September 9, 2016

TES No. 160599.001
Invoice No. 11747

Mr. Jonathan P. Jensen
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

Project: Englehart Avenue Bridge Replacement at
Reedley Main Canal
Fresno County, California

Subject: Foundation Report

Dear Mr. Jensen:

The attached Foundation Report presents the results of a geotechnical investigation for the design and construction of a reinforced concrete box culvert (RCB) planned on Englehart Avenue at Reedley Main Canal near Reedley, Fresno County, California. The report describes the study, findings, conclusions, and recommendations for use in project design and construction.

TECHNICON appreciates the opportunity to provide geotechnical engineering services to Cornerstone Structural Engineering Group during the design phase of this project. We trust this information meets your current needs. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully submitted,
TECHNICON Engineering Services, Inc.

Sarbjit Athwal, EIT
Project Engineer

Stephen P. Plauson, PE, GE
Geotechnical Engineering Manager

SS:SPP:mk

FOUNDATION REPORT
ENGLEHART AVENUE BRIDGE REPLACEMENT AT REEDLEY MAIN CANAL
FRESNO COUNTY, CALIFORNIA

Prepared For:

Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

September 9, 2016

TES No. 160599.001

GEOTECHNICAL & ENVIRONMENTAL ENGINEERING – CONSTRUCTION TESTING & INSPECTION

Prepared For:

Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

FOUNDATION REPORT
ENGLEHART AVENUE BRIDGE REPLACEMENT AT
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FRESNO COUNTY, CALIFORNIA

TECHNICON PROJECT
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Prepared by:

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Geotechnical Engineering Manager

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 9/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

TECHNICON Engineering Services, Inc.
4539 North Brawley Avenue, Suite 108
Fresno, California 93722
(559) 276-9311

September 9, 2016

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1 INTRODUCTION

1.1 GENERAL

This Foundation Report presents the results of a geotechnical investigation for a reinforced concrete box culvert (RCB) planned on Englehart Avenue at Reedley Main Canal near Reedley, Fresno County, California. The purpose of the investigation was to explore and evaluate the subsurface conditions at the site and prepare a Foundation Report containing recommendations to aid in project design and construction.

The Vicinity Map, presented on Figure 1, shows the location of the project and the Site Map, Figure 2, and Log of Test Boring Drawing (LOTB) show the proposed bridge replacement and the approximate boring location for this study.

1.2 PROJECT DESCRIPTION

The project involves the replacement of an existing bridge located on Englehart Avenue at Reedley Main Canal. The existing bridge is a 2 span, reinforced concrete flat slab bridge of approximately 29 feet long by 19 feet wide. The replacement bridge is anticipated to consist of a reinforced concrete box culvert (RCB) with a closed bottom. To accommodate the canal and roadway widths, the RCB will be approximately 58 feet in length and 24 feet in width. Based on preliminary information provided by Cornerstone Structural Engineering Group, the RCB will have a opening height of 6 feet and cover height equal to a typical asphalt concrete pavement section (e.g. less than 1.0 foot of cover) for a total height of approximately of 9 feet. The design will incorporate a concrete bottom slab and slab extensions up and down stream. Warped wing walls will form the transition of the bottom slab and side slopes.

It is anticipated that Caltrans Standards Plans will be utilized as the basis for design of the culvert and wingwalls.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to explore the site subsurface conditions to allow for development of recommendations and opinions to aid in project design. The report includes the following: A description of the proposed project including a vicinity map showing the location of the site and a site plan showing the locations of the exploration point for this study

- ❑ A description of the site surface and subsurface conditions encountered during the field investigation, including boring log
- ❑ A summary of the field exploration and laboratory testing program
- ❑ Discussion of regional and local geology including faults, seismicity, and liquefaction potential and associated effects
- ❑ Caltrans seismic design parameters
- ❑ Comments on the use of Caltrans Standard Plans for design of the box culvert and associated wingwalls
- ❑ Recommended Gross Nominal Bearing and Permissible Net Contact Stress for the box culvert foundation and anticipated settlement
- ❑ Recommended lateral earth pressures for design of the box culvert and wingwalls
- ❑ Comments on the corrosion potential of on-site soil
- ❑ Recommended pavement structural section for the design traffic index.
- ❑ Comments on site preparation and earthwork, including the use of on-site soils for engineered fill and recommended import fill specifications

The scope of services consisted of a field exploration program, laboratory testing, design analysis, and preparation of this written report as outlined in **TECHNICON'**s proposal dated April 20, 2016 (TES No. GP16-103).

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

The field exploration, conducted on July 15, 2016 consisted of drilling one (1) exploratory test boring and a site reconnaissance by a project engineer. The test boring was drilled with a CME 75 truck-mounted drill rig using hollow stem augers. The boring extended to a depth of 51.5 feet below the existing ground surface (bgs). The approximate location of the test boring is indicated on the Site Map, Figure 2, and the Log of Test Boring Drawing (LOTB), Sheet 2. In addition, a Dynamic Cone Penetration (DCP) Test was performed in the center of the canal to assess the depth of historic scour.

The soils encountered in the boring 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 as the number of blows per foot over the last 12 inches of sampler penetration on the LOTB. The blow counts listed in the LOTB have not been corrected for the effects of overburden pressure, rod length, boring diameter, sampler size, or hammer efficiency.

2.2 FIELD AND LABORATORY TESTING

Penetration rates, determined in general accordance with ASTM D-1586, were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

Laboratory tests were performed on selected near surface samples to evaluate their physical characteristics. The following laboratory tests were used to develop the design geotechnical parameters:

- ❑ Unit weight (ASTM D2937)
- ❑ Moisture content (ASTM D2216)
- ❑ Sieve Analysis (ASTM D422)
- ❑ Direct Shear (ASTM D3080)
- ❑ Soluble Sulfate, and Soluble Chloride Contents (California Test Method No's. 417 and 422)

- ❑ pH and Minimum Resistivity (California Test Method No. 643)
- ❑ Resistance Value (California Test Method No. 301)

The dry density and moisture content test results are shown on the LOTB in Appendix A. The soluble sulfate, soluble chloride, pH and minimum resistivity are discussed in the "Corrosion Potential" Section (Section 5.6). The remaining test results are provided in Appendix B.

3 SITE GEOLOGY AND CONDITIONS

3.1 SURFACE CONDITIONS

The subject bridge replacement is at the Englehart Avenue and Reedley Main Canal crossing. Englehart Avenue is a 2 lane asphalt paved road with unpaved shoulders and aligned north-south. Reedley Main Canal was unlined and at the time of the field investigation the canal flowing with a water depth of approximately 4 to 5 feet. The slopes of Reedley Main Canal were approximately 1:1 to 1/2:1 horizontal to vertical (H:V), with the canal crossing Englehart Avenue in a northeast to southwest direction at a skew of approximately 55 degrees. The bridge location is generally bounded by mature tree orchards to the west, northwest, and northeast, open fields to the southwest and southeast, and American Avenue to the south.

