTOS	PETER WEI
		FIELD INVESTIGATION BY:	PROJECT ENGINEER
CHECKED BY	PETER WEI	DATE: MAY 2015	

BIGGS CARDOSA ASSOCIATES INC
STRUCTURAL ENGINEERS
5250 N. Palm Avenue, Suite 211
Fresno, California 93704
559-449-8686

PROJECT
SAND CREEK BRIDGE REPLACEMENT ON ENNIS ROAD

ROAD NO. BRIDGE NO. 42C-0099, BRLO-5942(238)

THE COUNTY OF FRESNO 1856

DEPARTMENT OF PUBLIC WORKS AND PLANNING

LOG OF TEST BORINGS

DRAWING NO. SHEET NO. S-14 TOTAL

(2015134S1)
2015134

APPENDIX B

LABORATORY TESTS

Classification Tests
The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System. The results are presented in "Log of Test Borings", Appendix A.

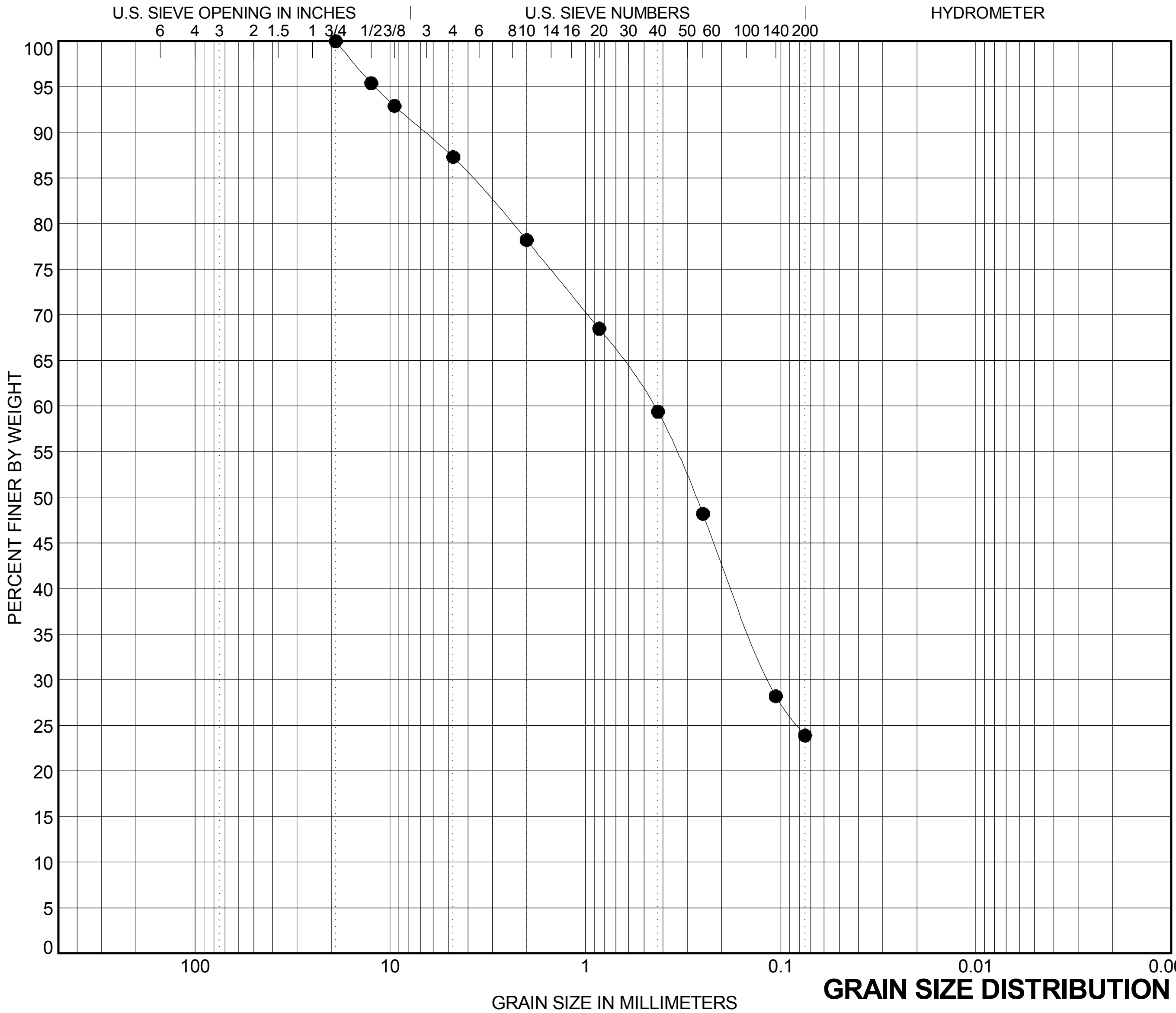
Moisture-Density
The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM D 2216. This information was used to classify and correlate the soils. The results are presented in "Log of Test Borings", Appendix A.

