August 4, 2017
Revised: July 17, 2020
Kleinfelder Project No.: 20161220.001A

Mr. Mark A. Weaver, MS, PE
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

SUBJECT: Foundation Report
W Manning Avenue over James Bypass West Channel
Bridge No. 42C-0691
San Joaquin, Fresno County, California

Dear Mr. Weaver:

The attached report presents the results of the geotechnical study for the West Manning Avenue Bridge (Bridge No. 42C-0691) over James Bypass West Channel project located east of San Joaquin, in Fresno County, California. This report describes our study and provides conclusions and recommendations for use in foundation design and construction.

We appreciate the opportunity to provide geotechnical engineering services to Cornerstone Structural Engineering Group, the County of Fresno, and other project designers. We trust this information meets your current needs. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully Submitted,

KLEINFELDER, INC.

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 09/30/2021
GEOTECHNICAL
STATE OF CALIFORNIA

Adam AhTye, PE
Staff Professional II

Stephen P. Plauson, PE, GE
Principal Geotechnical Engineer

AA/SPP:ct

FOUNDATION REPORT
W MANNING AVENUE OVER JAMES BYPASS
WEST CHANNEL
BRIDGE NO. 42C-0691
SAN JOAQUIN, FRESNO COUNTY, CALIFORNIA
KLEINFELDER PROJECT #20161220.001A

AUGUST 4, 2017
REVISED: JULY 17, 2020

A Report Prepared for:

Mr. Mark A. Weaver, MS, PE
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

FOUNDATION REPORT
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Adam AhTye, PE
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Stephen P. Plauson, PE, GE
Principal Geotechnical Engineer

KLEINFELDER, INC.
3731 W Aschcroft Ave
Fresno, California 93722
(559) 486-0750

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1 INTRODUCTION

1.1 GENERAL

This report presents the results of the geotechnical investigation for the proposed replacement of West Manning Avenue Bridge (Bridge No. 42C-0691) over James Bypass West Channel, located east of San Joaquin in Fresno County, California. The scope of services consisted of a field exploration program, laboratory testing, engineering analysis, and preparation of this written report. This report has been prepared in conjunction with the project 60 percent. The Foundation Report will be amended as design proceeds, with the final Foundation Report reflecting final design. A concurrent geotechnical study is also being performed for the West Manning Avenue Bridge (Bridge No. 42C-0692) over James Bypass East Channel located approximately 1,150 feet east of Bridge No. 42C-0691 and will be presented in a separate report.

1.2 SCOPE OF SERVICES

The purpose of the Foundation Report is to provide geotechnical recommendations and opinions to aid in project design. The report provides the following:

- A description of the proposed project;
- A summary of the field exploration and laboratory testing programs;
- Comments on the regional geology and site engineering seismology, including the recommended peak ground acceleration and Caltrans Seismic Design Criteria Version 1.7 ARS curve;
- Comments on liquefaction potential;
- Recommendations for pile foundations, including design and specified tip elevations;
- Recommended LPILE parameters for use in evaluating the pile response to lateral loads;

- Comments on initial soil stiffness and ultimate equivalent lateral pressure by Caltrans procedures for abutment end walls; and,
- Log of Test Borings drawing.

1.3 PROJECT DESCRIPTION

The existing structure is a 6-span reinforced concrete precast girder bridge. The structure is 36 feet 8 inches in width and 180 feet in length containing an asphalt concrete overlay. The span lengths center of bearing (c.o.b.) to c.o.b. are 30 feet between abutments and bents.

Construction of the replacement bridge will involve complete replacement of the existing structure. The new structure will be a 3-span bridge, 173 feet in length with a total width of 44 feet and span lengths of 72 feet between bents and 50.5 feet between the abutments and adjacent bents. The replacement is anticipated to include two, 48-inch CIDH (Cast-In-Drilled-Hole) piles at each abutment and bent. Construction is anticipated to be split into three phases. Piles, pile extensions, and cap beams would first be erected under the existing bridge, followed by the bridge's closure and demolition. Precast voided slab units would then be placed and paved with a polyester overlay.

Table 1.3-1

Bridge Replacement

Foundation Design Data

Support	Location (Sta. No.)[1]	Pile Type	Finished Grade Elev.[2] (ft)	Cut-off Elev. (ft)	Pile Cap Size (ft)		S_P[3]	No. Piles per Support
					B	L		
Abut 1	28+66.8	48 inch CIDH	183.74	174.24	5	69	1”	2
Bent 2	29+18.8	48 inch CIDH	183.84	174.34	5	69	1”	2
Bent 3	29+90.8	48 inch CIDH	183.99	174.49	5	69	1”	2
Abut 4	30+39.8	48 inch CIDH	184.07	174.57	5	69	1”	2

Notes: 1. Stations were based on layout drawings provided by Cornerstone Structural Engineering Group
2. Elevations based on project datum
3. Permissible settlement under service limit load

Table 1.3-2

Bridge Replacement

LRFD Loading Data

Support	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)		Extreme Event Limit State (Controlling Group, kips)	
	Total Load		Permanent Load	Compression		Compression	
	Per Support	Max. per Pile	Per Support	Per Support	Max. per Pile	Per Support	Max. per Pile
Abutment 1	1020	540	770	1500	800	1100	600
Bent 2	1470	780	1120	2200	1200	1600	810
Bent 3	1470	780	1120	2200	1200	1600	810
Abutment 4	1020	540	770	1500	800	1100	600

1.4 POLICY EXCEPTIONS

No known exceptions to Caltrans policy were made in the geotechnical evaluation for the foundations for this project.

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

The field exploration for the Manning Avenue Bridge No.42C-0691 replacement consisted of drilling 3 test borings on July 2 through July 9, 2015. The test borings were drilled with a CME 75 truck-mounted drill rig and CME 55 track-mounted drill rig using hollow stem auger and mud rotary techniques. Borings B-1 and B-3 were drilled near the abutments to depths of approximately 81 and 101 feet below ground surface and Boring B-2 was drilled in the canal to approximately 91 feet below ground surface. The approximate location of the test borings are shown on the Log of Test Borings drawing in Figures 2 and 3 of this report.

The earth materials encountered in the test borings were visually classified in the field and a continuous log was recorded. In-place samples of soil units were collected from the test boring at selected depths by driving a 2.5-inch I.D. split barrel sampler containing brass liners into the undisturbed soil with a 140-pound automatic safety hammer free falling a distance of 30-inches. In addition, an ASTM D1586 standard penetrometer without liners (barrel I.D. of 1.5 inches) was driven 18-inches in the same manner. This latter sampling procedure generally conformed to the ASTM D1586 test procedure. Resistance to sampler penetration over the last 12-inches is noted on the Log of Test Borings as the "Penetration Index". The penetration indices listed on the Log of Test Borings have not been corrected for the effects of overburden pressure, sampler size, rod length, or hammer efficiency. In addition, bulk samples were obtained from auger cuttings at selected borings.

Penetration rates determined in general accordance with ASTM D1586 were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

2.2 LABORATORY TESTS

Laboratory tests were performed on selected samples to evaluate certain characteristics and engineering properties. The laboratory testing program was designed with emphasis on the evaluation of geotechnical properties of foundation materials as they pertain to the proposed construction. The laboratory testing program included performing the following tests:

- Unit Weight (ASTM D2937)
- Moisture Content (ASTM D2216)
- Atterberg Limits (ASTM D4318)
- Grain Size Distribution (ASTM D422, without hydrometer)
- Material in Soils Finer than No. 200 (75-µm) Sieve (ASTM D1140)
- Resistance Value (California Test Method No. 301)
- Direct Shear (ASTM D3080)
- Soluble Sulfate and Chloride Content (California Test Method Nos. 417 and 422)
- pH and Minimum Resistivity (California Test Method No. 643)

The soluble sulfate, soluble chloride, pH, and minimum resistivity results are presented in Section 4.0 ("Corrosion Evaluation"). All other lab test results are presented in Appendix A.

Laboratory data was also used from recent borings drilled for the concurrent geotechnical study for the nearby West Manning Avenue Bridge (Bridge No. 42C-0692) over James Bypass East Channel.

2.3 GEOTECHNICAL DESIGN PARAMETERS

Soil conditions and characteristics are very similar along the West Manning Avenue alignment at the two bridge sites (Bridge Nos. 42C-0691 and 42C-0692). Design geotechnical parameters were based on site specific laboratory data for the entire project alignment and interpretation of the geology in the area. Consideration was also given to correlations with sample penetration rates. Tables 2.3-1 and 2.3-2 provide a summary of geotechnical design parameters and generalized soil profile used.

Table 2.3-1

Geotechnical Design Parameters, Abutments

Elevation (feet)	Material	γ_t (pcf)[1]	Φ (degrees)	c (psf)
184 – 167	SC	123	31	150
167 – 158	SP	122	38	0
158 – 140	SC/SM	128	34	150
140 – 128	SC/CL	127	30	500
Below 128	SP	124	36	0

Notes: [1] Total unit weight
[2] Buoyant unit weight

Table 2.3-2

Geotechnical Design Parameters, Bents

Elevation (feet)	Material	γ_t (pcf)[1]	Φ (degrees)	c (psf)
158 – 140	SC/SM	110	34	150
140 – 128	SC/CL	117	30	500
Below 128	SP	124	36	0

Notes: [1] Total unit weight
[2] Buoyant unit weight

3 SITE CONDITIONS

3.1 SURFACE CONDITIONS AND TOPOGRAPHY

Presently, Manning Avenue is a 2-lane paved road supported on a fill embankment, approximately 8 feet above the surrounding grade with about 2:1 (H:V) side slopes. At the time of investigation water was not present in the James Bypass West Channel. Some dried vegetation, debris and rip rap exist on the banks and bottom of the channel. The channel invert at the replacement is presently at approximately elevation 158 feet with slopes to the abutments. The unlined James Bypass Canal is about 100 feet west of the bridge. The James Bypass Canal access roads are at approximately elevation 183 feet.

3.2 REGIONAL GEOLOGY

The project site lies in the central portion of the San Joaquin Valley in the Great Valley geomorphic province in California. This province was formed by the filling of a large structural trough or downwarp in the underlying bedrock. The trough is situated between the Sierra Nevada Range on the east and south and the Coast Range on the west. Both of these mountain ranges were initially formed by uplifts that occurred during the Jurassic and Cretaceous periods of geologic time (greater than 65 million years ago). Renewed uplift began in the Sierra Nevada during the Tertiary time, and is continuing today. The trough that underlies the valley is asymmetrical, with the greatest depths of sediments near the western margin. The sediments that fill the trough originated as erosion material from the adjacent mountains and foothills.

3.3 EARTH MATERIALS

The following description provides a general summary of the subsurface conditions encountered during the field exploration and further validated by the laboratory testing program. For a more thorough description of the actual conditions encountered at the specific boring location, refer to the Log of Test Borings drawing presented in Figures 2 and 3. The soils encountered were classified according to the Unified Soil Classification System (ASTM D2487).

The upper natural earth material consists of Holocene age Great Valley basin deposits. Typical to these deposits, the upper soils are laterally discontinuous. Material at the abutments consists of clayey sand and sandy lean clay with laterally discontinuous poorly graded and silty sand layers. Below an elevation of 135 to 130 feet, poorly graded sands were encountered to the depth explored, about elevation 80. Material encountered at the boring near the bents consist of about 6 feet of poorly graded sand with silt underlain by silty sand to about elevation 137, sandy lean clay to about elevation 127, and poorly graded sand with laterally discontinuous layers of sandy lean clay and silt to the depth explored, elevation 67.

3.4 GROUNDWATER CONDITIONS

The California Department of Water Resources groundwater elevation contours from well data indicate the static groundwater elevation in the general project area is about elevation 150 feet. No free groundwater was encountered in any borings along West Manning Avenue at the two bridge sites. However, elevated moisture levels were detected.

Groundwater may be influenced by water in the James Bypass Canal, which contains water about 1 to 2 months of the year. It is understood the James Bypass West Channel is a flood channel that accepts excess flood water from the Kings River. Water in the channel is not likely to be sustained long enough to increase groundwater elevations sufficient to create buoyant effects, but it may be possible that a temporary perched water zone could develop in the upper 20 feet of soil below the channel bottom. The temporary perched water condition is anticipated to mound briefly below the channel bottom and dissipate as the perched water flows laterally away from the channel. Groundwater conditions at the site could change at some time in the future due to variations in rainfall, groundwater withdrawal, construction activities, channel flows, and/or other factors not apparent at the time the test borings were made.

3.5 CHANNEL SCOUR/DEGRADATION

Bridge maintenance reports indicate about 5 feet of channel degradation from 1956 to the present. Borings within the channel indicate a depth of historic scour to be about 6 feet below existing channel bottom. The mean grain size (D_{50}) and 90 percent passing grain size (D_{90}) of the soil anticipated to be exposed in the channel is about 0.34 mm and 8.1 mm, respectively.

A review of the GP for the project indicated potential scour at the supports. A summary of the design scour is presented in Table 3.5-1.

TABLE 3.5-1
SCOUR DATA

Support No.	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)
Abutment 1	N/A	7
Bent 2	151	7
Bent 3	151	7
Abutment 4	N/A	7

4 CORROSION EVALUATION

Two soil samples from Boring B-2 at depths of 0 to 5 feet and 20 to 21.5 feet and borings performed at the adjacent structure were tested to evaluate the soluble sulfate content, soluble chloride content, and pH and minimum resistivity. Specific test results are presented in Table 4.1-1.

Table 4.1-1
Corrosion Related Testing

Boring No.	Depth (ft)	Soluble Sulfate (mg/kg)	Soluble Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
B-2	0-5	199	35.8	9.6	4,200
B-2	20-21.5	3.2	15	8.2	41,500
B-5	0-5	15.7	24.1	8.2	10,500
B-6	30-31.5	7.3	21	8.6	17,100

Laboratory tests indicate the soluble sulfates, soluble chlorides, pH and resistivity are all outside the Caltrans threshold limits for corrosive soils. As such, the site may be considered as a non-corrosive environment with respect to steel and concrete foundations. Corrosion is dependent upon a complex variety of conditions, which are beyond the geotechnical practice. Consequently, a qualified corrosion engineer should be consulted if the owner desires more specific recommendations and material types and/or mitigation.

5 SEISMIC RECOMMENDATIONS

5.1 LOCAL FAULTING

There are no known faults, which cut through the local soil at the site. The project site is not located in an Alquist-Priolo Earthquake Fault Zone, as defined by Special Publication 42 (revised 2007) published by the California Geologic Survey (CGS). Numerous faults and shear zones within the region could influence the project site.

5.2 SEISMIC DESIGN CRITERIA

Seismic design parameters were developed in accordance with the Caltrans Seismic Design Criteria Version 1.7.

The project site is located in a region with the potential for relatively moderate seismic activity. The more significant faults that could influence the project site include the San Andreas (Creeping Section) (Fault ID No. 182), the Great Valley 13 (Coalinga) (Fault ID No. 205), and the San Andreas (Parkfield) (Fault ID No. 214). According to the Caltrans fault database, the San Andreas (Creeping Section and Parkfield) is a strike slip fault with a dip angle of 90 degrees and assigned a Maximum Magnitude (M_{Max}) of 7.9, and the Great Valley 13 (Coalinga) is reverse fault with a dip angle of 15 degrees toward the west and assigned a Maximum Magnitude (M_{Max}) of 7.0.

Based on the boring data, the site can be classified as Soil Profile Type D. A V_{s30} of 266 m/s was used for the evaluation. The site is not located within a California deep soil basin region, as defined by Caltrans, so $Z_{1.0}$ and $Z_{2.5}$ were considered not applicable. Site characteristics and governing deterministic faults are summarized in Table 5.2-1.

Table 5.2-1
Site Characteristics And
Governing Deterministic Faults Parameters

Site Coordinates	Lat = 36.603685 deg, Long = -120.130515 deg
Shear Wave Velocity	266 m/s
Depth to V_s=1.0 km/s, $Z_{1.0}$	N/A
Depth to V_s=2.5 km/s, $Z_{2.5}$	N/A
Fault Name and ID Number	**San Andreas (Creeping section), No. 182**
Maximum Magnitude (M_{Max})	7.9
Fault Type	Strike Slip
Fault Dip	90 degrees
Dip Direction	Vertical
Bottom of Rupture Plane	12.00 km
Top of Rupture Plane (Z_{tor})	0.00 km
R_{RUP}[1]	73.38 km
R_{jB}[2]	73.38 km
R_X[3]	73.38 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	0
Fault Name and ID Number	**Great Valley (Coalinga), No. 205**
Maximum Magnitude (M_{Max})	7.0
Fault Type	Reverse
Fault Dip	15 degrees
Dip Direction	West
Bottom of Rupture Plane	15.30 km
Top of Rupture Plane (Z_{tor})	9.10 km
R_{RUP}[1]	35.59 km
R_{jB}[2]	34.41 km
R_X[3]	33.90 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	1
Fault Name and ID Number	**San Andreas (Parkfield), No. 214**
Maximum Magnitude (M_{Max})	7.9
Fault Type	Strike Slip
Fault Dip	90 degrees
Dip Direction	Vertical
Bottom of Rupture Plane	6.00 km
Top of Rupture Plane (Z_{tor})	0.00 km
R_{RUP}[1]	80.12 km
R_{jB}[2]	80.12 km
R_X[3]	75.12 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	0

Notes: [1]R_{RUP} = Closest distance from the site to the fault rupture plane.
[2]R_{JB} = Joyner-Boore distance; the shortest horizontal distance to the surface projection of the rupture area.
[3]R_X = Horizontal distance from the site to the fault trace or surface projection of the top of the rupture plane.

