March 7, 2023
File No.: 20164632

Dewberry
11060 White Rock Road, Suite 200
Rancho Cordova, California 95670

Attention: Mr. Mike Pugh

SUBJECT: **Final Foundation Report (100% Submittal)**
Dry Creek on Burrough Valley Road Bridge Replacement
Tollhouse Road and Burrough Valley Road
Bridge No 42C-0134
Fresno County, California

Mr. Pugh:

The attached report presents the results of the geotechnical foundation study for the Dry Creek on Burrough Valley Road Bridge Replacement near Tollhouse Road in Fresno County, California. This report describes the study and provides conclusions and recommendations for use in design of the bridge.

Kleinfelder appreciates the opportunity to provide geotechnical engineering services to Dewberry and the County of Fresno. It is trusted this information will meet 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.

Stephen P. Plauson, PE, GE
Senior Principal Geotechnical Engineer

David L. Pearson, PE, GE
Senior Principal Engineer

FINAL FOUNDATION REPORT (100% SUBMITTAL)
DRY CREEK ON BURROUGH VALLEY ROAD BRIDGE REPLACEMENT
BRIDGE NO 42C-0134
FRESNO COUNTY, CALIFORNIA
KLEINFELDER PROJECT #: 20164632

MARCH 7, 2023

A Report Prepared for:

Dewberry
11060 White Rock Road, Suite 200
Rancho Cordova, California 95670

FINAL FOUNDATION REPORT (100% SUBMITTAL)
DRY CREEK ON BURROUGH VALLEY ROAD BRIDGE REPLACEMENT
TOLLHOUSE ROAD AND BURROUGH VALLEY ROAD
BRIDGE NO 42C-0134
FRESNO COUNTY, CALIFORNIA

Prepared by:

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 09/30/2023
GEOTECHNICAL
STATE OF CALIFORNIA

Stephen P. Plauson, PE, GE
Senior Principal Geotechnical Engineer

Reviewed by:

REGISTERED PROFESSIONAL ENGINEER
DAVID L. PEARSON
No. 674
Exp. 12/31/2023
GEOTECHNICAL
STATE OF CALIFORNIA

David L. Pearson, PE, GE
Senior Principal Engineer

KLEINFELDER, INC.
3649 W. Holland Ave. STE 105
Fresno, California 93722
(559) 486-0750

March 7, 2023
Kleinfelder Project No.: 20164632

TABLE OF CONTENTS

COVER LETTER

FIGURES

APPENDICES

1. INTRODUCTION

1.1. GENERAL

This report presents the results of a geotechnical study for the proposed Dry Creek Bridge Replacement (Bridge No. 42C-0134) on Burrough Valley Road in Fresno County, California.

1.2. SCOPE OF WORK

The scope of services consisted of a field reconnaissance, field exploration, laboratory testing, engineering analysis, and preparation of this written report. The purpose of this Foundation Report is to evaluate the general soil conditions, and provide geotechnical recommendations and opinions to aid in the project design. The report provides the following:

- A description of the proposed project;
- Discussion of the field and laboratory testing programs;
- Comments on the regional geology and site engineering seismology, including the recommended Caltrans Seismic Design Criteria Version 1.7 ARS curve;
- Comments on liquefaction potential;
- Recommended parameters for use in design of the selected foundation type. Pile Data Table for rock sockets;
- LPILE parameters for lateral evaluation of piles;
- Comments regarding erosion, scour, and degradation from the project hydraulic analysis;
- Comments on soil stiffness and ultimate equivalent lateral pressure for resisting dynamic loading of abutment walls.
- Comments on the corrosion potential of foundation soil.
- Recommended pavement sections for bridge approaches.
- Log of Test Borings drawings suitable for inclusion into the contract drawings.

1.3. PROJECT DESCRIPTION

The proposed Dry Creek Bridge Replacement will include a two-span structure on Burrough Valley Road about 80 feet east of Tollhouse Road. Planning indicates the bridge will have a total length of 114 feet and a total width of 34.96 feet. The bridge will utilize a cast-in-place/prestressed concrete slab. Tables 1.3-1 through 1.3-3 provide data on the bridges furnished by Dewberry, the Project Structural Design Engineer.

TABLE 1.3-1
FOUNDATION DATA SHEET

Support	Location	Road Grade Elev. (ft)	Cut Off Elev. (ft)	Pile Cap Size (ft)		S_P[1]	No. Piles per Support
Abutment 1	10+84.5	1556.0	1549.75	4.0	34.7	1”	5
Pier 2	11+40.0	1548.5	1547.0	4.7	34.7	1”	4
Abutment 3	11+95.5	1560.0	1553.75	4.0	34.7	1”	5

Note: (1) Permissible settlement under service load

TABLE 1.3-2
FOUNDATION DESIGN LOADS

Support No.	Rock Socket Diameter (feet)	Service Limit State (kips)			Strength Limit State (kips)				Extreme Event Limit State (kips)			
		Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
		Per Support	Max Per Pile	Per Support	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Abut 1	3	970	205	700	1370	290	0	0	N/A	N/A	N/A	N/A
Pier 2	2.5	1525	450	965	2470	695	0	0	925	325	TBD	TBD
Abut 3	3	970	205	700	1370	290	0	0	N/A	N/A	N/A	N/A

Note: Loading provided by Dewberry and extrapolated for other loading conditions.

1.4. POLICY EXCEPTIONS

Other than the planned 2H:1V approach side slopes, no known exceptions to Caltrans policy were made in the geotechnical evaluation for the foundations for this project.

2. FIELD AND LABORATORY PROGRAMS

2.1. FIELD INVESTIGATION AND TESTING

The field exploration for the project was conducted on April 4, 5, and 6, 2016 and consisted of drilling four (4) test borings at the proposed bridge crossing. The test borings were drilled with a CME-55 truck-mounted drill rig using hollow stem auger and coring techniques. The borings were excavated to depths ranging 15.1 to 51.5 feet below the existing ground surface. The approximate locations of the test borings are indicated on the Log of Test Borings (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. Relatively undisturbed samples 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 hammer free falling a distance of 30 inches. In addition, samples of the subsurface material were obtained using a 1.4-inch I.D. standard penetrometer, driven 18 inches in accordance with ASTM D1586 test procedures. The sampler was used without liners. Resistance to sampler penetration was noted on the boring logs as the number of blows per foot over the last 12 inches of sampler penetration. The blow counts listed in the boring logs have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. Rock cores were obtained using a 4-inch O.D. HQ-3 core barrel. Core runs and the RQD for the run are noted on the Log of Test Borings. In addition, near surface bulk samples were obtained from auger cuttings at the test borings.

A Kleinfelder engineer logged the earth materials encountered during the drilling operation. Soil and core samples obtained were taken to the laboratory for geotechnical testing.

2.2. LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected samples to evaluate certain 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:

- In-place density and moisture content, American Society for Testing and Materials (ASTM D2937)
- Moisture Content (ASTM D2216)
- Direct Shear (ASTM D3080)
- Unconfined Compressive Strength (ASTM D7012)
- Grain Size Distribution (ASTM D422, w/o Hydrometer)
- R-value (California Test Method No. 301)
- pH and Minimum Resistivity (California Test Method No. 643)
- Soluble Sulfates (California Test Method No.417)
- Soluble Chlorides (California Test Method No.422)

The soluble sulfate, soluble chloride, pH, and minimum resistivity results are presented in Section 4 ("Corrosion Evaluation"). The remaining test results are provided in Appendix B.

