

FOUNDATION REPORT

TRAVERS CREEK BRIDGE ON EAST MANNING AVENUE (REPLACE)
(BRIDGE NO.: 42C-0687)
FRESNO COUNTY, CALIFORNIA

For

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1.0 INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed Travers Creek Bridge (Br. No. 42C-0687) replacement project (Project) to be constructed in Fresno County, California. The bridge is on East Manning Avenue over Travers Creek, east of Reedley. The project location is shown on the Project Location Map, Plate No. 1.

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

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

Please note that the project was initiated in 2013, and a draft Foundation Report was submitted in 2016. This report was prepared with updated recommendations based on recent design guidelines, as discussed with the design team. This report supersedes the draft Foundation Report dated January 14, 2016.

2.0 PROJECT DESCRIPTION

The existing Travers Creek Bridge (Br. No. 42C0175) was built in 1925 and widened once in 1942. The Caltrans Bridge Inspection Report (2008) has rated the bridge as functionally obsolete. The County of Fresno plans to replace the existing bridge with a new bridge that will be designed to meet current AASHTO standards. Based on the plans provided by Dewberry (Designer), dated July 6, 2026, the new bridge will be a single-span structure of about 64.5 feet in length and 77 feet (skewed) in width. The new bridge will have two 12-foot-wide traffic lanes and two 25-foot-wide shoulders. The bridge deck will be composed of pre-stressed, precast voided concrete slabs.



3.0 EXCEPTIONS TO POLICIES AND PROCEDURES

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

4.0 GEOTECHNICAL INVESTIGATION

Two borings (R-14-001 and R-14-002) were drilled to a depth of approximately 81.5 feet below grade with a truck-mounted drill rig on January 27 and 28, 2014. The drilling was conducted with hollow-stem augers and the mud rotary wash method. Selected soil samples were obtained using either a 2.4-inch I.D. Modified California (MC) or 1.4-inch I.D. Standard Penetration Test (SPT) sampler at various depths. The samplers were driven into subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Log of Test Borings (LOTB) in Appendix A.

The drilling subcontractor was Technicon Engineering Services, Inc. of Fresno, California. Based on the hammer energy calibration information provided by the drilling company, the hammer energy ratio of the drill rig (CME 55) used is approx. 85%. Using a method suggested by Daniel, Howie, and Sy (2003), when correlating standard penetration data, the blow counts for the Modified California Sampler may be converted to equivalent SPT blow counts by multiplying a conversion factor of 0.65.

The soil samples were sealed and transported to our laboratory for further evaluation and testing. Two bulk soil samples within the upper about 5 feet of subgrade were also collected for R-value tests. The field investigation was conducted under the supervision of our field engineer, who logged the test borings and prepared the samples for subsequent laboratory testing and evaluation. The approximate boring locations are shown on the Site Plan, Plate No. 2.

5.0 LABORATORY TESTING PROGRAM

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



6.0 GEOTECHNICAL CONDITIONS

6.1 Geology

General geologic features pertaining to the bridge site were evaluated by reference to California Geological Survey (CGS 2010), Geologic Data Map No. 2, compiled by Charles W. Jennings (1977). Based on the publication, the project site and its vicinity are generally underlain by late Tertiary to Quaternary sediments including geologic units of Q and Qoa. Q represents Pleistocene to Holocene alluvium, lake, playa, and terrace deposits; unconsolidated and semi-consolidated. Qoa includes older Pleistocene to Holocene alluvium, lake, playa, and terrace deposits. A portion of the published Geologic Map covering the project site is attached as Plate No. 3.

6.2 Subsurface Conditions

The subsurface conditions were evaluated based on the results of the field explorations. Based on the Foundation Plan, dated July 6, 2026, the existing ground surface elevations are estimated to be at approximately Elev. 356 and 357 feet at Borings R-14-001 and R-14-002, respectively.

Boring R-14-001 encountered predominantly medium dense to very dense silty and clayey sand to the maximum depth drilled, approximately 81.5 feet below grade. Hard silt was encountered at depths approximately between 49 and 63 feet and between 73 and 78 feet.

Boring R-14-002 encountered primarily medium dense to very dense silty and clayey sand to the maximum depth drilled, approximately 81.5 feet below grade. Loose silty sand was encountered at the top about 9 feet. Hard silt was encountered at depths approximately between 24 and 29 feet and between 64 and 78 feet.

Groundwater was not encountered in the upper approximately 21.5 feet in Boring R-14-001 or 26.5 feet in Boring R-14-002 prior to advancing the borings using the rotary wash drilling method. Historical groundwater monitoring data from a California Department of Water Resources monitoring well (Well Site Code 366039N1194079W001) located approximately 550 feet west of the Project indicate that groundwater levels historically ranged from approximately 16.4 to 74.6 feet below ground surface. The monitoring record extends through March 18, 1998, when the final groundwater measurement of 39 feet below ground surface (bgs) was collected prior to destruction of the well. The shallowest recorded groundwater level was approximately 16.4 feet bgs in March 1970 and the lowest was 75 feet bgs in November 1992. Accordingly, groundwater in the vicinity of the Project may rise to within approximately 16 feet of the ground



surface under favorable hydrologic conditions. Groundwater elevations may fluctuate over time due to seasonal precipitation, irrigation practices, groundwater pumping, nearby surface water conditions, and other natural or anthropogenic factors that may not have been present during the subsurface investigation.

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

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

7.0 SCOUR DATA

According to the results of the scour Analysis by Avila and Associates (July 2026), the scour information is tabulated below:

Table 1 – Scour Data

Support Location	Long Term Scour (Degradation and Contraction) Elevation (feet)	Short Term Scour (Local) Depth (feet)
Abutment 1	343	4
Abutment 2	343	4

According to Table 3.7.5-1 of the AASHTO Load and Resistant Factor Design (LRFD) Bridge Design Specifications, Eighth Edition (AASHTO LRFD BDS 2017) with California Amendments, the effects due to 100% channel degradation/aggradation and contraction scour plus 100% local scour shall be considered at the Service limit state; 100% channel degradation/aggradation and contraction scour plus 50% local scour shall be considered in Strength limit state load combinations. For the Extreme Event I limit state, 100% degradation/aggradation and 100% contraction scour should be considered, but local scour should not be included in structural or geotechnical design.



8.0 CORROSION EVALUATION

The corrosion investigation for this project was performed on selected soil samples in general accordance with the provisions of California Test Methods 643, 417, and 422. A summary of the corrosion test results is presented in Table 2 below.

Table 2 – Corrosion Test Results

Boring No.	Depth (ft)	pH	Minimum Resistivity (ohms-cm)	Chloride Content (ppm)	Sulfate Content (ppm)	Corrosive?
R-14-001	21	7.59	2,950	8.8	44.8	No
R-14-002	3	7.44	6,700	5.4	21.0	No

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. Except for MSE, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,500 ohm-cm.

Based on the corrosion test results, the soils in the tested soil samples shown in the above table are not considered corrosive per Caltrans Corrosion Guideline Version 3.2, May 2021.

9.0 SEISMIC RECOMMENDATIONS

9.1 Seismic Sources

The project site is located in a seismically active part of northern California. Many faults in the region are capable of producing earthquakes, which may cause strong ground shaking at the site. The proposed bridge site is located at coordinates of approximately 36.6040 degrees north latitude and 119.4059 degrees west longitude. The information of the relevant faults in the area is summarized in Table 3. The maximum magnitudes represent the largest earthquake that a fault is capable of generating and is related to the seismic moment. The attached Caltrans ARS Online Map, Plate No. 4, presents the location of the fault system relative to the project site.

Table 3 – Caltrans ARS Online Information

Fault	Fault ID	Maximum Magnitude (Mmax)	Fault Type	Approx. Distance Rrup/Rx (miles)
Kern Canyon	189	7.5	N	54.9/54.9
Great Valley 13 (Coalinga)	205	7.0	R	50.7/50.2
San Andreas (Creeping Section) 2011 CFM	182	7.9	SS	75.6/74.9
San Andreas (Parkfield)	214	7.9	SS	75.5/75.5

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



9.2 Seismic Design Criteria

The Caltrans ARS Online program (V4.0) was used for producing acceleration response spectra. Development of the design ARS curve is based on site location (longitude and latitude) and the average shear wave velocity for the top 100 feet of soils (V_{s30}).

The shear wave velocities for the top 100 feet of soils at the bridge site were estimated using the established correlations and guidelines in Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (2012). An average shear wave velocity of 240 m/sec was adopted.

Table 4 – Ground Motion Parameters

Site Parameters			Design Ground Motion Parameters (1) (Return Period = 975 years)			Soil Profile Class ⁽³⁾
Locations		Shear-Wave Velocity ⁽²⁾ VS,30m, m/sec	Horizontal Peak Ground Acceleration (HPGA), g	Deaggregated Mean Earthquake Moment Magnitude for PGA	Deaggregated Mean Site-to-Fault Distance for 1.0 Period Spectral Acceleration, km	
Latitude, degrees	Longitude, degrees					
36.6040	-119.4059	240	0.24	6.58	92.14	S2

Notes:

- (1) Based on the Caltrans web tool ARS Online (Version 4.0)
- (2) Shear wave velocity determined by MASW results and SPT correlations.
- (3) Per Section 6.1 of the Caltrans Seismic Design Criteria Version 2.0 (SDC 2.0), "Soil Profile Classification."

The recommended ARS curve is presented on Plate No. 5.

9.3 Seismic Hazard

Faulting

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

Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and low-plastic silts of low relative density are the types of soils that are usually susceptible to liquefaction. Clay is generally not susceptible to liquefaction. The liquefaction potential was evaluated according to the procedure proposed by Youd et al. (2001).



Using the Caltrans ARS online tool, the peak ground acceleration (PGA) at the site was estimated to be 0.24g and the moment magnitude was estimated to be 6.58, representing a hazardous level of 5 percent exceedance in 50 years. The above seismic parameters were incorporated into the liquefaction analysis. Liquefaction potential calculations show that since the subsoils encountered in both borings are mainly composed of medium dense to very dense sand and hard silt, the liquefaction potential appears to be low. Therefore, it is our opinion that liquefaction potential does not exist, and the liquefaction should not be a design concern for this project. Liquefaction calculations are attached in Appendix C.

Dry Sand Settlement

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

10.0 AS-BUILT FOUNDATION DATA

The Caltrans Bridge Inspection Report (2008) indicates that the existing bridge (Br. No. 42C0175) was built in 1925 and widened once in 1942. The bridge is approximately 33.14 feet (10.1 meters) long and 30.18 feet (9.2 meters) wide with two traffic lanes. The structure is described to be *"simply supported single-span CIP/RC T girders (6), covered with a thick AC overlay, on RC cantilever abutments founded on RC spread footings."* The Caltrans report classifies the bridge as functionally obsolete.

11.0 FOUNDATION RECOMMENDATIONS

11.1 General

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

11.2 Foundations

Based on the subsurface conditions and discussions with the Designer, cast-in-drilled-hole (CIDH) concrete piles with a 30-inch diameter were selected for foundation support. Due to the presence of sandy soils and groundwater, cave-in conditions should be anticipated.



Temporary casing and/or fluid displacement method should be anticipated at all times for CIDH pile construction.

The loads from LRFD Strength state and Extreme Event limit state will be used for design of the foundations. A minimum pile spacing of three times the pile diameter (3D, center-to-center) is recommended. Pertinent foundation design information provided by the Designer, including Foundation Design Data and Foundation Factored Design Loads, is tabulated in Tables 4 and 5.

Table 5 – Foundation Design Data

Support	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size (ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	LRFD	30" CIDH	353.0	350.25	3.5	97.77	1	9
Abut 2	LRFD	30" CIDH	353.0	350.25	3.5	97.77	1	9

Table 6 – Foundation Design Loads

Support	Service-I Limit State (kips)			Strength / Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load		Perm. Loads	Compression		Tension		Compression		Tension	
	Per Support	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	1,830	n/a	1,520	3,120	390	n/a	n/a	1,520	262	n/a	n/a
Abut 2	1,830	n/a	1,520	3,120	390	n/a	n/a	1,520	262	n/a	n/a

11.3 Axial Pile Capacity Analysis

The computer program SHAFT by Ensoft Inc. was used for axial pile capacity analyses. The capacity of CIDH piles is estimated based on the procedures presented in the publication of the U.S. Department of Transportation, Drilled Shafts: Construction Procedures and LRFD Design Methods (FHWA, 2010). The procedures are proposed by O'Neil and Reese (1999), which utilize an α factor for clay and a β factor for sand. The α factor is a function of the undrained shear strength normalized with the atmospheric pressure. The β factor is a function of the effective overburden stress. The pile capacity of CIDH piles is derived primarily from frictional resistance along the pile shaft, and end bearing is not included when estimating the pile capacity.



The SHAFT computer calculation results are presented in Appendix C. The lateral pile capacity was analyzed by the Designer. The design pile tip elevations are presented in Tables 6 and 7 below.

Table 7 – Foundation Design Recommendations

Support No.	Pile Type	Cut-off Elev. (ft)	Service-I Limit State Load (kips) per Support		Total Permissible Support Settlement (in)	Required Factored Nominal Resistance (kips)			
			Total	Perm.		Strength/Constr.		Extreme Event	
						Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1.0)	Tension (φ=1.0)
Abut 1	30" CIDH	350.25	1,830	1,520	1	390	n/a	262	n/a
Abut 2	30" CIDH	350.25	1,830	1,520	1	390	n/a	262	n/a

Table 8 – Pile Data Table

Support No.	Pile Type	Nominal Resistance (kips)		Design Tip Elev. (ft)	Specified Tip Elev. (ft)
		Compression	Tension		
Abut 1	30" CIDH	560	n/a	291 (a); 335 (c); 319 (d)	289
Abut 2	30" CIDH	560	n/a	291 (a); 325 (c); 319 (d)	289

Notes:

- (1) Design tip elevations are controlled by: (a) Compression, (c) Settlement, (d) Lateral
- (2) The specified tip elevation shall not be raised above the lowest design tip elevation.
- (3) Slurry displacement method may be used for concrete placement of the CIDH piles. Therefore, the specified pile tip elevations are two feet below the design tip elevations to account for the zone of untested concrete due to the limitations of the gamma-gamma logging (GGL) equipment (Caltrans Geotechnical Manual, "CIDH Concrete Pile Design Considerations" Module, dated March 2021.

11.4 Lateral Pile Capacity Analysis

The lateral pile capacity will be analyzed by the Designer. The geotechnical parameters presented in Tables 9 and 10 were adopted for lateral capacity analysis.

For group effect, p -multipliers of 0.4, 0.6, 0.7, and 0.8 should be applied respectively for piles spacing of two, three, four, and five times pile diameter, if the load direction is parallel to the row; p -multipliers of 0.8, 0.9 and 1.0 can be used for a single row of piles spacing of two and half, three and four times pile diameter, respectively, if the load direction is perpendicular to the row. The y -multiplier is taken as 1.0. The final tolerable pile top movement may be determined by the Designer considering the overall design conditions.