5.2.1 Deterministic Response Spectrum

The deterministic response spectrum was developed using ARS Online. The deterministic response spectrum from the Minimum Spectrum for California governed.

5.2.2 Probabilistic Response Spectrum

The probabilistic response spectrum was developed using ARS Online.

5.2.3 Design Response Spectrum

The upper envelope of the deterministic and probabilistic spectral values determines the design response spectrum. The probabilistic response spectra were found to govern for all periods. The recommended acceleration and displacement design response spectra are presented graphically and numerically in Appendix B.

5.2.4 References

Caltrans. Caltrans ARS Online, http://dap3.dot.ca.gov/ARS_Online

Caltrans. Geotechnical Services Manual.

Caltrans. Seismic Design Criteria, Appendix B Design Spectrum

Caltrans. Website http://dap3.dot.ca.gov/shake_stable/v2/technical.php

5.3 LIQUEFACTION AND SEISMIC SETTLEMENT

In order for liquefaction of soils due to ground shaking to occur, it is generally accepted that four conditions will exist:

- The subsurface soils are in a relatively loose state,
- The soils are saturated,
- The soils are non-plastic, and
- Ground motion is of sufficient intensity to act as a triggering mechanism.

Based on the ARS curve, the design Peak Horizontal Ground Acceleration (PHGA) is 0.34g, with a Moment Magnitude of 6.5. The potential for liquefaction was evaluated using Youd et. al. The depth of ground water would preclude the occurrence of liquefaction.

Dynamic compaction, or seismic settlement, can occur in unsaturated, loose granular soils or in poorly-compacted fill soils. The potential seismic induced settlement of unsaturated sand sediments at this site was evaluated using data from the current borings and the methodology described by Tokimatsu and Seed (1987). The results of settlement calculations indicated that the estimated seismic settlement in the unsaturated soil is estimated to be about ½ to 1¾ inches as a result of a magnitude 6.5 earthquake. This potential seismic settlement may induce downdrag on piles.

6 FOUNDATION RECOMMENDATIONS

6.1 GENERAL

Based on the field exploration, laboratory testing, and geotechnical analyses, the soils at the site are suitable for supporting the planned bridge structure. Due to the scour potential, loading conditions and other constraints, pile foundations were considered appropriate. Driven piles have been precluded as an option due to the presence of a nearby underground utility that could be damaged by vibrations from pile driving. Therefore, Cast-In-Drilled-Hole (CIDH) piles are considered a suitable alternative.

It is understood designers are planning complete replacement of the existing structure. The replacement process will utilize two, 48-inch CIDH piles at each abutment and bent. The following sections discuss conclusions and recommendations to support design of the CIDH pile foundations.

6.2 PILE FOUNDATIONS

6.2.1 Axial Capacity

Table 6.2-1 provides the unfactored downdrag load on the 48-inch diameter piles due to seismic dry settlement of the upper 14 feet of the abutments soil profile and the upper 11 feet of the bents soil profile. Table 6.2-2 provides the recommended design and specified tip elevations for the abutments and bents and includes the downdrag. Piles should be spaced at a minimum distance of 3 pile diameters center to center. Reduced axial capacities would be applicable for piles spaced closer than 3 pile diameters.

Table 6.2-1

Seismically Induced Downdrag

Support	Unfactored Nominal Downdrag on 48-inch CIDH Pile (kips)
Abutment 1	52
Bents 2 & 3	78
Abutment 4	46

Table 6.2-2

Foundation Recommendations for Abutments and Bents

Support	Pile	Cut-off Elev. (ft)	Service Limit State Max. Load (kips) per Pile	S_P	Required Factored Nominal Resistance per Pile (kips)				Design Tip Elev.[1] (ft)	Specified Tip Elev.[2] (ft)
					Strength Limit		Extreme Event			
					Comp. (ϕ=0.7)	Tens. (ϕ=0.7)	Comp (ϕ=1)	Tens (ϕ=1)		
Abut 1	48" CIDH	173.1	540	1"	800	0	600	0	138 (a) 136 (a-I) 147 (a-II), 149 (c), TBD (d)[4]	136
Bent 2	48" CIDH	159.0	780	1"	1200	0	810	0	95 (a) 92 (a-I) 116 (a-II), 117 (c), TBD (d)	92
Bent 3	48" CIDH	161.0	780	1"	1200	0	810	0	95 (a) 92 (a-I) 116 (a-II), 117 (c), TBD (d)	92
Abut 4	48" CIDH	174.7	540	1"	800	0	600	0	129 (a) 127 (a-I) 145 (a-II), 146 (c), TBD (d)	127

Notes: (1) Design tip elevations are controlled by: (a) Compression (Service Limit), (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load.
(2) The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.
(3) The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the potentially liquefied soil layers, which do not contribute to the design resistance.
(4) Design tip elevation for lateral loading to be provided by Cornerstone.

6.2.2 Foundation Construction Considerations

Pile borings will primarily expose relatively clean to silty sand, which will be susceptible to caving. Construction of the CIDH piles should utilize a casing oscillator, permanent or temporary casing, or slurry assisted drilling techniques. If permanent casing is used, Special Provisions should require the casing be tight to the soil (similar to Caltrans Standard Specification for temporary casing).

6.2.3 Lateral Capacity

The lateral response of pile foundations can be evaluated using LPILE Plus Version 5.0, or greater, for Windows (computer software developed by Ensoft Inc.). The geotechnical parameters summarized in Tables 6.2-3 and 6.2-4 can be used for evaluation of lateral loading of piles at abutments and bents, respectively.

Table 6.2-3

LPILE Parameters, Abutments

Elevation [1] (feet)	Recommended P-Y Curve	γ (pci)	ϕ (degree)	c (psi)	k (pci)	ε_{50}
184 – 167	Silt (Cemented c-phi Soil)	0.071	31	1.04	300	0.005
167 – 158	Sand (Reese)	0.071	38	--	300	--
158 – 140	Silt (Cemented c-phi Soil)	0.074	34	1.04	300	0.005
140 – 128	Silt (Cemented c-phi Soil)	0.073	30	3.47	200	0.005
Below 128	Sand (Reese)	0.072	36	--	230	--

Notes: [1] Assumes Elevation 184 for bridge deck

Table 6.2-4

LPILE Parameters, Bents

Elevation [1] (feet)	Recommended P-Y Curve	γ (pci)	ϕ (degree)	c (psi)	k (pci)	ε_{50}
158 – 140	Silt (Cemented c-phi Soil)	0.064	34	1.04	310	0.005
140 – 128	Silt (Cemented c-phi Soil)	0.068	30	3.47	200	0.005
Below 128	Sand (Reese)	0.072	36	--	330	--

Notes: [1] Assumes Elevation 158 for channel bottom

When considering the lateral capacity of a pile group, it will be necessary to reduce the single pile capacity of trailing piles. The reduction in capacity due to the effects of shaft interaction will be dependent upon the center-to-center (CTC) pile spacing. It is recommended that the capacity of individual trailing piles in a laterally loaded group be reduced according to the data in Table 6.2-5.

Table 6.2-5

Group Affect for Laterally Loaded Pile

CTC Spacing (In-line Loading)	Ratio of Lateral Resistance of Trailing Pile in Group to Isolated Single Pile
3B	0.6
4B	0.8
5B	1.0

Note: B is pile width

If desired, a more precise lateral group effect can be evaluated based on final geometry.

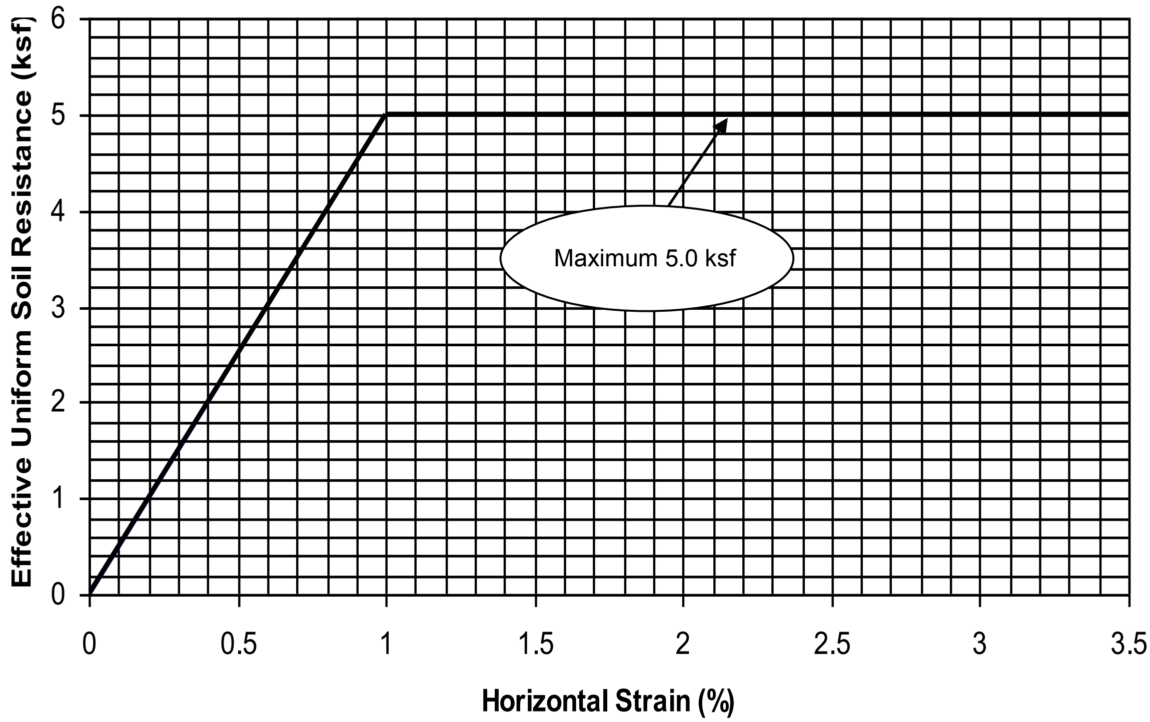
6.3 DYNAMIC LOADING

6.3.1 Abutment Dynamic Lateral Resistance

For backfill at abutments constructed in accordance with applicable provisions of the Caltrans Standard Specifications (2015), an initial abutment soil stiffness of 50 kip/in/ft is recommended. The ultimate lateral resistance that may be applied against abutments to resist seismic loading will be dependent on the deflection that occurs (which mobilizes shear resistance in the soil). Figure 6.3-1 presents the ultimate equivalent uniform lateral soil resistance as a function of horizontal strain (deflection/height) for the abutments. The maximum resistance for strain in excess of 1.0 percent is 5.0 ksf, when the height of the wall that is buried below the horizontal ground surface is equal to, or greater than, 5.5 feet. When the abutment height is less than 5.5 feet, the maximum equivalent uniform lateral soil resistance shall be reduced proportionately by H/5.5, where H is the endwall height in feet.

Figure 6.3-1

Unfactored Nominal Lateral Bearing for Seismic Loading at Abutments



6.4 LATERAL EARTH PRESSURES

Table 6.4-1 provides the lateral earth pressures against retaining walls. The recommended values do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be adequately drained. Data are presented for active, braced, and at-rest conditions for walls supporting level ground surface. The values consider the anticipated strength of backfill and the performance of existing structures. Lateral earth pressures are strain related. The active pressure would be applicable to walls capable of rotating 0.0005 radians. The at-rest pressures are applicable to walls fully fixed against translation or rotation. The at-rest pressures include the Jaky solution for normally consolidated soil plus consideration for the locked-in pressure associated with the pre-stressing due to backfill compaction (over-consolidation).

Table 6.4-1

Retaining Wall Parameters

Condition	Earth Pressures
Active	35 psf/ft
Braced[1]	23H psf
At Rest	85 psf/ft
Dynamic Increments	13 psf/ft

Note: [1] H is the wall height in feet.

The above values are less than those used for Caltrans Standard Plan walls. Therefore, Standard Plan retaining walls could be used.

6.5 EARTHWORK

In general, any required fill or backfill should be constructed in accordance with the latest revisions of Caltrans Standard Specifications. It is anticipated minor fills may be associated with the project, and are anticipated to not have significant post-construction settlement.

6.6 FLEXIBLE PAVEMENT

Soil samples were obtained along Manning Avenue alignment at the two bridge sites (Bridge Nos. 42C-0691 and 42C-0692). The site subgrade soil consists of sandy lean clays and clayey sands at B-3 and B-4 with R-values of 8 and 6, respectively. A design R-value of 5 was used for West Manning Avenue. Table 6.6-1 provides optional conventional flexible pavement sections for Traffic Indexes of 8.5 and 9.0, based on an anticipated daily traffic provided by Cornerstone. Analysis is based on Caltrans procedures.

Table 6.6-1

Conventional Flexible Pavement Sections

(Based on Subgrade R-Value = 5)

Traffic Index	Optional Sections (feet)	
	2-Layer Section	3-Layer Section
8.5	0.40 HMA / 1.65 AB	0.40 HMA / 0.55 AB / 1.20 ASB
9.0	0.45 HMA / 1.70 AB	0.45 HMA / 0.55 AB / 1.30 ASB

Table 6.6-2 provides overlay recommendations for existing pavement. Sections are based on the anticipated R-Value of existing subgrade soil, the furnished Traffic Index (TI) for the road segment, and a 10-year and 20-year design life. It is understood the County of Fresno is planning to mill and overlay the existing pavement to match previous overlays west and east of the project site. The proposed overlay thickness is anticipated to be 0.45 feet. Considering the R-value and design TI, the proposed overlay thickness is not anticipated to provide a 20-year design life, but is anticipated to have a similar design life to other portions of Manning Avenue that have recently had similar overlays.

Table 6.6-2

Recommended Design Pavement Sections

(Based on Subgrade R-Value = 5)

Traffic Index	Minimum Overlay (feet)	
	10-year Design	20-year Design
8.5	0.85 HMA	0.95 HMA
9.0	0.95 HMA	1.00 HMA

Hot mix asphalt (HMA), Class 2 aggregate base (AB) and Class 2 aggregate subbase (AS) should conform to, and be placed in accordance with, the latest revision of Caltrans Standard Specifications. Considering the clayey nature of the subgrade, it is recommended subgrade be scarified to a depth of 12 inches, moisture conditioned to 2 percent above optimum moisture and compacted to at least 90 percent, but not more than 95 percent of maximum density. It is recommended the maximum density for subgrade be based on ASTM D1557.

Alternatively, full depth reclamation with cement (FDR-C) of subgrade and HMA could be considered. A minimum FDR-C section of 18 inches would be recommended. If FDR-C is chosen as the design alternative, a mix design can be prepared. For preliminary purposes, a portland cement content of 5 to 7 percent amendment is typical in sandy lean clay materials to result in a design unconfined compressive strength of 400 psi. The minimum HMA thickness would be 0.35 and 0.40 feet for TIs of 8.5 and 9, respectively. The "weak link" of the section is the HMA wearing surface, which is the easiest and most economical layer to maintain, repair, or rehabilitate to meet future needs. The full depth thickness, versus the minimum thickness, is less vulnerable to brittle fatigue or shrinkage cracking. The use of 1.5 feet of cement treated subgrade (CTS) will result in the overall HMA/CTS being suitable for a TI of 9.7 and 10.2, with the HMA surface being the only element requiring future rehabilitation. Traffic can be diverted onto the compacted full depth CTS after finish grading. With the minimum CTS thickness, automobile traffic could be placed on the subgrade after finish grading. Care must be exercised to not over stress the minimum CTS with truck traffic until the first HMA lift is placed.

Lime treatment could be considered to reduce aggregate base thickness for asphalt pavement sections. Previous work performed in the area has indicated that lime treatment using an engineered process, which includes testing for soil chemistry, necessary lime content and R-Value, and development of QC/QA procedures, can successfully improve the stability of subgrade materials. If lime treatment is desired, the additional testing and recommendations can be provided.