3.2 SUBSURFACE CONDITIONS

The natural site soil consists of nonmarine deposits with a geologic age of Pleistocene. The general earth material profile depicted by the subsurface exploration consisted primarily of silty sand in the upper 3 feet, followed by poorly graded sand with silt and poorly graded sand to 16 feet and underlain by laterally discontinuous layers of clayey sand, silty sand, poorly graded sand, poorly graded sand with silt, and sandy clay to the depth explored, 51.5 feet bgs. The granular soil generally had a relative consistency of loose to very dense while the fine grained soil generally had a relative consistency of hard.

The above is a general description of the earth material profile. A more detailed representation of the stratigraphy at the specific exploration location is provided on the LOTB included in Appendix A.

3.3 GROUNDWATER CONDITIONS

Groundwater was encountered at depth ranging from 4 to 15 feet bgs at the test boring location. The water encountered appears to be perched due to water flow in the canal. The State of California Department of Water Resources, “Lines of Equal Elevation of Water in Wells”, Spring 2011 indicates the regional depth to groundwater exceeds 50 feet. Additional research utilizing the California Department of Water Resources (DWR) website indicates the nearest monitored well to be approximately ¼ of a mile to the southeast (Well No. 14S23E36R001M). Based on the groundwater elevation data collected at this well, the historic high groundwater depth was recorded at 16 feet bgs in the early 1980's and the current recorded groundwater depth is approximately 55 feet bgs.

The groundwater elevation at the bridge site is likely is more likely influenced by flow or recency of flow within Reedley Main Canal and could affect construction. Depending on the flow or recency of flow in Reedley Main Canal at the time of construction, earthwork and construction may be impacted by soft/yielding subgrade and/or saturated conditions. It is assumed that construction may occur during the winter months shortly after closure of the canal. Therefore, it should be anticipated that the canal bottom and sides of the canal could be saturated and may not provide a stable bottom for construction activities.

4 SEISMIC RECOMMENDATIONS

4.1 SEISMIC SOURCES

The project site and its vicinity are located in an area traditionally characterized by relatively low seismic activity. The site is not located in an Alquist-Priolo Earthquake Fault Zone as established by the Alquist-Priolo Fault Zoning Act (Section 2622 of Chapter 7.5, Division 2 of the California Public Resources Code).

Review of the Caltrans Deterministic PGA Map (September 2007), indicates there are no existing major fault systems within 25 miles of the project vicinity. Based on review of published data and current understanding of the geologic framework and tectonic setting of the proposed improvements, the primary sources of seismic shaking at this site are listed in Table 4.1-1. A major seismic event on these or other nearby faults may cause ground shaking at the site. Based on the deterministic ground acceleration, the San Andreas Fault is considered the governing fault.

TABLE 4.1-1
LOCAL FAULTS AND ESTIMATED MOMENT MAGNITUDES

Fault	Approximate Distance from Site (km)	Maximum Credible Earthquake (Moment Magnitude, M_w)	Peak Ground Acceleration (g)
San Andreas Fault	129	8.0	0.091
Independence	99	7.1	0.084
Round Valley	97	7.0	0.081
Coast Ranges Sierran Block	89	6.5	0.067

4.2 SEISMIC DESIGN CRITERIA

Development of a site specific Acceleration Response Spectra (ARS) curve was undertaken in accordance Caltrans Geotechnical Design Manual (Ver. 2.3.07, March 2016) and the Caltrans Seismic Design Criteria (Ver. 1.7, November 2013).

The Wahtoke, California 7½-minute Quadrangle Topographic Map indicates the proposed replacement Englehart Avenue Bridge Replacement lies on the southeast part of Section 31, T14S, R24E. Furthermore, the average shear wave velocity for the upper 30m (100 feet) of the subsurface soil and rock at the bridge site was estimated by using established correlations and procedures presented in the Caltrans Geotechnical Design Manual. The estimated shear wave velocity is provided below.

Site Location: Latitude: 36.66253° N / Longitude: -119.41258° W
Shear Wave Velocity: Vs(30) = 311 m/s

ARS curves for the bridge site were determined based on the Caltrans Deterministic PGA Map (September 2007), Caltrans ARS Online (Ver. 2.3.07), the shear wave velocity of the soil, and the latitude/longitude at the bridge location. A Site Specific ARS curve was developed for the project and is included in Appendix D for use in the seismic analysis of the bridge. The recommended Design ARS curve consists of the envelope of the Caltrans Minimum Deterministic ARS and Caltrans Online Probabilistic ARS. The results of the 2008 USGS Deaggregation Tool (Beta) do not govern, since the shear wave velocity exceeds 300 m/s.

4.3 SEISMIC HAZARDS

Review of the Caltrans Deterministic PGA Map (September 2007) indicates that no mapped active faults cross or project toward the site. Additionally, no evidence of active faulting was visible on the site during our site reconnaissance. Therefore, it is our opinion that the potential for fault-related surface rupture at the proposed bridge site is very low. Furthermore, the Caltrans Deterministic PGA Map (September 2007) indicates the site is located relatively far from active faults, as such, the possibility for the site to experience strong ground shaking may be considered low.

4.4 SEISMICALLY INDUCED GROUND FAILURE

4.4.1 Design Ground Motion

For the purpose of evaluating liquefaction, a probabilistic seismic hazards analysis (PSHA) procedure was performed using the 2008 USGS Deaggregation Tool (Beta) to estimate the earthquake magnitude. The program allows user input of the project site coordinates and produces the expected peak ground motions for the site for selected probability of exceedance (e.g. return periods). The USGS Deaggregation Tool, based on a probability of exceedance of 2 percent in 50 years, determined a weighted magnitude of Mw = 6.08. The peak ground acceleration was assessed using ARS Online and found to be 0.227g

4.4.2 Liquefaction

In order for liquefaction, and possible associated effects, 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 fine, granular, and uniform,
- Ground shaking of sufficient intensity should occur 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).

Saturated granular sediments can experience liquefaction if subject to seismically induced ground motion of sufficient intensity and duration. Based on the ground shaking which may be expected at this site, the relative density and geologic age of the sediments, analysis utilizing Youd (2001) indicates liquefaction, seismically induced settlement, or bearing loss is considered unlikely.

4.4.3 Dynamic Compaction

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Considering that problematic soils were not identified in the borings drilled for this study, seismically induced dry sand settlement is anticipated to be minimal. Calculations indicate that seismically induced dry sand settlement is negligible.