Table 9 – LPILE Parameters for Abut 1 (per Boring R-14-001, West of Bridge)

Approx. Elevation (ft)	Generalized Soil Profile	LPILE Soil Type	Soil Strength	K (pci)	E50 (in/in)	Effective Unit Wt. (pcf)
356 to 351	Silty Sand	Sand (Reese)	$\phi = 30^\circ$	Default	N/A	120
351 to 342	Silty Sand	Sand (Reese)	$\phi = 38^\circ$	Default	N/A	120
342 to 317	Silty Sand / Clayey Sand	Sand (Reese)	$\phi = 40^\circ$	Default	N/A	60
317 to 293	Silty Sand / Poorly Graded Sand / Silt with Sand	Sand (Reese)	$\phi = 36^\circ$	Default	N/A	60
293 to 283	Silty Sand	Sand (Reese)	$\phi = 38^\circ$	Default	N/A	60
283 to 278	Silt	Stiff Clay w/o Free Water (Reese)	C = 4,000 psf	N/A	Default	60
278 to 274.5	Silty Sand	Sand (Reese)	$\phi = 36^\circ$	Default	N/A	60

Table 10 – LPILE Parameters for Abut 2 (per Boring R-14-002, East of Bridge)

Approx. Elevation (ft)	Generalized Soil Profile	LPILE Soil Type	Soil Strength	K (pci)	E50 (in/in)	Effective Unit Wt. (pcf)
357 to 348	Silty Sand	Sand (Reese)	$\phi = 30^\circ$	Default	N/A	120
348 to 343	Silty Sand	Sand (Reese)	$\phi = 36^\circ$	Default	N/A	120
343 to 333	Silty Sand	Sand (Reese)	$\phi = 36^\circ$	Default	N/A	60
333 to 323	Silty Sand / Silt with Sand	Sand (Reese)	$\phi = 34^\circ$	Default	N/A	60
323 to 313	Silty Sand	Sand (Reese)	$\phi = 36^\circ$	Default	N/A	60
313 to 293	Silty Sand	Sand (Reese)	$\phi = 38^\circ$	Default	N/A	60
293 to 279	Silt	Stiff Clay w/o Free Water (Reese)	C = 4,000 psf	N/A	Default	60
279 to 275.5	Silty Sand	Sand (Reese)	$\phi = 36^\circ$	Default	N/A	60

11.5 Lateral Earth Pressures

Cantilever walls which are free to rotate at least 0.004 radian may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharges (dead or live loads) should be added to the preceding lateral earth pressures.



Abutment and wing walls should be designed to resist the lateral earth pressures as recommended below.

Active Earth Pressure: A minimum active earth pressure of 36 pcf EFP is recommended for design for level backfill for the yielding wall and compacted Structure Backfill per Caltrans Standard Specifications. An active earth pressure coefficient, K_a , of 0.3 can be used to estimate additional loads due to the potential surcharge. A total seismic lateral pressure coefficient, K_{ae} , of 0.35 is recommended.

At-Rest Earth Pressure: An at-rest earth pressure of 55 pcf EFP is recommended for design for level backfill for the non-yielding wall and compacted Structure Backfill per Caltrans Standard Specifications. An at-rest earth pressure coefficient, K_o , of 0.5 can be used to estimate additional loads due to the potential surcharge. A total seismic lateral pressure coefficient, K_{oe} , of 0.6 is recommended.

These values were developed based on compacted Structure Backfill per Caltrans Standard Specifications (ϕ of 34 degrees and no cohesion), no hydrostatic pore pressure buildup behind the walls, level backslope, and a seismic coefficient, K_h , of 0.12 ($1/2 \times \text{PGA}$).

An equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf should be used if the traffic is within a horizontal distance of the wall height. The horizontal earth pressure in front of the abutment walls should be ignored.

12.0 PAVEMENT SECTIONS

Pavement design for flexible pavement sections using hot mix asphalt (HMA) is based on Caltrans empirical design method. The tests on the sample from the existing subgrade produced an R-value of 60. Since the pavement design will be controlled by import fill, it is recommended to use an R-value of 15 for design. Table 9 below presents the recommendations for structural pavement sections based on a Traffic Index (TI) of 9 for a design life of 20 years.

Table 11 – Structural Pavement Sections

TI	R-value	Structural Pavement Section (ft)					
		Option 1	Option 2		Option 3		
		Full-Depth HMA	HMA	AB	HMA	AB	AS
9	15	1.10	0.50	1.50	0.50	0.70	0.80

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



13.0 GRADING

All grading and compaction operations should be performed in accordance with the project specifications and Section 19 “Earthwork” of Caltrans Standard Specifications (2010). A representative from this office or regulating agency should observe all excavated areas during grading and perform moisture and density tests on prepared subgrade and compacted fill material.

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

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

The embankment fill slope gradient should not be steeper than 2H:1V (horizontal to vertical). It should be noted that local irregularities such as loose layers and pockets and seepage might require flatter slopes. This office should review the final grading plans prior to grading to see that the intent of our recommendations is included in the plans.

14.0 CONSTRUCTION CONSIDERATIONS

14.1 General

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

14.2 Construction of CIDH Piles

The Caltrans standard specifications and Standard Special Provisions (SSP) for “Cast-in-Place Concrete Piling” should be used for construction of CIDH piles. The contractor should carefully examine the subsurface conditions and make their own



interpretation and perform an independent study on the constructability of the piles. Hard drilling conditions should be anticipated in very dense sandy soils.

Vertical inspection pipes for acceptance testing should be provided in all CIDH piles that are 24 inches in diameter or larger, except when the holes are dry or when the holes are dewatered without use of temporary casing to control groundwater. The acceptance test should include Gamma-Gamma Logging. Gamma-Gamma Logging should be performed in accordance with California Test Method 233 Standard (CT 233) to check the integrity of CIDH piles. CT 233 defines pile rejection criteria based on the statistical principles of mean and three standard deviations to analyze the homogeneity of a pile. Crosshole sonic logging should be considered to aid in refining the extent of anomalies. Anomalies detected should be evaluated by the Designer for their significance and potential impact on design and to see if mitigation plans are required. Details of the acceptance testing and Gamma-Gamma Logging are contained in Caltrans SSP and CT 233.

Due to the presence of granular material and groundwater, raveling or caving is expected, which may require additional drilling and cleaning effort and may increase the concrete volume for the piles. The use of temporary steel casing and fluid displacement method should be anticipated at all times to maintain the integrity of the piles. Oscillator steel casing method may be required if full depth casing is necessary. It is prudent to make the contractor aware of these conditions so that they take appropriate steps to comply with the standards and maintain the integrity of the CIDH concrete piles. Mitigation and repair procedures for CIDH anomaly should be anticipated. All pile excavations should be observed by a geotechnical engineer prior to the placement of reinforcement and concrete so that if conditions differ from those anticipated, appropriate recommendations can be made.

14.3 Waiting Period

It is anticipated that less than five feet of fill for new approach embankments and roadway widening is expected, which needs to be confirmed once grading plan is made available. Since no saturated soft materials were encountered in our borings, the settlement due to fill is anticipated to be minor and should generally occur during construction. A waiting period is not required. However, it is recommended that the approach embankment be constructed before starting installation of the pile foundation.

14.4 Construction Dewatering

Groundwater may cause instability of excavation walls and bottom (piping, erosion, blow-outs, etc.) and difficult working conditions. For excavation below the groundwater



table, construction dewatering will be required. The contractor should evaluate the subsurface conditions before selecting a dewatering method, which may include shoring, sumps, or tremie slabs. Groundwater should be lowered to at least 2 feet below the bottom of the excavation to provide workable conditions. Designing the dewatering system should be the contractor's responsibility.

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

Since the site is within the farmland, possible hazardous materials such as pesticides may be drawn where construction dewatering is performed. An investigation for subsurface environmental contamination was beyond the scope of our services.

14.5 Temporary Excavation and Shoring

Excavation will be required for installation of foundations. It is possible that unknown old buried utilities are located at the site. It might require special equipment and additional efforts to remove these buried objects.

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



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

14.6 Working Platform

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

15.0 NOTES TO DESIGNER

The foundation recommendations presented in Section 11.3 are based on the load demands at limits states. The lateral pile capacity analysis will be conducted by the structural engineer. It is recommended that the structural engineer verify the pile tip elevations when finalizing the pile data table. Final specified pile tip elevations should be the lower of the design tip elevations resulting from the axial and lateral pile capacity analysis.

It should be noted that the lateral resistance estimated is based on the assumption that the abutments are set back an adequate distance from the creek banks, and/or the channel banks are protected from scour and erosion.

16.0 PLAN REVIEW

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



17.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

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

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



Dewberry

Travers Creek Bridge (Replace)

Job No. 2013-137-FIN

July 30, 2026

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

Respectfully submitted,

PARIKH CONSULTANTS, INC.



Frank Y. Wang, PE, GE 2862
Sr. Project Engineer



Y. David Wang, PhD. PE 52911
Project Manager



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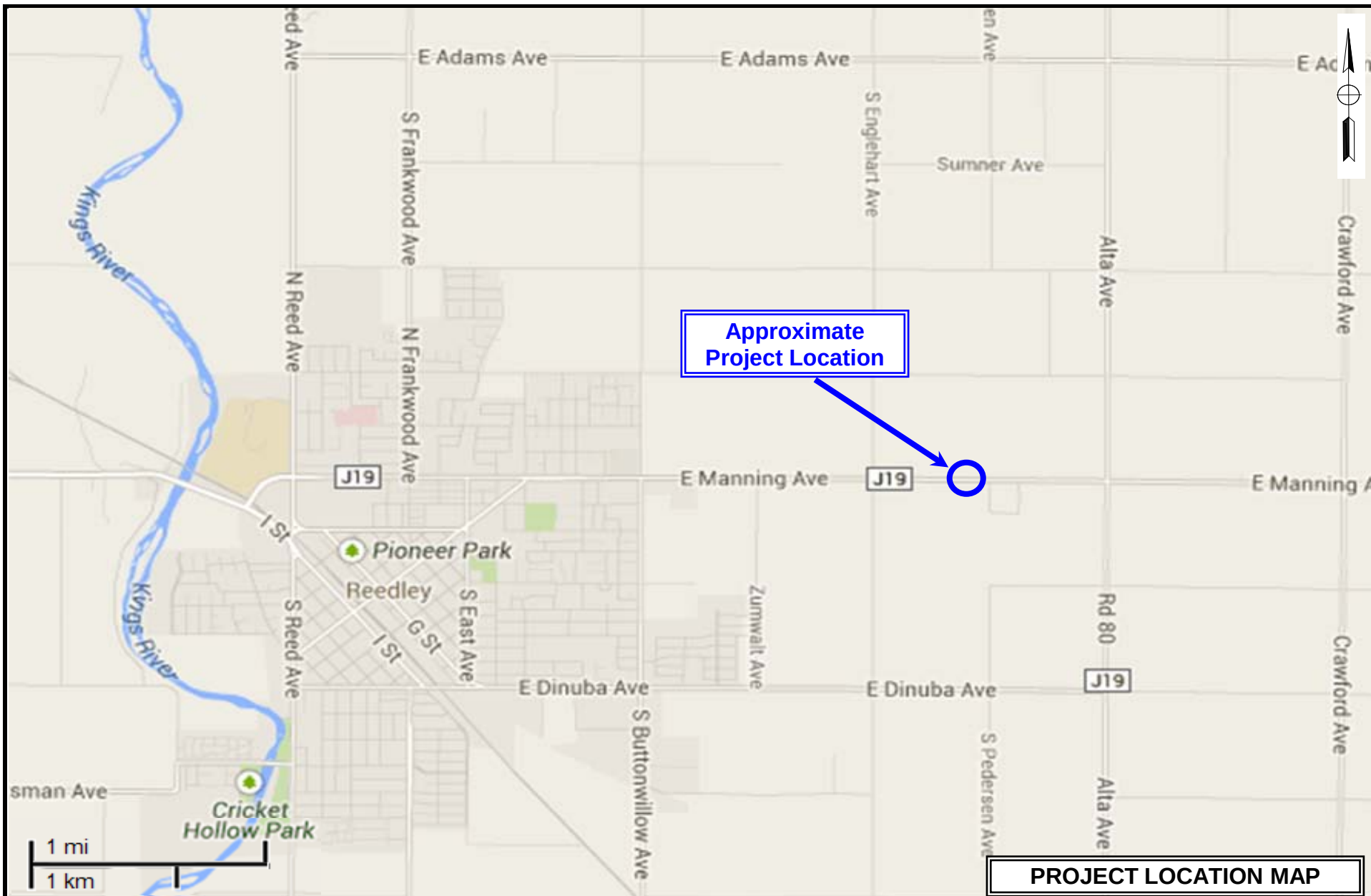
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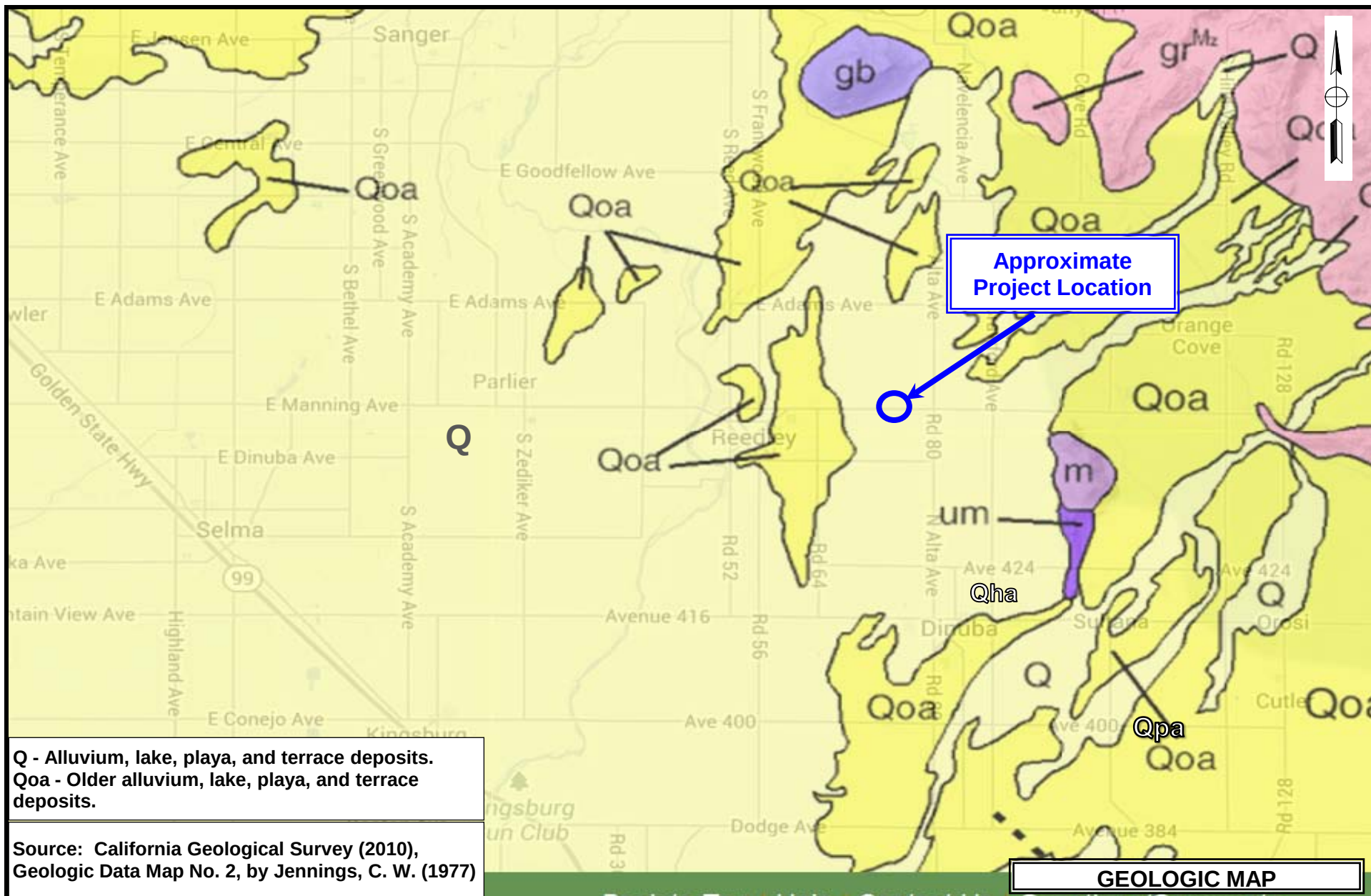
PROJECT LOCATION MAP