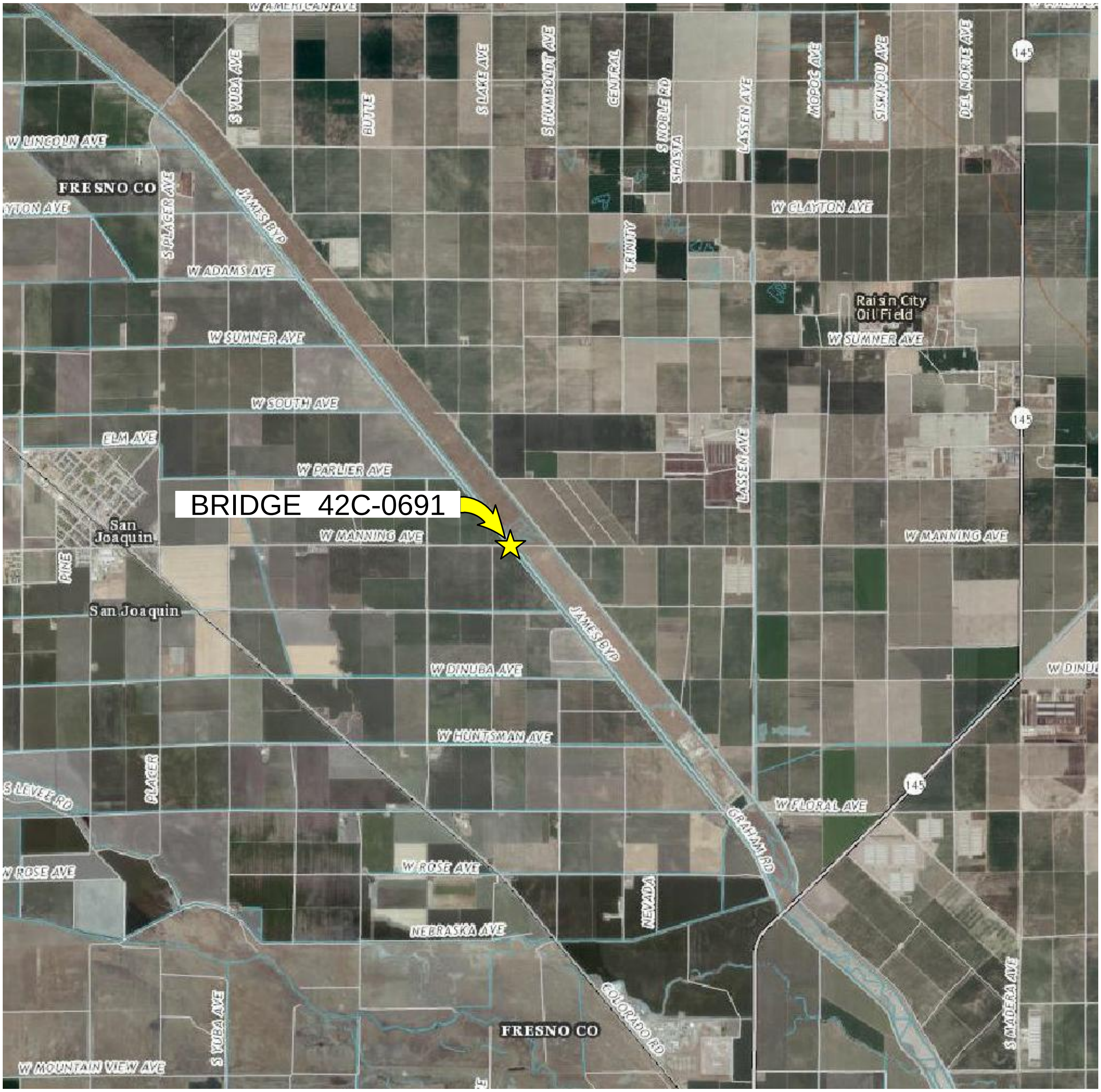
7 CLOSURE

The conclusions and recommendations in this report are for the design of the proposed West Manning Avenue Bridge (Bridge No. 42C-0691) replacement across the James Bypass West Channel, located in Fresno County, California, as described in the text of this report. The findings, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No warranty, express or implied, is made. The field exploration program and this report were based on the proposed project information provided to Kleinfelder. If any change (i.e., structure type, location, etc.) is implemented which materially alters the project, additional geotechnical services may be required, which could include revisions to the recommendations given herein.

This report is intended for use by Cornerstone Structural Engineering Group, County of Fresno, and their subconsultants, within a reasonable time from its issuance. Noncompliance with the recommendations of the report or misuse of the report will release Kleinfelder from any liability.

The scope of the geotechnical services did not include an environmental site assessment for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater or atmosphere, or the presence of wetlands.

FIGURES



SOURCE: U.S.G.S. / THE NATIONAL MAP, AERIAL SERIES, SAN JOAQUIN, CALIFORNIA, DATED 4/2014.

N

6,000 3,000 0 6,000

APPROXIMATE SCALE (feet)

CAD FILE: L:CADD\2015\20161220\JamesBypassBridge-ManningAve_7-2015\EasternBridge\ LAYOUT: F1 PLOTTED: 15 Jul 2015, 9:33pm, MGriffin

Images: USGS_TheNationalMap_Aerial_4-mil_3-2015_SanJoaquin-CA.JPG

ATTACHED IMAGES:
ATTACHED XREFS:
LONG BEACH, CA

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PROJECT NO.	20161220
DRAWN:	7/2015
DRAWN BY:	MRG
CHECKED BY:	NP
FILE NAME:	20161220-fig1_SVM-EBrg.dwg

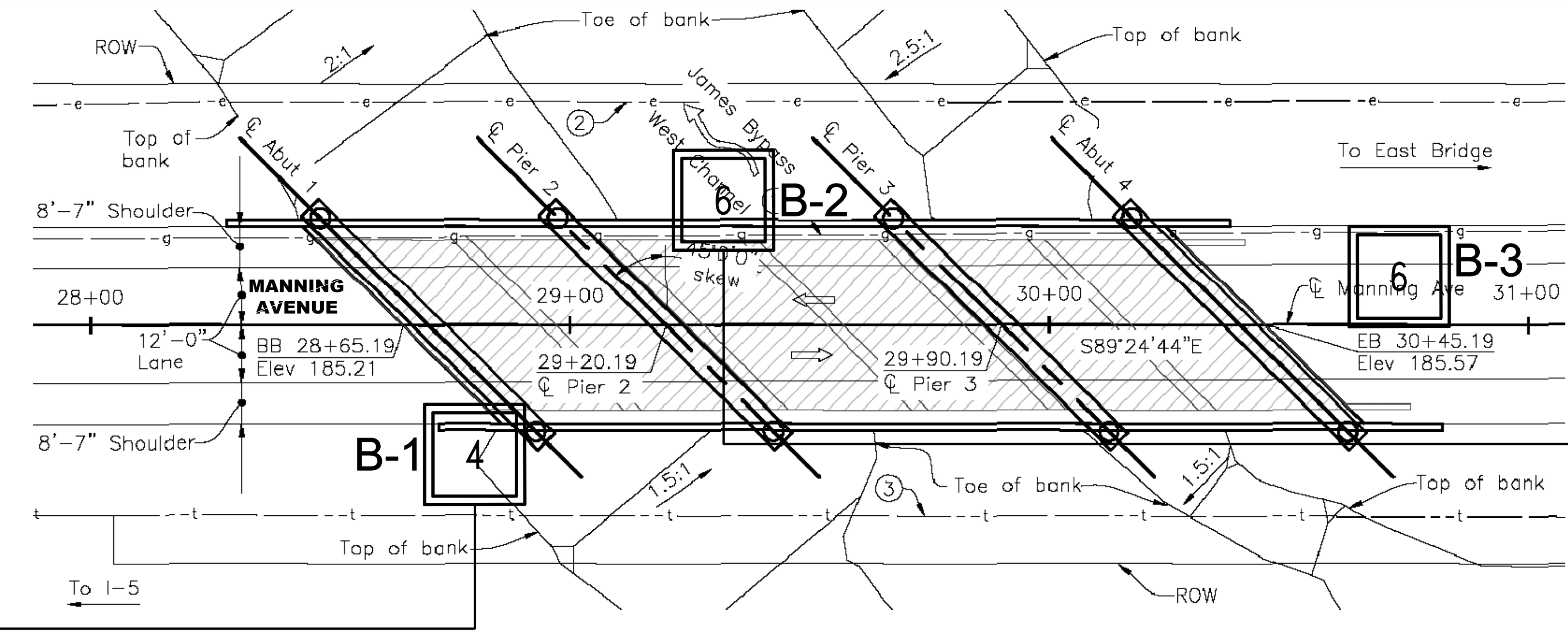
SITE VICINITY MAP

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0691
SAN JOAQUIN, FRESNO COUNTY, CALIFORNIA

FIGURE
1

NOTES:

1. 1.5-INCH DIAMETER SAMPLES WERE TAKEN USING A STANDARD PENETRATION TEST (SPT) SPLIT BARREL SAMPLER WITH AN INSIDE DIAMETER (ID) OF 1.5 INCHES AND AN OUTSIDE DIAMETER (OD) OF 2.0 INCHES.
2. 2.5-INCH DIAMETER RING SAMPLES WERE TAKEN USING A CALIFORNIA SPLIT BARREL SAMPLER WITH AN ID OF 2.5 INCHES AND AN OD OF 3.0 INCHES.
3. ALL DRIVE SAMPLES WERE DRIVEN WITH 140 LB HAMMER WITH A FALLING HEIGHT OF 30 INCHES.
4. ELEVATION BASED ON PLANS FROM QUINCY ENGINEERING, DATED 11/4/2013.



PLAN

SCALE: 1"=30'

PROFILE

SCALE: 1"=10' HORIZONTAL
SCALE: 1"=5' VERTICAL

MANNING AVENUE (WEST BRIDGE)

28+50 29+00 29+50

90% SUBMITTAL - NOT FOR CONSTRUCTION

ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

FIELD INVESTIGATION	T. DeSOUZA	DATE	As Shown
DRAWN	D. FAHRNEY		–
CHECKED	N. POPENOE		–
REVISION			

Scale: AS SHOWN

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3731 W Ashcroft Ave
Fresno, California 93722
559.486.0750

FOR R/W DATA AND ACCURATE ACCESS DETERMINATION SEE R/W RECORDS AT PUBLIC WORKS

REGISTERED PROFESSIONAL ENGINEER STEPHEN P. PLASSON No. 2731 Exp. 09/30/2021 GEOTECHNICAL STATE OF CALIFORNIA

PROJECT
JAMES BYPASS WEST CHANNEL BRIDGE AT MANNING AVENUE

Road No. – Bridge No. 42C0691

THE COUNTY OF FRESNO 1856

DEPARTMENT OF PUBLIC WORKS & PLANNING

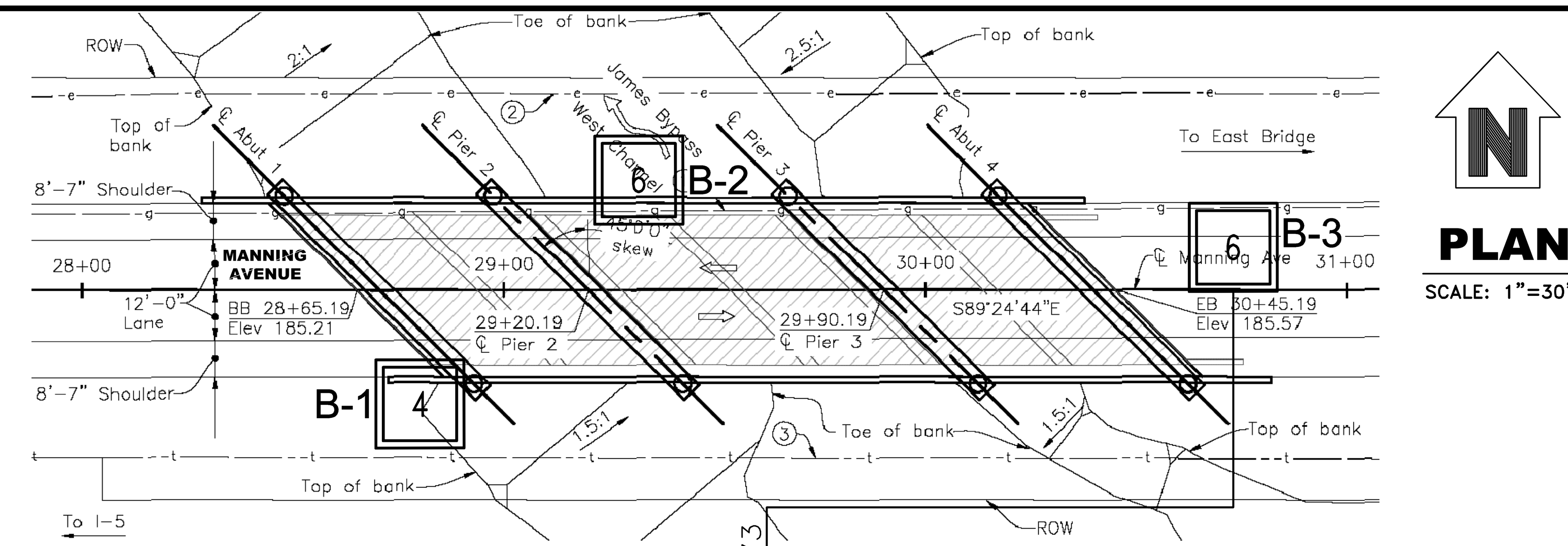
LOG OF TEST BORINGS
BR. No. 42C0691

Drawing No. S15 Sheet No. 24 of 37

U:\Projects\CADD\CADD 2016\20161220\20161220_LOTBs.dwg – 2/24/2017 10:46 AM

NOTES:

1. 1.5–INCH DIAMETER SAMPLES WERE TAKEN USING A STANDARD PENETRATION TEST (SPT) SPLIT BARREL SAMPLER WITH AN INSIDE DIAMETER (ID) OF 1.5 INCHES AND AN OUTSIDE DIAMETER (OD) OF 2.0 INCHES.
2. 2.5–INCH DIAMETER RING SAMPLES WERE TAKEN USING A CALIFORNIA SPLIT BARREL SAMPLER WITH AN ID OF 2.5 INCHES AND AN OD OF 3.0 INCHES.
3. ALL DRIVE SAMPLES WERE DRIVEN WITH 140 LB HAMMER WITH A FALLING HEIGHT OF 30 INCHES.
4. ELEVATION BASED ON PLANS FROM QUINCY ENGINEERING, DATED 11/4/2013.



90% SUBMITTAL - NOT FOR CONSTRUCTION

ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

FIELD INVESTIGATION T. DeSOUZA	DATE As Shown	Scale	
DRAWN D. FAHRNEY	–	AS SHOWN	
CHECKED N. POPENOE	–		
REVISION			

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Fresno, California 93722
559.486.0750

FOR R/W DATA AND ACCURATE ACCESS DETERMINATION SEE R/W RECORDS AT PUBLIC WORKS

REGISTERED PROFESSIONAL ENGINEER STEPHEN P. PLAISON No. 2731 Exp. 09/30/2021 GEOTECHNICAL STATE OF CALIFORNIA

PROJECT

JAMES BYPASS WEST CHANNEL BRIDGE AT MANNING AVENUE

Road No. – Bridge No. 42C0691

THE COUNTY OF FRESNO 1856

DEPARTMENT OF PUBLIC WORKS & PLANNING

LOG OF TEST BORING
BR. No. 42C0691

Drawing No. S16 Sheet No. 25 of 37

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APPENDIX A

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	0.0 - 5.0	CLAYEY SAND (SC)					48				
B-1	5.0	CLAYEY SAND (SC)	19.2	104.4							**Direct Shear=**
											Peak Cohesion: 150 psf
											Peak Friction Angle: 33.0°
											20% Stress Cohesion: 150 psf
											20% Stress Friction Angle: 33.0°
B-1	10.0	CLAYEY SAND (SC)	18.5	111.3							
B-1	20.0	POORLY-GRADED SAND (SP)	3.8	109.5							
B-1	25.0	CLAYEY SAND (SC)					27				
B-1	30.0	CLAYEY SAND (SC)	22.9	105.7							
B-1	40.0	SILTY SAND (SM)	9.9	108.7							
B-1	50.0	CLAYEY SAND (SC)	20.0	109.9							
B-1	55.0	POORLY-GRADED SAND (SP)	28.1	101.3							
B-2	0.0 - 5.0	POORLY-GRADED SAND WITH SILT (SP-SM)				100	9.3				**pH=** 9.6
											Resistivity= 4200
											Sulfates= 199
											Chlorides= 35.8
B-2	5.0	POORLY-GRADED SAND WITH SILT (SP-SM)	1.7	102.3							
B-2	10.0	SILTY SAND (SM)					18				
B-2	15.0	SILTY SAND (SM)	6.1	103.3							
B-2	20.0	SILTY SAND (SM)									**pH=** 8.2
											Resistivity= 41500
											Sulfates= 3.2
											Chlorides= 15
B-2	25.0	SANDY LEAN CLAY (CL)	3.8	112.2				29	16	13	
B-2	35.0	POORLY-GRADED SAND (SP)	30.7	95.0							**Direct Shear=**
											Peak Cohesion: 0 psf
											Peak Friction Angle: 35.0°

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic

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PROJECT NO.: 20161220
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CHECKED BY:
DATE:
REVISED: -

LABORATORY TEST RESULT SUMMARY

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0691
SAN JOAQUIN, FRESNO COUNTY, CA

FIGURE
A-1

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
											20% Stress Cohesion: 550 psf
											20% Stress Friction Angle: 24.0°
B-3	0.33	SANDY LEAN CLAY (CL)									**R-Value=** 8
B-3	5.0	CLAYEY SAND (SC)	12.8	120.8							
B-3	15.0	CLAYEY SAND (SC)	10.7	111.2							
B-3	20.0	POORLY-GRADED SAND (SP)					5.6				
B-3	25.0	POORLY-GRADED SAND (SP)	19.5	109.7							
B-3	35.0	SILTY SAND (SM)	9.7	112.1							
B-3	45.0	CLAYEY SAND (SC)	28.3	95.6							