5 DESIGN RECOMMENDATIONS

5.1 GENERAL

Based on the laboratory data, field exploration, and geotechnical analyses conducted for this study, it is geotechnically feasible to construct the proposed RCB as currently envisioned. Provided that the recommendations presented in this report are incorporated into the project design and construction, use of a closed bottom RCB with bottom mat/slab bearing on recompacted native soil or approved engineered fill prepared in accordance with Caltrans Standard Specifications, Section 19 are considered appropriate for structure support. Recommendations regarding the geotechnical aspects of design are presented in subsequent sections.

5.2 SCOUR EVALUATION

TECHNICON performed a gradational analysis of the sediments within the test boring at the elevation of the Reedley Main Canal bottom to aid in the hydraulic evaluation of the channel scour by others.

To evaluate the canal bottom for scour, **TECHNICON** performed Dynamic Cone Penetration (DCP) Test to determine the historic scour depth. The DCP test was performed by dropping a 15-lb slide hammer from a height of 20 inches driving a 1.5-inch cone pointed rod. Observations and hand exploration indicates the Reedley Main Canal channel has undergone localized scour within isolated areas of the existing bridge. It is estimated that the scour depth has extended to a depths of approximately 12 to 18 inches below the current canal bottom elevation. A summary of the DCP Test results can be seen in Appendix C.

5.3 SLOPE STABILITY

Slope stability using dimensionless parameters by Janbu for permanent and temporary slopes was calculated for a canal and temporary slope height of 8 feet. It was determined that permanent slopes configured at 1½:1 H:V should be stable with regard to gross (deep seated) and surficial slope failure modes (factor of safety greater than 1.5, respectively). Temporary slopes configured at 1¼:1 H:V should be stable with regard to gross (deep seated) failure mode (factor of safety greater than 1.25).

5.4 BOX CULVERT DESIGN

5.4.1 Bearing and Settlement

Based on the field exploration, laboratory testing, and geotechnical analyses, the soils at the site are suitable for supporting the RCB. The General Plan indicates the proposed RCB length is approximately 58 feet and the width is approximately 24 feet. The opening height of the RCB is 6 feet and the overall structure height including pavement is estimated to be 9.0 feet.

Considering the base dimensions of the RCB and the shear strength of the on-site soils, the Gross Nominal Bearing Resistance is high. Table 5.4-1 “Footing Data Table” provides the bearing resistance and settlement.

TABLE 5.4-1
FOOTING DATA TABLE

Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth (ft)	Total Permissible Support Settlement (inches)	Service Limit State	Strength or Construction Limit State ϕ_b=0.45
L	B				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
58	24	392.57	1	1	3.8	18.9

Based on the Gross Maximum Bearing Stress of 3.0 ksf provided by the structural engineer for the RCB, the total settlement of the RCB is approximately 0.8-inch. Differential settlement is anticipated to be reduced to half of the total settlement across the length/width of the RCB.

The design bearing stress/resistance given in Table 5.2-1 requires that the RCB will be placed on unyielding native soil or approved engineered fill. Any soft, unsuitable sediment in the channel bottom should be excavated to expose firm undisturbed soil and removed from project site. Based on observations and DCP testing performed in the canal bottom, for preliminary planning it should be anticipated that a general excavation depth of 12 to 18 inches may be required to remove unsuitable soil. However, isolated deeper areas deemed unsuitable could exist, which may require deeper excavation.

If unstable foundation conditions are encountered it will be necessary to stabilize the area prior to foundation construction. Stabilization options include placing a minimum of 12 inches of either a lean concrete slurry or ¾-inch diameter crushed gravel. If the crushed gravel is utilized, an engineering fabric conforming to the requirements of Section 88 of the Caltrans Standard

Specifications should be placed on the subgrade prior to rock placement to prevent migration of fines into the rock. The fabric is necessary to add reinforcement and prevent migration of subgrade soil into the open spaces of the gravel. **TECHNICON** should be contacted to observe and approve the exposed subgrade prior to stabilizing the working/foundation area.

5.4.2 Lateral Earth Pressures

Caltrans Standard Plans, May 2010, for RCB's are based on the soil surrounding the planned RCB having minimum and maximum lateral earth pressures equal to 42 lb/ft^3 and 100 lb/ft^3. In addition, the maximum cover density is to be limited to 140 lb/ft^3. Based on the analysis of the native soil, the soil will exhibit an earth cover density of approximately 131 lb/ft^3. The minimum and maximum restrained lateral earth pressures of the native soil, backfilled in accordance with Caltrans Standard Specifications, Section 19 are 68.5 lb/ft^3 and 93 lb/ft^3. Consequently, the use of Caltrans Standard Plans for design of the RCB would be appropriate. Table 5.4-2 provides active and at-rest pressures and the dynamic incremental increase of the earth pressure against retaining walls considering earthquake loading. The pressures are based on the use of on-site soils for wall backfill.

TABLE 5.4-2
LATERAL EARTH PRESSURES

Loading Condition	Lateral Earth Pressure (psf/ft of Wall Height)		Earth Pressure Coefficient
	Drained	Undrained	
Active Pressure (psf/ft of depth)	36	19 + Hydrostatic	0.27
At-Rest Pressure (psf/ft of depth)	57	30.5 + Hydrostatic	0.43
Dynamic Active Incremental Increase (psf/ft of depth)	16.0		
Dynamic At-Rest Incremental Increase (psf/ft of depth)	8.0		

The Special Provisions requires that backfill placed within a 1:1 zone extending upward from the base of the RCB consist of low expansion granular fill (Expansion Index less than 10).

Should retaining walls be influenced by surcharge loads, the surcharge against the walls can be evaluated by multiplying the surcharge pressure by the earth pressure coefficient. Surcharge loads should be modeled as a uniform pressure against the wall by multiplying the surcharge load by the earth pressure coefficient.

5.4.3 Resistance to Lateral Loading

Lateral loads applied to RCB can be resisted by a combination of passive lateral bearing and sliding resistance. The allowable and ultimate passive pressures and frictional resistance for the RCB are presented in Table 5.4-3.

TABLE 5.4-3
PASSIVE BEARING AND SLIDING RESISTANCE

	WSD		LRFD	
	Static	Total Combined	Nominal	Strength Limit
Frictional Coefficient (Sliding)	0.47	0.56	0.70	0.56
Passive Pressure (psf/ft of depth)	250	335	500	250
Lateral Translation Needed to Develop Passive Pressure	0.007D	0.015D	0.03D	0.007D

Note: D is the depth of the zone providing resistance.
WSD = Working Stress Design, LRFD = Load/Resistance Factor Design

5.4.4 Bottom Slab Cutoff Wall

Extensions of the culvert bottom slab are planned up and down stream of the proposed RCB. Based on the granular nature of the anticipated bottom sediments and presence of flowing water, it is recommended that a cutoff wall be constructed at the ends of the concrete channel lining. The cutoff wall could be designed in accordance with Caltrans Standard Plans and have a minimum embedment of 4 feet below the bottom of the RCB. The final embedment of the cutoff wall should be extended as dictated by the scour conditions.