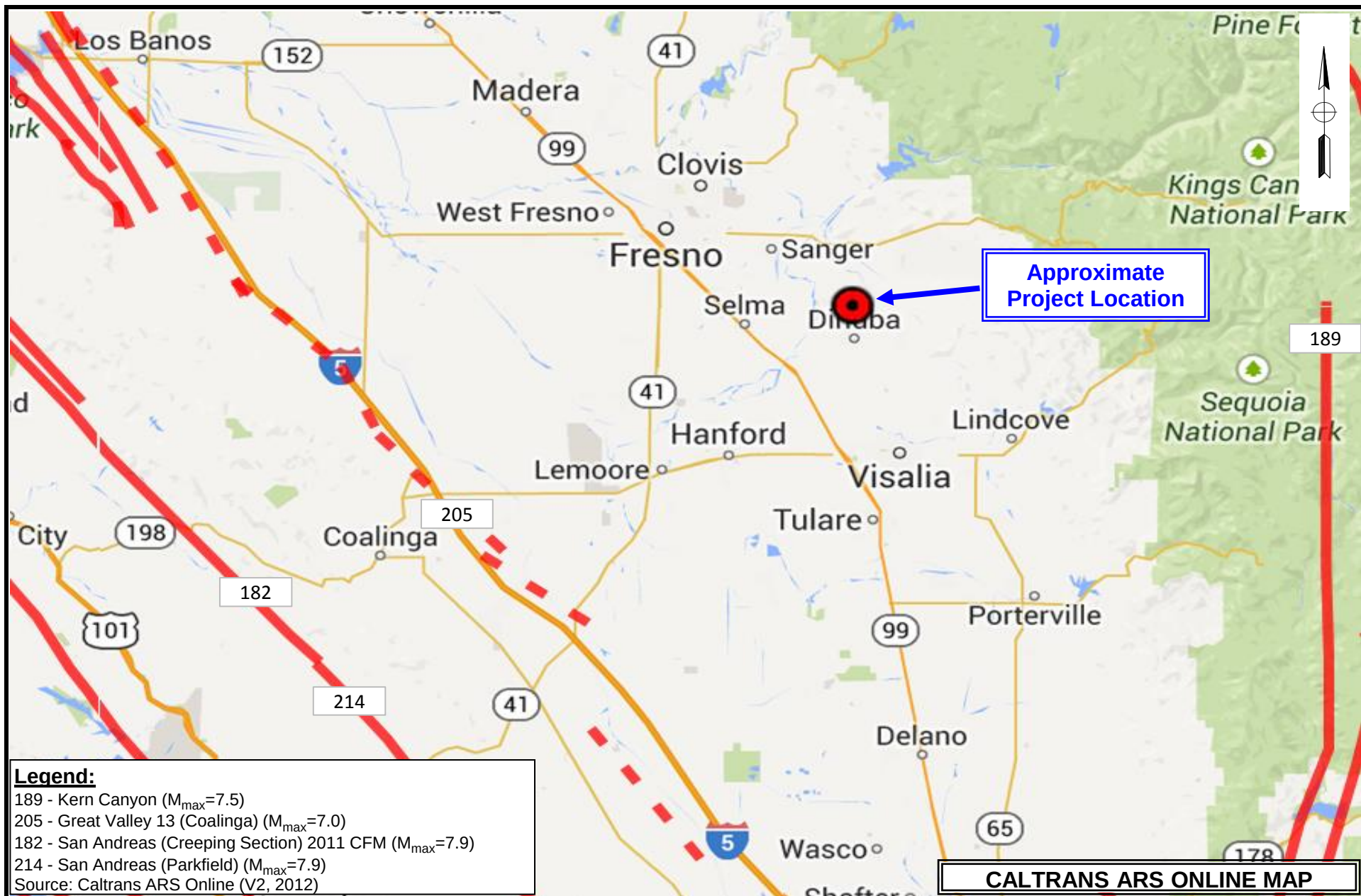
LEGEND:



Approx. Boring Location







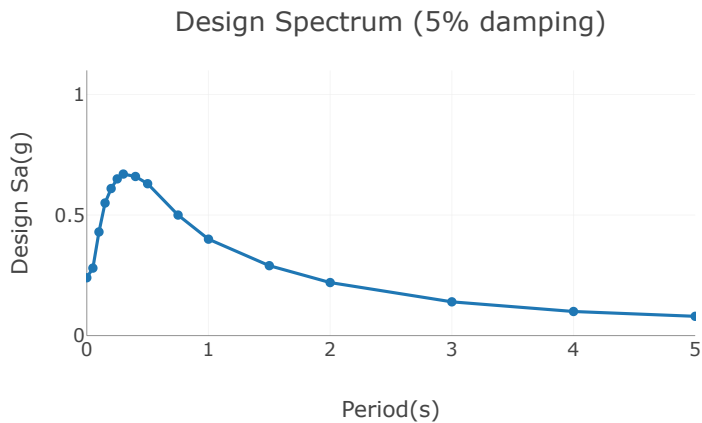


ARS Online V4.0 results for Latitude: 36.6040° Longitude: -119.4059° Vs30: 240 m/s

Return period: 975 years (5% damping)

Period(s)	Sa(g)	Near-fault amp	Design Sa(g)
0	0.24	1	0.24
0.05	0.28	1	0.28
0.10	0.43	1	0.43
0.15	0.55	1	0.55
0.20	0.61	1	0.61
0.25	0.65	1	0.65
0.30	0.67	1	0.67
0.40	0.66	1	0.66
0.50	0.63	1	0.63
0.75	0.5	1	0.5
1.0	0.4	1	0.4
1.5	0.29	1	0.29
2.0	0.22	1	0.22
3.0	0.14	1	0.14
4.0	0.1	1	0.1
5.0	0.08	1	0.08
7.5	0.05	1	0.05
10.0	0.04	1	0.04

Copy table



Deaggregation Results

Period(s)	M _{mean}	R _{mean} (km)	ε
PGA	6.58	45.97	0.96
1.0	7.31	92.14	1.34

Note: Peak ground acceleration (PGA) is equal to the spectral acceleration (Sa) at a period of 0.

M_{mean} = Mean Moment Magnitude

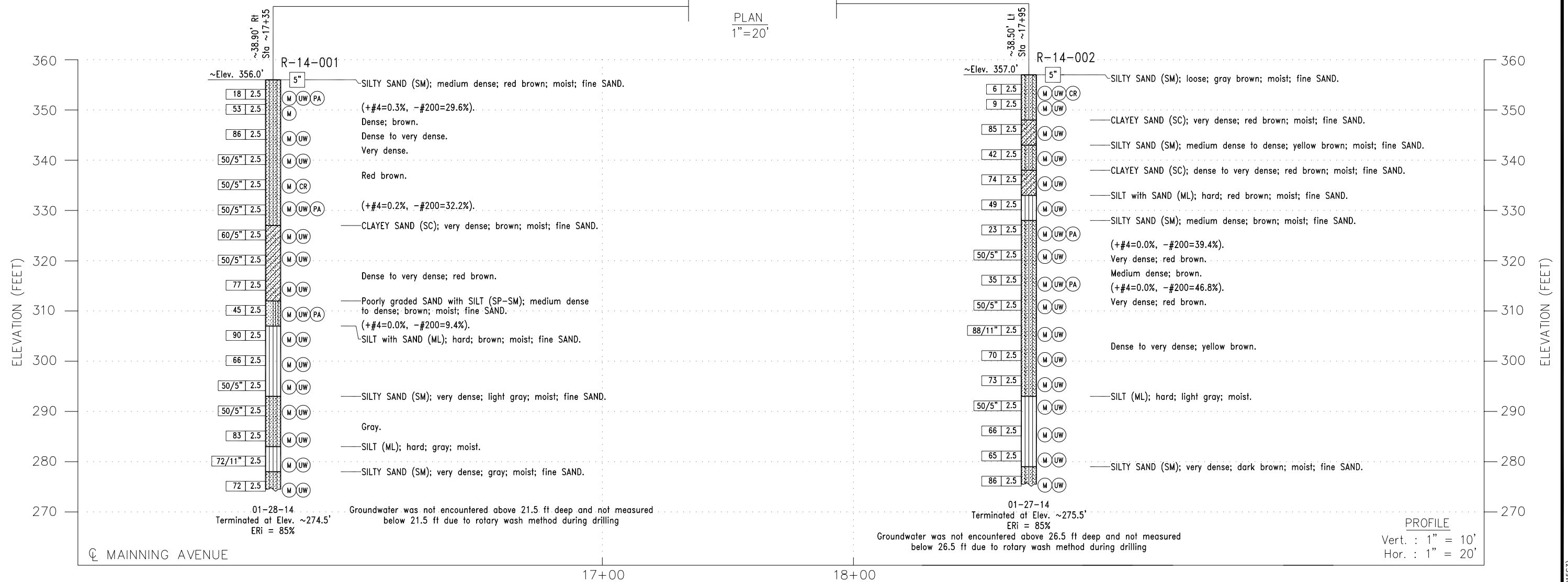
R_{mean} = Mean site-source distance

ε = number of standard deviations for logarithmic spectral acceleration

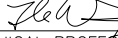
New Analysis

APPENDIX A

All dimensions are in feet unless otherwise shown.



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
61	FR	LOCAL			


 7/6/2026
 GEOTECHNICAL PROFESSIONAL DATE

PLANS APPROVAL DATE

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

PARIKH CONSULTANTS, INC.
 1497 N Milpitas Blvd.
 MILPITAS, CA 95035

REGISTERED PROFESSIONAL ENGINEER
 YUAN-KAI WANG
 2862
 GEOTECHNICAL
 STATE OF CALIFORNIA

DRAWN BY		KIM OUYANG	V.SANTOS	FRANK WANG	 PARIKH Practicing in the Geosciences PARIKH CONSULTANTS, INC. 1497 N MILPITAS BLVD. MILPITAS, CA 95035	PROJECT			DEPARTMENT OF PUBLIC WORKS AND PLANNING		
CHECKED BY		PETER WEI	FIELD INVESTIGATION BY: DATE: JANUARY 2014	PROJECT ENGINEER		TRAVERS CREEK BRIDGE ON MANNING AVENUE			LOG OF TEST BORINGS		
						ROAD NO.	BRIDGE NO. 42C-0175, BRLS-5942 (198)		DRAWING NO.	SHEET NO.	TOTAL

(csc10c107) c10c107

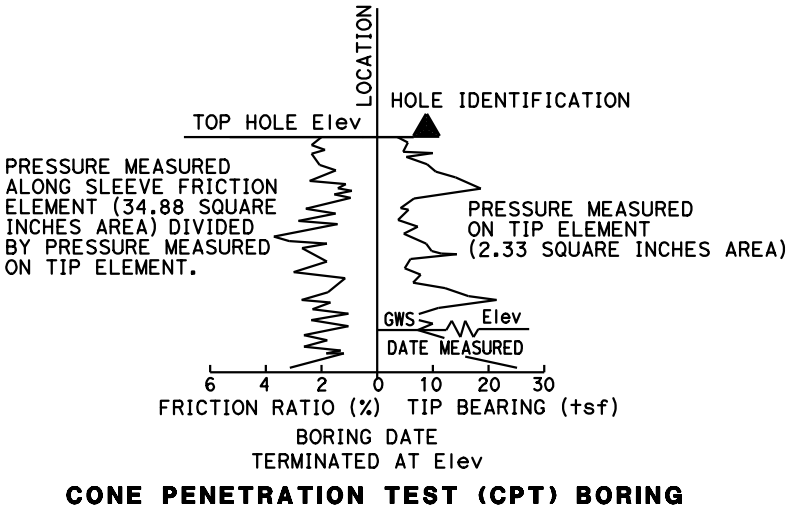
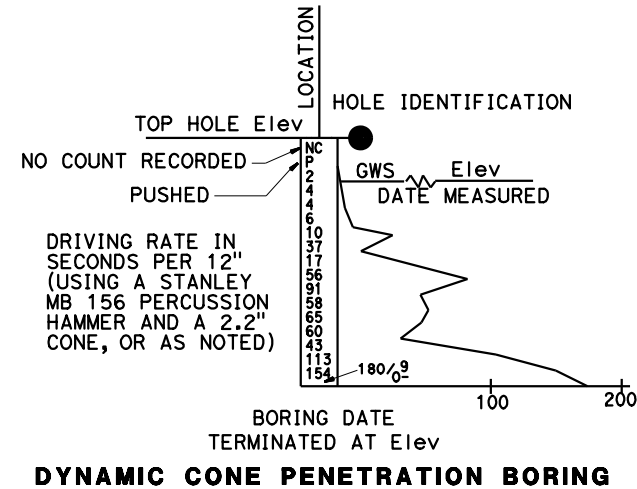
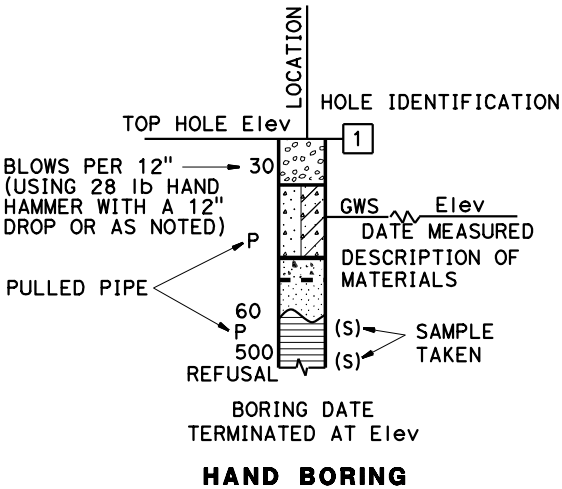
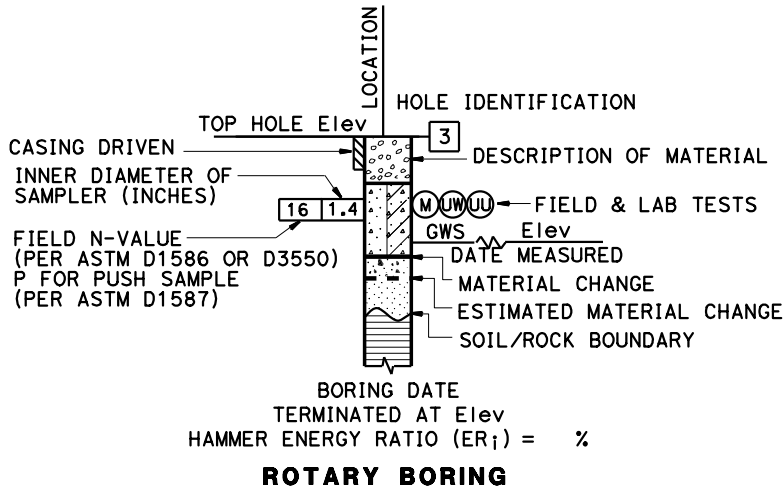
CEMENTATION	
DESCRIPTION	CRITERIA
WEAK	CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE.
MODERATE	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE.
STRONG	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE.