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic

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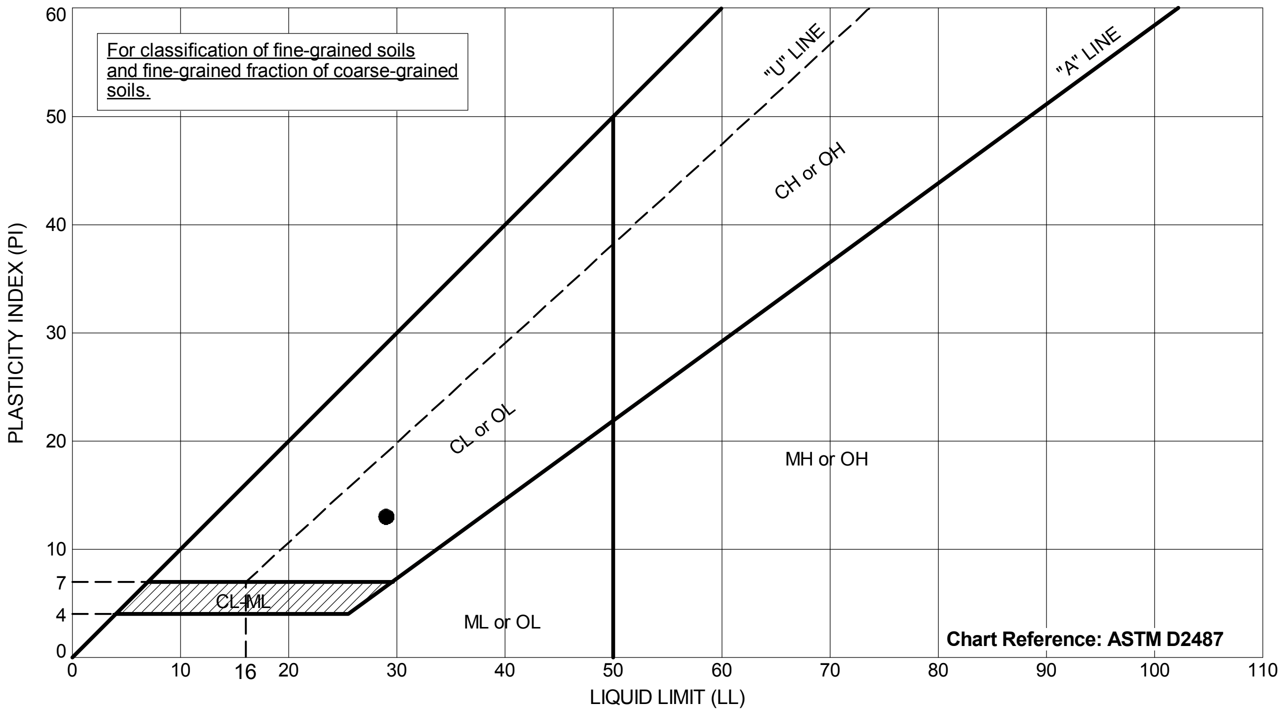
PROJECT NO.: 20161220
DRAWN BY:
CHECKED BY:
DATE:
REVISED: -

LABORATORY TEST
RESULT SUMMARY

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0691
SAN JOAQUIN, FRESNO COUNTY, CA

FIGURE
A-2



	Exploration ID	Depth (ft.)	Sample Description	Passing #200	LL	PL	PI
●	B-2	25	SANDY LEAN CLAY (CL)	NM	29	16	13

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NM = Not Measured

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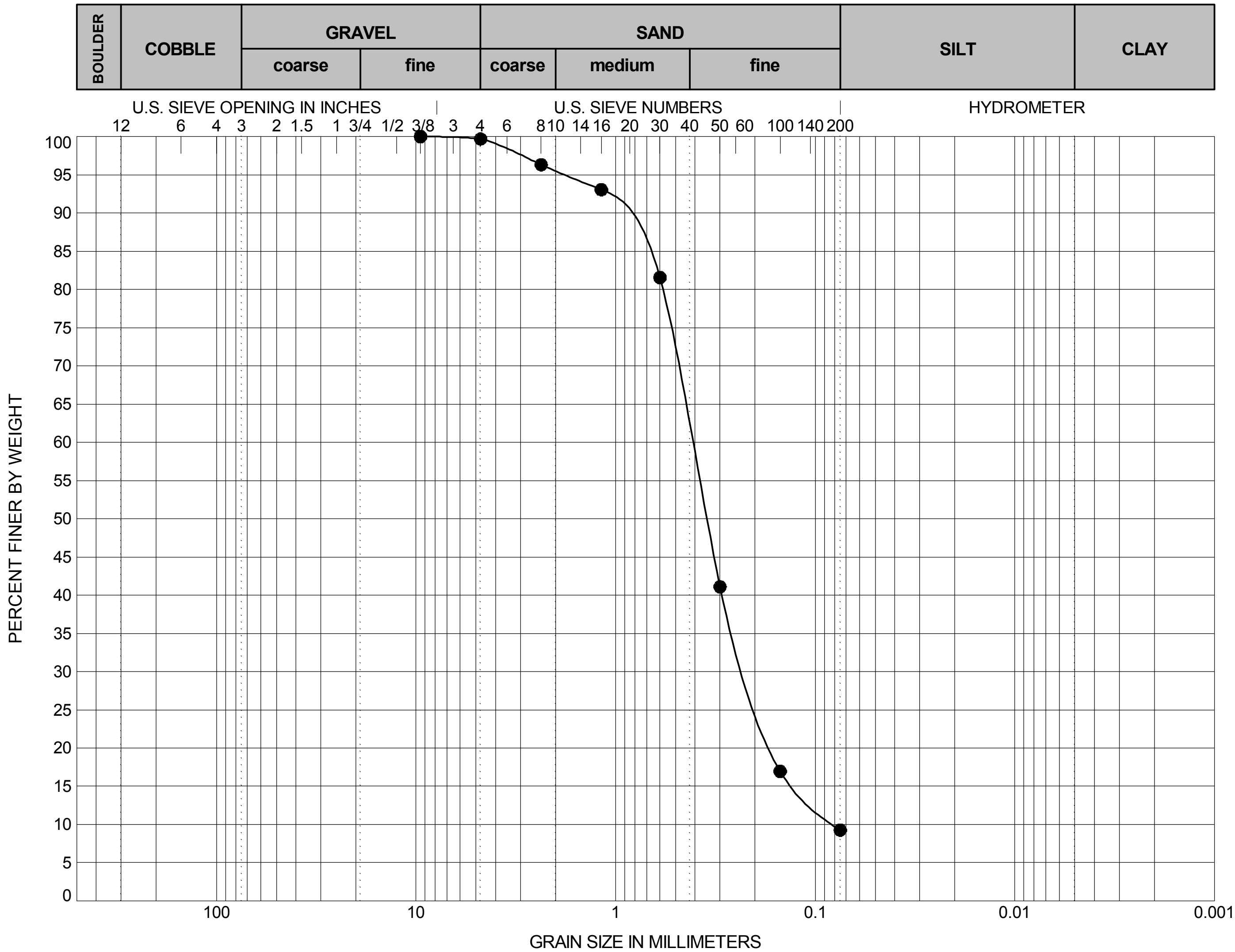
PROJECT NO.: 20161220	ATTERBERG LIMITS	FIGURE
DRAWN BY: TD		
CHECKED BY: NMP	MANNING AVENUE OVER JAMES BYPASS BRIDGE NO. 42C-0691 SAN JOAQUIN, FRESNO COUNTY, CA	A-3
DATE: 7/10/2015		
REVISED: -		



	Exploration ID	Depth (ft.)	Sample Description	LL	PL	PI
●	B-2	0 - 5	POORLY-GRADED SAND WITH SILT (SP-SM)	NM	NM	NM

	Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
●	B-2	0 - 5	9.5	0.415	0.218	0.08	1.43	5.18		100	9.3	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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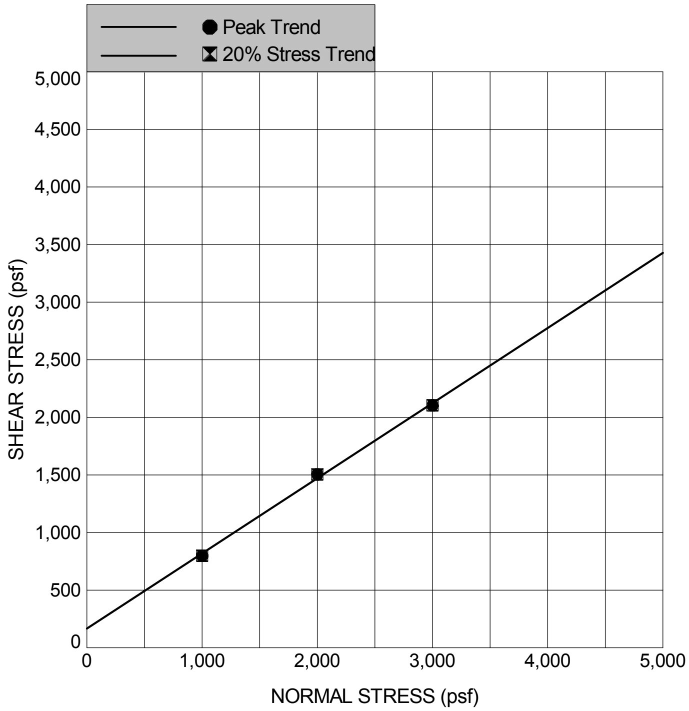
PROJECT NO.: 20161220	SIEVE ANALYSIS	FIGURE
DRAWN BY: TD		
CHECKED BY: NMP	MANNING AVENUE OVER JAMES BYPASS BRIDGE NO. 42C-0691 SAN JOAQUIN, FRESNO COUNTY, CA	A-4
DATE: 7/10/2015		
REVISED: -		



Exploration ID	Depth (ft.)	Sample Description
B-1	5	CLAYEY SAND (SC)

Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity
NM	NM	NM	NM	NM	

	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1	19.2	102.9			4.60	1.00
	2	19.2	99.6			4.60	1.00
	3	19.2	98.8			4.60	1.00

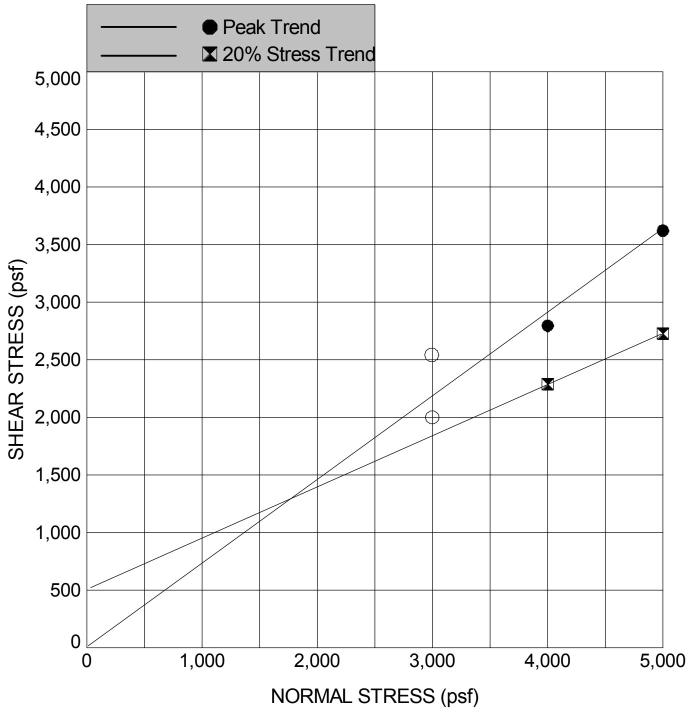
	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
At Test	1						
	2						
	3						

Specimen No.	Peak Shear Stress (psf)	20% Stress Shear Stress (psf)	Horizontal Displacement (in)	Normal Stress (psf)	Strain Rate (in/min)
1	799.9	799.9		1000	
2	1506	1506		2000	
3	2105	2105		3000	

Results	Cohesion (psf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	150	33	
20% Stress	150	33	

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NM = Not Measured

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20161220	DIRECT SHEAR	FIGURE
	DRAWN BY: TD		A-5
	CHECKED BY: NMP	MANNING AVENUE OVER JAMES BYPASS BRIDGE NO. 42C-0691 SAN JOAQUIN, FRESNO COUNTY, CA	
	DATE: 7/10/2015		
	REVISED: -		



TEST CONDITIONS: Cohesionless

Exploration ID	Depth (ft.)	Sample Description
B-2	35	POORLY-GRADED SAND (SP)

Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity
NM	NM	NM	NM	NM	

	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1	9.2	90.3			4.60	1.00
	2	9.2	89.4			4.60	1.00
	3	9.2	88.9			4.60	1.00

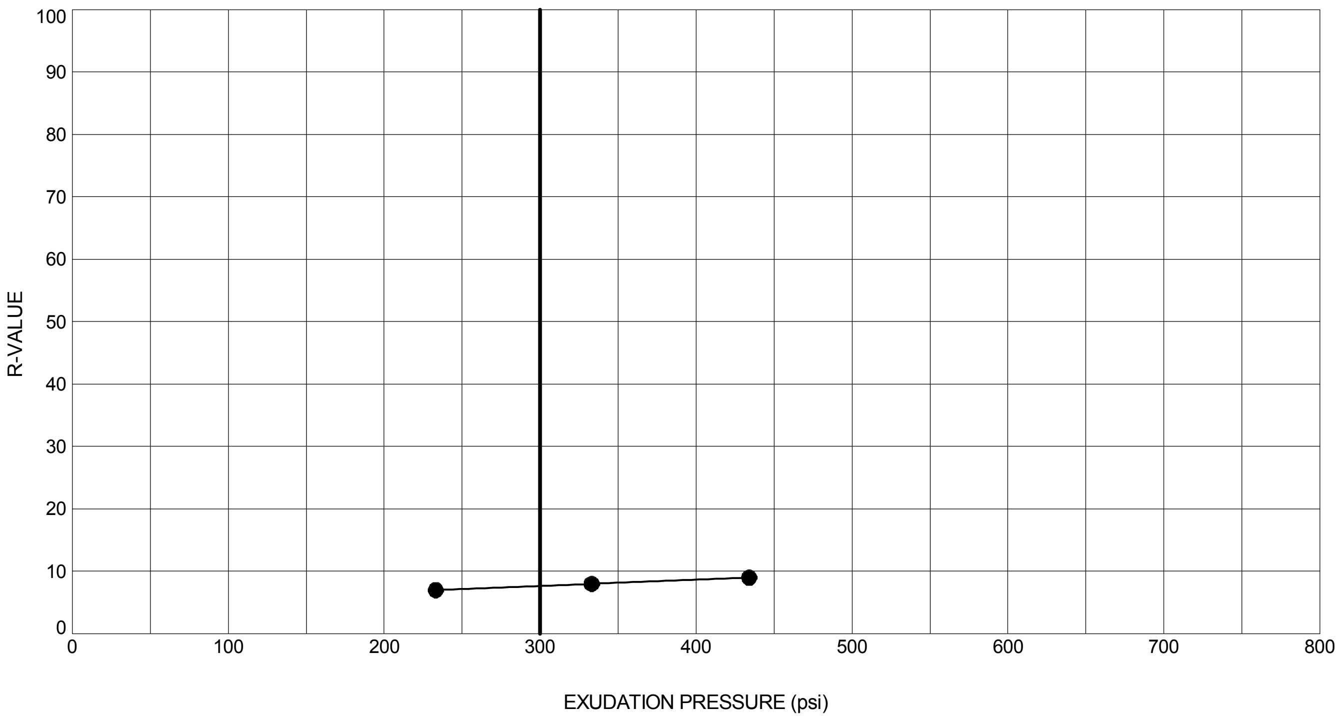
	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
At Test	1	30.6				4.60	1.00
	2	30.4				4.60	1.00
	3	29.8				4.60	1.00

Specimen No.	Peak Shear Stress (psf)	20% Stress Shear Stress (psf)	Horizontal Displacement (in)	Normal Stress (psf)	Strain Rate (in/min)
1	2573.7	1999.76	0.4900	3000	0.005
2	2796.2	2291.47	0.4900	4000	0.005
3	3621.11	2728.73	0.4900	5000	0.005

Results	Cohesion (psf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	0	35	
20% Stress	550	24	

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NM = Not Measured
◯ = Unused Data Points

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20161220 DRAWN BY: TD CHECKED BY: NMP DATE: 7/10/2015 REVISED: -	DIRECT SHEAR MANNING AVENUE OVER JAMES BYPASS BRIDGE NO. 42C-0691 SAN JOAQUIN, FRESNO COUNTY, CA	FIGURE A-6



Exploration ID	Depth (ft.)	Sample Description	R-Value @ 300 psi Exudation Pressure
B-3	0.33 - 5	SANDY LEAN CLAY (CL)	8

Specimen No.	Moisture at Time of Test (%)	Dry Unit Weight (pcf)	Expansion Pressure (psi)	Exudation Pressure (psi)	Corrected Resistance Value
1	13.7	118.1	0	434	9
2	14.6	115.8	0	333	8
3	16.0	113.0	0	233	7

Testing perfomed in general accordance with ASTM D2844.