Design geotechnical parameters were based on site specific laboratory data for project alignment and interpretation of the geology in the area. Consideration was also given to correlations with sample penetration rates. Tables 2.2-1 through 2.2-3 provide a summary of geotechnical design parameters and generalized soil profile used to characterize the site at Abutment 1, Pier 2, and Abutment 3, respectively.

TABLE 2.2-1
DESIGN PARAMETERS, ABUTMENT 1

Elevation (feet)	Material	γ_{total} (pcf)	Φ (°)	Unconfined Compressive Strength (psi)
1562 – 1544	SM	125	35	-
1544– 1536	Decomposed Granitic	140	40	-
Below 1536	Igneous Granitic Rock	150	-	1,100

TABLE 2.2-2
DESIGN PARAMETERS, PIER 2

Elevation (feet)	Material	γ_{total} (pcf)	Φ (°)	Unconfined Compressive Strength (psi)
1565 - 1545	SM	125	35	-
1545 – 1536	Decomposed Granitic	140	40	-
Below 1536	Igneous Granitic Rock	150	-	1,100

TABLE 2.2-3
DESIGN PARAMETERS, ABUTMENT 3

Elevation (feet)	Material	γ_{total} (pcf)	Φ (°)	Unconfined Compressive Strength (psi)
1566 - 1545	SM/SP	125	35	-
1545 - 1530	Decomposed Granitic	140	40	-
Below 1530	Igneous Granitic Rock	150	-	1,100

3. SITE GEOLOGY AND SUBSURFACE CONDITIONS

3.1. SURFACE CONDITIONS AND TOPOGRAPHY

The bridge site is located on the Burrough Valley Road about 80 feet east of Tollhouse Road in the foothills of the Sierra Nevadas. The natural terrain in the project area is relatively flat with gentle slopes towards Dry Creek. Dry Creek is a southeasterly flowing drainage. The elevation of the existing roadway at the existing bridge site is about 1559 to 1554 feet above sea level. The existing bridge is a 64-foot long three-span timber bridge over Dry Creek. The new alignment is planned along the existing bridge Vegetation in the project area consists of sparse to moderate growths of brush, and grass and scattered trees.

3.2. REGIONAL GEOLOGY

The project site is located along the western perimeter of the Sierra Nevada geomorphic province, between the Great Valley and Basin and Range Geomorphic provinces of California. More specifically, the site is located within the foothills that have formed at the base of the Sierra Nevada Mountain Range; a west-tilted fault block with comparatively little internal deformation and significant variation in topography. The Sierra Nevada Mountains were formed by the intrusion of the Sierra Nevada Batholith that began in the Jurassic time period and continued into the Cretaceous. The intrusion of the Sierra Nevada Batholith is represented by the granitic rocks that are currently visible. Uplift along normal faults of the Sierra Nevada frontal fault zone on the eastern side of the range and continued basin and range extension have resulted in the current topography of the area and continued building of the Sierra Nevadas.

3.3. EARTH MATERIALS

At the location of the proposed bridge, the native sediments in the project area have been mapped by Matthews et al., 1965 (Fresno 2° geologic sheet) by the California Geological Survey (CGS) as Mesozic granitic rocks.

The near surface soil consisted of silty sands varying in depths ranging from 7 to 11 feet below ground surface (bgs) for all borings, except for boring B-2. Boring B-2 began at the bridge deck where casing was driven into the creek channel 4.5' bgs. The top of the bridge deck to the channel surface below measured to be 9 feet. The near surface silty sand was underlain by poorly graded sand varying in depth from 13.5 to 15 feet bgs, except for boring B-1B. At boring B-1B, the near surface soil of silty sand was underlain by moderately weathered and intensely fractured granitic igneous rock beginning at a depth of 7' bgs. Drilling conditions seemed to get easier at 27' bgs and again became harder at 32' bgs. At boring B-2, the poorly graded sand was underlain by moderately weathered to intensely fractured decomposed igneous granitic rock to 23' bgs. Slightly weathered, granitic igneous rock was found beginning at 23' bgs and extending to 53' bgs where the boring was terminated. At B-3, the poorly graded sand was underlain by decomposed granite beginning at 15' bgs and extending to approximately 30' bgs. Slightly to moderately weathered, moderately fractured igneous granitic rock was encountered below the decomposed granite layer.

The above is a general summary of the earth material profile encountered in the borings drilled for this investigation. A more detailed description of the materials encountered in the test borings is noted on the Log of Test Borings drawing in Appendix A.

3.4. GEOLOGIC HAZARDS

Based on the relatively shallow bedrock, subsidence and landslides are not anticipated to be problematic to the structures.

The soils encountered at the site have a low expansion potential. The potential for heaving at the site is considered low.

3.5. GROUNDWATER CONDITIONS

Groundwater was encountered at about elevation 1544, 1550, and 1545 feet at B-1, B-2, and B-3, respectively. Water was present in the channel, at about elevation 1550 feet. It is anticipated that the local ground water will be perched on the bedrock and will coincide relatively close to the water level in Dry Creek.

3.6. SCOUR POTENTIAL

A preliminary hydraulic analysis was performed by Avila and Associates, dated June 14, 2016. Results indicate no long term degradation and contraction scour, and local pier scour dependent on the pile diameter. For Abutment 1, Pier 2, and Abutment 3 with a 36 inch, 30 inch, and 36 inch pile, respectively, pier scour is anticipated to be 13, 7.5, and 17 feet, respectively. This is the equivalent of elevation 1543 and 1541 at Abutments and Pier 2, respectively.

It is anticipated decomposed granitic rock is subject to scour. The non-erodible surface of the granitic bedrock is anticipated to be at approximately elevation 1536 feet at Abutment 1 and Pier 2, and elevation 1530 feet at Abutment 3.

4. CORROSION EVALUATION

Kleinfelder has completed laboratory testing to provide data regarding corrosivity of onsite soils. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services.

Bulk soil samples were obtained from boring B-3 at a depth of 15 feet and was tested to evaluate the pH, minimum resistivity, soluble sulfate content, and soluble chloride content. Specific test results are presented in Table 4.4-1. If fill materials will be imported to the project site, similar corrosion potential laboratory testing should be completed on the imported material.

TABLE 4.4-1
CORROSION RELATED TESTING

Boring and Depth	pH	Minimum Resistivity (ohm-cm)	Soluble Sulfate (mg/kg)	Soluble Chloride (mg/kg)
B-3 @ 15 ft.	7.9	1769	6.7	11.8

These laboratory tests indicate the resistivity, pH, soluble sulfates, and soluble chlorides are all outside the Caltrans threshold limits. Consequently, the site would be considered to be a non-corrosive environment with respect to 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 on material types and/or the possible need for mitigation.

5. SEISMIC RECOMMENDATIONS

5.1. LOCAL FAULTING

There are no known faults, which cut through 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).