ABBREVIATION:

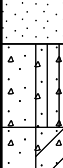
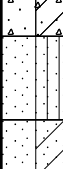
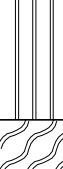
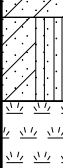
GWS - GROUND WATER SURFACE

BOREHOLE IDENTIFICATION		
SYMBOL	HOLE TYPE	DESCRIPTION
	A	AUGER BORING (HOLLOW OR SOLID STEM BUCKET)
	R	ROTARY DRILLED BORING (CONVENTIONAL)
	RW	ROTARY DRILLED WITH SELF-CASING WIRE-LINE
	RC	ROTARY CORE WITH CONTINUOUSLY-SAMPLED, SELF-CASING WIRE-LINE
	P	ROTARY PERCUSSION BORING (AIR)
	R	ROTARY DRILLED DIAMOND CORE
	RC	ROTARY DRILLED DIAMOND CORE, CONTINUOUSLY SAMPLED
	HD	HAND DRIVEN (1-INCH SOIL TUBE)
	HA	HAND AUGER
	D	DYNAMIC CONE PENETRATION BORING
	CPT	CONE PENETRATION TEST (ASTM D5778)
	O	OTHER (NOTE ON LOTB)
NOTE: SIZE IN INCHES.		

CONSISTENCY OF COHESIVE SOILS				
DESCRIPTION	SHEAR STRENGTH (tsf)	POCKET PENETROMETER MEASUREMENT, PP, (tsf)	TORVANE MEASUREMENT, TV, (tsf)	VANE SHEAR MEASUREMENT, VS, (tsf)
VERY SOFT	LESS THAN 0.12	LESS THAN 0.25	LESS THAN 0.12	LESS THAN 0.12
SOFT	0.12 - 0.25	0.25 - 0.5	0.12 - 0.25	0.12 - 0.25
MEDIUM STIFF	0.25 - 0.5	0.5 - 1	0.25 - 0.5	0.25 - 0.5
STIFF	0.5 - 1	1 - 2	0.5 - 1	0.5 - 1
VERY STIFF	1 - 2	2 - 4	1 - 2	1 - 2
HARD	GREATER THAN 2	GREATER THAN 4	GREATER THAN 2	GREATER THAN 2



STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
LEGEND - SOIL
(SHEET 1 OF 2)
NO SCALE

GROUP SYMBOLS AND NAMES							
GRAPHIC/SYMBOL		GROUP NAMES		GRAPHIC/SYMBOL		GROUP NAMES	
	GW	WELL-GRADED GRAVEL WELL-GRADED GRAVEL WITH SAND			CL	LEAN CLAY LEAN CLAY WITH SAND LEAN CLAY WITH GRAVEL SANDY LEAN CLAY SANDY LEAN CLAY WITH GRAVEL GRAVELLY LEAN CLAY GRAVELLY LEAN CLAY WITH SAND	
	GP	POORLY-GRADED GRAVEL POORLY-GRADED GRAVEL WITH SAND					
	GW-GM	WELL-GRADED GRAVEL WITH SILT WELL-GRADED GRAVEL WITH SILT AND SAND			CL-ML	SILTY CLAY SILTY CLAY WITH SAND SILTY CLAY WITH GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY WITH GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY WITH SAND	
	GW-GC	WELL-GRADED GRAVEL WITH CLAY (OR SILTY CLAY) WELL-GRADED GRAVEL WITH CLAY AND SAND (OR SILTY CLAY AND SAND)					
	GP-GM	POORLY-GRADED GRAVEL WITH SILT POORLY-GRADED GRAVEL WITH SILT AND SAND			ML	SILT SILT WITH SAND SILT WITH GRAVEL SANDY SILT SANDY SILT WITH GRAVEL GRAVELLY SILT GRAVELLY SILT WITH SAND	
	GP-GC	POORLY-GRADED GRAVEL WITH CLAY (OR SILTY CLAY) POORLY-GRADED GRAVEL WITH CLAY AND SAND (OR SILTY CLAY AND SAND)					
	GM	SILTY GRAVEL SILTY GRAVEL WITH SAND			OL	ORGANIC LEAN CLAY ORGANIC LEAN CLAY WITH SAND ORGANIC LEAN CLAY WITH GRAVEL SANDY ORGANIC LEAN CLAY SANDY ORGANIC LEAN CLAY WITH GRAVEL GRAVELLY ORGANIC LEAN CLAY GRAVELLY ORGANIC LEAN CLAY WITH SAND	
	GC	CLAYEY GRAVEL CLAYEY GRAVEL WITH SAND					
	GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL WITH SAND			OL	ORGANIC SILT ORGANIC SILT WITH SAND ORGANIC SILT WITH GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT WITH GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT WITH SAND	
	SW	WELL-GRADED SAND WELL-GRADED SAND WITH GRAVEL					
	SP	POORLY-GRADED SAND POORLY-GRADED SAND WITH GRAVEL			CH	FAT CLAY FAT CLAY WITH SAND FAT CLAY WITH GRAVEL SANDY FAT CLAY SANDY FAT CLAY WITH GRAVEL GRAVELLY FAT CLAY GRAVELLY FAT CLAY WITH SAND	
	SW-SM	WELL-GRADED SAND WITH SILT WELL-GRADED SAND WITH SILT AND GRAVEL					
	SW-SC	WELL-GRADED SAND WITH CLAY (OR SILTY CLAY) WELL-GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY AND GRAVEL)			MH	ELASTIC SILT ELASTIC SILT WITH SAND ELASTIC SILT WITH GRAVEL SANDY ELASTIC SILT SANDY ELASTIC SILT WITH GRAVEL GRAVELLY ELASTIC SILT GRAVELLY ELASTIC SILT WITH SAND	
	SP-SM	POORLY-GRADED SAND WITH SILT POORLY-GRADED SAND WITH SILT AND GRAVEL					
	SP-SC	POORLY-GRADED SAND WITH CLAY (OR SILTY CLAY) POORLY-GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY AND GRAVEL)			OH	ORGANIC FAT CLAY ORGANIC FAT CLAY WITH SAND ORGANIC FAT CLAY WITH GRAVEL SANDY ORGANIC FAT CLAY SANDY ORGANIC FAT CLAY WITH GRAVEL GRAVELLY ORGANIC FAT CLAY GRAVELLY ORGANIC FAT CLAY WITH SAND	
	SM	SILTY SAND SILTY SAND WITH GRAVEL					
	SC	CLAYEY SAND CLAYEY SAND WITH GRAVEL			OH	ORGANIC ELASTIC SILT ORGANIC ELASTIC SILT WITH SAND ORGANIC ELASTIC SILT WITH GRAVEL SANDY ORGANIC ELASTIC SILT SANDY ORGANIC ELASTIC SILT WITH GRAVEL GRAVELLY ORGANIC ELASTIC SILT GRAVELLY ORGANIC ELASTIC SILT WITH SAND	
	SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND WITH GRAVEL					
	PT	PEAT			OL/OH	ORGANIC SOIL ORGANIC SOIL WITH SAND ORGANIC SOIL WITH GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL WITH GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL WITH SAND	
		COBBLES COBBLES AND BOULDERS BOULDERS					

FIELD AND LABORATORY TESTING	
	CONSOLIDATION
	CORROSIVITY TESTING
	CONSOLIDATED UNDRAINED TRIAXIAL
	DIRECT SHEAR
	EXPANSION INDEX
	HYDRAULIC CONDUCTIVITY
	ORGANIC CONTENT-%
	PARTICLE SIZE ANALYSIS
	PLASTICITY INDEX LIQUID LIMIT
	POINT LOAD INDEX
	UNCONFINED COMPRESSION - SOIL
	UNCONFINED COMPRESSION - ROCK
	UNCONSOLIDATED UNDRAINED TRIAXIAL
	200 SIEVE TEST

APPARENT DENSITY OF COHESIONLESS SOILS	
DESCRIPTION	SPT N ₆₀ (BLOWS/12 INCHES)
VERY LOOSE	0 - 5
LOOSE	5 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	GREATER THAN 50

MOISTURE	
DESCRIPTION	CRITERIA
DRY	NO DISCERNABLE MOISTURE
MOIST	MOISTURE PRESENT, BUT NO FREE WATER
WET	VISIBLE FREE WATER

PERCENT OR PROPORTION OF SOILS	
DESCRIPTION	CRITERIA
TRACE	PARTICLES ARE PRESENT BUT ESTIMATED TO BE LESS THAN 5%
FEW	5% - 10%
LITTLE	15% - 25%
SOME	30% - 45%
MOSTLY	50% - 100%

PARTICLE SIZE		
DESCRIPTION		SIZE
BOULDER		GREATER THAN 12"
COBBLE		3" - 12"
GRAVEL	COARSE	¾" - 3"
	FINE	⅕" - ¾"
SAND	COARSE	⅓" - ⅕"
	MEDIUM	⅓" - ⅓"
	FINE	1/300" - ⅓"
SILT AND CLAY		LESS THAN 1/300"

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
LEGEND - SOIL
(SHEET 2 OF 2)
NO SCALE

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM D 2216. This information was used to classify and correlate the soils. The results are presented in the summary table on Plate B-2.

Grain Size Classification

Grain size classification tests (ASTM D 422) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plate B-3, “Grain Size Distribution Curves”.

Corrosion Tests

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

R-value Test

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

Borehole	Sample Number	Depth	Classification	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Unconfined Compressive Strength (tsf)
R-14-001	1	3.0	SM	4.8	115.4				0.3	29.6	
R-14-001	2	6.0	SM	6.5	-						
R-14-001	3	11.0	SM	11.9	123.1						
R-14-001	4	15.5	SM	7.8	95.6						
R-14-001	5	20.5	SM	8.4	-						
R-14-001	6	25.0	SM	8.9	126.4				0.2	32.2	
R-14-001	7	30.5	SC	9.0	132.4						
R-14-001	8	35.0	SC	7.9	126.5						
R-14-001	9	41.0	SC	11.9	124.0						
R-14-001	10	46.0	SP-SM	4.2	110.5					9.4	
R-14-001	11	51.0	ML	13.7	121.5						
R-14-001	12	56.0	ML	19.1	107.3						
R-14-001	13	60.5	ML	18.0	109.9						
R-14-001	14	65.5	SM	15.5	110.8						
R-14-001	15	71.0	SM	16.2	116.2						
R-14-001	16	76.0	ML	32.5	89.0						
R-14-001	17	81.0	SM	24.6	99.5						
R-14-002	1	3.0	SM	7.3	115.2						
R-14-002	2	6.0	SM	6.2	111.5						
R-14-002	3	11.0	SC	11.9	123.3						
R-14-002	4	16.0	SM	14.2	119.3						
R-14-002	5	21.0	SC	13.0	121.8						
R-14-002	6	26.0	ML	19.9	109.7						
R-14-002	7	31.0	SM	17.1	116.7				0.0	39.4	
R-14-002	8	35.5	SM	12.4	124.9						
R-14-002	9	41.0	SM	16.5	117.6				0.0	46.8	
R-14-002	10	45.5	SM	11.1	113.1						
R-14-002	11	51.0	SM	13.5	123.5						
R-14-002	12	56.0	SM	15.3	118.9						
R-14-002	13	61.0	SM	14.7	117.4						
R-14-002	14	65.5	ML	22.0	103.7						
R-14-002	15	71.0	ML	20.4	109.8						
R-14-002	16	76.0	ML	39.8	82.3						
R-14-002	17	81.0	SM	20.3	110.9						

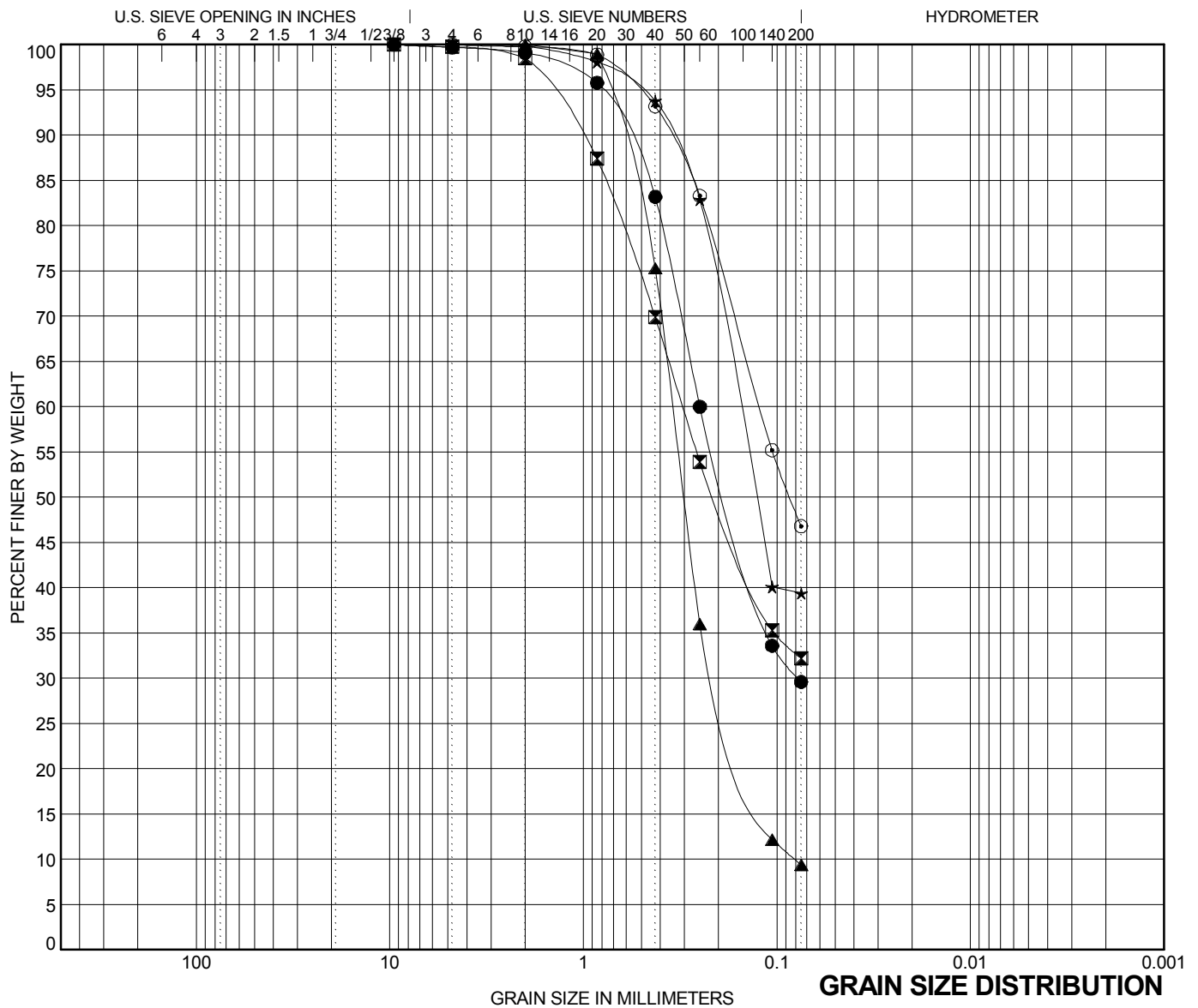


PARIKH CONSULTANTS, INC.
 GEOTECHNICAL CONSULTANTS
 MATERIALS ENGINEERING

**TRAVERS CREEK BRIDGE ON EAST MANNING AVENUE
 FRESNO COUNTY, CALIFORNIA**

JOB NO: 2013-137-FDN

PLATE NO: B-2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

BORING	SAMPLE #	DEPTH	Classification			LL	PL	PI	Cc	Cu
●	R-14-001	1	3.0	SILTY SAND						
⊠	R-14-001	6	25.0	SILTY SAND						
▲	R-14-001	10	46.0	Poorly graded SAND with SILT					1.45	4.28
★	R-14-002	7	31.0	SILTY SAND						
⊙	R-14-002	9	41.0	SILTY SAND						
BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	R-14-001	1	3.0	9.5	0.25	0.078	0.3	70.1	29.6	
⊠	R-14-001	6	25.0	9.5	0.306		0.2	67.6	32.2	
▲	R-14-001	10	46.0	2	0.346	0.201	0.081	0.0	90.6	9.4
★	R-14-002	7	31.0	4.75	0.158			0.0	60.6	39.4
⊙	R-14-002	9	41.0	4.75	0.123			0.0	53.2	46.8

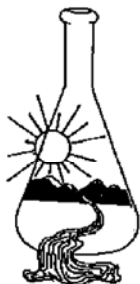


PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

TRAVERS CREEK BRIDGE ON EAST MANNING AVENUE
FRESNO COUNTY, CALIFORNIA

JOB NO: 2013-137-FDN

PLATE NO: B-3



Sunland Analytical
11353 Pyrites Way
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 02/07/14
Date Submitted 02/04/14

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

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

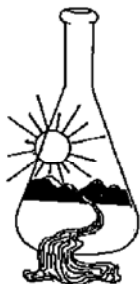
The reported analysis was requested for the following:
Location : 2013-137-FDN-TRAVER Site ID: R14-001#5 @ 21
Thank you for your business.