5.4.5 Warped Wingwalls

Proposed warped wingwalls shall be supported on approved undisturbed native soil channel slopes or properly engineered fill as well as the bottom slab extension. The native soils have strength characteristics that result in design earth pressures compatible with Caltrans Standard Plans. Provided that the Special Provisions specify that imported backfill consist of soil similar to the native soil or soil having a ϕ angle of at least 35 degrees, Caltrans Standard Plans design could be used.

5.4.6 Construction Observations

The culvert excavation should be observed by a representative of the Geotechnical Engineer. The purpose of these observations is to check that the bearing soils exposed in the excavation are similar to those on which the recommendations are based.

5.5 PAVEMENT DESIGN

Bulk soil samples were tested at two locations for R-value for pavement design. The test results are presented in Table 5.5-1. Pavement recommendations will be provided in the “Final” Foundation Report for the design Traffic Index (TI) to be provided by Mark Thomas & Company.

TABLE 5.5-1
SUMMARY OF R-VALUE TESTS

Sample Location	Depth (ft)	Soil Type	R-Value by Exudation
RV-1	0-2	Silty SAND (SM)	53
RV-2	0-2	Silty SAND (SM)	62

5.6 CORROSION POTENTIAL

Two (2) soil samples obtained from the site were tested to evaluate pH, minimum electrical resistivity, and soluble sulfate and chloride content. Provided in Table 5.6-1 are the pH, minimum electrical resistivity and soluble sulfate and chloride content.

TABLE 5.6-1
CORROSION POTENTIAL

Depth (ft)	Location	Soil Type	pH	Minimum Resistivity (ohm-cm)	Soluble Sulfate (ppm)	Soluble Chloride (ppm)
0 to 3	B-1	Silty Sand (SM)	7.38	12,780	5	5
10 to 16	B-1	Clayey Sand (SC)	7.82	2,237	5	5

These values are all outside the Caltrans threshold limits. Consequently, the site would be considered to be a non-corrosive environment with respect to foundations.

These values are generally representative of an environment that would be mildly corrosive to buried unprotected metals. An example of the potential soil corrosion is provided by utilizing methods provided in Caltrans California Test 643, “Method for Estimating the Service Life of Steel Culverts”. The method indicates a 1-gauge steel zinc-coated culvert is estimated to have a maintenance-free service life (years to perforation) provided in Table 5.6-2. Therefore, if project improvements will involve metal that comes into contact with the on-site soil (e.g. steel barriers etc.), the design should consider the potential soil corrosiveness described.

TABLE 5.6-2
ESTIMATED SERVICE LIFE OF BURIED STEEL
"UTILIZING CALIFORNIA TEST METHOD 643"

Depth	Location	Maintenance-Free Service Life (Years to Perforation)
0 to 3	B-1	70
10 to 16	B-1	34

5.7 EARTHWORK

5.7.1 Grading

All grading operations should be performed in accordance with the project specifications and within the intent of applicable items of Section 19 of the Caltrans Standard Specifications, 2010. It is recommended that relative compaction be based on dry weight methodology for Caltrans 216 and 231. Where culvert and wingwall fill is place against the existing Reedley Main Canal canal slopes, benches having horizontal dimensions of 2 vertical should be excavated to remove unsuitable/disturbed soil and expose competent subgrade.

5.7.2 Engineered Fill

All engineered fill soils should be non-expansive, relatively granular soil that is nearly free of, rubble, organics or other deleterious debris, and less than 3 inches in maximum dimension. Excavated on-site soil may be used as engineered fill, provided they meet the above criteria. Any imported soil shall meet also meet these criteria. Imported fill materials to be used for engineered fill should be sampled and tested by a representative of the project Geotechnical Engineer prior to being transported to the site.

6 ADDITIONAL SERVICES

6.1 DESIGN REVIEW AND CONSULTATION

It is recommended that **TECHNICON** be retained to review those portions of the contract drawings and specifications that pertain to earthwork, foundations, and pavements prior to finalization to determine whether they are consistent with our recommendations.

6.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that a representative of **TECHNICON** observe the excavation, earthwork, foundation, and pavement phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design. **TECHNICON** can conduct the necessary field testing and provide results on a timely basis so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of the observations, field testing, and conclusions regarding the conformance of the completed work to the intent of the plans and specifications will be provided. This additional service is not part of this current contractual agreement. **TECHNICON** firm will not be responsible for establishing or confirming building or foundations depths or locations unless retained to do so.

7 LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of our field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of the variations between borings may not become evident until construction. If variations or undesirable conditions are encountered during construction, our firm should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. The unexpected conditions frequently require additional expenditures for proper construction of the project. **TECHNICON Engineering Services, Inc.** will not assume any responsibility for errors or omissions if the final extent and depth of earthwork is not determined by our firm at the time of construction due to said variations or undesirable conditions encountered.