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 low to moderate seismic activity. The more significant faults that could influence the project site include the Creeping Section of the San Andreas Fault (Fault ID No. 182), the Parkfield Section of the San Andreas Fault (Fault ID No. 214), and the Kern Canyon Fault (Fault ID No. 189). According to the Caltrans fault database, all sections of the San Andreas Fault are right lateral strike slip faults with dip angles of 90 degrees and assigned Maximum Magnitudes (M_{Max}) of 7.9. The Kern Canyon Fault is a normal fault with a dip angle of 60 degrees to the east and assigned Maximum Magnitudes of 7.5. The characteristics of these three faults are summarized in Table 5.2-1.

Based on Caltrans SDC 1.7, the estimate V_{s30} for the site is 302 m/s, which indicates the Soil Profile Type is D. A V_{s30} of 302 m/s was used for the ARS 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 = 36.992647 deg, Long = -119.413436 deg
Shear Wave Velocity	302 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) fault, No. 182
Maximum Magnitude (M_{Max})	7.9
Fault Type	Right Lateral Strike Slip
Fault Dip	90 degrees
Dip Direction	Vertical
Bottom of Rupture Plane	12 km
Top of Rupture Plane (Z_{tor})	0 km
R_{RUP}[1]	149.924 km
R_{jB}[2]	149.924 km
R_X[3]	149.923 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	0
Fault Name and ID Number	San Andreas fault (Parkfield), No. 214
Maximum Magnitude (M_{Max})	7.9
Fault Type	Right Lateral Strike Slip
Fault Dip	90 degrees
Dip Direction	Vertical
Bottom of Rupture Plane	6 km
Top of Rupture Plane (Z_{tor})	0 km
R_{RUP}[1]	151.210 km
R_{jB}[2]	151.210 km
R_X[3]	150.696 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	0
Fault Name and ID Number	Kern Canyon fault, No. 189
Maximum Magnitude (M_{Max})	7.5
Fault Type	Normal
Fault Dip	60 degrees
Dip Direction	E
Bottom of Rupture Plane	14.6 km
Top of Rupture Plane (Z_{tor})	0 km
R_{RUP}[1]	98.480 km
R_{jB}[2]	98.480 km
R_X[3]	88.472 km
F_{norm} (1 for normal, 0 for others)	1
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. The Minimum Spectrum for California is associated with a Moment Magnitude 6.5 earthquake about 12 km from the site.

5.2.2 Probabilistic Response Spectrum

The probabilistic response spectrum was developed using the ARS Online and verified with the 2008 USGS Deaggregation website.

5.2.3 Design Response Spectrum

The upper envelope of the deterministic and probabilistic spectral values determines the design response spectrum. The design ARS curve is governed by Minimum Deterministic Spectrum for California for periods up 0.60 seconds and by the probabilistic spectrum for periods greater than 0.60 seconds. 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/shake_stable/v2/.
Caltrans. Geotechnical Services Manual.
Caltrans. Seismic Design Criteria, Appendix B Design Spectrum
Caltrans. Website http://dap3.dot.ca.gov/shake_stable/v2/technical.php
USGS. Website http://geohazards.usgs.gov/deaggint/2008/

5.3. LIQUEFACTION POTENTIAL

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.

Geologic age also influences the potential for liquefaction. Sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are even more resistant; and pre-Pleistocene sediments are generally immune to liquefaction (Youd, 2001).

The Caltrans design peak horizontal ground acceleration (PHGA) for the site is 0.23g. Based on the anticipated subsurface conditions and ground shaking at the site, liquefaction or seismically induced settlement, lateral spread or bearing loss is considered unlikely.

6. FOUNDATION RECOMMENDATIONS

6.1. GENERAL

The bridge site is underlain by granitic bedrock. Based on the conditions encountered in the test borings, Table 6.1-1 provides the estimated decomposed granitic and bedrock elevation for each support.

TABLE 6.1-1
ESTIMATED BEDROCK ELEVATIONS

Support	Location	Estimated Decomposed Bedrock Elevation (feet)	Estimated Bedrock Elevation (feet)
Abutment 1	10+84.5	1544	1536
Pier 2	11+40.0	1545	1536
Abutment 3	11+95.5	1545	1530

At present, it is anticipated that scour elevations would preclude the use of spread footings. Therefore, design is based on rock socket foundations.

6.2. PILE FOOTINGS

6.2.1. Axial Capacity

Table 6.2-1 provides the design tip elevations.

TABLE 6.2-1
FOUNDATION RECOMMENDATIONS

Support Location	Rock Socket Diameter (feet)	Required Factored Nominal Resistance per Pile (kips)			Design Pile Tip Elevation (ft)	Specified Tip Elevation (ft)
		Service Limit ?=0.5	Strength Limit ?=0.7	Extreme Limit ?=1.0		
Abut 1	3	205	290	NA	1533(a), 1533(a-1), 1534(c), 1528(d)	1528
Pier 2	2.5	450	695	325	1529(a), 1528(a-1), 1532(c), 1528(d)	1528
Abut 3	3	205	290	NA	1527(a), 1527(a-1), 1528(c), 1522(d)	1522

Notes:

(1) Design tip elevations are controlled by: (a) Compression (Service Limit), (a-1) compression (Strength Limit), (a-11) compression (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 lateral tip elevation is based on less than 0.01 inch tip deflection under fixed head condition.

(4) Tip elevations are based on estimated bedrock elevations. Piles shall penetrate the same embedment depth into competent granitic bedrock, as determined by a geologic observation, as estimated by design.

6.2.2. <u>Foundation Construction Considerations</u>

Pile borings below groundwater will require temporary casing, or use of a slurry seal.

6.2.3. <u>Lateral Capacity</u>

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 6.2-2 through 6.2-4 can be used for evaluation of lateral loading of piles.

TABLE 6.2-2
LPILE PARAMETERS – ABUTMENT 1

Elev. (feet)	Recommended P-Y Curve	⍰' (pcf)	q_u (psi)	E_{RI} (ksi)	RQD (%)	k_{rm}	⍰ (°)	k (pci)
1562 to 1545	Sand (Reese)	125	-	-	-	-	35	110
1545 to 1536	Sand (Reese)	78	-	-	-	-	40	140
Below 1536	Weak Rock (Reese)	88	1100	1900	0	0.0001	-	-

TABLE 6.2-3
LPILE PARAMETERS – PIER 2

Elev. (feet)	Recommended P-Y Curve	⍰' (pcf)	q_u (psi)	E_{RI} (ksi)	RQD (%)	k_{rm}	⍰ (°)	k (pci)
1550 to 1545	Sand (Reese)	63	-	-	-	-	35	110
1545 to 1536	Sand (Reese)	78	-	-	-	-	40	140
Below 1536	Weak Rock (Reese)	88	1100	1900	0	0.0001	-	-