* For future reference to this analysis please use SUN # 66315 - 137262

EVALUATION FOR SOIL CORROSION

Soil pH	7.59	
Minimum Resistivity	2.95	ohm-cm (x1000)
Chloride	8.8 ppm	0.0009 %
Sulfate-S	44.8 ppm	0.0045 %

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



Sunland Analytical
11353 Pyrites Way
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 02/07/14
Date Submitted 02/04/14

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

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

The reported analysis was requested for the following:
Location : 2013-137-FDN-TRAVER Site ID: R14-002#1 @ 3FT
Thank you for your business.

* For future reference to this analysis please use SUN # 66315 - 137263

EVALUATION FOR SOIL CORROSION

Soil pH	7.44		
Minimum Resistivity	6.70	ohm-cm (x1000)	
Chloride	5.4 ppm	0.0005	%
Sulfate-S	21.0 ppm	0.0021	%

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



R-VALUE REPORT

Parikh Consultants, Inc.

ASTM D2844 or CTM 301

(408) 452-9000

Project Name: Travers Creek Bridge on East Manning Avenue

Date: 2/4/2014

Client: Biggs Cardosa Associates

Project #: 2013-137-FDN

Sample #: R-14-001 Depth: 0'-5'

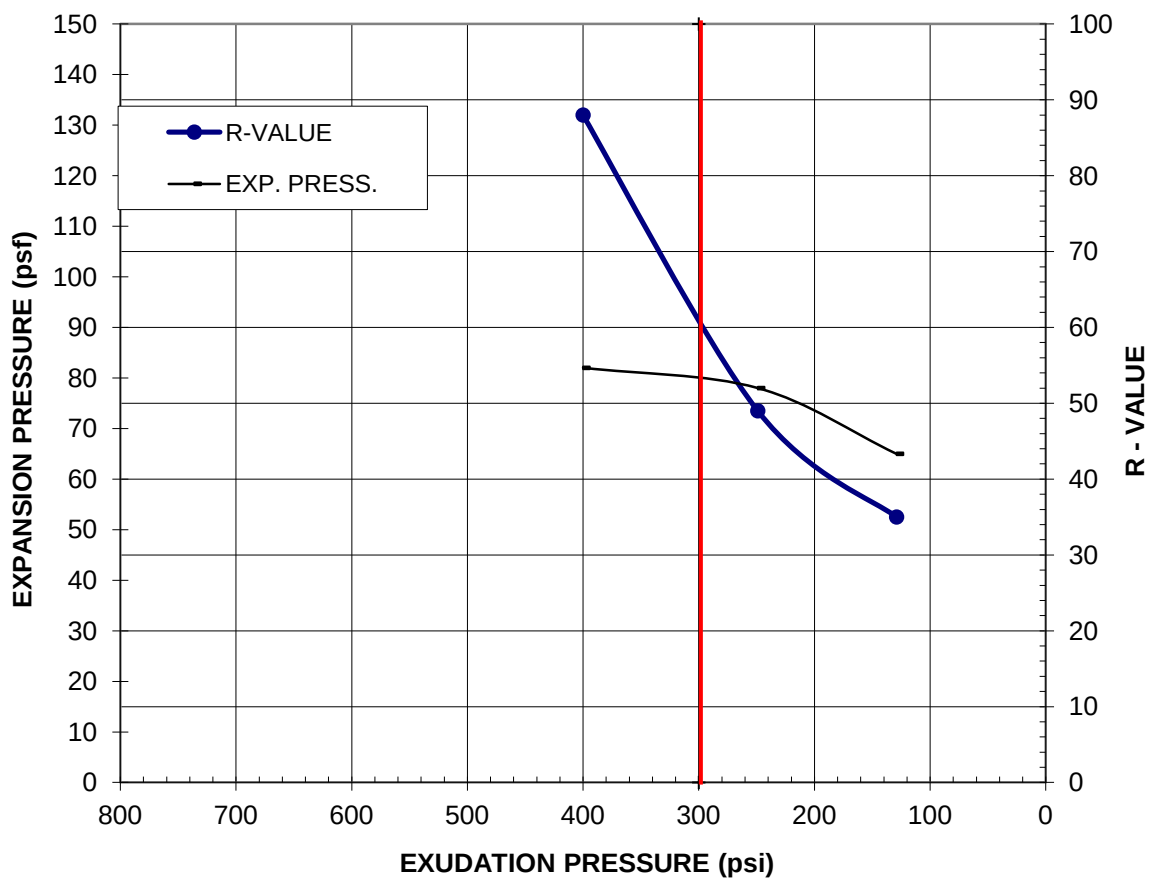
Lab #: M926

Location / Source: Onsite / Native

Sample Date:

Material : Silty fine Sand, brown

Sampled By:



Specimen No.

A

B

C

Exudation Pressure, psi

129

249

400

Expansion Pressure, psf

65

78

82

R-Value

35

49

88

Moisture Content at Test, %

21.0

19.1

17.2

Dry Density at Test, pcf

115.6

118.6

119.5

R-Value @ 300 psi Exudation Pressure = 60

Expansion Pressure @300 psi Exudation, psf = 80

Minimum R-Value Requirement:

Comments:

Report By: Prav Dayah

PLATE NO.: B-5

APPENDIX C

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME **Travers Creek Bridge**
PROJECT NO. **2013-137-FDN**
BORING NO. **R-14-001**

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

FAULT INFO

$a_{max} (g) = 0.23$
 $FAULT M_w = 7.8$
 $MSF = 0.94$

BOREHOLE DIA (in) 5
GW DEPTH (ft) 20

HAMMER ENERGY = 80%

Sample No	Depth (ft)	Soil Type	Blow Count	Sample Type	CYCLIC STRESS RATIO (CSR)				LIQUEFACTION RESISTANCE (CRR _{7.5})										S _r =(CRR _{7.5} /CSR)*MSF*K σ*K				
					σ _v (psf)	σ' _v (psf)	γ _d	CSR	SPT-N _{eq}	C _E	C _R	C _S	C _B	N ₆₀	C _N	(N ₁) ₆₀	F.C.	(N ₁) _{60, CS}	CRR _{7.5}	Kσ	Kα	F.S.	Volumetric Strain (%)
1	2	1	18	MC	250	250	1.00	0.15	11.7	1.3	0.75	1.0	1.0	11.7	1.66	19.4	30%	27.0	0.34	1.00	1		
2	5	1	53	MC	625	625	0.99	0.15	34.5	1.3	0.75	1.0	1.0	34.5	1.45	50.1		50.1		1.00	1		
3	10	1	86	MC	1250	1250	0.98	0.15	55.9	1.3	0.80	1.0	1.0	59.6	1.21	71.9		71.9		1.00	1		
4	15	1	100	MC	1875	1875	0.97	0.14	65.0	1.3	0.85	1.0	1.0	73.7	1.03	75.8		75.8		1.00	1		
5	20	1	100	MC	2500	2500	0.96	0.14	65.0	1.3	0.95	1.0	1.0	82.3	0.90	73.9		73.9		0.91	1		
6	25	1	100	MC	3125	2813	0.94	0.16	65.0	1.3	0.95	1.0	1.0	82.3	0.84	69.5	32%	86.3		0.87	1		
7	30	1	100	MC	3750	3125	0.92	0.17	65.0	1.3	1.00	1.0	1.0	86.7	0.80	69.0		69.0		0.84	1		
8	35	1	100	MC	4375	3438	0.89	0.17	65.0	1.3	1.00	1.0	1.0	86.7	0.75	65.3		65.3		0.81	1		
9	40	1	77	MC	5000	3750	0.85	0.17	50.1	1.3	1.00	1.0	1.0	66.7	0.72	47.7		47.7		0.78	1		
10	45	1	45	MC	5625	4063	0.80	0.17	29.3	1.3	1.00	1.0	1.0	39.0	0.68	26.6	9%	27.7	0.36	0.76	1	1.54	
11	50	1	90	MC	6250	4375	0.75	0.16	58.5	1.3	1.00	1.0	1.0	78.0	0.65	50.7		50.7		0.73	1		
12	55	1	66	MC	6875	4688	0.70	0.15	42.9	1.3	1.00	1.0	1.0	57.2	0.62	35.5		35.5		0.71	1		
13	60	1	100	MC	7500	5000	0.66	0.15	65.0	1.3	1.00	1.0	1.0	86.7	0.59	51.5		51.5		0.69	1		
14	65	1	100	MC	8125	5313	0.62	0.14	65.0	1.3	1.00	1.0	1.0	86.7	0.57	49.4		49.4		0.68	1		
15	70	1	83	MC	8750	5625	0.59	0.14	54.0	1.3	1.00	1.0	1.0	71.9	0.55	39.4		39.4		0.66	1		
16	75	1	72	MC	9375	5938	0.57	0.13	46.8	1.3	1.00	1.0	1.0	62.4	0.53	32.9		32.9		0.65	1		
Total Liquefaction Settlement (in.)=																					0.0		

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, C_N = 2.2/(1.2 + σ'_v/P_a) with a maximum value of 1.7 per Kayen et al. (1992) as cited in Youd et al. (2001).
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60CS} = \alpha + \beta (N_1)_{60}$

where α and β = coefficients determined from the following relationships

for FC ≤ 5% $\alpha = 0$, $\beta = 1.0$
for 5% < FC < 35% $\alpha = \exp(1.76 - (190/FC^2))$, $\beta = (0.99 + (FC^{1.5}/1000))$
for FC ≥ 35% $\alpha = 5.0$, $\beta = 1.2$

Reference: Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME **Travers Creek Bridge**
 PROJECT NO. **2013-137-FDN**
 BORING NO. **R-14-002**

SOIL GROUPS
 1. GRAVELS, SANDS AND NONPLASTIC SILTS
 2. CLAYS AND PLASTIC SILTS

FAULT INFO

$a_{max} (g) = 0.23$
 $FAULT M_w = 7.8$
 $MSF = 0.94$

BOREHOLE DIA (in) 5
 GW DEPTH (ft)= 20

HAMMER ENERGY = 80%

Sample No	Depth (ft)	Soil Type	Blow Count	Sample Type	CYCLIC STRESS RATIO (CSR)				LIQUEFACTION RESISTANCE (CRR _{7.5})										S _r =(CRR _{7.5} /CSR)*MSF*K _σ *K _α					Volumetric Strain (%)
					σ _v (psf)	σ' _v (psf)	γ _d	CSR	SPT-N _{eq}	C _E	C _R	C _S	C _B	N ₆₀	C _N	(N ₁) ₆₀	F.C.	(N ₁) _{60, CS}	CRR _{7.5}	K _σ	K _α	F.S.		
1	2	1	6	MC	250	250	1.00	0.15	3.9	1.3	0.75	1.0	1.0	3.9	1.66	6.5		6.5	0.08	1.00	1			
2	5	1	9	MC	625	625	0.99	0.15	5.9	1.3	0.75	1.0	1.0	5.9	1.45	8.5		8.5	0.10	1.00	1			
3	10	1	85	MC	1250	1250	0.98	0.15	55.3	1.3	0.80	1.0	1.0	58.9	1.21	71.0		71.0		1.00	1			
4	15	1	42	MC	1875	1875	0.97	0.14	27.3	1.3	0.85	1.0	1.0	30.9	1.03	31.8		31.8		1.00	1			
5	20	1	74	MC	2500	2500	0.96	0.14	48.1	1.3	0.95	1.0	1.0	60.9	0.90	54.7		54.7		0.91	1			
6	25	1	49	MC	3125	2813	0.94	0.16	31.9	1.3	0.95	1.0	1.0	40.3	0.84	34.1		34.1		0.87	1			
7	30	1	23	MC	3750	3125	0.92	0.17	15.0	1.3	1.00	1.0	1.0	19.9	0.80	15.9	39%	24.0	0.27	0.87	1	1.35		
8	35	1	100	MC	4375	3438	0.89	0.17	65.0	1.3	1.00	1.0	1.0	86.7	0.75	65.3		65.3		0.81	1			
9	40	1	35	MC	5000	3750	0.85	0.17	22.8	1.3	1.00	1.0	1.0	30.3	0.72	21.7	47%	31.0		0.80	1			
10	45	1	100	MC	5625	4063	0.80	0.17	65.0	1.3	1.00	1.0	1.0	86.7	0.68	59.0		59.0		0.75	1			
11	50	1	88	MC	6250	4375	0.75	0.16	57.2	1.3	1.00	1.0	1.0	76.3	0.65	49.5		49.5		0.73	1			
12	55	1	70	MC	6875	4688	0.70	0.15	45.5	1.3	1.00	1.0	1.0	60.7	0.62	37.7		37.7		0.71	1			
13	60	1	73	MC	7500	5000	0.66	0.15	47.5	1.3	1.00	1.0	1.0	63.3	0.59	37.6		37.6		0.69	1			
14	65	1	100	MC	8125	5313	0.62	0.14	65.0	1.3	1.00	1.0	1.0	86.7	0.57	49.4		49.4		0.68	1			
15	70	1	66	MC	8750	5625	0.59	0.14	42.9	1.3	1.00	1.0	1.0	57.2	0.55	31.4		31.4		0.66	1			
16	75	1	65	MC	9375	5938	0.57	0.13	42.3	1.3	1.00	1.0	1.0	56.3	0.53	29.7		29.7	0.45	0.65	1	2.04		
Total Liquefaction Settlement (in.)=																					0.0			

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, C_N = 2.2/(1.2 + σ_v/P_a) with a maximum value of 1.7 per Kayen et al. (1992) as cited in Youd et al. (2001).
3. The influence of Fines Contents are expressed by the following correction: (N₁)_{60CS} = α + β (N₁)₆₀

where α and β = coefficients determined from the following relationships

for FC ≤ 5% α = 0, β = 1.0
 for 5% < FC < 35% α = exp(1.76-(190/FC²)), β = (0.99+(FC^{1.5}/1000))
 for FC ≥ 35% α = 5.0, β = 1.2

Reference: Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

Foundation Design Recommendations & Pile Data Table

Structure: Travers Creek Br (Replace)
Segment: East Manning Av.
Project No.: 2013-137-FIN

Foundation Design Data Sheet

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size (ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	LRFD	30" CIDH	353.00	350.25	3.5	97.77	1	9
Abut 2	LRFD	30" CIDH	353.00	350.25	3.5	97.77	1	9

Foundation Factored Design Loads

Support No.	Service-I Limit State (kips)		Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load Per Support	Permanent Loads Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	1830	1520	3120	390	N/A	N/A	1520	262	N/A	N/A
Abut 2	1830	1520	3120	390	N/A	N/A	1520	262	N/A	N/A

Foundation Design Recommendations

Location	Pile Type	Cut-off Elevation (ft)	Service-I Limit State (kips) per support		Total Permissible Support Settlement (in)	Required Nominal Resistance (kips)				Design Tip Elev. (ft)	Specified Tip Elevation (ft)
						Strength Limit		Extreme Event			
			Total	Permanent		Comp. (qs=0.7)	Tension (qs=0.7)	Comp. (qs=1.0)	Tension (qs=1.0)		
Abut 1	30" CIDH	350.25	1,830	1,520	1	560	N/A	270	N/A	291 (a-I) 314 (a-II) 335 (c) 319 (d)	289
Abut 2	30" CIDH	350.25	1,830	1,520	1	560	N/A	270	N/A	291 (a-I) 315 (a-II) 325 (c) 319 (d)	289

Notes:

(1) Design tip elevations are controlled by (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (c) Settlement, (d) Lateral Load

(2) The specified tip elevation shall not be raised above the lowest design tip elevation.