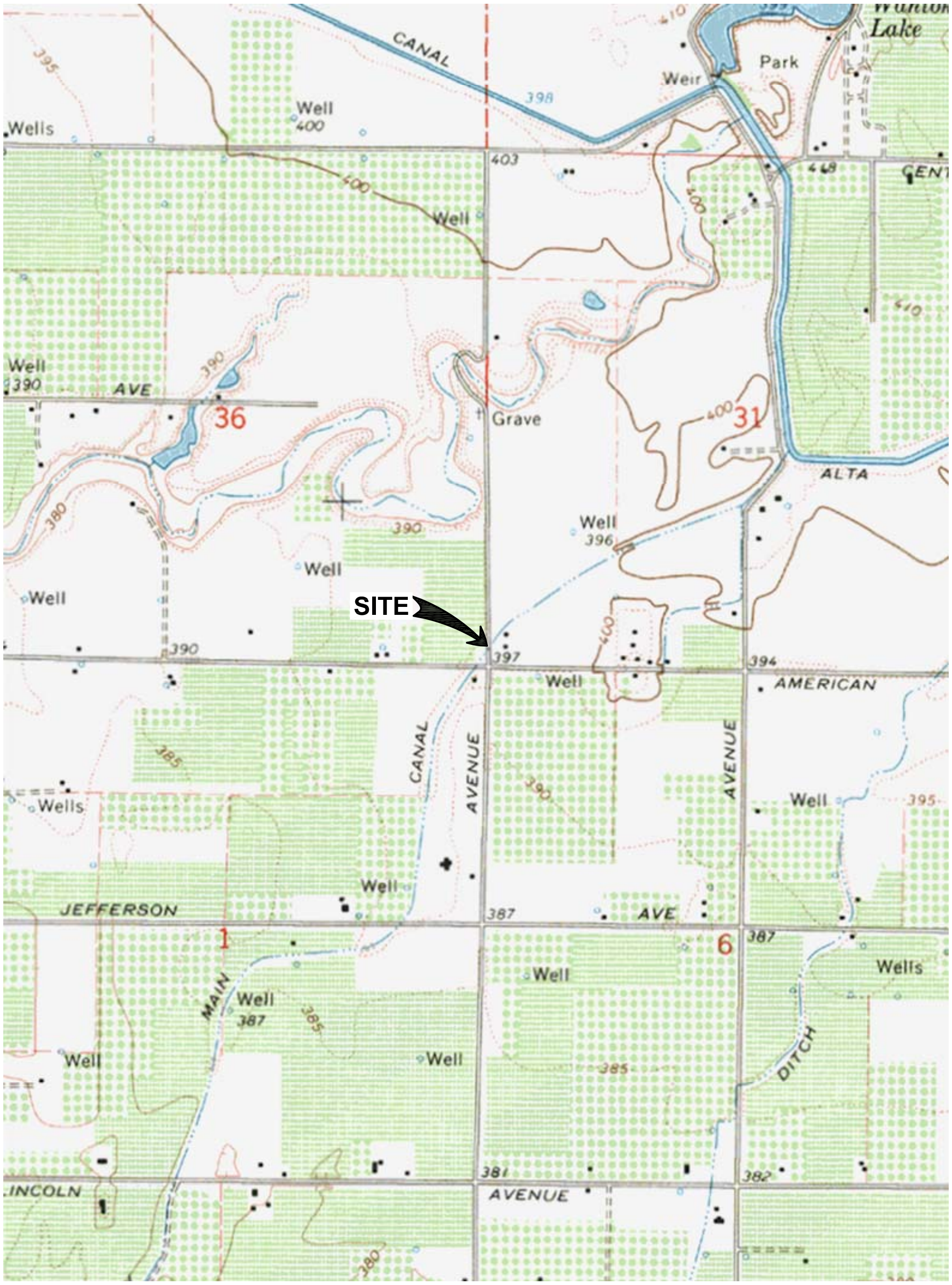
If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes, or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing. Such conditions may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. This report does not relieve the contractors of responsibility for temporary excavation construction, bracing and shoring in accordance with CAL OSHA requirements.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. This report should not be construed as an environmental audit or study.

This report has been prepared for the sole use by Cornerstone Structural Engineering Group and their designated consultants for the Englehart Avenue Bridge Replacement at the Reedley Main Canal near Reedley, in Fresno County, California. Recommendations presented herein should not be extrapolated to other areas or used for other projects without prior review. This report has been prepared with the intent that the firm of **TECHNICON** will be performing the construction testing and observation for the complete project. If, however, another firm or individual(s) should be retained or employed to use this Foundation Report for the purpose of construction testing and observation, notice is hereby given that **TECHNICON** will not assume any responsibility for errors or omissions, if any, which may occur and which could have been avoided, corrected, or mitigated if **TECHNICON**, had performed the work. This notice also applies to the misuse or misinterpretation of the conclusions and recommendations outlined in this report. Furthermore, the other firm or individual(s) performing construction testing and observation should accept transfer of responsibility of the work, as required by the California Building Code, in writing to the project owner and **TECHNICON**. The firm accepting transfer of responsibility should perform additional investigation(s) as may be necessary to develop their own conclusions, evaluations, and recommendations for design and construction.