TABLE 6.2-4
LPILE PARAMETERS – ABUTMENT 3

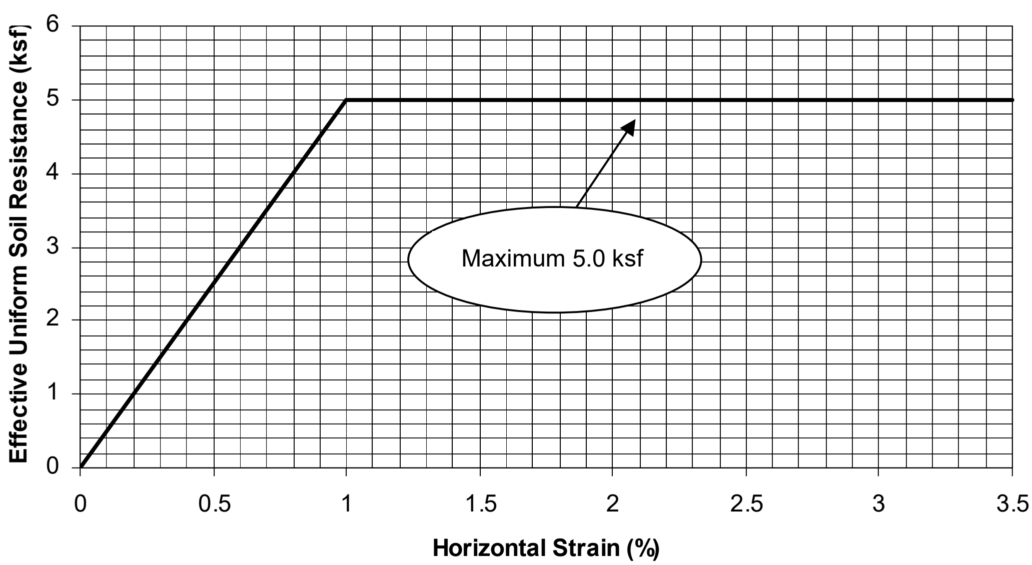
Elev. (feet)	Recommended P-Y Curve	⍰' (pcf)	q_u (psi)	E_{RI} (ksi)	RQD (%)	k_{rm}	⍰ (°)	k (pci)
1565 to 1545	Sand (Reese)	125	-	-	-	-	35	110
1545 to 1530	Sand (Reese)	78	-	-	-	-	40	140
Below 1530	Weak Rock (Reese)	88	1100	1900	0	0.0001	-	-

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, an initial abutment soil stiffness of 50 kip/in/ft is recommended. The ultimate lateral resistance that may be applied against abutment 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% 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 ft. When the abutment height is less than 5.5 ft, 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. EARTH WORK

Any required earth work should be performed in accordance with Section 19 of the latest version of the Caltrans Standard Specifications.

6.5 PAVEMENT

The subgrade Resistance-value (R-value) for the proposed roadway subgrade was evaluated in the laboratory on near surface soil samples obtained from the test boring B-1. Testing was in conformance with California Test Method 301. Results indicate a R-value of 51. A design R-value of 50 is recommended.

Flexible pavement sections have been determined for a Traffic Index (TI) of 8.5 as designated by Fresno County. Estimated structural sections for asphalt concrete (HMA) are provided in Table 6.5-1. The pavement design recommendations presented are based upon the California Department of Transportation (Caltrans) design procedures, including the gravel equivalent safety factor on the wearing surface.

TABLE 6.5-1
PRELIMINARY PAVEMENT SECTIONS

Assumed TI	Design R-value	Pavement Structural Section
8.5	50	0.40’ HMA / 0.55’AB

The HMA should conform to, and be placed in accordance with, Section 39 of the latest revision of the Caltrans Standard Specifications (CSS). Class 2 aggregate base (AB) should be in conformance with Section 26 of the CSS. AB and at least the upper 0.65 feet of subgrade should be compacted to 95% of maximum density. Subgrade should be compacted in accordance with Fresno County Standards, if more stringent.

7. CLOSURE

The conclusions and recommendations in this report are for the design of the proposed Dry Creek on Burrough Valley Road Bridge Replacement near Tollhouse Road in Fresno County, California, as described in the text of this report. The findings, conclusions, and recommendations presented in this report are based on the test borings performed, data developed, and site observations and were prepared in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions, and at the date the services are provided. Conditions may vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, expressed, or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided. 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 Dewberry, County of Fresno, and project 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

CAD FILE: L:CADD\2016\20164632\BurroughValleyRd-Bridge_5-2016\ LAYOUT: 1 PLOTTED: 24 May 2016, 9:44am, MGriffin

SOURCE: U.S.G.S./A.A.S.G. AERIAL, FRESNO COUNTY, CALIFORNIA, DATED 4-2015.

EXPLANATION

APPROXIMATE SITE LOCATION

N

2,000 70 0 2,000

APPROXIMATE SCALE (feet)

The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. Kleinfelder makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.

KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

IRVINE, CA

PROJECT NO.	20164632
DRAWN:	5/2016
DRAWN BY:	MRG
CHECKED BY:	NP
FILE NAME:	20164632-F1_SVM.dwg

SITE VICINITY MAP

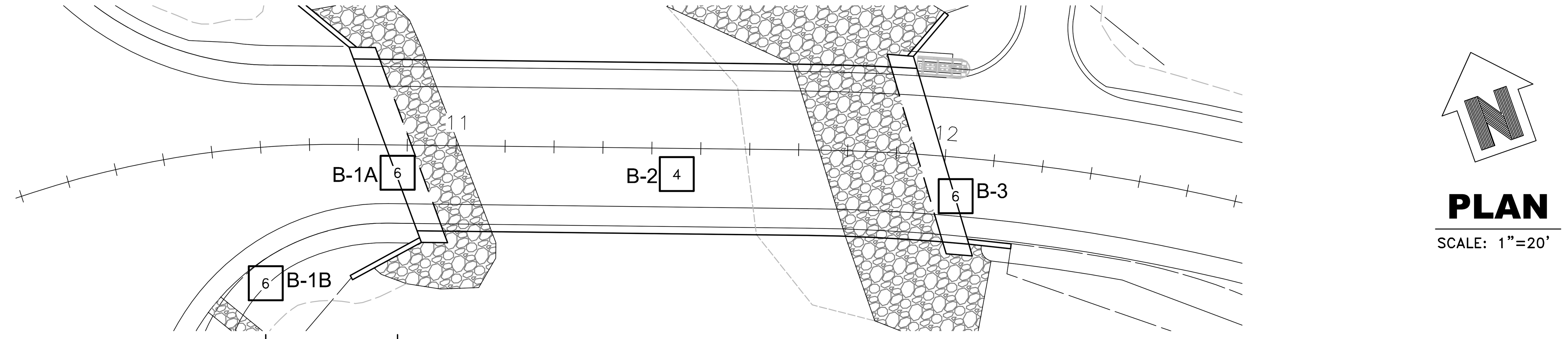
BURROUGH VALLEY ROAD BRIDGE
(OVER DRY CREEK)
FRESNO COUNTY, CALIFORNIA

FIGURE

1

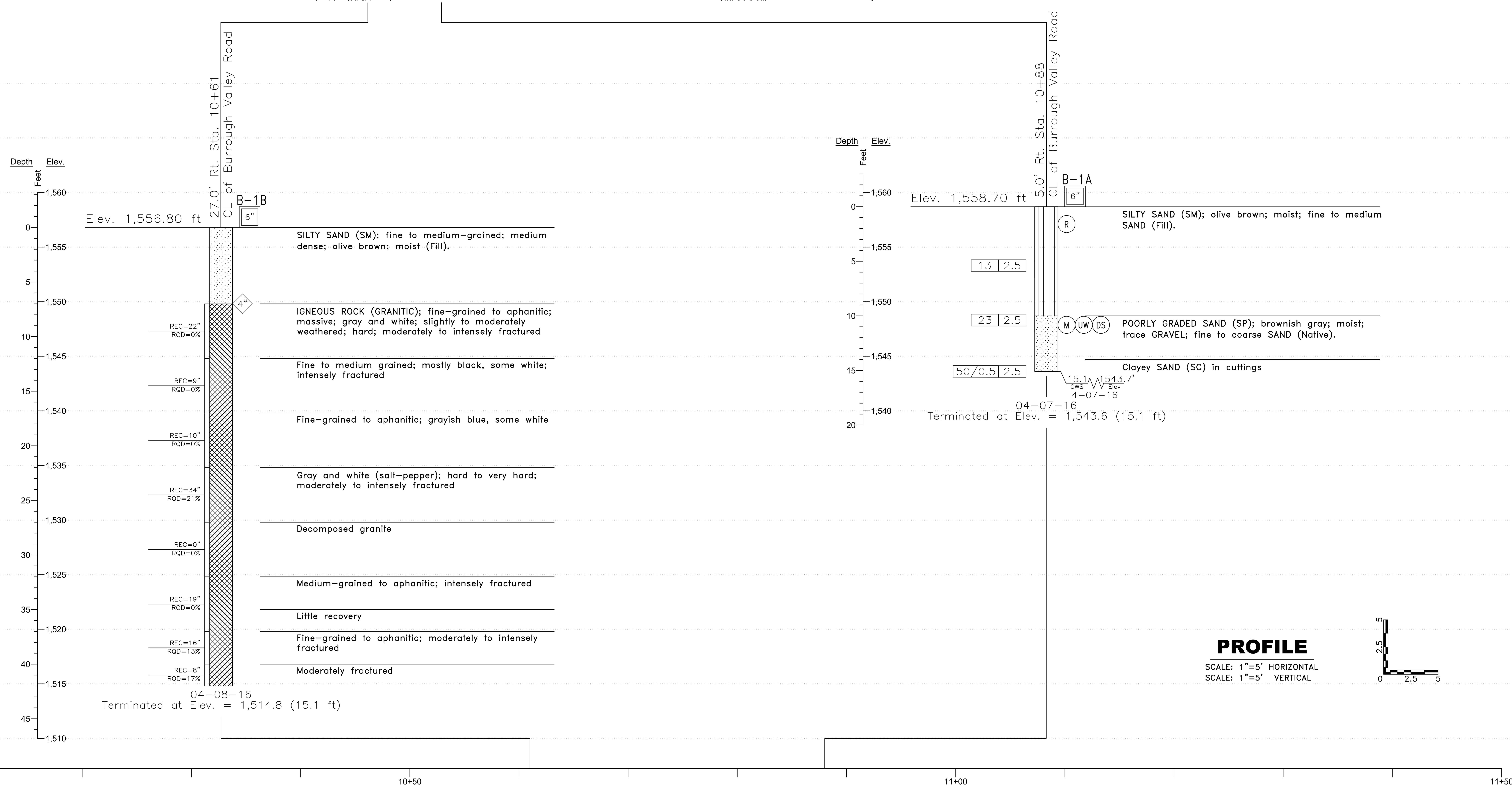
NOTES:

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. ELEVATIONS BASED ON TOPOGRAPHIC MAP PROVIDED BY DRAKE HAGLAN & ASSOCIATES.



PLAN

SCALE: 1"=20'



PROFILE

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

	DATE	RECORD DRAWING		SCALE
DESIGNED:		RESIDENT ENGINEER	DATE	AS SHOWN
DRAWN: D. FAHRNEY				
CHECKED: N. DAHLEN				

FOR RIGHT OF WAY DATA AND ACCURATE ACCESS DETERMINATION, SEE DOCUMENTS IN THE DEPARTMENT OF PUBLIC WORKS AND PLANNING.