(3) Slurry displacement method may be used for concrete placement of the CIDH piles. Therefore, the specified pile tip elevations are two feet below the design tip elevations to account for the zone of untested concrete due to the limitations of the gamma-gamma logging (GGL) equipment (Caltrans Geotechnical Manual, "CIDH Concrete Pile Design Considerations" Module, dated March 2021.

Foundation Design Recommendations & Pile Data Table

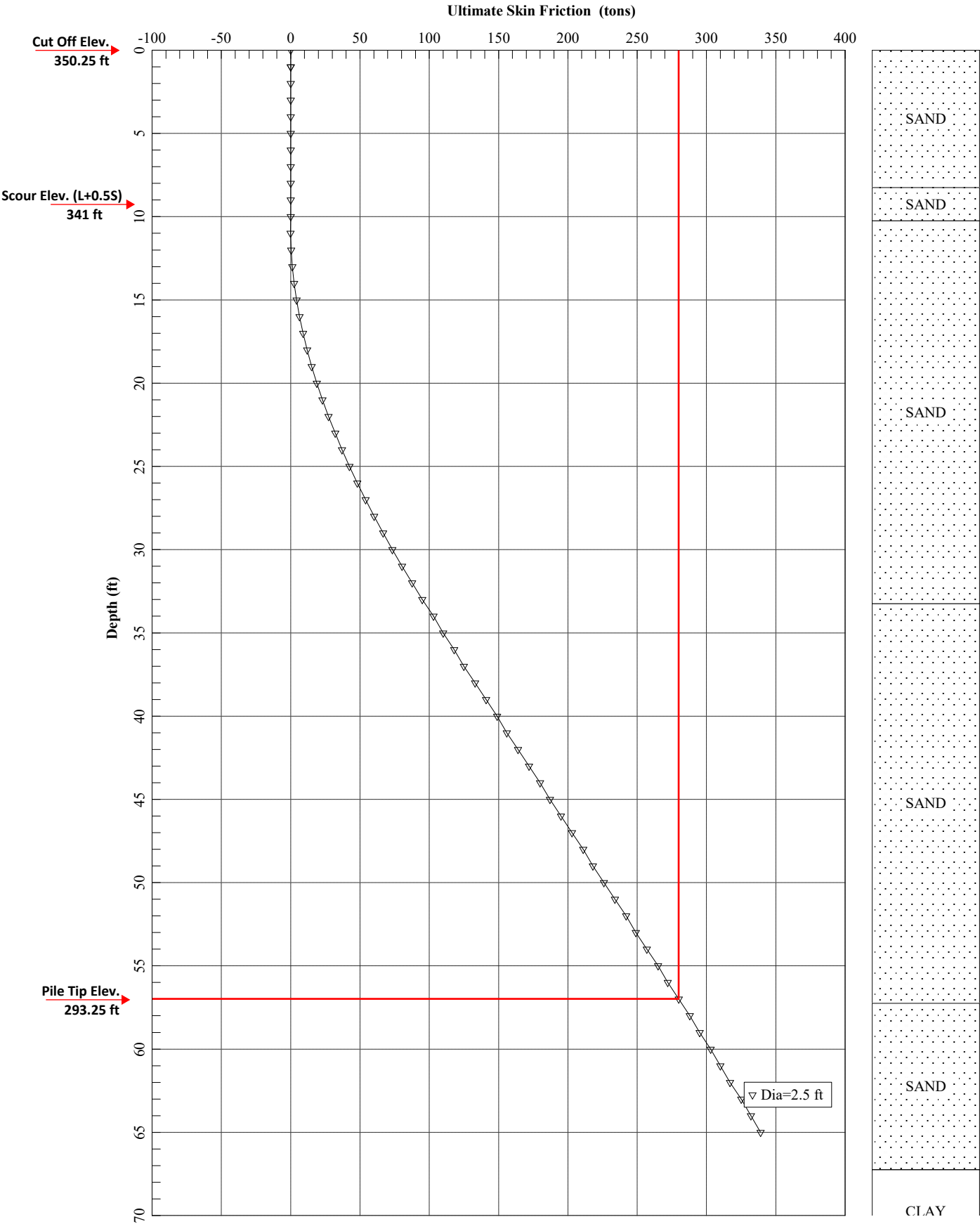
Structure: Travers Creek Br (Replace)
Segment: East Manning Av.
Project No.: 2013-137-FIN

Pile Data Table

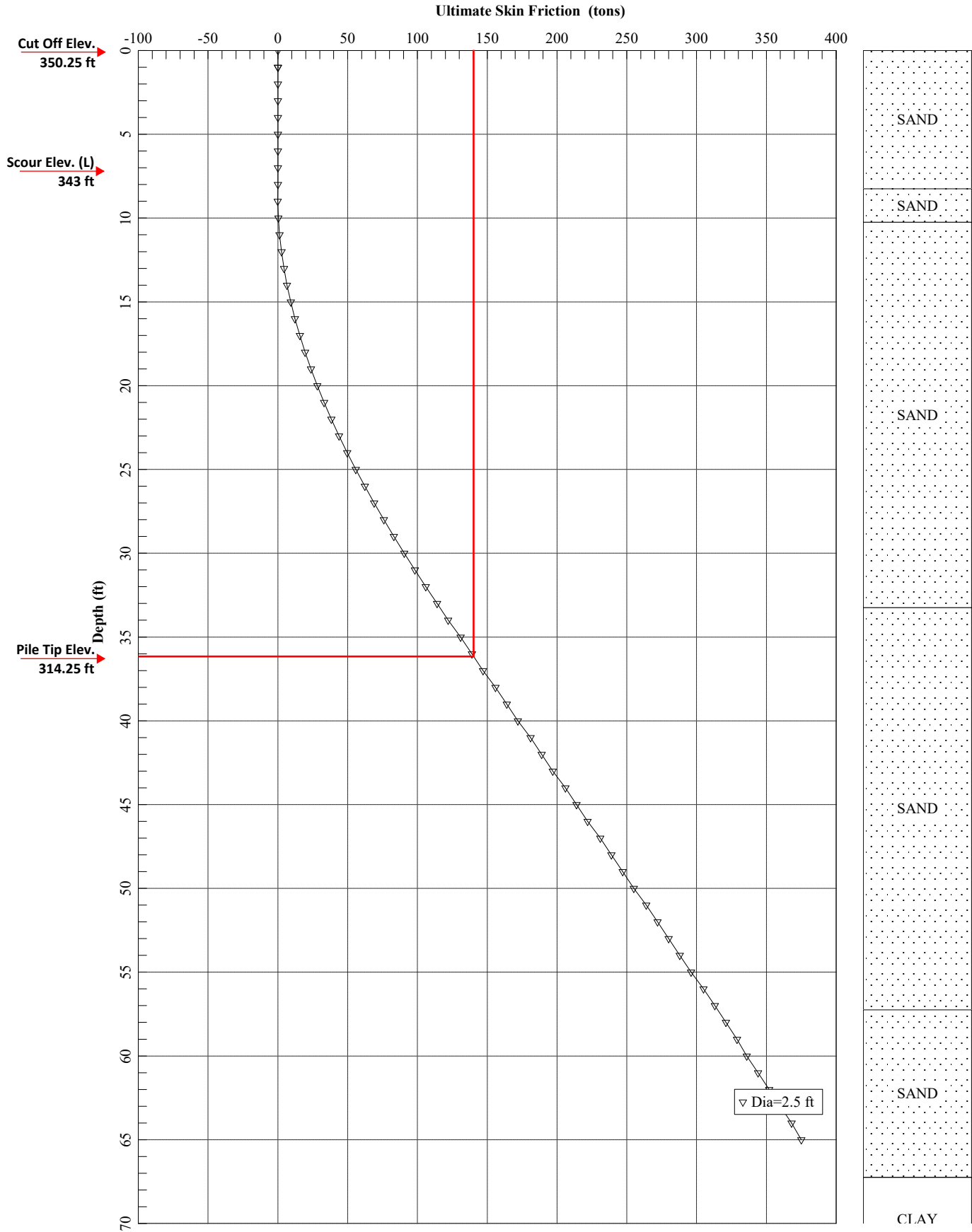
Location	Design Method	Pile Type	Nominal Resistance (kips)		Design Tip Elev. (ft)	Specified Tip Elevation (ft)
			Compression	Tension		
Abut 1	LRFD	30" CIDH	560	N/A	291 (a); 335 (c); 319 (d)	289.0
Abut 2	LRFD	30" CIDH	560	N/A	291 (a); 325 (c); 319 (d)	289.0

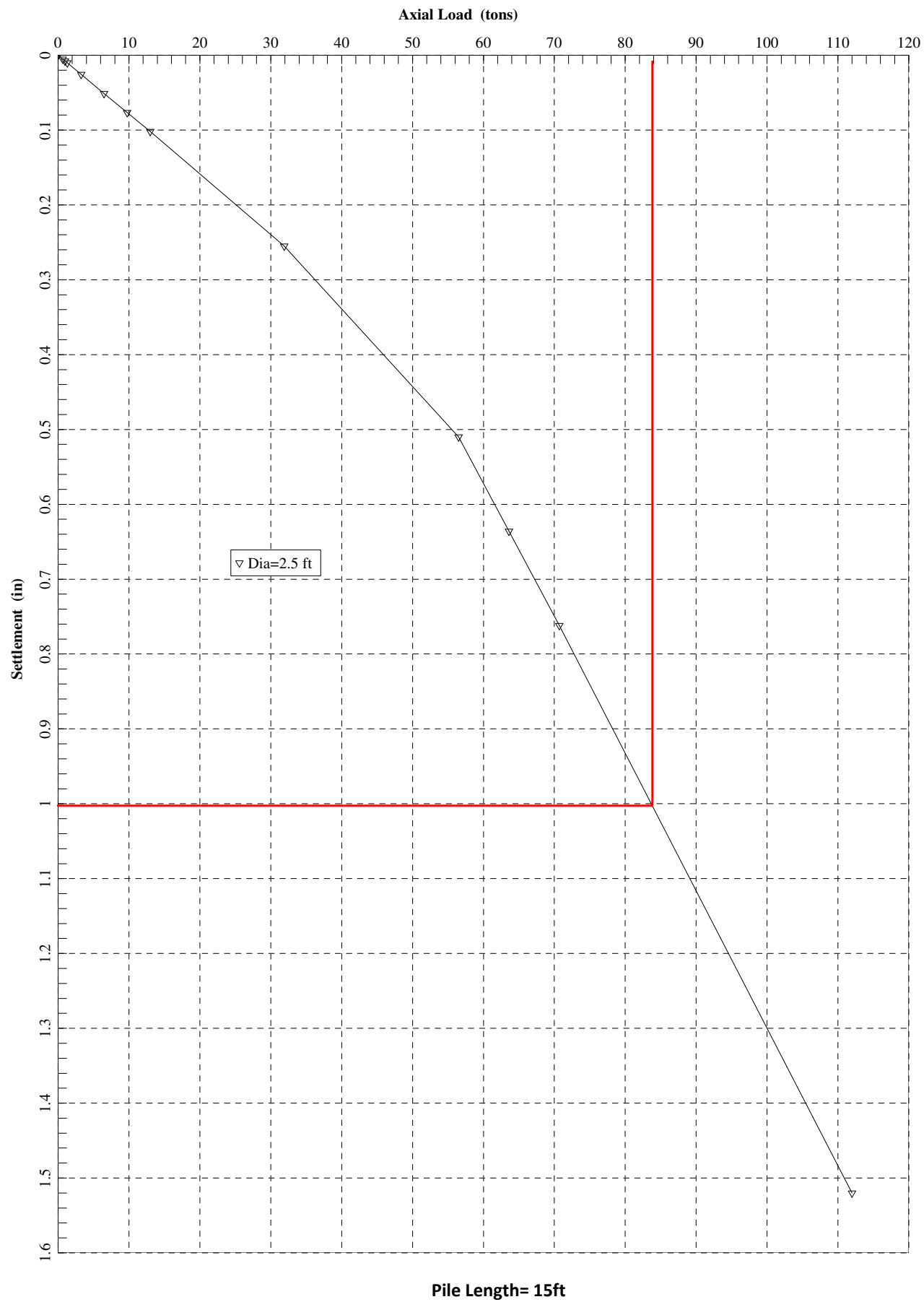
Notes:
(1) Design tip elevations are controlled by: (a) Compression, (c) Settlement, (d) Lateral
(2) The specified tip elevation shall not be raised above the lowest design tip elevation.
(3) Slurry displacement method may be used for concrete placement of the CIDH piles. Therefore, the specified pile tip elevations are two feet below the design tip elevations to account for the zone of untested concrete due to the limitations of the gamma-gamma logging (GGL) equipment (Caltrans Geotechnical Manual, "CIDH Concrete Pile Design Considerations" Module, dated March 2021.