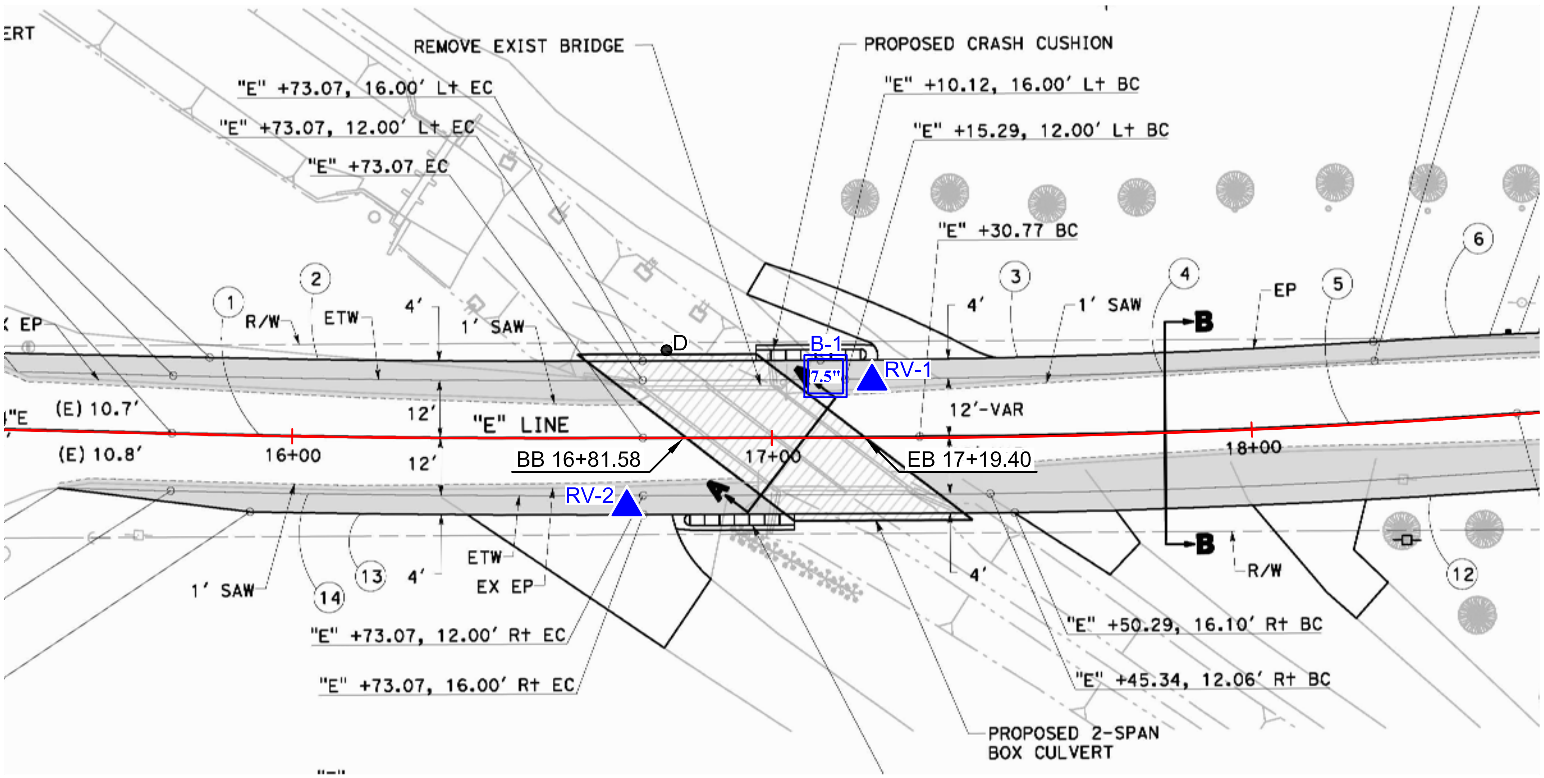
FIGURE 1 & 2



NORTH

LAT.: 36.66253°N, LONG.: 119.41258°W, 31-T14S-R24E & 36-T14S-R23E, MDB&M, USGS MAP: WAHTOKE, DATE: 1966

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 160599	VICINITY MAP ENGLEHART AVENUE BRIDGE REPLACEMENT AT REEDLEY MAIN CANAL COUNTY OF FRESNO, CALIFORNIA	FIGURE 1
	SOURCE: USGS TOPOGRAPHIC MAPS		NTS



B-1 7.5" =SOIL BORING LOCATION

RV-2 =R-VALUE LOCATIONS

D =DYNAMIC CONE PENETRATION TEST

NORTH

SCALE: 1"=40'

0' 20' 40'

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 160599	DATE: 8/30/16	SITE MAP ENGLEHART AVENUE BRIDGE REPLACEMENT AT REEDLEY MAIN CANAL COUNTY OF FRESNO, CALIFORNIA	FIGURE 2
	SOURCE: CORNERSTONE	APPROVED BY: SA		

LOG TEST BORINGS

APPENDIX A

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL 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
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY)
GP-GC	Poorly-graded GRAVEL with SILT Poorly-graded GRAVEL with SILT and 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
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL
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)
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)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic/Symbol	Group Names
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
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
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT 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
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
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
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
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
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
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

FIELD AND LABORATORY TESTING

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 5333)
CP	Compaction Curve (CTM 216)
CR	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content-% (ASTM D 2974)
P	Permeability (CTM 220)
PA	Particle Size Analysis (ASTM D 422)
PI	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
R	R-Value (CTM 301)
SA	Sieve Analysis
SE	Sand Equivalent (CTM 217)
SL	Shrinkage Limit (ASTM D 427)
SW	Swell Potential (ASTM D 4546)
UC	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 4767)

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

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 with finger pressure.

BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A R RW RC P	Auger Boring (hollow or solid stem bucket) Rotary drilled boring (conventional) Rotary drilled with self-casing wire-line Rotary core with continuously-sampled, self-casing wire-line Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hard driven(1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778)
	O	Other (note on LOTB)

Note: Size in inches

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 in.)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Greater than 50

MOISTURE

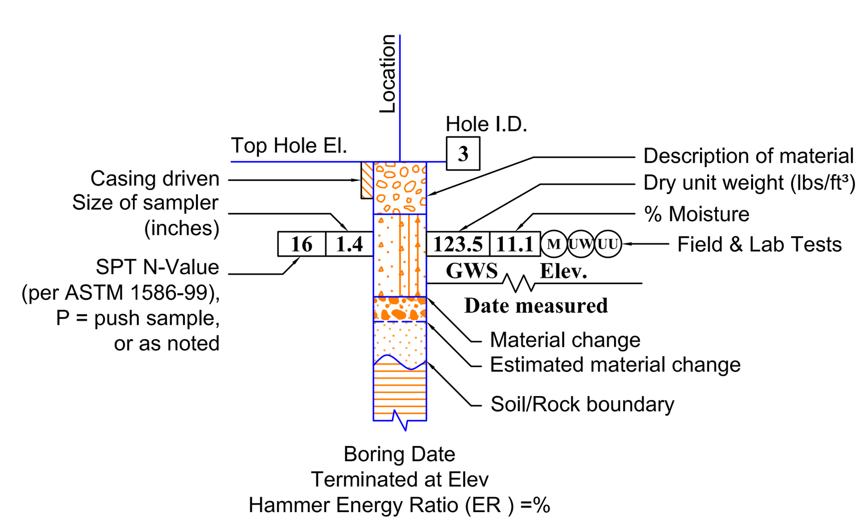
Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

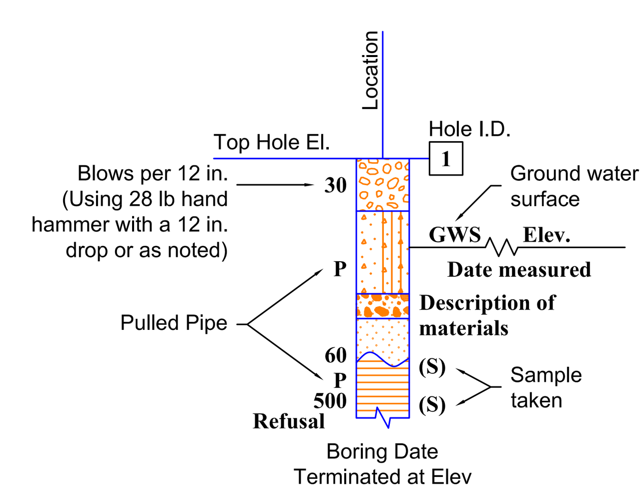
Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5% - 10%
Little	15% - 25%
Some	30% - 45%
Mostly	50% - 100%

PARTICLE SIZE

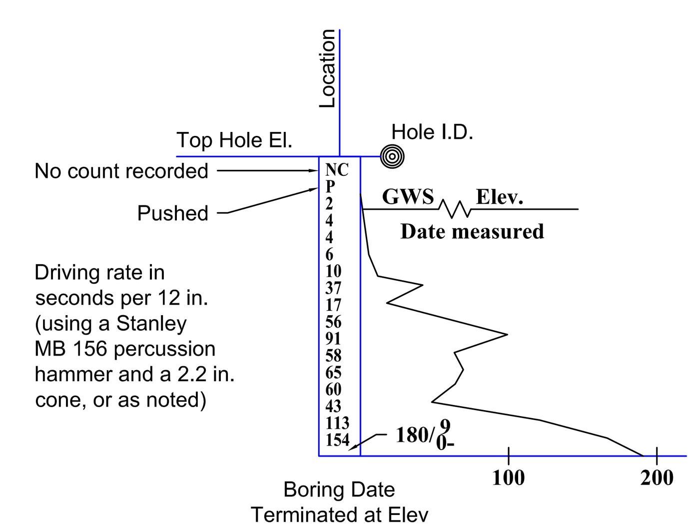
Description		Size (in.)
Boulder		Greater than 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		Less than 1/300