STEPHEN P. PLAUSON — SUPERVISING ENGINEER — 2/15/23 DATE

REGISTERED PROFESSIONAL ENGINEER STEPHEN P. PLAUSON No. 2731 Exp. 09/30/2023 GEOTECHNICAL STATE OF CALIFORNIA

PROJECT
DRY CREEK ON BURROUGH VALLEY ROAD
BRIDGE REPLACEMENT
BRIDGE NO. 42C-0710

ROAD NO. BRIDGE NO. 42C-0710

THE COUNTY OF FRESNO 1856

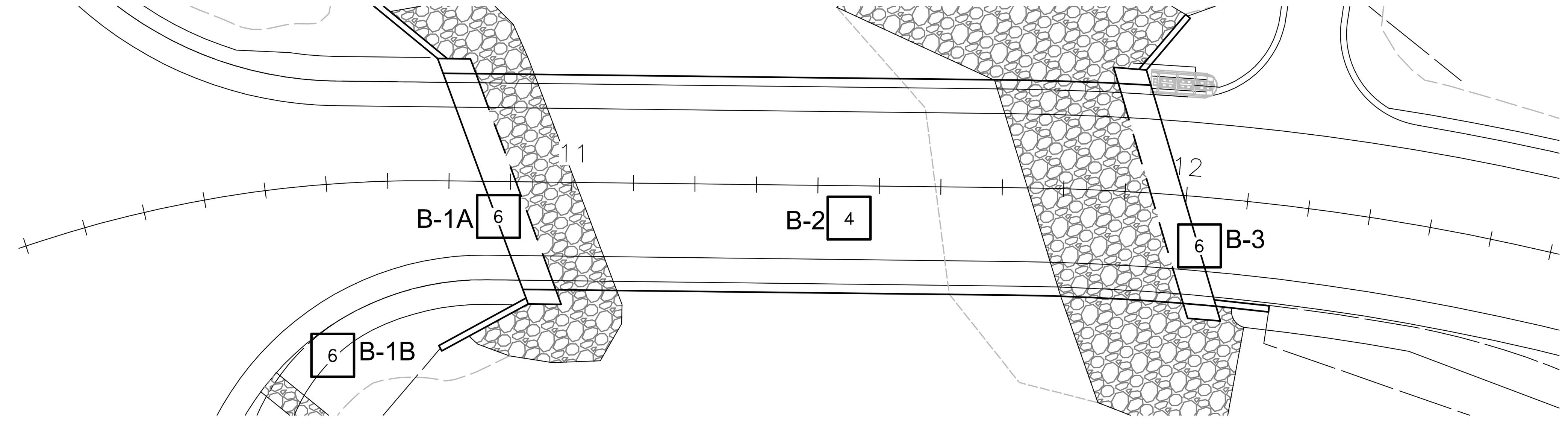
DEPARTMENT OF PUBLIC WORKS AND PLANNING

LOG OF TEST BORINGS
1 OF 2

DRAWING NO. 11278 SHEET NO. TOTAL

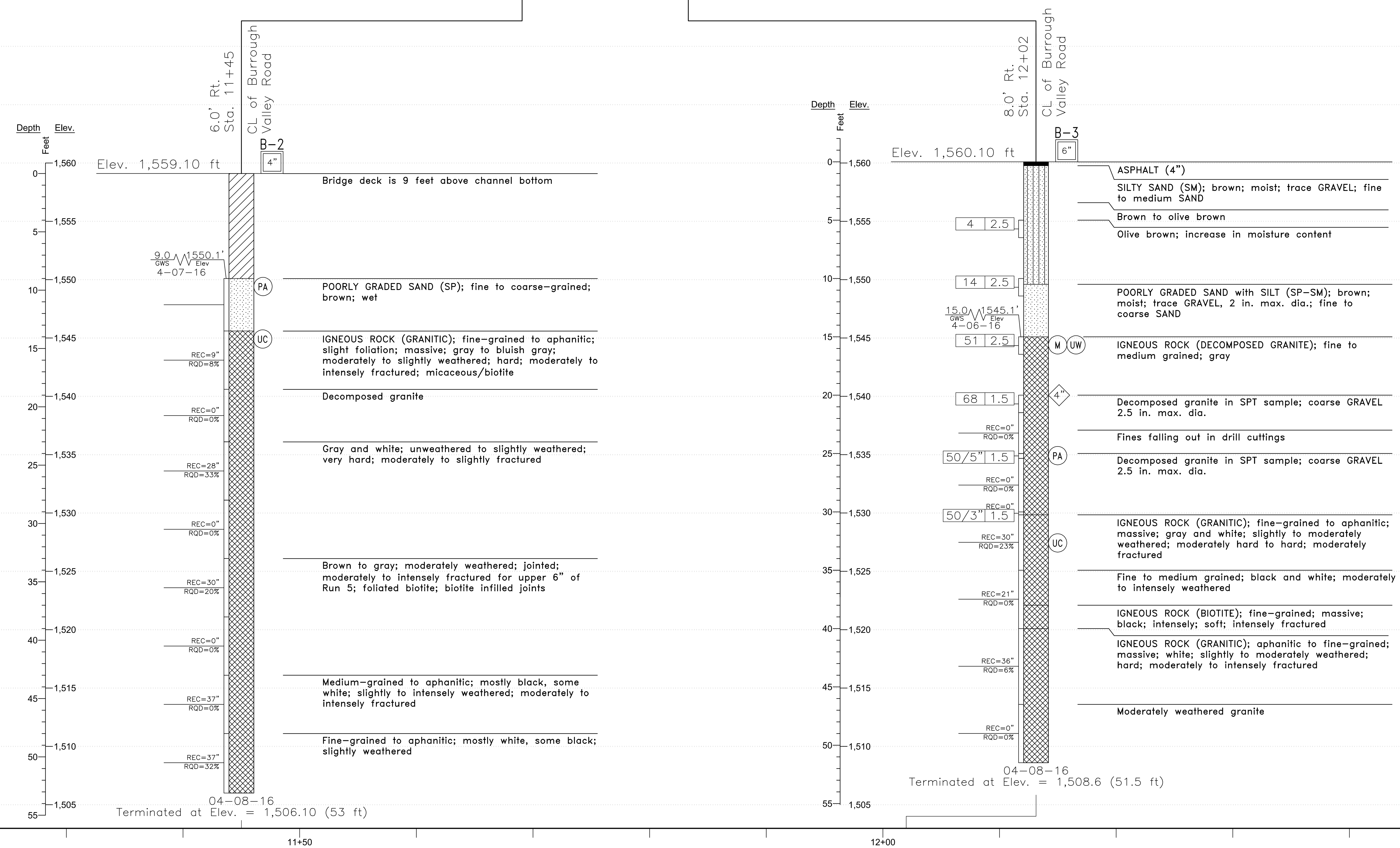
NOTES:

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. ELEVATIONS BASED ON TOPOGRAPHIC MAP PROVIDED BY DRAKE HAGLAN & ASSOCIATES.



PLAN

SCALE: 1"=20'



PROFILE

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

	DATE	RECORD DRAWING		SCALE			PROJECT		DEPARTMENT OF PUBLIC WORKS AND PLANNING
DESIGNED:		RESIDENT ENGINEER	DATE	AS SHOWN	STEPHEN P. PLAUSON, SUPERVISING ENGINEER	2/15/23 DATE	DRY CREEK ON BURROUGH VALLEY ROAD BRIDGE REPLACEMENT BRIDGE NO. 42C-0710		LOG OF TEST BORINGS 2 OF 2
DRAWN: D. FAHRNEY									
CHECKED: N. DAHLEN							ROAD NO.	BRIDGE NO. 42C-0710	DRAWING NO. 11278 SHEET NO. TOTAL

FOR RIGHT OF WAY DATA AND ACCURATE ACCESS DETERMINATION, SEE DOCUMENTS IN THE DEPARTMENT OF PUBLIC WORKS AND PLANNING.

REGISTERED PROFESSIONAL ENGINEER STEPHEN P. PLAUSON No. 2731 Exp. 09/30/2023 GEOTECHNICAL STATE OF CALIFORNIA

THE COUNTY OF FRESNO 1856

APPENDIX A

LABORATORY TESTS

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-1A	0.0 - 5.0	SILTY SAND (SM)									**R-Value=** 51
B-1A	10.0	POORLY-GRADED SAND (SP)									**Direct Shear=**
											Peak Cohesion: 0.51 tsf
											Peak Friction Angle: 40.0°
B-2	10.0	POORLY-GRADED SAND (SP)				100	5.6				
B-3	15.0	DECOMPOSED GRANITE	10.9	140.0							**pH=** 7.92
											Resistivity= 17690
											Sulfates= 6.7
											Chlorides= 11.8
B-3	25.0	DECOMPOSED GRANITE				97	6.1				

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

KLEINFELDER
Bright People. Right Solutions.