Abutment 1
30" CIDH Pile
Strength Limit: 560kips (280 tons)

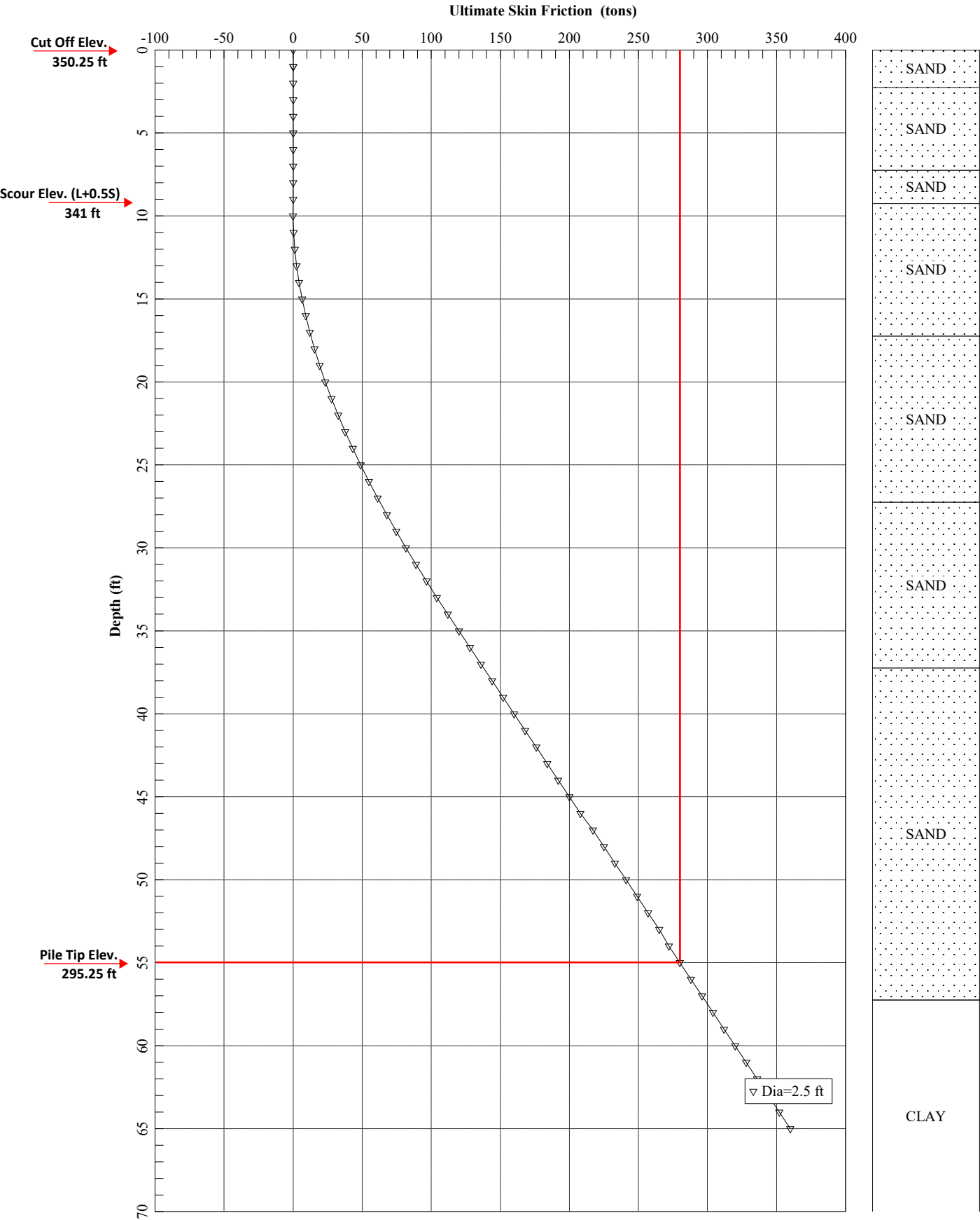


Abutment 1
30" CIDH Pile
Extreme Limit: 270 kips (135 tons)

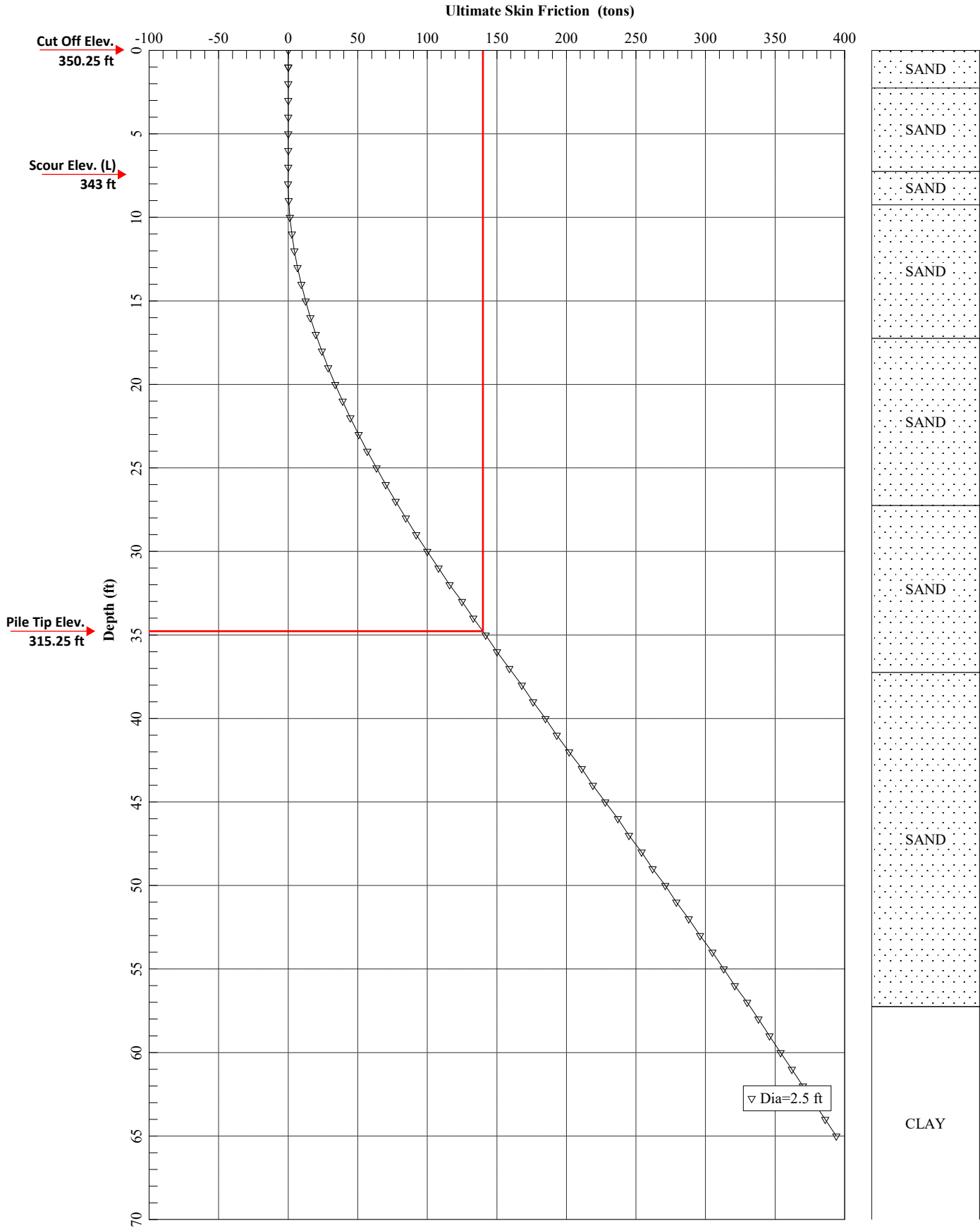


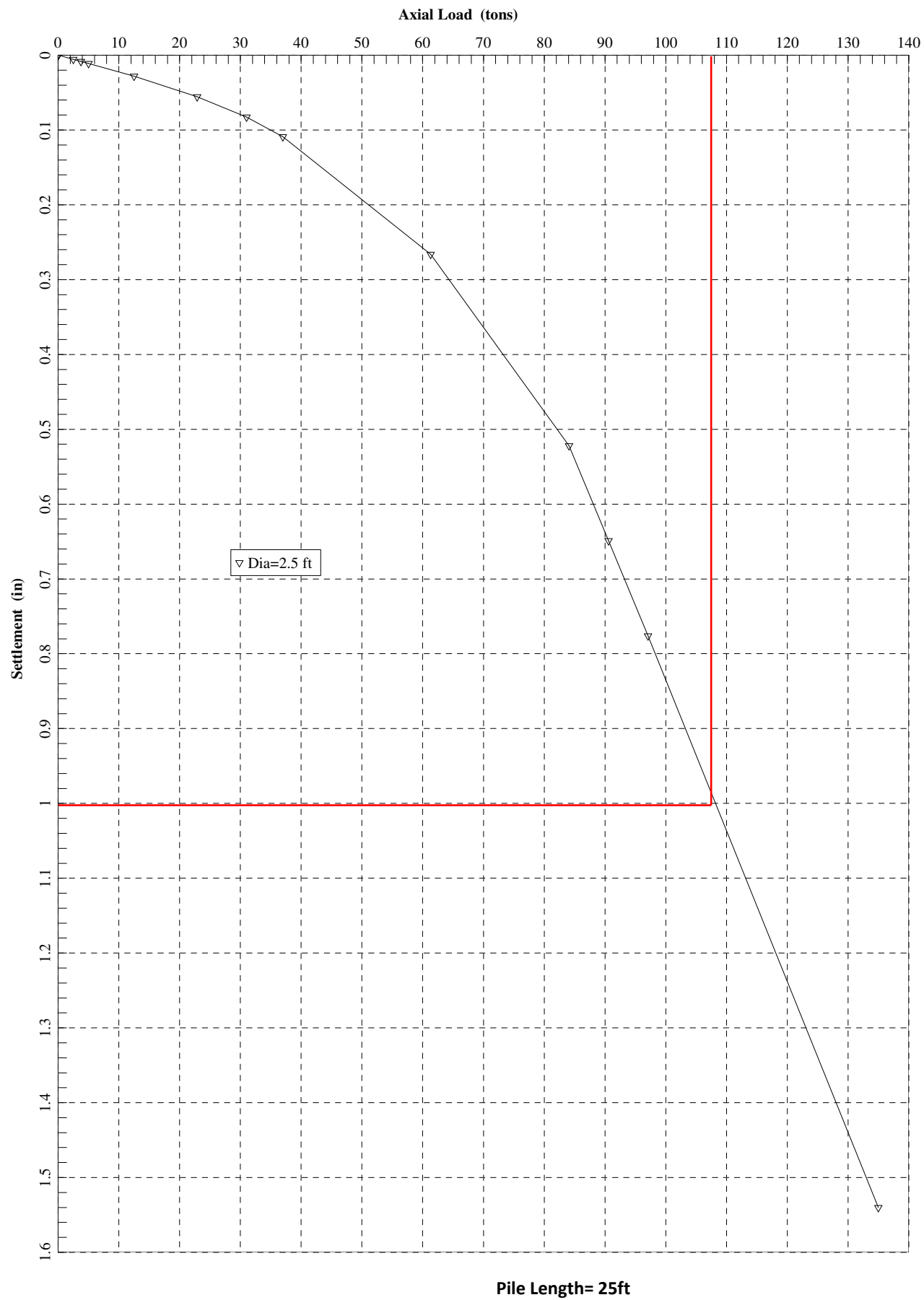


Abutment 2
30" CIDH Pile
Strength Limit: 560kips (280 tons)

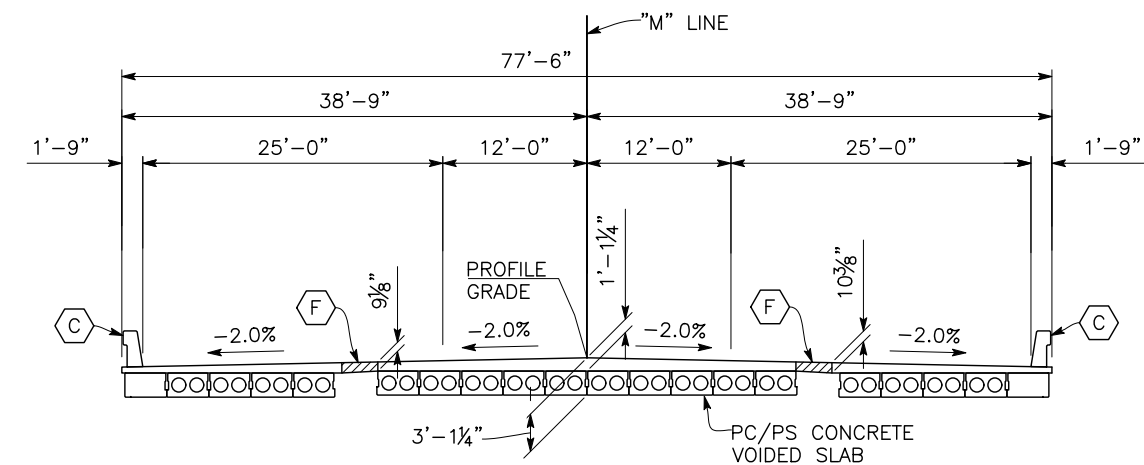
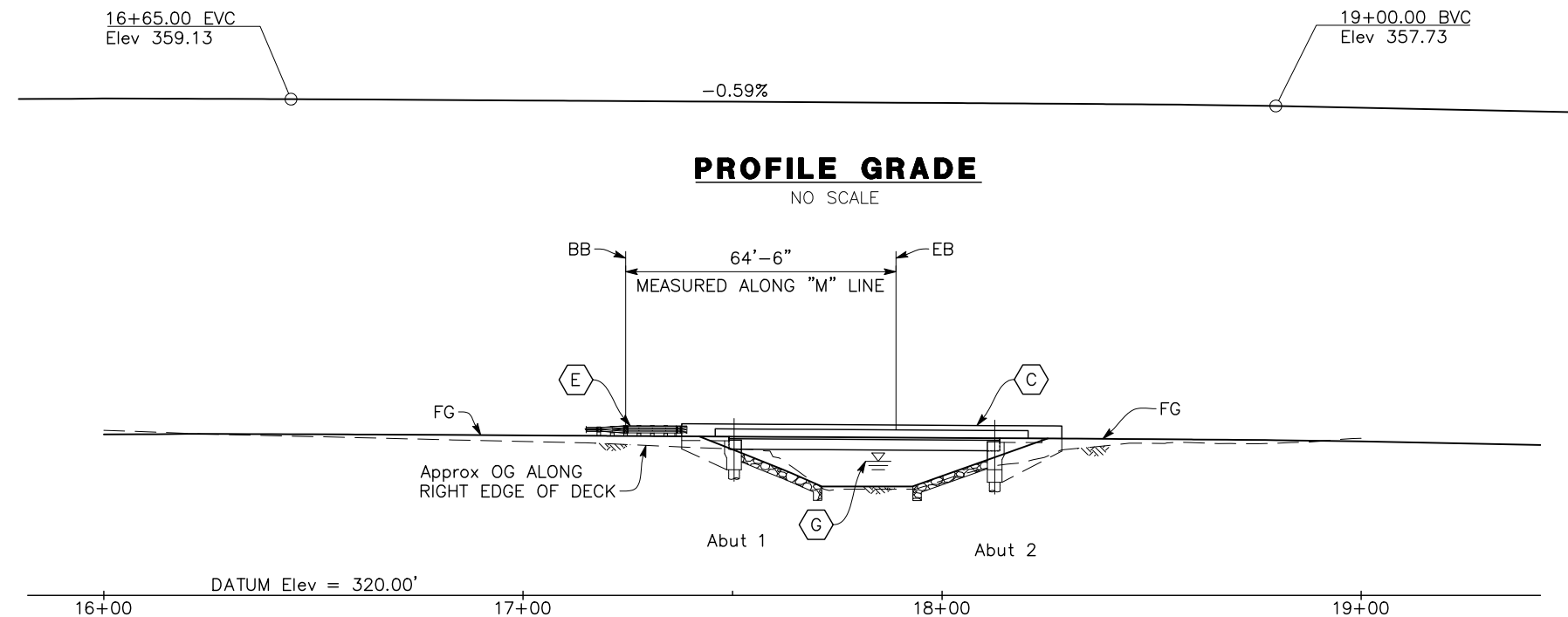


Abutment 1
30" CIDH Pile
Strength Limit: 270kips (135 tons)