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PROJECT NO.: 20161220		R-VALUE	FIGURE
DRAWN BY: TD			
CHECKED BY: NMP		MANNING AVENUE OVER JAMES BYPASS BRIDGE NO. 42C-0691 SAN JOAQUIN, FRESNO COUNTY, CA	A-7
DATE: 7/10/2015			
REVISED: -			

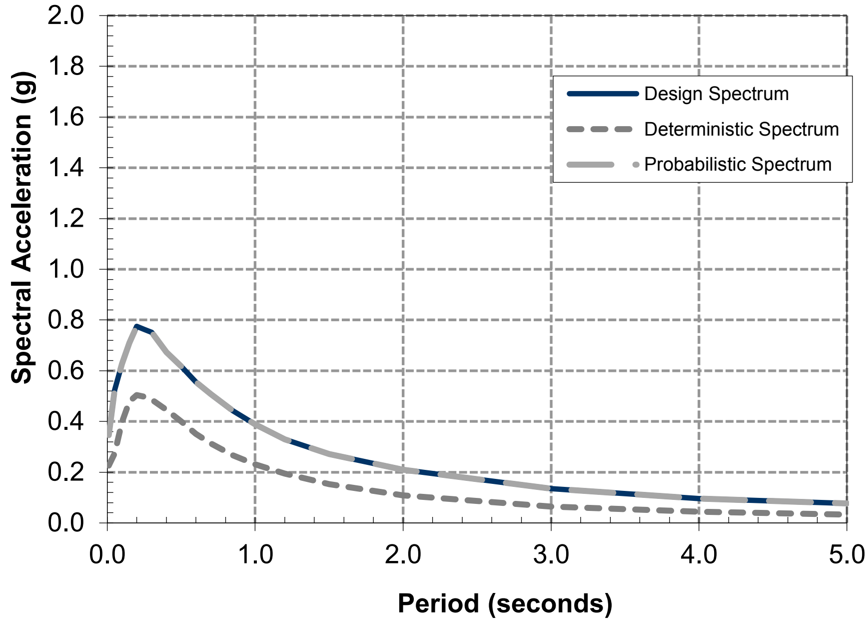
APPENDIX B

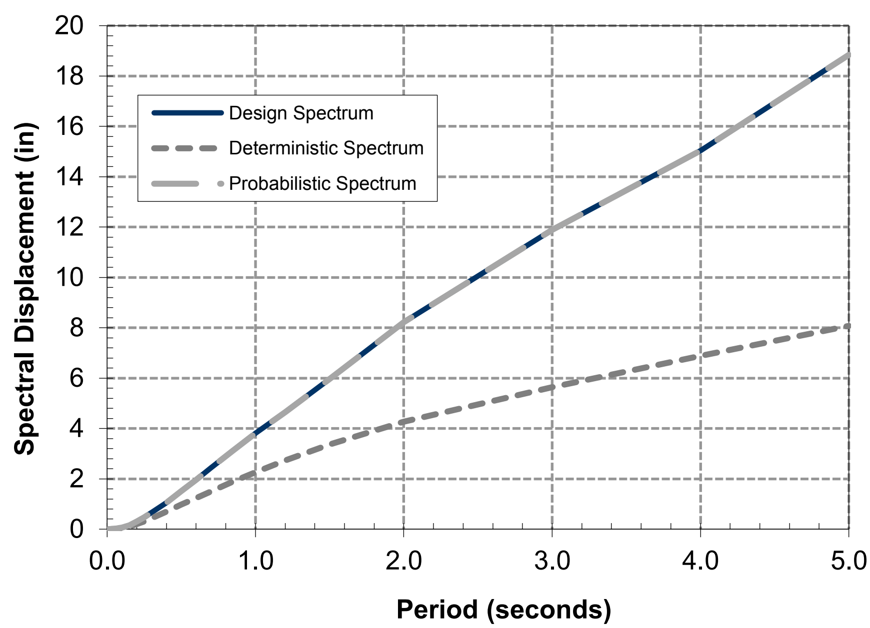
ARS CURVE

SITE DATA

Latitude:	36.6037	Shear Wave Velocity	266 m/s
Longitude:	-120.13051	Depth to Vs = 1.0 km/s:	N/A
		Depth to Vs = 2.5 km/s:	N/A

Period (s)	SA (g)	SD (in)
0.01 (PGA)	0.346	0.00
0.05	0.524	0.01
0.1	0.627	0.06
0.15	0.710	0.16
0.2	0.775	0.30
0.25	0.762	0.47
0.3	0.751	0.66
0.4	0.673	1.05
0.5	0.618	1.51
0.6	0.556	1.96
0.7	0.508	2.44
0.85	0.442	3.13
1	0.389	3.81
1.2	0.330	4.65
1.5	0.271	5.97
2	0.210	8.22
3	0.135	11.89
4	0.096	15.03
5	0.077	18.84





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PROJECT NO	20161220
DRAWN:	8/3/17
DRAWN BY:	TD
CHECKED BY:	NMP
FILE NAME:	

CALTRANS ARS CURVES

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0691
SAN JOAQUIN, FRESNO COUNTY, CALIFORNIA

FIGURE

B-1

August 4, 2017
Revised: July 17, 2020
Kleinfelder Project No.: 20161220.001A

Mr. Mark A. Weaver, MS, PE
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

SUBJECT: Foundation Report
W Manning Avenue over James Bypass West Channel
Bridge No. 42C-0692
San Joaquin, Fresno County, California

Dear Mr. Weaver:

The attached report presents the results of the geotechnical study for the West Manning Avenue Bridge (Bridge No. 42C-0692) over James Bypass East Channel located east of San Joaquin, in Fresno County, California. This report describes our study and provides conclusions and recommendations for use in foundation design and construction.

We appreciate the opportunity to provide geotechnical engineering services to Cornerstone Structural Engineering Group, the County of Fresno, and other project designers. We trust this information meets your current needs. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully Submitted,

KLEINFELDER, INC.

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 09/30/2021
GEOTECHNICAL
STATE OF CALIFORNIA

Adam AhTye, PE
Staff Professional II

Stephen P. Plauson, PE, GE
Principal Geotechnical Engineer

AA/SPP:ct

KLEINFELDER 3731 West Ashcroft Avenue, Fresno, CA 93722 **p|** 559-486-0750 **f|** 559-442-5081

FOUNDATION REPORT
W MANNING AVENUE OVER JAMES BYPASS EAST CHANNEL
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CALIFORNIA
KLEINFELDER PROJECT #20161220.001A

AUGUST 4, 2017
REVISED: JULY 17, 2020

A Report Prepared for:

Mr. Mark A. Weaver, MS, PE
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

FOUNDATION REPORT
W MANNING AVENUE OVER JAMES BYPASS WEST CHANNEL
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CALIFORNIA

Prepared by:

Adam AhTye, PE
Staff Professional II

Stephen P. Plauson, PE, GE
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 09/30/2021
GEOTECHNICAL
STATE OF CALIFORNIA

KLEINFELDER, INC.
3731 W Aschcroft Ave
Fresno, California 93722
(559) 486-0750

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1 INTRODUCTION

1.1 GENERAL

This report presents the results of the geotechnical investigation for the proposed replacement of West Manning Avenue Bridge (Bridge No.42C-0692) over James Bypass East Channel, located east of San Joaquin in Fresno County, California. The scope of services consisted of a field exploration program, laboratory testing, engineering analysis, and preparation of this written report. This report has been prepared in conjunction with the project 60 percent. The Foundation Report will be amended as design proceeds, with the final Foundation Report reflecting final design. A concurrent geotechnical study is also being performed for the West Manning Avenue Bridge (Bridge No. 42C-0691) over James Bypass West Channel located approximately 1,150 feet west of Bridge No. 42C-0692 and will be presented in a separate report.

1.2 SCOPE OF SERVICES

The purpose of the Foundation Report is to provide geotechnical recommendations and opinions to aid in project design. The report provides the following:

- A description of the proposed project;
- A summary of the field exploration and laboratory testing programs;
- Comments on the regional geology and site engineering seismology, including the recommended peak ground acceleration and Caltrans Seismic Design Criteria Version 1.7 ARS curve;
- Comments on liquefaction potential;
- Recommendations for pile foundations, including design and specified tip elevations;
- Recommended LPILE parameters for use in evaluating the pile response to lateral loads;
- Comments on initial soil stiffness and ultimate equivalent lateral pressure by Caltrans procedures for abutment end walls; and,

- Log of Test Borings drawing.

1.3 PROJECT DESCRIPTION

The existing structure is a 3-span reinforced concrete precast girder bridge. The structure is 36 feet 8 inches in width and 70 feet in length containing an asphalt concrete overlay. The span lengths center of bearing (c.o.b.) to c.o.b. are 30 feet between bents and 20 feet between abutments and adjacent bents.

Construction of the replacement bridge will involve complete replacement of the existing structure. The new structure will have a total width of 44 feet, with a single span length of 64.75 feet. The replacement process is anticipated to include two, 48-inch CIDH (Cast-In-Drilled-Hole) piles at each abutment. Construction is anticipated to be split into three phases. Piles, pile extensions, and cap beams would first be erected under the existing bridge, followed by the bridge's closure and demolition. Precast voided slab units would then be placed and paved with a polyester overlay.

Table 1.3-1

Bridge Replacement

Foundation Design Data

Support	Location (Sta. No.)[1]	Pile Type	Finished Grade Elev.[2] (ft)	Cut-off Elev. (ft)	Pile Cap Size (ft)		S_P[3]	No. Piles per Support
					B	L		
Abut 1	42+33.88	48 inch CIDH	183.49	174.49	5	50	2”	2
Abut 2	42+96.17	48 inch CIDH	183.33	174.33	5	50	2”	2

Notes: [1] Stations were based on layout drawings provided by Cornerstone Structural Engineering Group
[2] Elevations based on project datum
[3] Permissible settlement under service limit load

Table 1.3-2

Bridge Replacement

Foundation Design Data

Support	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)		Extreme Event Limit State (Controlling Group, kips)	
	Total Load		Permanent Load	Compression		Compression	
	Per Support	Max. per Pile	Per Support	Per Support	Max. per Pile	Per Support	Max. per Pile
Abutment 1	1020	540	740	1500	840	1060	570
Abutment 2	1020	540	740	1500	840	1060	570

1.4 POLICY EXCEPTIONS

No known exceptions to Caltrans policy were made in the geotechnical evaluation for the foundations for this project.

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION AND TESTING

The field exploration for the Manning Avenue Bridge No.42C-0692 replacement consisted of drilling 3 test borings on July 7 through July 9, 2015. The test borings were drilled with a CME 75, truck-mounted drill rig using hollow stem auger and hand auger techniques. Borings B-4 and B-6 were drilled to depths of approximately 61 feet below ground surface and Boring B-5 was drilled in the channel to approximately 5 feet below ground surface. The approximate location of the test borings are shown on the Log of Test Borings drawing in Figure 2 of this report.

The earth materials encountered in the test borings were visually classified in the field and a continuous log was recorded. In-place samples of soil units were collected from the test boring at selected depths by driving a 2.5-inch I.D. split barrel sampler containing brass liners into the undisturbed soil with a 140-pound automatic safety hammer free falling a distance of 30-inches. In addition, an ASTM D1586 standard penetrometer without liners (barrel I.D. of 1.5 inches) was driven 18-inches in the same manner. This latter sampling procedure generally conformed to the ASTM D1586 test procedure. Resistance to sampler penetration over the last 12-inches is noted on the Log of Test Borings as the "Penetration Index". The penetration indices listed on the Log of Test Borings have not been corrected for the effects of overburden pressure, sampler size, rod length, or hammer efficiency. In addition, bulk samples were obtained from auger cuttings at selected borings.

Penetration rates determined in general accordance with ASTM D1586 were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

2.2 LABORATORY TESTS

Laboratory tests were performed on selected samples to evaluate certain characteristics and engineering properties. The laboratory testing program was designed with emphasis on the evaluation of geotechnical properties of foundation materials as they pertain to the proposed construction. The laboratory testing program included performing the following tests:

- Unit Weight (ASTM D2937)

- Moisture Content (ASTM D2216)
- Grain Size Distribution (ASTM D422, without hydrometer)
- Material in Soils Finer than No. 200 (75-µm) Sieve (ASTM D1140)
- Resistance Value (California Test Method No. 301)
- Direct Shear (ASTM D3080)
- Unconfined Compressive Strength (ASTM D2166)
- Soluble Sulfate and Chloride Content (California Test Method Nos. 417 and 422)
- pH and Minimum Resistivity (California Test Method No. 643)

The soluble sulfate, soluble chloride, pH, and minimum resistivity results are presented in Section 4.0 ("Corrosion Evaluation"). All other lab test results are presented in Appendix A.

Laboratory data was also used to the extent from recent borings drilled for the concurrent geotechnical study for the nearby West Manning Avenue Bridge (Bridge No. 42C-0691) over James Bypass West Channel.

2.3 GEOTECHNICAL DESIGN PARAMETERS

Soil conditions and characteristics are very similar along the West Manning Avenue alignment at the two bridge sites (Bridge Nos. 42C-0691 and 42C-0692). Design geotechnical parameters were based on site specific laboratory data for the entire project alignment and interpretation of the geology in the area. Consideration was also given to correlations with sample penetration rates. Table 2.3-1 provides a summary of geotechnical design parameters and generalized soil profile used.

Table 2.3-1
Geotechnical Design Parameters

Elevation (feet)	Material	γ_t (pcf)[1]	γ_b (pcf)[2]	Φ (degrees)	c (psf)
184 – 168	SC/CL	127	65	33	275
168 – 158	SC/SM	123	61	35	150
158 – 146	CL	129	67	27	1800
Below 146	SP	130	68	40	0

Notes: [1] Total unit weight
[2] Buoyant unit weight

3 SITE CONDITIONS

3.1 SURFACE CONDITIONS AND TOPOGRAPHY

Presently, Manning Avenue is a 2-lane paved road supported on a fill embankment, approximately 8 feet above the surrounding grade with about 2:1 (H:V) side slopes. At the time of investigation, water was not present in the James Bypass East Channel. Some dried vegetation, debris and rip rap exist on the banks and bottom of the channel. The channel invert at the replacement is presently at approximately elevation 169 feet with slopes to the abutments. The debris and rip rap appear to retain sediments in the channel bottom under the bridge and north of the bridge. The channel bottom drops in elevation south of the bridge.

3.2 REGIONAL GEOLOGY

The project site lies in the central portion of the San Joaquin Valley in the Great Valley geomorphic province in California. This province was formed by the filling of a large structural trough or downwarp in the underlying bedrock. The trough is situated between the Sierra Nevada Range on the east and south and the Coast Range on the west. Both of these mountain ranges were initially formed by uplifts that occurred during the Jurassic and Cretaceous periods of geologic time (greater than 65 million years ago). Renewed uplift began in the Sierra Nevada during the Tertiary time, and is continuing today. The trough that underlies the valley is asymmetrical, with the greatest depths of sediments near the western margin. The sediments that fill the trough originated as erosion material from the adjacent mountains and foothills.

3.3 EARTH MATERIALS

The following description provides a general summary of the subsurface conditions encountered during the field exploration and further validated by the laboratory testing program. For a more thorough description of the actual conditions encountered at the specific boring location, refer to the Log of Test Borings presented in Figure 2. The soils encountered were classified according to the Unified Soil Classification System (ASTM D2487).

The upper natural earth material consists of Holocene age Great Valley basin deposits. Typical to these deposits, the upper soils are laterally discontinuous. The upper 37 feet of soil consisted of clayey sand and sandy lean clay with layers of laterally discontinuous poorly graded sand and silty sand. Below an elevation of about 145 to 147 feet, medium dense to dense poorly graded sands were encountered to the depths explored, about elevation 121 feet.

3.4 GROUNDWATER CONDITIONS

The California Department of Water Resources groundwater elevation contours from well data indicate the static groundwater elevation in the general project area is about elevation 150 feet. No free groundwater was encountered in any borings along West Manning Avenue at the two bridge sites. However, elevated moisture levels were detected.

It is understood the James Bypass East Channel is a flood channel that accepts excess flood water from the Kings River. Water in the channel is not likely to be sustained long enough to increase groundwater elevations sufficient to create buoyant effects, but it may be possible that a temporary perched water zone could develop in the upper 20 feet of soil below the channel bottom. The temporary perched water condition is anticipated to mound briefly below the channel bottom and dissipate as the perched water flows laterally away from the channel. Groundwater conditions at the site could change at some time in the future due to variations in rainfall, groundwater withdrawal, construction activities, channel flows, and/or other factors not apparent at the time the test borings were made.

3.5 CHANNEL SCOUR/DEGRADATION

Bridge maintenance reports indicate about 1 to 2 feet of channel degradation from 1956 to the present, and measured 0.65 feet of degradation of the channel bottom between 2005 to 2011. The boring within the channel was inconclusive to estimate a depth of historic scour. The mean grain size (D_{50}) and 90 percent passing grain size (D_{90}) of the soil anticipated to be exposed in the channel is about 0.15 mm and 0.47 mm, respectively.

A review of the GP for the project indicated potential scour at the supports. A summary of the design scour is presented in Table 3.5-1.

TABLE 3.5-1
SCOUR DATA

Support No.	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)
Abutment 1	N/A	7
Abutment 2	N/A	7

4 CORROSION EVALUATION

Soil samples from Borings B-5 (0 to 5 feet), B-6 (30 to 31.5 feet), and borings performed at the adjacent structure were tested to evaluate the soluble sulfate content, soluble chloride content, and pH and minimum resistivity. Specific test results are presented in Table 4.1-1.

Table 4.1-1
Corrosion Related Testing

Boring No.	Depth (ft)	Soluble Sulfate (mg/kg)	Soluble Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
B-2	0-5	199	35.8	9.6	4,200
B-2	20-21.5	3.2	15	8.2	41,500
B-5	0-5	15.7	24.1	8.2	10,500
B-6	30-31.5	7.3	21	8.6	17,100

Laboratory tests indicate the soluble sulfates, soluble chlorides, pH and resistivity are anticipated to be outside the Caltrans threshold limits for corrosive soils. As such, the site may be considered as a non-corrosive environment with respect to steel and concrete foundations. Corrosion is dependent upon a complex variety of conditions, which are beyond the geotechnical practice. Consequently, a qualified corrosion engineer should be consulted if the owner desires more specific recommendations and material types and/or mitigation.