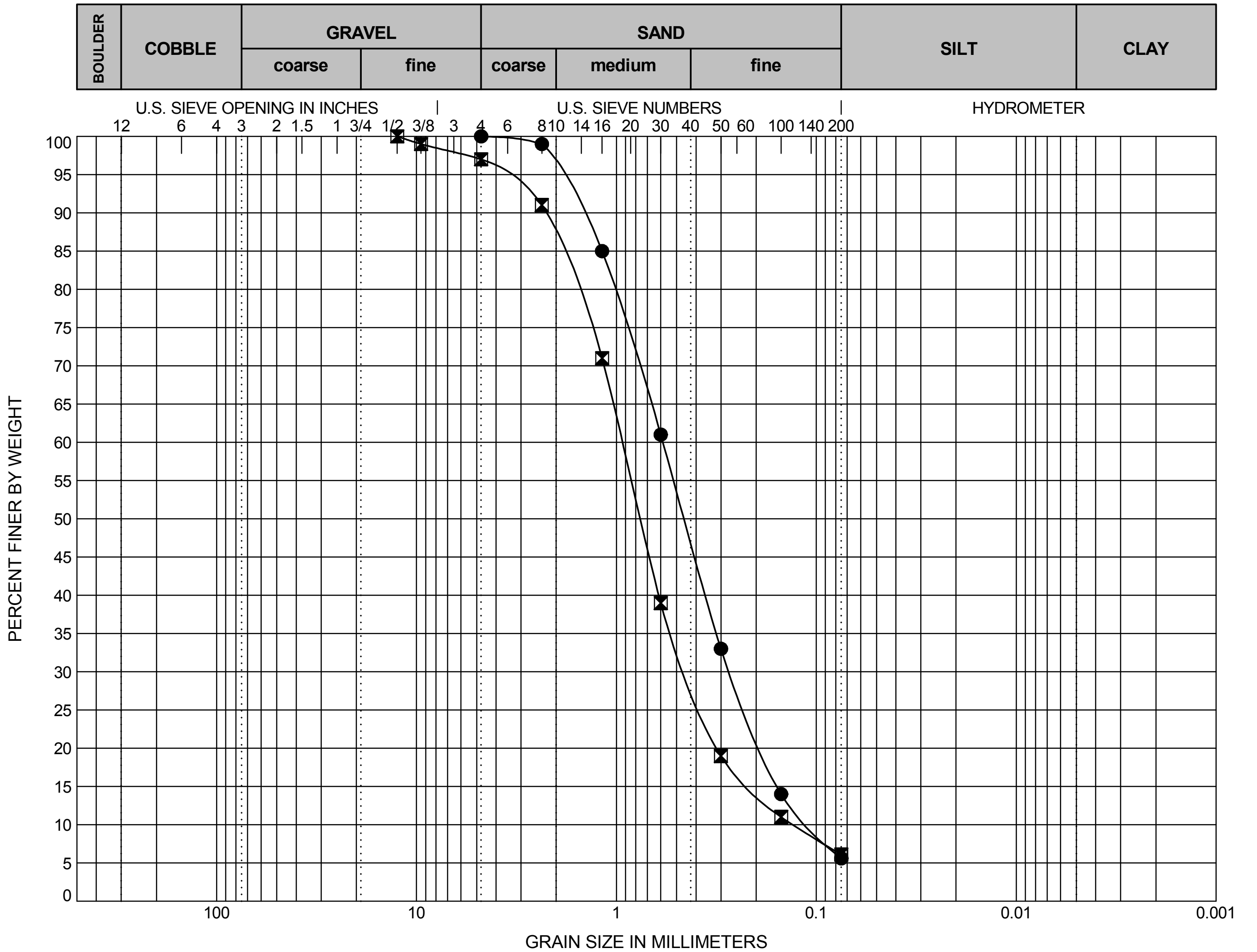
PROJECT NO.: 20164632
DRAWN BY:
CHECKED BY:
DATE:
REVISED: -

LABORATORY TEST RESULT SUMMARY

BURROUGH VALLEY ROAD BRIDGE
(OVER DRY CREEK)
FRESNO COUNTY, CALIFORNIA

FIGURE

A-1



Exploration ID		Depth (ft.)	Sample Description	LL	PL	PI
●	B-2	10	POORLY-GRADED SAND (SP)	NM	NM	NM
⧗	B-3	25	DECOMPOSED GRANITE	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	10	4.75	0.585	0.269	0.108	1.15	5.43		100	5.6	NM	NM
⧗	B-3	25	12.5	0.935	0.439	0.13	1.58	7.18		97	6.1	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

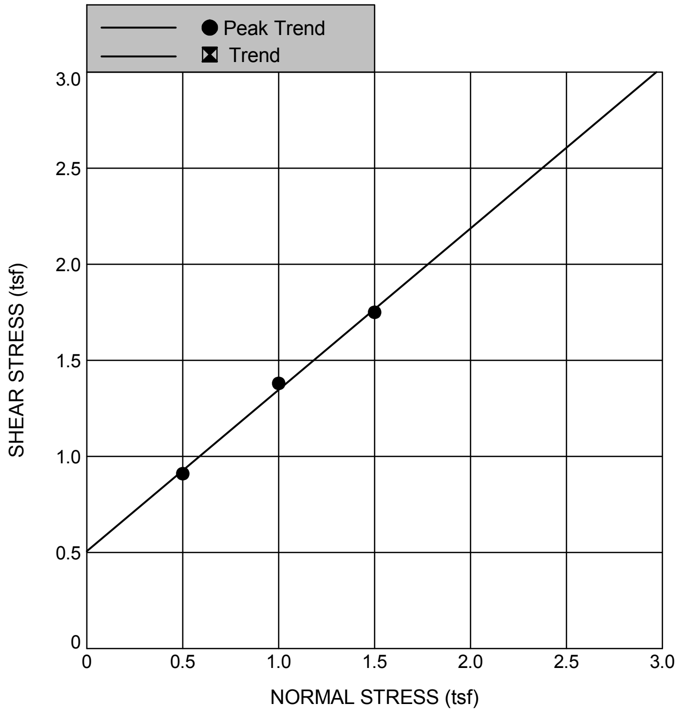
KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.: 20164632
DRAWN BY: INR
CHECKED BY:
DATE:
REVISED: -

SIEVE ANALYSIS

BURROUGH VALLEY ROAD BRIDGE
(OVER DRY CREEK)
FRESNO COUNTY, CALIFORNIA

FIGURE
A-2



Exploration ID	Depth (ft.)	Sample Description
B-1A	10	POORLY-GRADED SAND (SP)

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

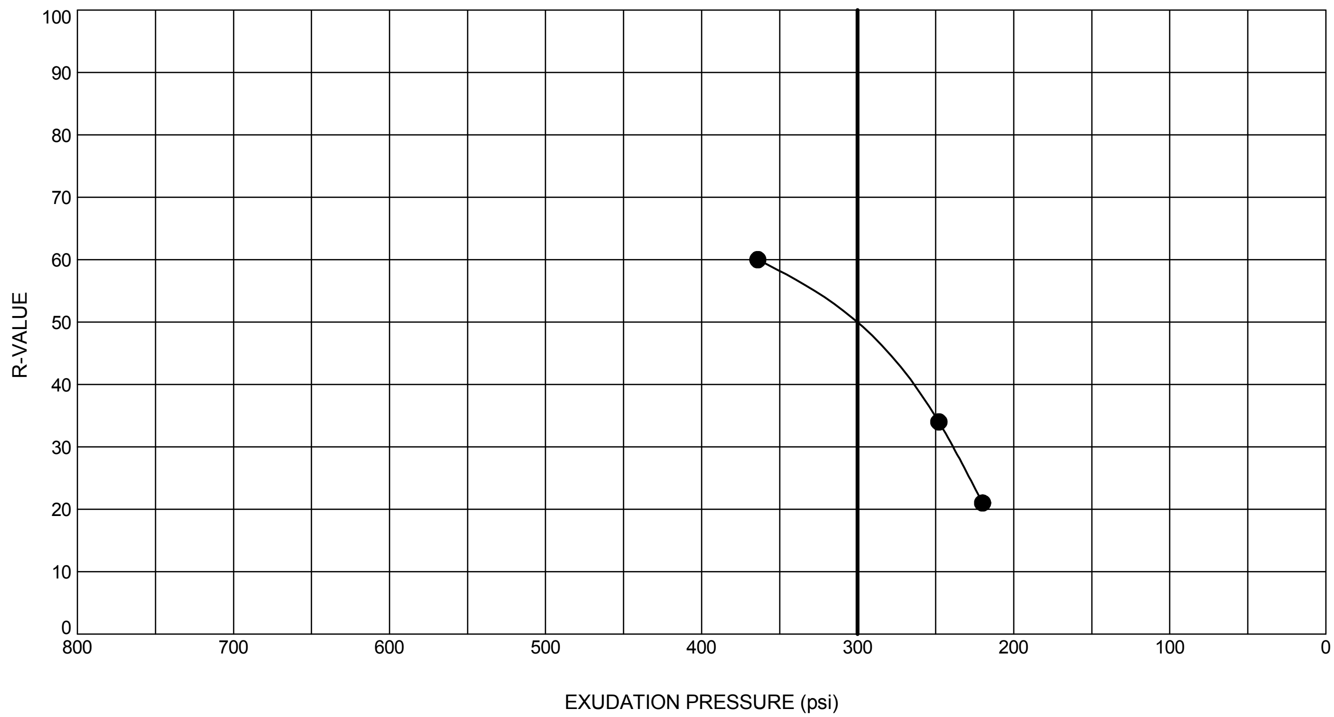
	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1	12.8	107.0	58.5	0.603	4.60	1.00
	2	13.8	108.4	65.0	0.583	4.60	1.00
	3	15.0	108.6	71.2	0.581	4.60	1.00
	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
At Test	1	19.6	107.5	94.5	0.572	4.60	0.98
	2	21.6	109.2	100.1	0.593	4.60	1.01
	3	19.1	112.8	96.9	0.541	4.60	0.97