Grain Size Classification
Grain size classification tests (ASTM D 422) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plate B-2, Grain Size Distribution Curves.

Unconfined Compression Tests
Unconfined Compressive Strength tests were performed on selected rock core samples. The tests were performed in general accordance with ASTM C 42. The results are presented on Plate B-3.

Corrosion Tests
Corrosion tests were performed on selected samples to determine the corrosion potential of the soils according to California Test Methods 643, 417 and 422. The tests were performed by Sunland Analytical. The test results are presented on Plates B-4A and B-4B.

R-value Test
R-value test was performed on representative bulk sample for pavement design. The test was performed according to California Test Method 301. The test results are presented on Plate B-5.



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
● **R-15-002**	**1**	**2.0**	**SILTY SAND**					

BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **R-15-002**	**1**	**2.0**	**19**	**0.445**	**0.115**		**12.7**	**63.4**	**23.9**	

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO: 2015-115-FDN | **PLATE NO: B-2**

PARIKH
Practicing in the Geosciences

COMPRESSIVE STRENGTH TEST FOR ROCK CORE SAMPLES (ASTM C 42)

Project Name: Sand Creek Bridge On Ennis Road
Project Number: 2015-115-FDN
Boring Number: R-15-001
Core Run Number: 2
Approx. Depth of Core Sample (ft): 9-10
Rock Type: Granite

Average Length (in)	Average Diameter (in)	Core Weight (gms)	Calculated Density (pcf)	Correction Factor	Max. Load (lbs)	Compressive Strength (psi)	Corrected Strength (psi)
4.6	2.37	937.6	175.95	1	65,630	14,877	14,880

2360 Qume Drive, Suite A, San Jose, CA 95131 | P (408) 452-9000 | F (408) 452-9004 | www.PARIKHnet.com
San Jose ♦ Oakland ♦ Walnut Creek ♦ Sacramento ♦ Fresno ♦ Los Angeles

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 06/05/2015
Date Submitted 06/01/2015

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 2015-115-FDN Site ID : R-15-001 at 2 ft
Thank you for your business.

* For future reference to this analysis please use SUN # 69649-145065.

EVALUATION FOR SOIL CORROSION

Soil pH	7.04	
Minimum Resistivity	7.50 ohm-cm (x1000)	
Chloride	14.5 ppm	00.00145 %
Sulfate	7.8 ppm	00.00078 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 06/05/15
Date Submitted 06/01/15

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA, 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 2015-115-FDN Site ID: R-15-002 at 5 ft
Thank you for your business.