5 SEISMIC RECOMMENDATIONS

5.1 LOCAL FAULTING

There are no known faults, which cut through the local soil at the site. The project site is not located in an Alquist-Priolo Earthquake Fault Zone, as defined by Special Publication 42 (revised 2007) published by the California Geologic Survey (CGS). Numerous faults and shear zones within the region could influence the project site.

5.2 SEISMIC DESIGN CRITERIA

Seismic design parameters were developed in accordance with the Caltrans Seismic Design Criteria Version 1.7.

The project site is located in a region with the potential for relatively moderate seismic activity. The more significant faults that could influence the project site include the San Andreas (Creeping Section) (Fault ID No. 182), the Great Valley 13 (Coalinga) (Fault ID No. 205), and the San Andreas (Parkfield) (Fault ID No. 214). According to the Caltrans fault database, the San Andreas (Creeping Section and Parkfield) is a strike slip fault with a dip angle of 90 degrees and assigned a Maximum Magnitude (M_{Max}) of 7.9, and the Great Valley 13 (Coalinga) is reverse fault with a dip angle of 15 degrees toward the west and assigned a Maximum Magnitude (M_{Max}) of 7.0.

Based on the boring data, the site can be classified as Soil Profile Type D. A V_{s30} of 266 m/s was used for the evaluation. The site is not located within a California deep soil basin region, as defined by Caltrans, so $Z_{1.0}$ and $Z_{2.5}$ were considered not applicable. Site characteristics and governing deterministic faults are summarized in Table 5.2-1.

Table 5.2-1

Site Characteristics And

Governing Deterministic Faults Parameters

Site Coordinates	Lat = 36.603685 deg, Long = -120.130515 deg
Shear Wave Velocity	266 m/s
Depth to V_s=1.0 km/s, $Z_{1.0}$	N/A
Depth to V_s=2.5 km/s, $Z_{2.5}$	N/A
Fault Name and ID Number	**San Andreas (Creeping section), No. 182**
Maximum Magnitude (M_{Max})	7.9
Fault Type	Strike Slip
Fault Dip	90 degrees
Dip Direction	Vertical
Bottom of Rupture Plane	12.00 km
Top of Rupture Plane (Z_{tor})	0.00 km
R_{RUP}[1]	73.38 km
R_{jB}[2]	73.38 km
R_X[3]	73.38 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	0
Fault Name and ID Number	**Great Valley (Coalinga), No. 205**
Maximum Magnitude (M_{Max})	7.0
Fault Type	Reverse
Fault Dip	15 degrees
Dip Direction	West
Bottom of Rupture Plane	15.30 km
Top of Rupture Plane (Z_{tor})	9.10 km
R_{RUP}[1]	35.59 km
R_{jB}[2]	34.41 km
R_X[3]	33.90 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	1
Fault Name and ID Number	**San Andreas (Parkfield), No. 214**
Maximum Magnitude (M_{Max})	7.9
Fault Type	Strike Slip
Fault Dip	90 degrees
Dip Direction	Vertical
Bottom of Rupture Plane	6.00 km
Top of Rupture Plane (Z_{tor})	0.00 km
R_{RUP}[1]	80.12 km
R_{jB}[2]	80.12 km
R_X[3]	75.12 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	0

Notes: [1]R_{RUP} = Closest distance from the site to the fault rupture plane.
[2]R_{JB} = Joyner-Boore distance; the shortest horizontal distance to the surface projection of the rupture area.
[3]R_X = Horizontal distance from the site to the fault trace or surface projection of the top of the rupture plane.

5.2.1 Deterministic Response Spectrum

The deterministic response spectrum was developed using ARS Online. The deterministic response spectrum from the Minimum Spectrum for California governed.

5.2.2 Probabilistic Response Spectrum

The probabilistic response spectrum was developed using ARS Online.

5.2.3 Design Response Spectrum

The upper envelope of the deterministic and probabilistic spectral values determines the design response spectrum. The probabilistic response spectra were found to govern for all periods. The recommended acceleration and displacement design response spectra are presented graphically and numerically in Appendix B.

5.2.4 References

Caltrans. Caltrans ARS Online, http://dap3.dot.ca.gov/ARS_Online

Caltrans. Geotechnical Services Manual.

Caltrans. Seismic Design Criteria, Appendix B Design Spectrum

Caltrans. Website http://dap3.dot.ca.gov/shake_stable/v2/technical.php

5.3 LIQUEFACTION AND SIESMIC SETTLEMENT

In order for liquefaction of soils due to ground shaking to occur, it is generally accepted that four conditions will exist:

- The subsurface soils are in a relatively loose state,
- The soils are saturated,
- The soils are non-plastic, and
- Ground motion is of sufficient intensity to act as a triggering mechanism.

Based on the ARS curve, the design Peak Horizontal Ground Acceleration (PHGA) is 0.34g, with a Moment Magnitude of 6.5. The potential for liquefaction was evaluated using Youd et. al. The depth of ground water would preclude the occurrence of liquefaction.

Dynamic compaction, or seismic settlement, can occur in unsaturated, loose granular soils or in poorly-compacted fill soils. The potential seismic induced settlement of unsaturated sand sediments at this site was evaluated using data from the current borings and the methodology described by Tokimatsu and Seed (1987). The results of settlement calculations indicated that the estimated seismic settlement in the unsaturated soil is estimated to be up to approximately ½- to ¾-inch in Boring B-4 at Abutment 1 and less than ¼-inch in Boring B-6 at Abutment 2 as a result of a magnitude 6.5 earthquake. This potential seismic settlement may induce downdrag on piles at Abutment 1.

6 FOUNDATION RECOMMENDATIONS

6.1 GENERAL

Based on the field exploration, laboratory testing, and geotechnical analyses, the soils at the site are suitable for supporting the planned bridge structure. Due to the scour potential, loading conditions and other constraints, pile foundations were considered appropriate. Driven piles have been precluded as an option due to the presence of a nearby underground utility that could be damaged by vibrations from pile driving. Therefore, Cast-In-Drilled-Hole (CIDH) piles are considered a suitable alternative.

Construction of the bridge will involve complete replacement of the existing structure. The replacement process will utilize two, 48-inch CIDH piles at each abutment locations. The following sections discuss conclusions and recommendations to support design of the CIDH pile foundations.

6.2 PILE FOUNDATIONS

6.2.1 Axial Capacity

Table 6.2-2 provides the recommended design and specified tip elevations for the abutments. Piles should be spaced at a minimum distance of 12 feet center to center. Reduced axial capacities would be applicable for piles spaced closer than 12 feet.

Table 6.2-2

Foundation Recommendations for Abutments and Bents

Support	Pile	Cut-off Elev. (ft)	Service Limit State Max. Load (kips) per Pile	S_P	Required Factored Nominal Resistance per Pile (kips)				Design Tip Elev.[1] (ft)	Specified Tip Elev.[2] (ft)
					Strength Limit		Extreme Event			
					Comp. (ϕ=0.7)	Tens. (ϕ=0.7)	Comp (ϕ=1)	Tens (ϕ=1)		
Abut 1	48” CIDH	174.49	540	2”	840	0	570	0	132 (a) 128 (a-I) 149 (a-II), 150 (c), TBD (d)	128
Abut 2	48” CIDH	174.33	540	2”	840	0	570	0	138 (a) 134 (a-I) 153 (a-II), 152 (c), TBD (d)	134

Notes: (1) Design tip elevations are controlled by: (a) Compression (Service Limit), (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load.
(2) The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.
(3) The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the potentially liquefied soil layers, which do not contribute to the design resistance.
(4) Design tip elevation for lateral loading to be provided by Cornerstone.

6.2.2 Foundation Construction Considerations

Pile borings will primarily expose relatively clean to silty sand, which will be susceptible to caving. Construction of the CIDH piles should utilize a casing oscillator, permanent or temporary casing, or slurry assisted drilling techniques. If permanent casing is used, Special Provisions should require the casing be tight to the soil (similar to Caltrans Standard Specification for temporary casing).

6.2.3 Lateral Capacity

The lateral response of pile foundations can be evaluated using LPILE Plus Version 5.0, or greater, for Windows (computer software developed by Ensoft Inc.). The geotechnical parameters summarized in Table 6.2-3 can be used for evaluation of lateral loading of piles at abutment locations.

TABLE 6.2-3
LPILE Parameters, Abutments

Elevation [1] (feet)	Recommended P-Y Curve	γ (pci)	ϕ (degree)	c (psi)	k (pci)	ε_{50}
184 – 168	Silt	0.073	33	1.91	300	0.005
168 – 158	Silt	0.071	35	1.04	250	0.005
158 – 146	Stiff Clay w/o Free Water	0.075	--	12.5	--	0.007
Below 146	Sand (Reese)	0.075	40	--	200	--

Notes: [1] Assumes Elevation 184 for bridge deck

When considering the lateral capacity of a pile group, it will be necessary to reduce the single pile capacity of trailing piles. The reduction in capacity due to the effects of shaft interaction will be dependent upon the center-to-center (CTC) pile spacing. It is recommended that the capacity of individual trailing piles in a laterally loaded group be reduced according to the data in Table 6.2-4.

Table 6.2-4

Group Affect for Laterally Loaded Pile

CTC Spacing (In-line Loading)	Ratio of Lateral Resistance of Trailing Pile in Group to Isolated Single Pile
3B	0.6
4B	0.8
5B	1.0

Note: B is pile width

If desired, a more precise lateral group effect can be evaluated based on final geometry.

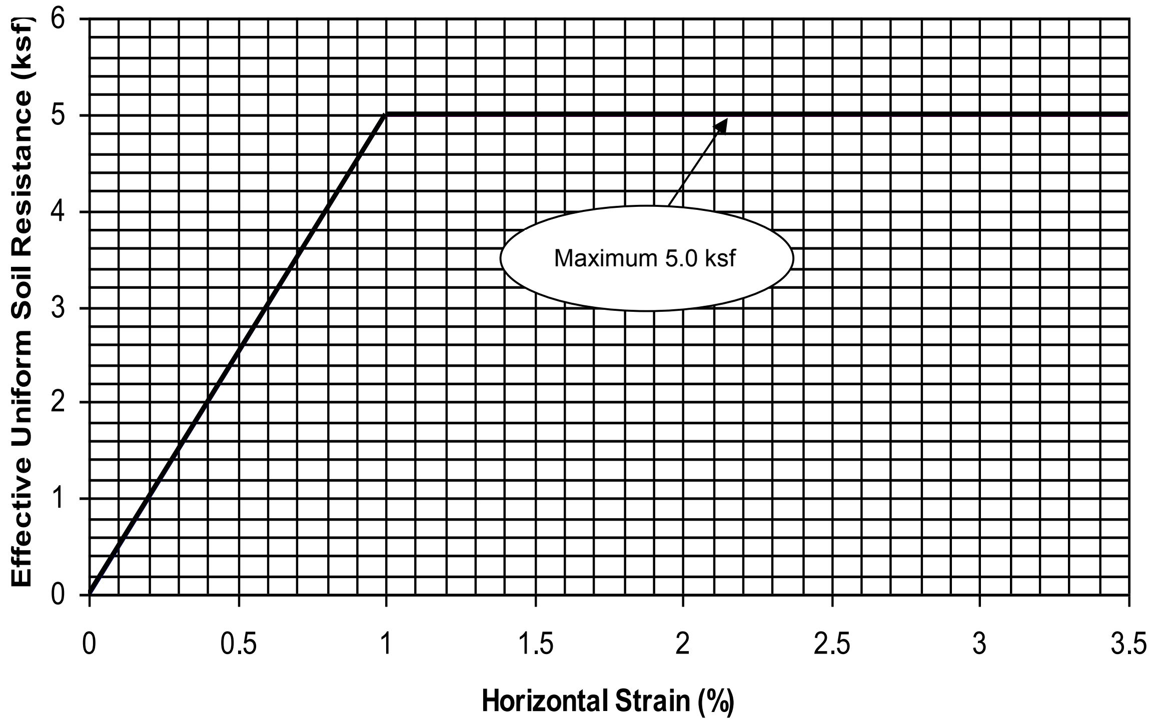
6.3 DYNAMIC LOADING

6.3.1 Abutment Dynamic Lateral Resistance

For backfill at abutments constructed in accordance with applicable provisions of the Caltrans Standard Specifications (2015), an initial abutment soil stiffness of 50 kip/in/ft is recommended. The ultimate lateral resistance that may be applied against abutments to resist seismic loading will be dependent on the deflection that occurs (which mobilizes shear resistance in the soil). Figure 6.3-1 presents the ultimate equivalent uniform lateral soil resistance as a function of horizontal strain (deflection/height) for the abutments. The maximum resistance for strain in excess of 1.0 percent is 5.0 ksf, when the height of the wall that is buried below the horizontal ground surface is equal to, or greater than, 5.5 feet. When the abutment height is less than 5.5 feet, the maximum equivalent uniform lateral soil resistance shall be reduced proportionately by H/5.5, where H is the endwall height in feet.

Figure 6.3-1

Unfactored Nominal Lateral Bearing for Seismic Loading at Abutments



6.4 LATERAL EARTH PRESSURES

Table 6.4-1 provides the lateral earth pressures against retaining walls. The recommended values do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be adequately drained. Data are presented for active, braced, and at-rest conditions for walls supporting level ground surface. The values consider the anticipated strength of backfill and the performance of existing structures. Lateral earth pressures are strain related. The active pressure would be applicable to walls capable of rotating 0.0005 radians. The at-rest pressures are applicable to walls fully fixed against translation or rotation. The at-rest pressures include the Jaky solution for normally consolidated soil plus consideration for the locked-in pressure associated with the pre-stressing due to backfill compaction (over-consolidation).

Table 6.4-1
Retaining Wall Parameters

Condition	Earth Pressures
Active	35 psf/ft
Braced1	23H psf
At Rest	85 psf/ft
Dynamic Increments	13 psf/ft

Note: [1] H is the wall height in feet.

The above values are less than those used for Caltrans Standard Plan walls. Therefore, Standard Plan retaining walls could be used.

6.5 EARTHWORK

In general, any required fill or backfill should be constructed in accordance with the latest revisions of Caltrans Standard Specifications. It is anticipated minor fills may be associated with the project, and are anticipated to not have significant post-construction settlement.

6.6 FLEXIBLE PAVEMENT

Soil samples were obtained along Manning Avenue alignment at the two bridge sites (Bridge Nos. 42C-0691 and 42C-0692). The site subgrade soil consists of sandy lean clays and clayey sands at B-3 and B-4 with R-values of 8 and 6, respectively. A design R-value of 5 was used for Manning Avenue. Table 6.6-1 provides optional conventional flexible pavement sections for Traffic Indexes of 8.5 and 9.0, based on an anticipated daily traffic provided by Cornerstone. Analysis is based on Caltrans procedures.

Table 6.6-1
Conventional Flexible Pavement Sections
(Based on Subgrade R-Value = 5)

Traffic Index	Optional Sections (feet)	
	2-Layer Section	3-Layer Section
8.5	0.40 HMA / 1.65 AB	0.40 HMA / 0.55 AB / 1.20 ASB
9.0	0.45 HMA / 1.70 AB	0.45 HMA / 0.55 AB / 1.30 ASB

Table 6.6-2 provides overlay recommendations for existing pavement. Sections are based on the anticipated R-Value of existing subgrade soil, the furnished Traffic Index (TI) for the road segment, and a 10-year and 20-year design life. It is understood the County of Fresno is planning to mill and overlay the existing pavement to match previous overlays west and east of the project site. The proposed overlay thickness is anticipated to be 0.45 feet. Considering the R-value and design TI, the proposed overlay thickness is not anticipated to provide a 20-year design life, but is anticipated to have a similar design life to other portions of Manning Avenue that have recently had similar overlays.

Table 6.6-2
Recommended Design Pavement Sections
(Based on Subgrade R-Value = 5)

Traffic Index	Minimum Overlay (feet)	
	10-year Design	20-year Design
8.5	0.85 HMA	0.95 HMA
9.0	0.95 HMA	1.00 HMA

Hot mix asphalt (HMA), Class 2 aggregate base (AB) and Class 2 aggregate subbase (AS) should conform to, and be placed in accordance with, the latest revision of Caltrans Standard Specifications. Considering the clayey nature of the subgrade, it is recommended subgrade be scarified to a depth of 12 inches, moisture conditioned to 2 percent above optimum moisture and compacted to at least 90 percent, but not more than 95 percent of maximum density. It is recommended the maximum density for subgrade be based on ASTM D1557.

Alternatively, full depth reclamation with cement (FDR-C) of subgrade and HMA could be considered. A minimum FDR-C section of 18 inches would be recommended. If FDR-C is chosen as the design alternative, a mix design can be prepared. For preliminary purposes, a portland cement content of 5 to 7 percent amendment is typical in sandy lean clay materials to result in a design unconfined compressive strength of 400 psi. The minimum HMA thickness would be 0.35 and 0.40 feet for TIs of 8.5 and 9, respectively. The “weak link” of the section is the HMA wearing surface, which is the easiest and most economical layer to maintain, repair, or rehabilitate to meet future needs. The full depth thickness, versus the minimum thickness, is less vulnerable to brittle fatigue or shrinkage cracking. The use of 1.5 feet of cement treated subgrade (CTS) will result in the overall HMA/CTS being suitable for a TI of 9.7 and 10.2, with the HMA surface being the only element requiring future rehabilitation. Traffic can be diverted onto the compacted full depth CTS after finish grading. With the minimum CTS thickness, automobile traffic could be placed on the subgrade after finish grading. Care must be exercised to not over stress the minimum CTS with truck traffic until the first HMA lift is placed.