Specimen No.	Peak Shear Stress (tsf)	Shear Stress (tsf)	Horizontal Displacement (in)	Normal Stress (tsf)	Strain Rate (in/min)
1	0.91		0.0800	0.5	0.01
2	1.38		0.1400	1	0.01
3	1.75		0.1000	1.5	0.01

Results	Cohesion (tsf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	0.51	40.03	

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

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164632 DRAWN BY: INR CHECKED BY: DATE: REVISED: -	DIRECT SHEAR BURROUGH VALLEY ROAD BRIDGE (OVER DRY CREEK) FRESNO COUNTY, CALIFORNIA	FIGURE A-3



Exploration ID	Depth (ft.)	Sample Description	R-Value @ 300 psi Exudation Pressure
B-1A	0 - 5	SILTY SAND (SM)	51

Specimen No.	Moisture at Time of Test (%)	Dry Unit Weight (pcf)	Expansion Pressure (psi)	Exudation Pressure (psi)	Corrected Resistance Value
1	12.7	120.9	0	220	21
2	11.8	122.5	0	248	34
3	10.9	123.5	0	364	60

Testing perfomed in general accordance with ASTM D2844.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20164632 DRAWN BY: INR CHECKED BY: DATE: REVISED: -	R-VALUE BURROUGH VALLEY ROAD BRIDGE (OVER DRY CREEK) FRESNO COUNTY, CALIFORNIA	FIGURE A-4

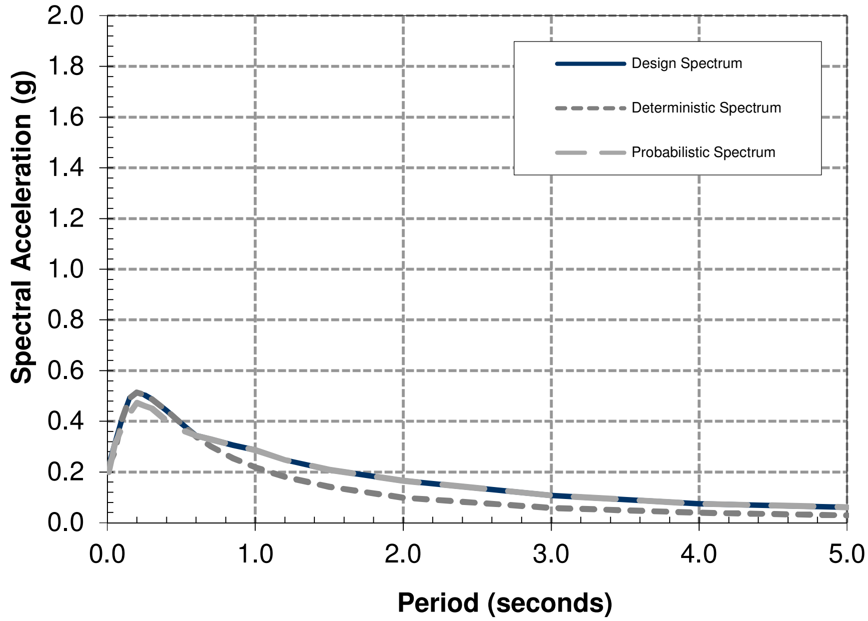
APPENDIX B

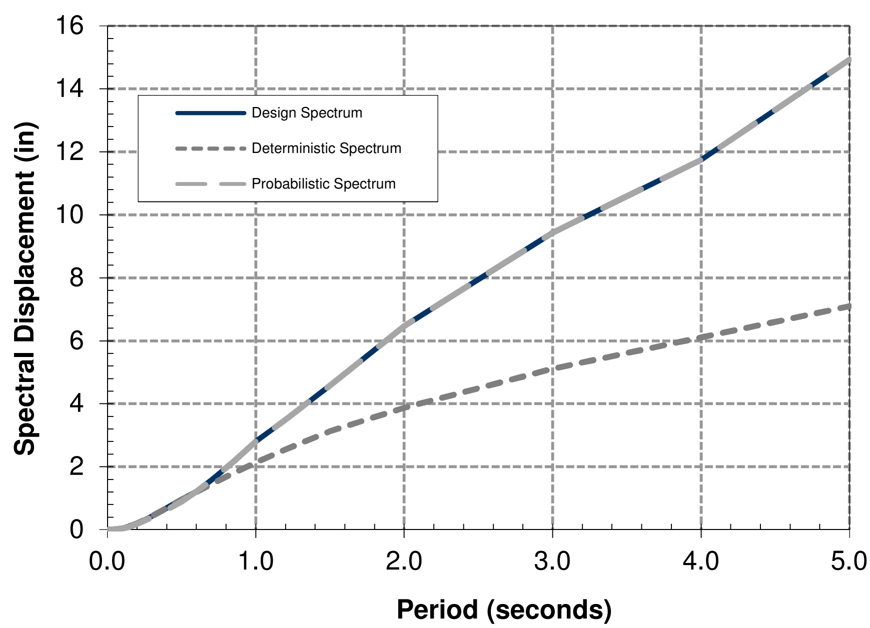
ARS CURVE

SITE DATA

Latitude:	36.9926	Shear Wave Velocity	302 m/s
Longitude:	-119.4134	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)	0.227	0.00
0.05	0.310	0.01
0.1	0.409	0.04
0.15	0.491	0.11
0.2	0.514	0.20
0.25	0.504	0.31
0.3	0.487	0.43
0.4	0.442	0.69
0.5	0.392	0.96
0.6	0.344	1.21
0.7	0.328	1.57
0.85	0.305	2.16
1	0.287	2.81
1.2	0.248	3.50
1.5	0.208	4.58
2	0.165	6.46
3	0.107	9.43
4	0.075	11.75
5	0.061	14.93





KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com

PROJECT NO	20164623	**CALTRANS ARS CURVES**	FIGURE
DRAWN:	6/14/16	BURROUGH VALLEY ROAD BRIDGE (OVER DRY CREEK) FRESNO COUNTY, CALIFORNIA	**B-1**
DRAWN BY:	INR		
CHECKED BY:	NMP		

APPENDIX C

GBA GEOTECHNICAL REPORT INFORMATION

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will <u>not</u> likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do <u>not</u> rely on an executive summary. Do <u>not</u> read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org