* For future reference to this analysis please use SUN # 69649 - 145066

EVALUATION FOR SOIL CORROSION

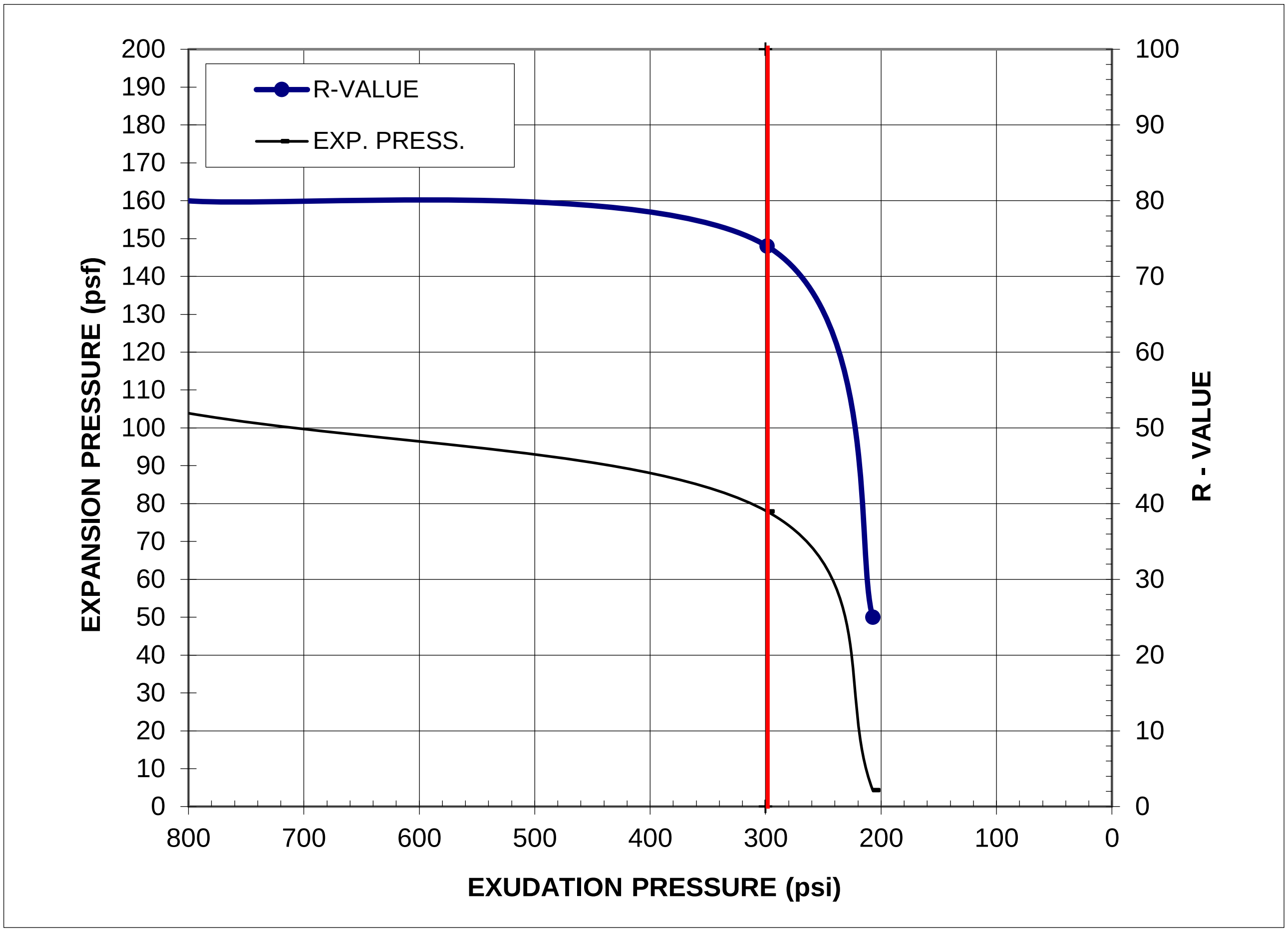
Soil pH	6.46		
Minimum Resistivity	10.45	ohm-cm (x1000)	
Chloride	12.5 ppm	0.0013	%
Sulfate-S	3.5 ppm	0.0004	%

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	Sand Creek Bridge On Ennis Road (Replace)	Date:	6/2/2015
Client:	BKF Engineers	Project #:	2015-115-FDN
Sample #:	R-3 Depth: 0-5'	Lab #:	M943
Location / Source:	Onsite / Native	Sample Date:	
Material :	Sand, gray	Sampled By:	NA



Specimen No.	A	B	C
Exudation Pressure, psi	207.2	298.8	801
Expansion Pressure, psf	4.33	77.94	103.92
R-Value	25	74	80
Moisture Content at Test, %	9.4	8.2	7.7
Dry Density at Test, pcf	128.1	131.7	131.8

R-Value @ 300 psi Exudation Pressure = 74

Expansion Pressure @300 psi Exudation, psf = **78**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

PLATE NO.: B-5

APPENDIX C

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN

PLATE NO.: C-1A

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN	PLATE NO.: C-1B

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN	PLATE NO.: C-1C

PARIKH
Practicing in the Geosciences

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

JOB NO.: 2015-115-FDN	PLATE NO.: C-1D

SAND CREEK BRIDGE ON ENNIS ROAD (REPLACE)
FRESNO COUNTY, CALIFORNIA

PARIKH
Practicing in the Geosciences

JOB NO.: 2015-115-FDN	PLATE NO.: C-2A

APPENDIX D

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

SUBJECT Bearing Capacity

PROJECT NO. 2015-115-FDN
PROJECT NAME Sandy Creek Bridge
CALCULATED BY P.W. DATE 10-28-16
CHECKED BY DATE
VERIFIED BY DATE
BACK CHECKED BY DATE

Ref. AASHTO (2012), sec. 10.6

Assume $\gamma = 120$ pcf, $\phi = 36°$, $c = 0$

$D_f = 5$ ft, $B = 10$ ft

$$q_n = c N_c + \gamma D_f N_q C_{wq} + 0.5 \gamma B N_\gamma C_{w\gamma}$$

$N_c = 50.6$ $C_{wq} = 0.5$
$N_q = 37.8$ $C_{w\gamma} = 0.5$
$N_\gamma = 56.3$

$$q_n = 0 + 120 \times 5 \times 37.8 \times 0.5 + 0.5 \times 120 \times 10 \times 56.3 \times 0.5$$
$$= 11{,}340 + 16{,}890 = 28{,}230 \text{ psf}$$

use 28 ksf, Resistance factor 0.45