Lime treatment could be considered to reduce aggregate base thickness for asphalt pavement sections. Previous work performed in the area has indicated that lime treatment using an engineered process, which includes testing for soil chemistry, necessary lime content and R-Value, and development of QC/QA procedures, can successfully improve the stability of subgrade materials. If lime treatment is desired, the additional testing and recommendations can be provided.

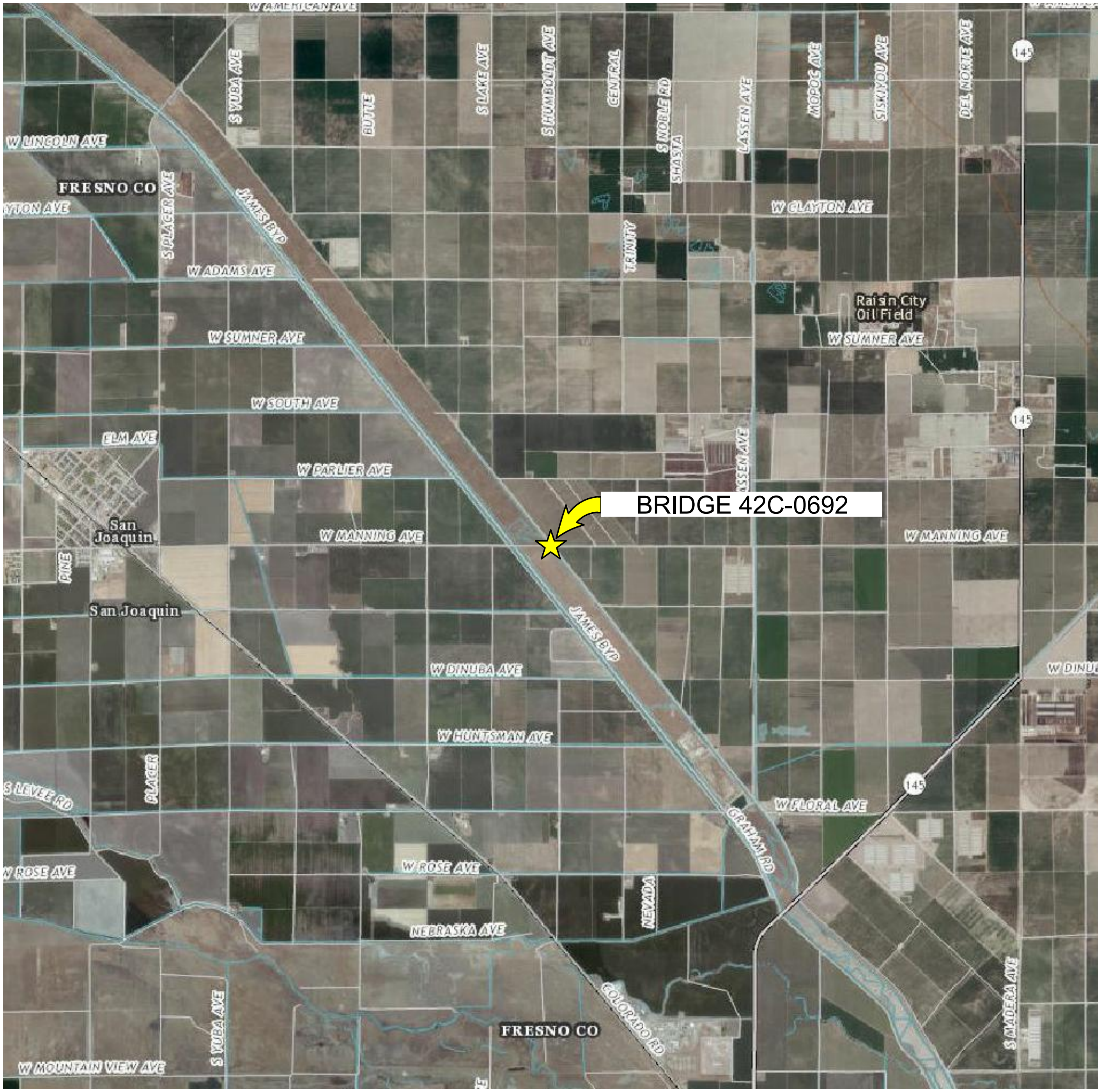
7 CLOSURE

The conclusions and recommendations in this report are for the design of the proposed West Manning Avenue Bridge (Bridge No. 42C-0692) replacement across the James Bypass East Channel, located in Fresno County, California, as described in the text of this report. The findings, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No warranty, express or implied, is made. The field exploration program and this report were based on the proposed project information provided to Kleinfelder. If any change (i.e., structure type, location, etc.) is implemented which materially alters the project, additional geotechnical services may be required, which could include revisions to the recommendations given herein.

This report is intended for use by Cornerstone Structural Engineering Group, County of Fresno, and their subconsultants, within a reasonable time from its issuance. Noncompliance with the recommendations of the report or misuse of the report will release Kleinfelder from any liability.

The scope of the geotechnical services did not include an environmental site assessment for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater or atmosphere, or the presence of wetlands.

FIGURES



SOURCE: U.S.G.S. / THE NATIONAL MAP, AERIAL SERIES, SAN JOAQUIN, CALIFORNIA, DATED 4/2014.

N

6,000 3,000 0 6,000

APPROXIMATE SCALE (feet)

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PROJECT NO.	20161220
DRAWN:	7/2015
DRAWN BY:	MRG
CHECKED BY:	NP
FILE NAME: 20161220-fig1_SVM-WBrg.dwg	

SITE VICINITY MAP

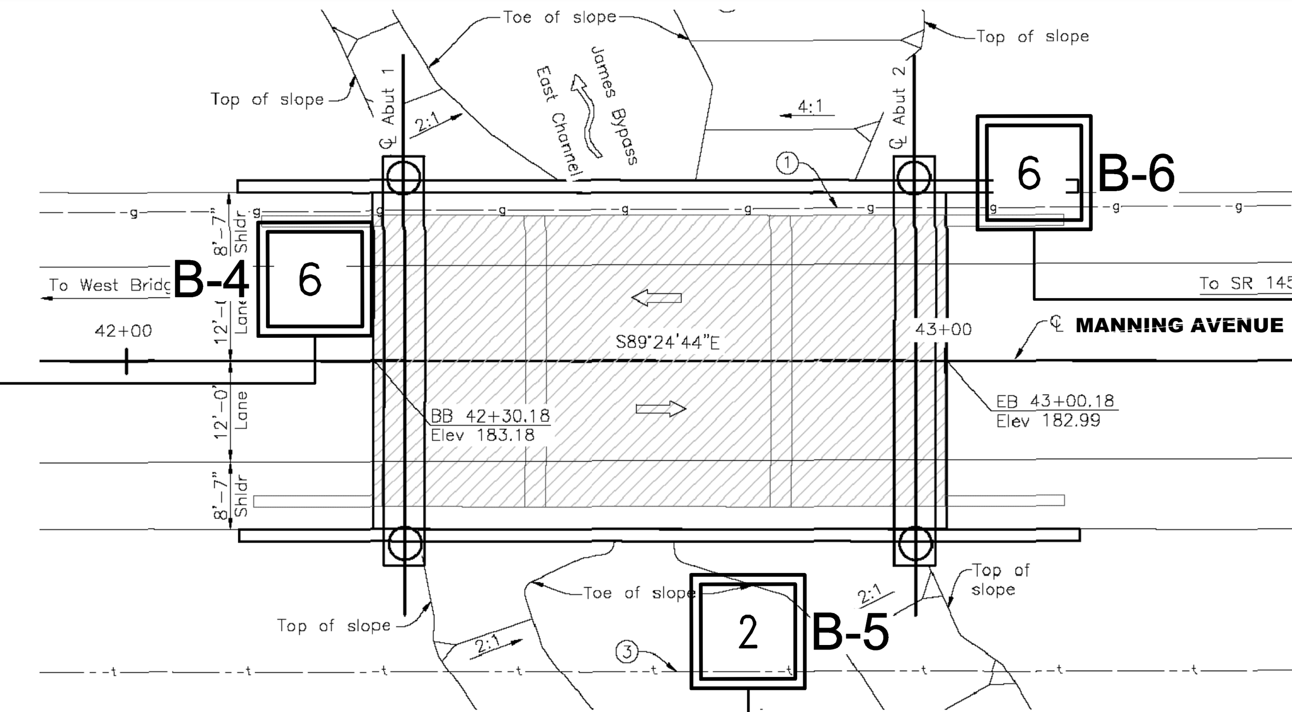
MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CALIFORNIA

FIGURE
1

ATTACHED IMAGES: Images: USGS_TheNationalMap_Aerial_4-mil_3-2015_SanJoaquin-CA.JPG
ATTACHED XREFS:
LONG BEACH, CA
CAD FILE: L:CADD\2015\20161220\JamesBypassBridge-ManningAve_7-2015\WesternBridge\ LAYOUT: PLOTTED: 15 Jul 2015, 9:35pm, MGriffin

NOTES:

1. 1.5–INCH DIAMETER SAMPLES WERE TAKEN USING A STANDARD PENETRATION TEST (SPT) SPLIT BARREL SAMPLER WITH AN INSIDE DIAMETER (ID) OF 1.5 INCHES AND AN OUTSIDE DIAMETER (OD) OF 2.0 INCHES.
2. 2.5–INCH DIAMETER RING SAMPLES WERE TAKEN USING A CALIFORNIA SPLIT BARREL SAMPLER WITH AN ID OF 2.5 INCHES AND AN OD OF 3.0 INCHES.
3. ALL DRIVE SAMPLES WERE DRIVEN WITH 140 LB HAMMER WITH A FALLING HEIGHT OF 30 INCHES.
4. ELEVATION BASED ON PLANS FROM QUINCY ENGINEERING, DATED 11/4/2013.



N

PLAN

SCALE: 1"=20'

PROFILE

SCALE: 1"=10' HORIZONTAL
SCALE: 1"=5' VERTICAL

90% SUBMITTAL - NOT FOR CONSTRUCTION

ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

FIELD INVESTIGATION T. DeSOUZA	DATE As Shown	Scale	
DRAWN D. FAHRNEY	–	AS SHOWN	
CHECKED N. POPENOE	–		
REVISION			

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FOR R/W DATA AND ACCURATE ACCESS DETERMINATION SEE R/W RECORDS AT PUBLIC WORKS

REGISTERED PROFESSIONAL ENGINEER STEPHEN P. PLAGMAN No. 2731 Exp. 09/30/2021 GEOTECHNICAL STATE OF CALIFORNIA

PROJECT
JAMES BYPASS EAST CHANNEL BRIDGE AT MANNING AVENUE
Road No. – Bridge No. 42C0692

THE COUNTY OF FRESNO 1856

DEPARTMENT OF PUBLIC WORKS & PLANNING
LOG OF TEST BORINGS
BR. No. 42C0692

Drawing No. S28 Sheet No. 37 of 37

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APPENDIX A

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-4	0.67 - 5.0	SANDY LEAN CLAY (CL)									**R-Value=** 6
B-4	5.0	CLAYEY SAND (SC)	15.5	107.9							**Direct Shear=**
											Peak Cohesion: 275 psf
											Peak Friction Angle: 35.0°
											20% Stress Cohesion: 200 psf
											20% Stress Friction Angle: 33.0°
B-4	10.0	CLAYEY SAND (SC)	26.5	86.1							
B-4	15.0	CLAYEY SAND (SC)					46				
B-4	20.0	CLAYEY SAND (SC)	13.9	113.5							
B-4	30.0	SILTY SAND (SM)	14.5	102.3							
B-4	35.0	SILTY SAND (SM)					25				
B-4	40.0	POORLY-GRADED SAND (SP)	5.2	92.7							**Direct Shear=**
											Peak Cohesion: 25 psf
											Peak Friction Angle: 36.0°
											20% Stress Cohesion: 75 psf
											20% Stress Friction Angle: 32.0°
B-5	0.0 - 5.0	SILTY SAND (SM)					29				**pH=** 8.2
											Resistivity= 10500
											Sulfates= 15.7
											Chlorides= 24.1
B-6	5.0	SANDY LEAN CLAY (CL)	17.5	113.6							
B-6	10.0	SANDY LEAN CLAY (CL)					73				
B-6	15.0	SILTY SAND (SM)	8.9	114.1							
B-6	20.0	SILTY SAND (SM)					28				
B-6	25.0	SILTY SAND (SM)	22.4	104.0							**Unconfined Compressive Strength=**
											q_u: 3580 psf Strain at Failure: 4.3%
B-6	30.0	SANDY LEAN CLAY (CL)									**pH=** 8.6
											Resistivity= 17000

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic

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PROJECT NO.: 20161220
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DATE:
REVISED: -

LABORATORY TEST RESULT SUMMARY

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CA

FIGURE
A-1

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
											Sulfates= 7.3
											Chlorides= 21
B-6	35.0	SANDY LEAN CLAY (CL)	16.0	109.1							

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic

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PROJECT NO.: 20161220
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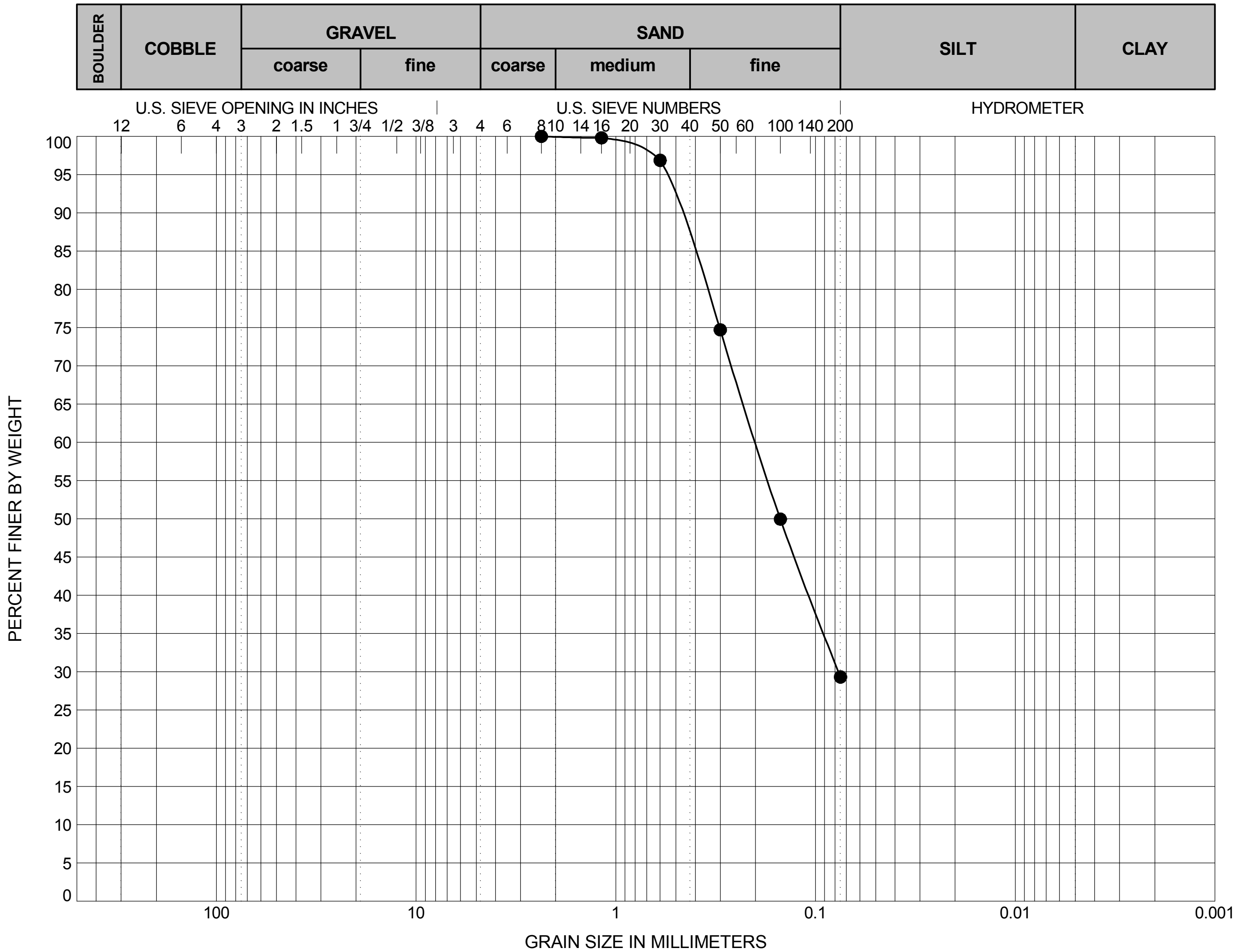
DATE:
REVISED: -

LABORATORY TEST
RESULT SUMMARY

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CA

FIGURE
A-2



	Exploration ID	Depth (ft.)	Sample Description	LL	PL	PI
●	B-5	0 - 5	SILTY SAND (SM)	NM	NM	NM

	Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
●	B-5	0 - 5	2.36	0.199	0.077	NM	NM	NM			29	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60}\ D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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PROJECT NO.:	20161220
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SIEVE ANALYSIS