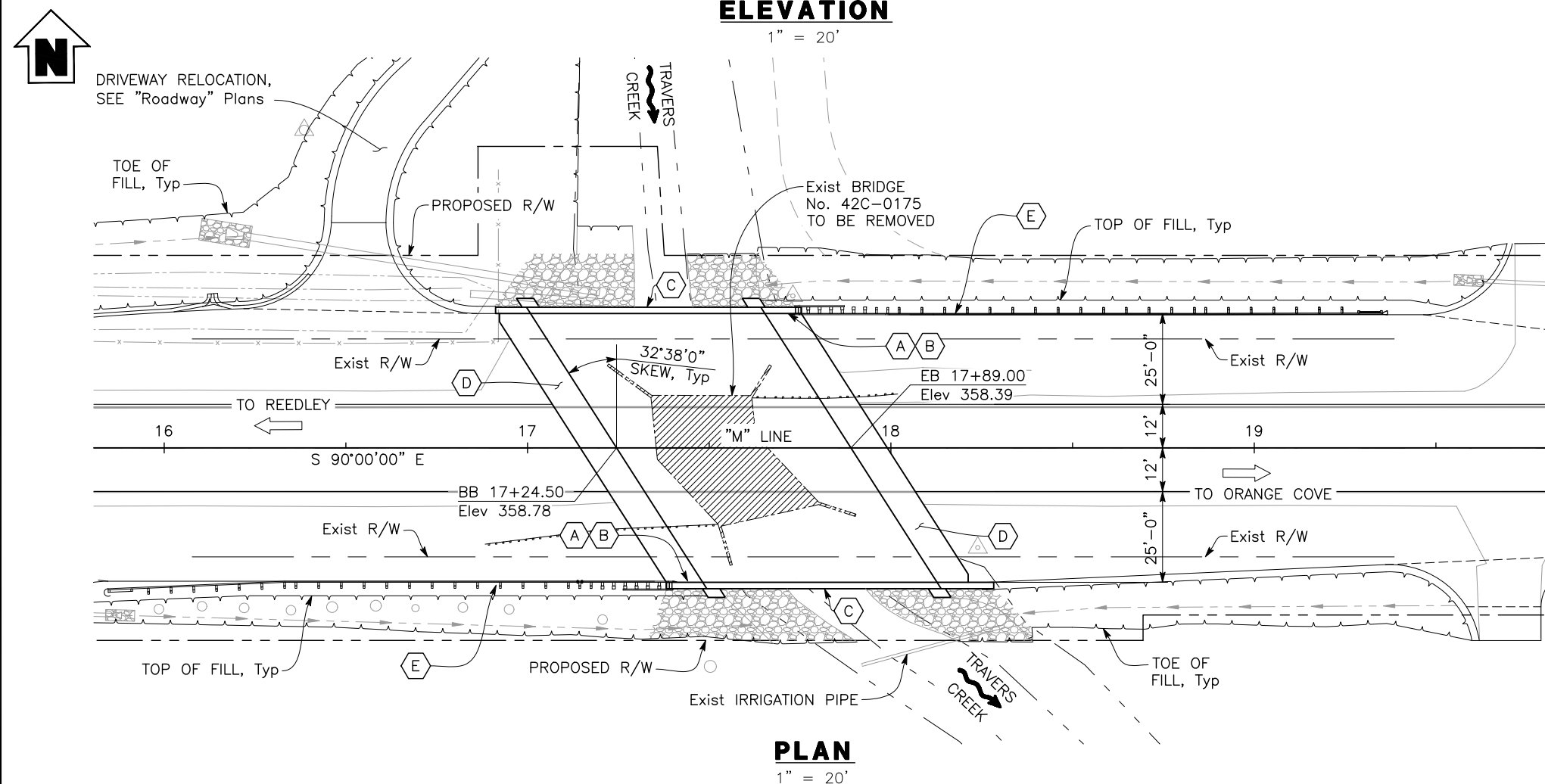




APPENDIX D



TYPICAL SECTION
1/8" = 1'-0"



- KEY NOTES:**
- (A) Paint "Travers Creek Bridge"
 - (B) Paint "Bridge No. 42C0687 FCBN: 10-103" and "Year Constructed"
 - (C) Concrete Barrier Type 836
 - (D) Structure Approach Slab, Type EQ (10)
 - (E) MGS, see Roadway Plans
 - (F) Closure Pour
 - (G) For "HYDRAULIC SUMMARY" see "FOUNDATION PLAN" sheet

- NOTES:**
1. For the location of "GENERAL NOTES", "QUANTITIES" and "INDEX TO PLANS" see the "INDEX TO PLANS" sheet.
 2. For "PILE DATE TABLE", see "FOUNDATION PLAN" sheet.

- LEGEND:**
- Indicates direction of water flow
 - Indicates direction of traffic
 - Indicates Existing Bridge
 - Indicates New Structure
 - Indicates Rock Slope Protection, see Roadway Plans
 - Indicates 100 Yr Water Surface Elevation = 354.1'

		DATE	RECORD DRAWING		SCALE			 Dewberry® DEWBERRY ENGINEERING INC. 11060 WHITE ROCK ROAD SUITE 200 RANCHO CORDOVA, CA 95670	PROJECT			DEPARTMENT OF PUBLIC WORKS AND PLANNING												
DESIGNED: F. CASIMIRO		7/6/26	RESIDENT ENGINEER		DATE	TRAVERS CREEK BRIDGE ON MANNING AVENUE						GENERAL PLAN												
DRAWN: E. CISNEROS		7/6/26				ROAD NO. S0900						DRAWING NO. 11256												
CHECKED: T. DUBOVIK		7/6/26													BRIDGE NO. 42C-0687					SHEET NO. ST-1				
																							TOTAL	
FOR RIGHT OF WAY DATA AND ACCURATE ACCESS DETERMINATION, SEE DOCUMENTS IN THE DEPARTMENT OF PUBLIC WORKS AND PLANNING.																								

FILE => U:\13017 - Manning Avenue over Travers Creek Bridge - sub B01\400 Project Design Files\430 Bridges & Structures\Plans\TC-S-GP.dwg

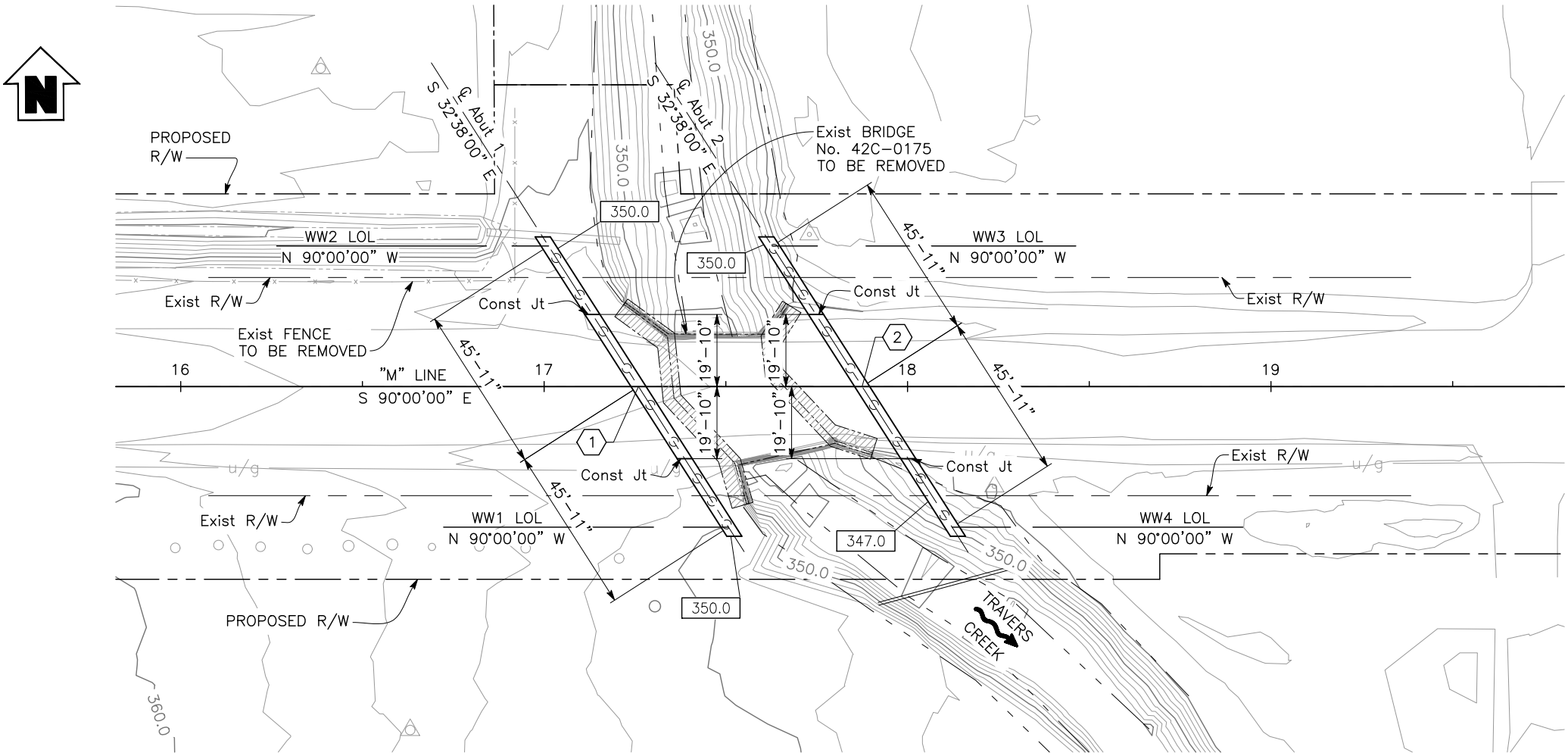
BENCH MARK AND DATUM				
Monument	Coordinates		Elevation (ft)	Description/ Location
	North	East		
5003	10024.65	12367.36	356.79	"FD FC 5/8" RBR
5004	10041.87	12084.05	357.16	"FD FC 5/8" RBR
5006	9969.10	11738.10	363.56	"FD FC 5/8" RBR
5020	9984.71	12090.66	358.08	FD BM L0119
5021	10017.07	11438.02	358.07	SET 80/D

PILE DATA TABLE					
LOCATION	PILE TYPE	NOMINAL RESISTANCE (kips)		DESIGN TIP ELEVATIONS (ft)	SPECIFIED TIP ELEVATIONS (ft)
		COMPRESSION	TENSION		
Abut 1	30" CIDH	560	N/A	289.0 (a) 319.0 (b)	289.0
Abut 2	30" CIDH	560	N/A	289.0 (a) 319.0 (b)	289.0

PILE NOTES:
1. Design Tip Elevation is controlled by the following demands: (a) Compression, (b) Lateral.
2. The CIDH spacificd tip elevation shall not be raised.

HYDROLOGIC SUMMARY		
Drainage Area: 33.9 Square Miles		
Frequency (Years)	Design Flood	Base Flood
	50	100
Discharge (cubic feet per second)	1,090	1,340
Water Surface (elevation at bridge), (feet)	353.3	354.1
Velocity (feet per second)	3.4	3.4

Flood plain data is based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the County and interested or affected parties should make their own investigations.



PLAN
1" = 20'

SCOUR DATA TABLE		
Support Number	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)
Abut 1	343	4
Abut 2	343	4

- NOTES:
- The Contractor shall verify that WWLOL matches edge of precast slab considering construction tolerances in precast slab units have been fabricated.
 - For proposed and existing utilities, see "UTILITY PLAN" in Roadway Plans.

- LEGEND:
- Indicates Bottom of Footing Elevation
 - Indicates Direction of Water Flow
 - Indicates Existing Bridge
 - Indicates New Structure
 - Indicates 30" Dia CIDH Pile

		NORTHING	EASTING
1	Abut 1 Sta 17+25.98	10000.00	12037.04
2	Abut 2 Sta 17+87.52	10000.00	12098.58

DESIGNED: F. CASIMIRO	DATE: 7/6/26	RESIDENT ENGINEER	DATE:
DRAWN: E. CISNEROS	7/6/26		
CHECKED: T. DUBOVIK	7/6/26		

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

SCALE

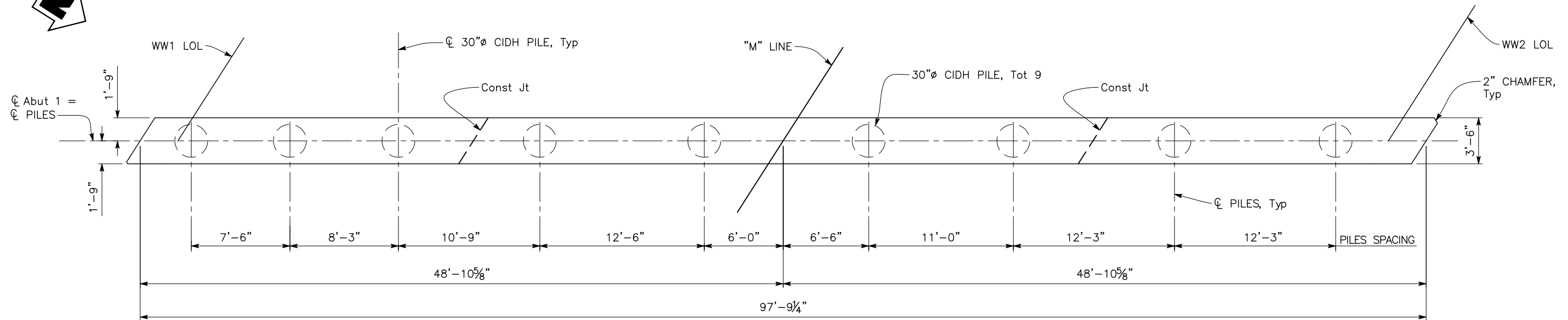
AS SHOWN

Dewberry
DEWBERRY ENGINEERING INC.
11060 WHITE ROCK ROAD SUITE 200
RANCHO CORDOVA, CA 95670

PROJECT
TRAVERS CREEK BRIDGE ON MANNING AVENUE
ROAD NO. S0900 BRIDGE NO. 42C-0687

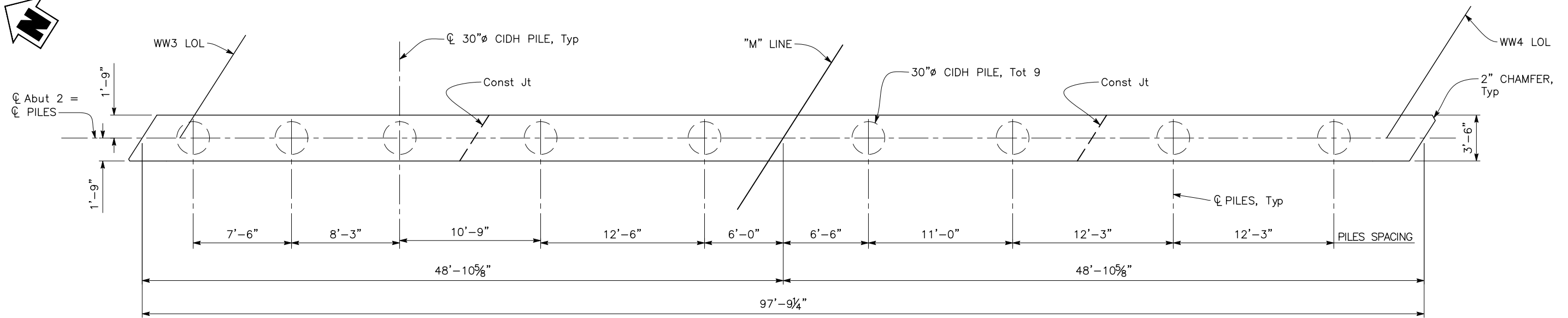
DEPARTMENT OF PUBLIC WORKS AND PLANNING
FOUNDATION PLAN
DRAWING NO. 11256 SHEET NO. ST-4 TOTAL 62

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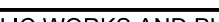
ABUTMENT 1 PILE LAYOUT

1/4" = 1'-0"



ABUTMENT 2 PILE LAYOUT

1/4" = 1'-0"

		DATE	RECORD DRAWING		SCALE			 Dewberry DEWBERRY ENGINEERING INC. 11060 WHITE ROCK ROAD SUITE 200 RANCHO CORDOVA, CA 95670	PROJECT			DEPARTMENT OF PUBLIC WORKS AND PLANNING			
DESIGNED: F. CASIMIRO		7/6/26	RESIDENT ENGINEER		DATE				TRAVERS CREEK BRIDGE ON MANNING AVENUE			ABUTMENT DETAILS NO. 1			
DRAWN: E. CISNEROS		7/6/26													
CHECKED: T. DUBOVIK		7/6/26													
FOR RIGHT OF WAY DATA AND ACCURATE ACCESS DETERMINATION, SEE DOCUMENTS IN THE DEPARTMENT OF PUBLIC WORKS AND PLANNING.															
ROAD NO. S0900		BRIDGE NO. 42C-0687								DRAWING NO. 11256		SHEET NO. ST-8		TOTAL 62	