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CA

FIGURE
A-3

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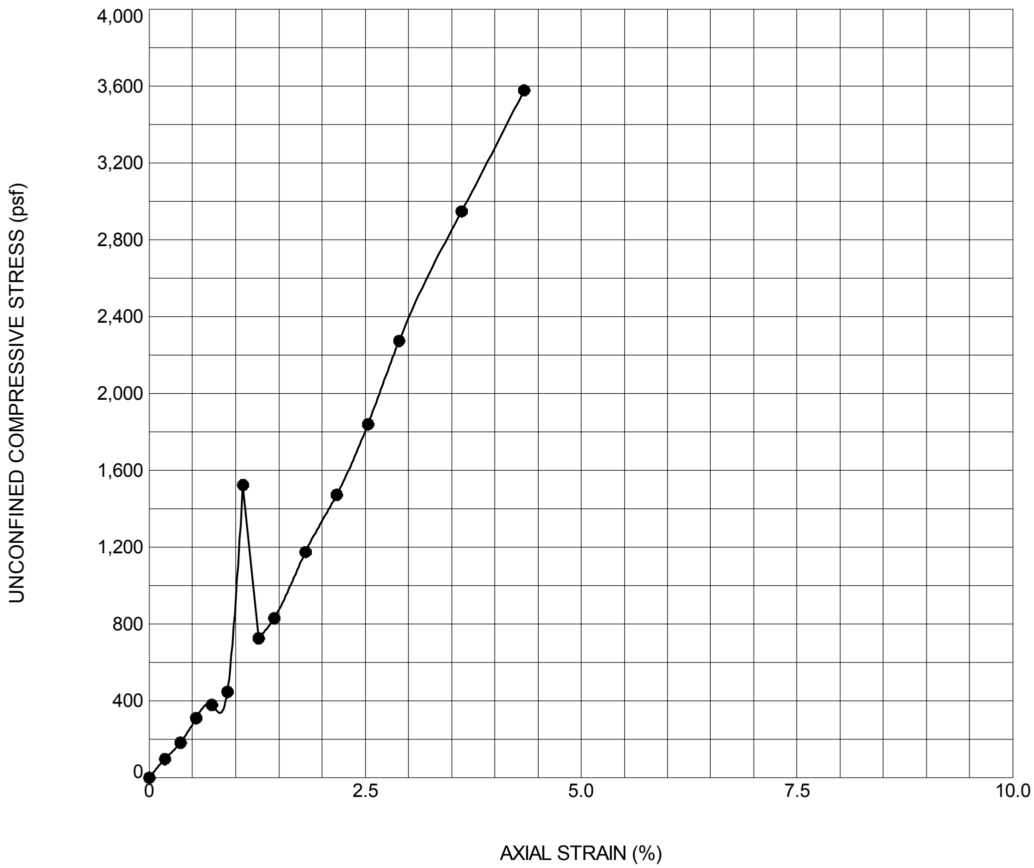

Exploration ID	Depth (ft.)	Sample Description
B-6	25	SILTY SAND (SM)

Time to Failure (min)	Average Rate of Strain to Failure (%/min)	Shear Strength (psf)	Unconfined Compressive Strength (psf)	Strain at Failure (%)
NM	NM	NM	3580	4.3

Specimen No.	Sample Condition Type	Initial Sample Dia. (mm)	Initial Height (mm)	Initial Height to Diameter Ratio	Initial Water Content (%)	Initial Dry Unit Wt. (pcf)	Initial Saturation (%)	Initial Void Ratio	Specific Gravity	Passing #200	LL	PL	PI
1	Undisturbed	61.2	140.5	2.3:1	22.4	104.1	NM	NM	NM	NM	NM	NM	NM

Testing perfomed in general accordance with ASTM D2166.
NP = Nonplastic
NM = Not Measured

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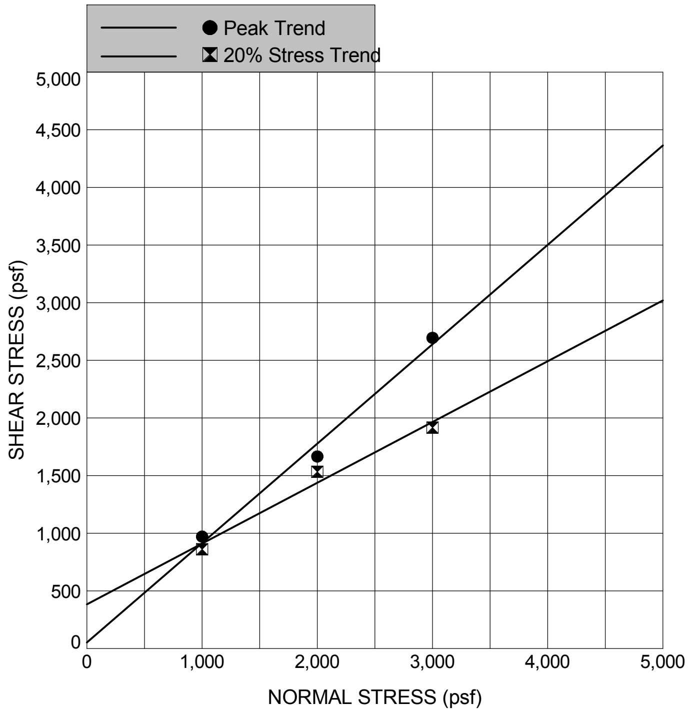
UNCONFINED COMPRESSIVE STRENGTH

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CA

FIGURE
A-4



TEST CONDITIONS: Cohesionless

Exploration ID	Depth (ft.)	Sample Description
B-4	5	CLAYEY SAND (SC)

Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity
NM	NM	NM	NM	NM	

	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1	15.5	90.3			4.60	1.00
	2	15.5	89.4			4.60	1.00
	3	15.5	88.9			4.60	1.00
At Test	1	30.6	73.4			4.60	1.00
	2	30.4	82.5			4.60	1.00
	3	29.8	81.0			4.60	1.00

Specimen No.	Peak Shear Stress (psf)	20% Stress Shear Stress (psf)	Horizontal Displacement (in)	Normal Stress (psf)	Strain Rate (in/min)
1	971.15	861.79	0.4900	1000	0.005
2	1666.02	1533.37	0.4800	2000	0.005
3	2695.18	1917.05	0.4800	3000	0.005

Results	Cohesion (psf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	275	35	
20% Stress	200	33	

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NM = Not Measured

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PROJECT NO.:	20161220
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DATE:	7/10/2015
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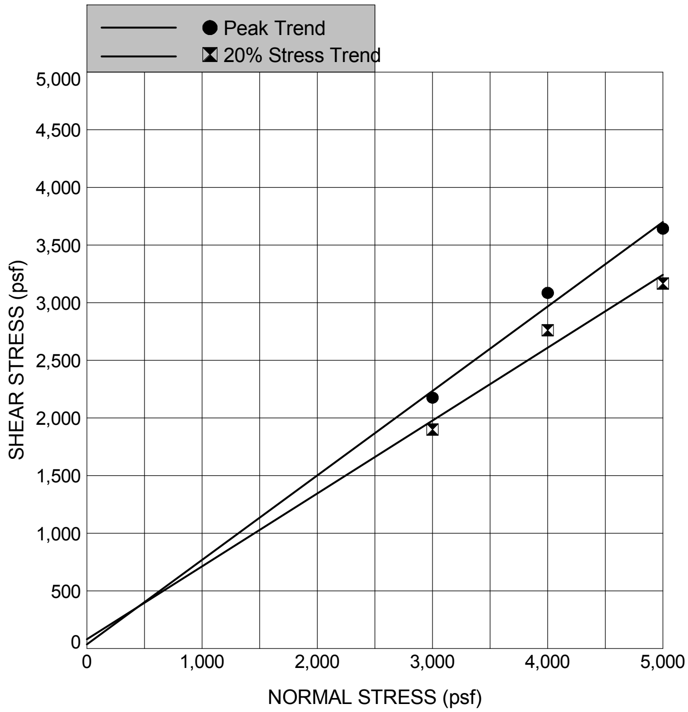
DIRECT SHEAR

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CA

FIGURE
A-5



TEST CONDITIONS: Cohesionless

Exploration ID	Depth (ft.)	Sample Description
B-4	40	POORLY-GRADED SAND (SP)

Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity
NM	NM	NM	NM	NM	

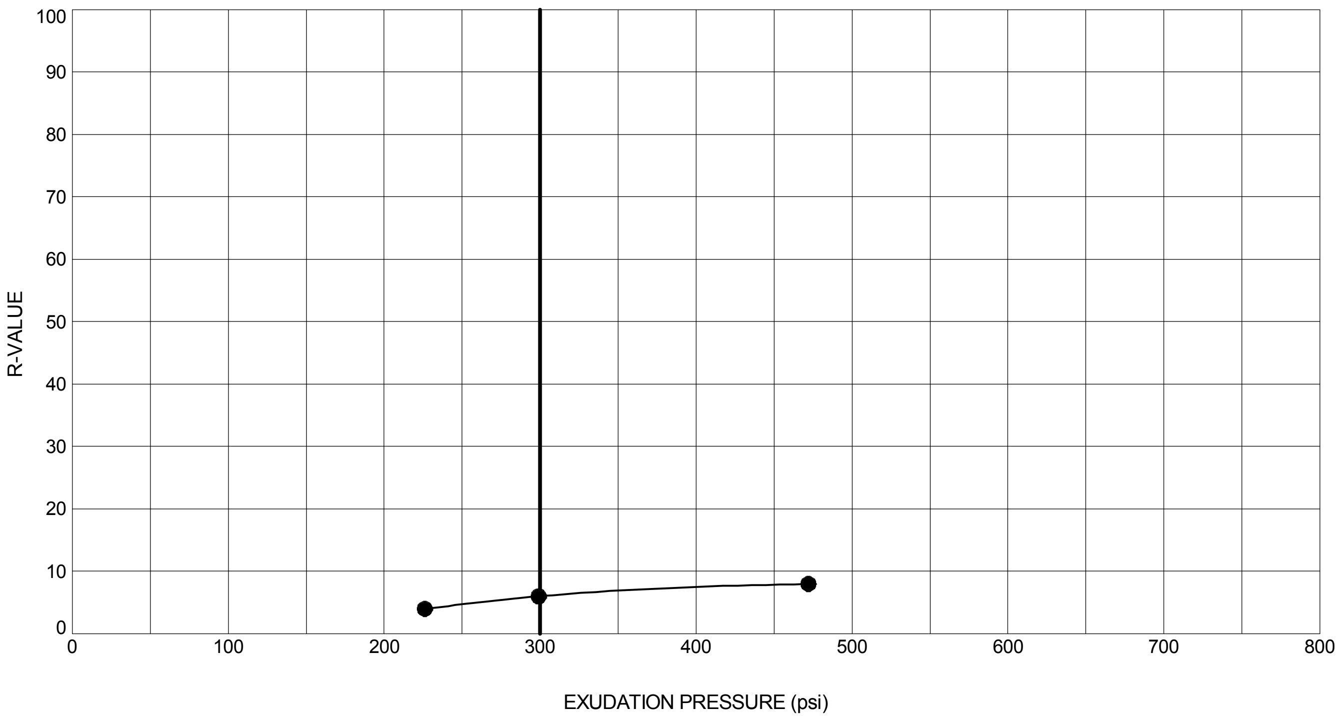
	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1	5.2	88.5			4.60	1.00
	2	5.2	88.0			4.60	1.00
	3	5.2	89.7			4.60	1.00
At Test	1	30.3	68.4			4.60	1.00
	2	31.0	74.4			4.60	1.00
	3	32.0	63.4			4.60	1.00

Specimen No.	Peak Shear Stress (psf)	20% Stress Shear Stress (psf)	Horizontal Displacement (in)	Normal Stress (psf)	Strain Rate (in/min)
1	2176.52	1900.85	0.4800	3000	0.005
2	3085.06	2760.25	0.4800	4000	0.005
3	3642.78	3165.65	0.4800	5000	0.005

Results	Cohesion (psf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	25	36	
20% Stress	75	32	

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NM = Not Measured

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20161220	DIRECT SHEAR
	DRAWN BY: TD	
	CHECKED BY: NMP	MANNING AVENUE OVER JAMES BYPASS BRIDGE NO. 42C-0692 SAN JOAQUIN, FRESNO COUNTY, CA
	DATE: 7/10/2015	FIGURE A-6
	REVISED: -	



Exploration ID	Depth (ft.)	Sample Description	R-Value @ 300 psi Exudation Pressure
B-4	0.67 - 5	SANDY LEAN CLAY (CL)	6

Specimen No.	Moisture at Time of Test (%)	Dry Unit Weight (pcf)	Expansion Pressure (psi)	Exudation Pressure (psi)	Corrected Resistance Value
1	15.6	113.8	0	299	6
2	16.9	110.6	0	226	4
3	14.3	116.1	0	472	8

Testing perfomed in general accordance with ASTM D2844.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20161220 DRAWN BY: TD CHECKED BY: NMP DATE: 7/10/2015 REVISED: -	R-VALUE MANNING AVENUE OVER JAMES BYPASS BRIDGE NO. 42C-0692 SAN JOAQUIN, FRESNO COUNTY, CA	FIGURE A-7

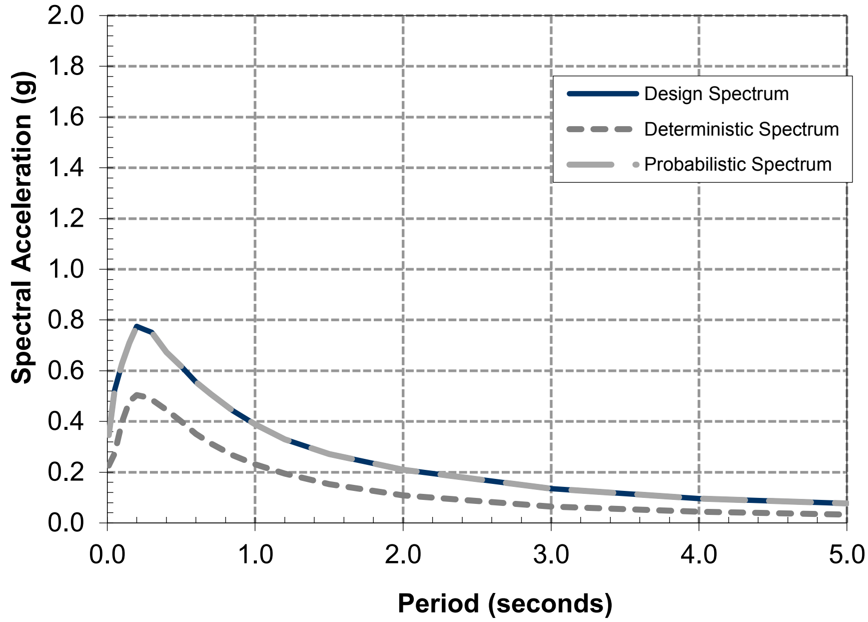
APPENDIX B

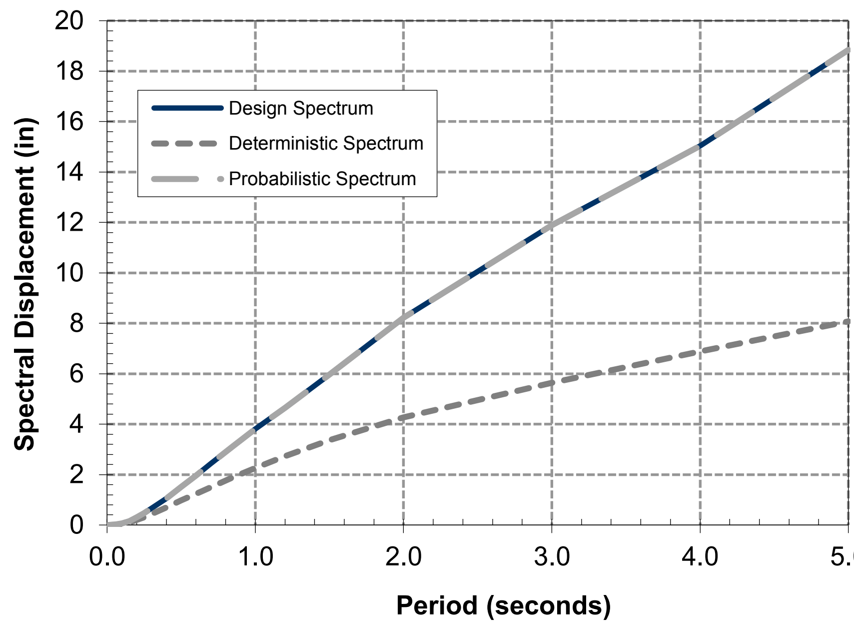
ARS CURVE

SITE DATA

Latitude:	36.6037	Shear Wave Velocity	266 m/s	
Longitude:	-120.13051!	Depth to Vs = 1.0 km/s:	N/A	
		Depth to Vs = 2.5 km/s:	N/A	

Period (s)	SA (g)	SD (in)
0.01 (PGA)	0.346	0.00
0.05	0.524	0.01
0.1	0.627	0.06
0.15	0.710	0.16
0.2	0.775	0.30
0.25	0.762	0.47
0.3	0.751	0.66
0.4	0.673	1.05
0.5	0.618	1.51
0.6	0.556	1.96
0.7	0.508	2.44
0.85	0.442	3.13
1	0.389	3.81
1.2	0.330	4.65
1.5	0.271	5.97
2	0.210	8.22
3	0.135	11.89
4	0.096	15.03
5	0.077	18.84





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CALTRANS ARS CURVES

MANNING AVENUE OVER JAMES BYPASS
BRIDGE NO. 42C-0692
SAN JOAQUIN, FRESNO COUNTY, CALIFORNIA

FIGURE

B-1