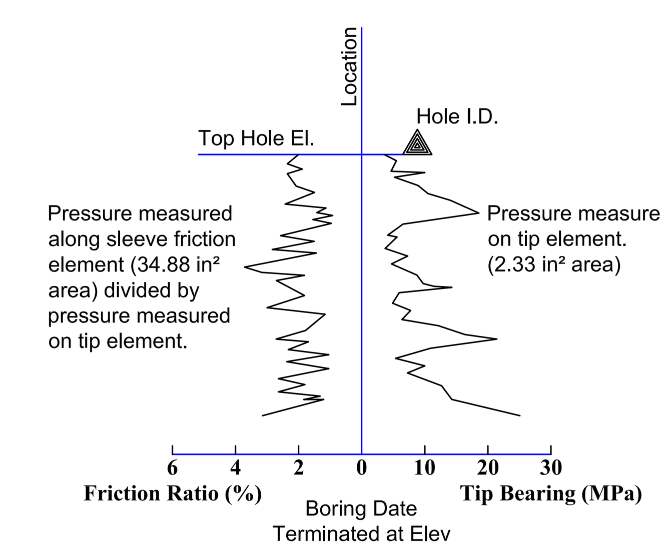
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) BORING

DESIGNED S. Athwal	DATE 7/15/16		
DRAWN M. Heraz	8/30/16		
CHECKED S. Plauson			
REVISION		FOR R/W DATA AND ACCURATE ACCESS DETERMINATION SEE R/W RECORDS AT PUBLIC WORKS	

TECHNICON
ENGINEERING SERVICES, INC.
4539 N. BRAWLEY AVENUE - SUITE 108
FRESNO, CALIFORNIA 93722

PROJECT
ENGLEHART AVENUE BRIDGE REPLACEMENT AT REEDLEY MAIN CANAL
COUNTY OF FRESNO, CA

Road No. Bridge No.

THE COUNTY OF FRESNO 1856

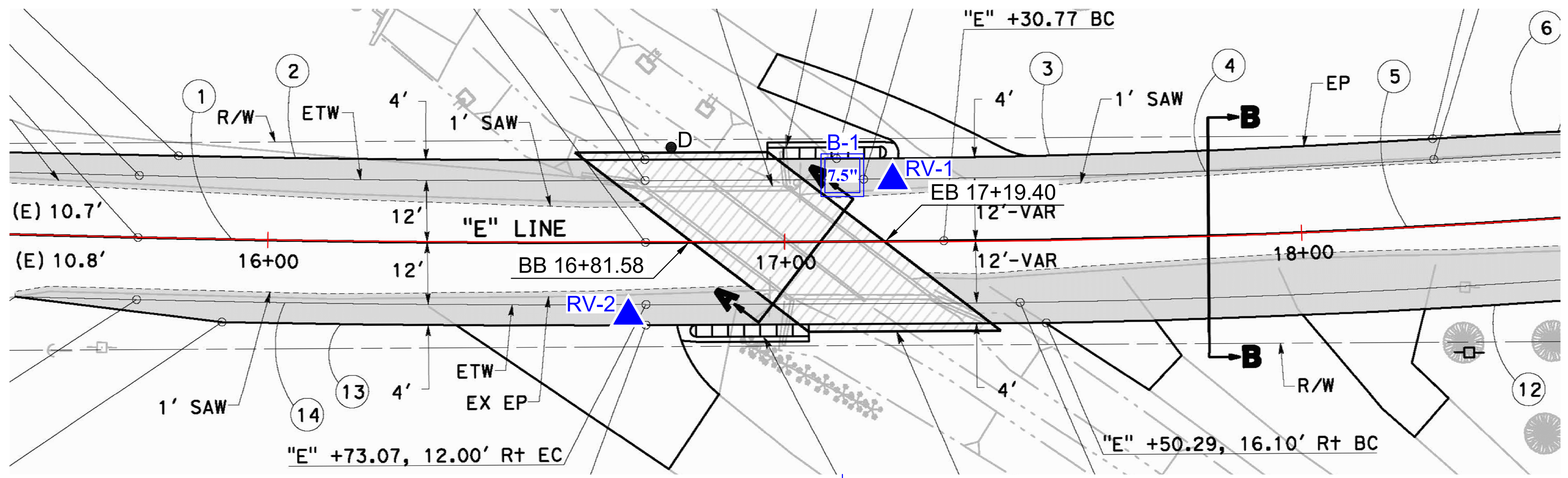
DEPARTMENT OF PUBLIC WORKS & PLANNING

LOG OF TEST BORINGS

Drawing No. 160599 Sheet No. 1 Total 2

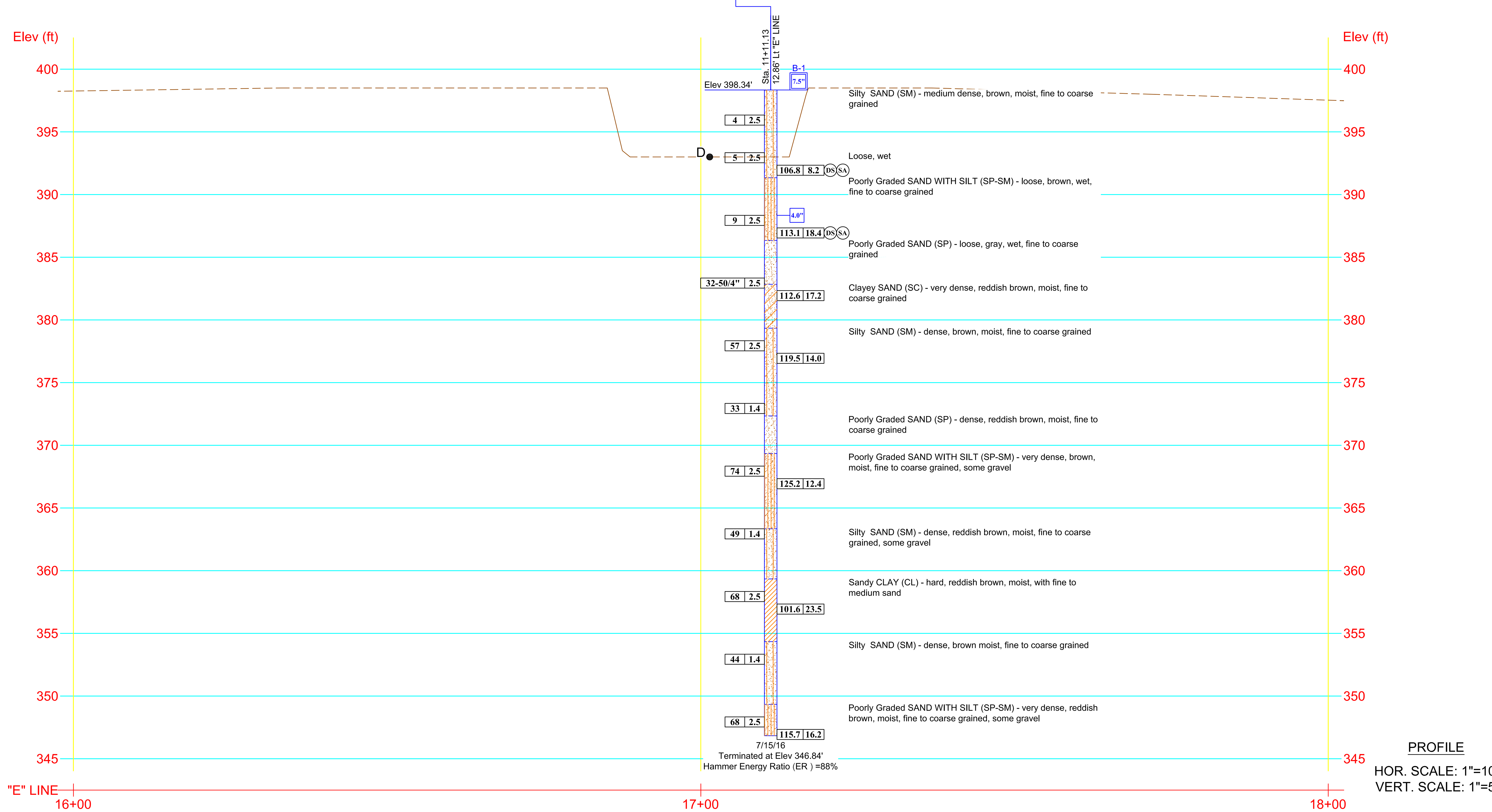
NOTES:

1. Purge water from the canal ranging at Elev 394.34' to Elev 383.34'.
2. Hammer type - CME Automatic 140 pound with 30-inch drop for all samples.
3. All dimensions are in feet unless otherwise noted.
4. This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Preparation Manual (June 2010).



PLAN

SCALE: 1"=20'



PROFILE

HOR. SCALE: 1"=10'

VERT. SCALE: 1"=5'

DESIGNED	S. Athwal	7/15/16
DRAWN	M. Heraz	8/30/16
CHECKED	S. Plauson	
REVISION		

DATE

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

Scale in Feet: 0 20 40

TECHNICON ENGINEERING SERVICES, INC.
4539 N. BRAWLEY AVENUE - SUITE 108
FRESNO, CALIFORNIA 93722

PROJECT

ENGLEHART AVENUE BRIDGE REPLACEMENT AT REEDLEY MAIN CANAL

COUNTY OF FRESNO, CA

Road No. Bridge No.

THE COUNTY OF FRESNO 1856

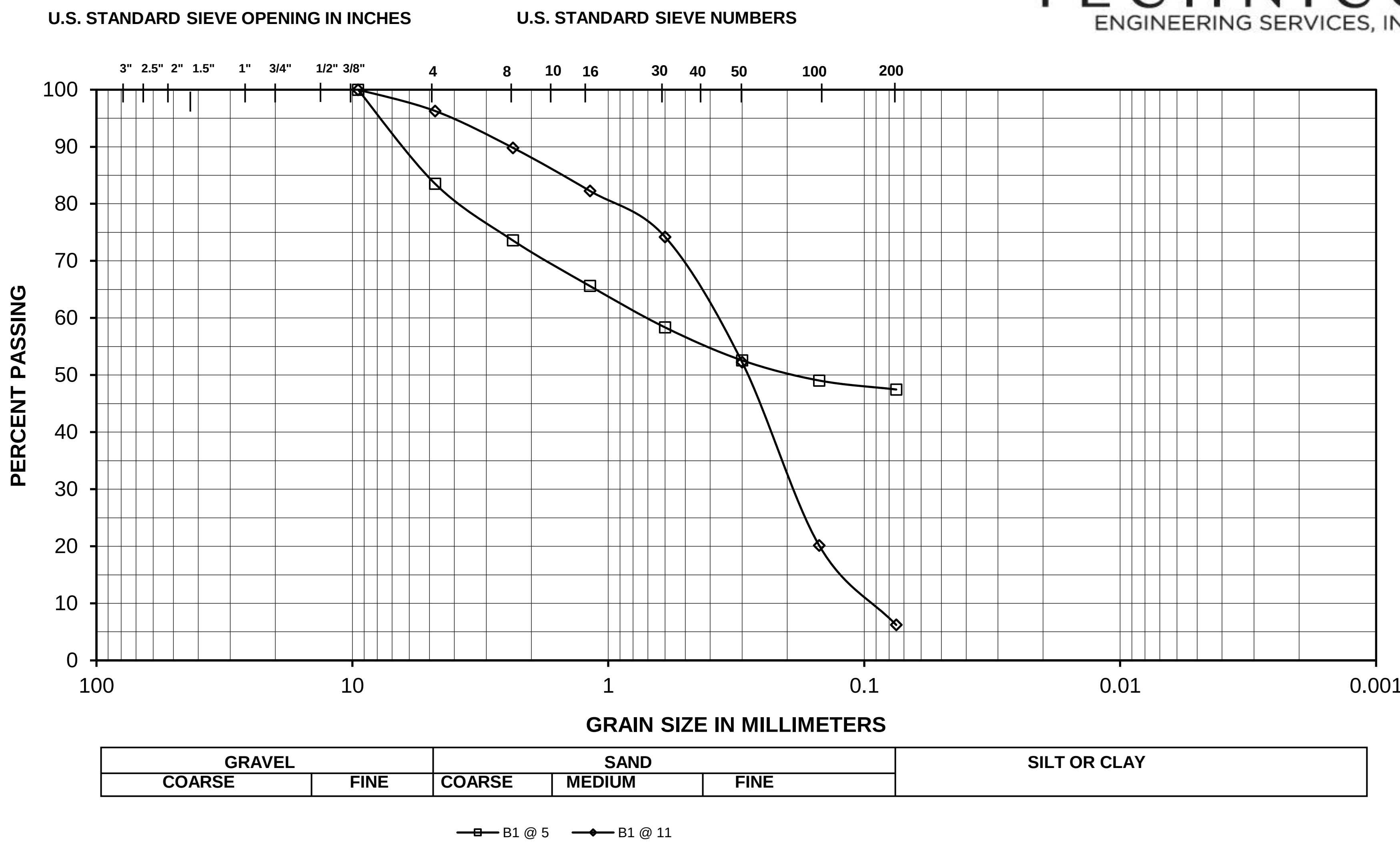
DEPARTMENT OF PUBLIC WORKS & PLANNING

LOG OF TEST BORINGS

Drawing No. 160599 Sheet No. 2 Total 2

LABORATORY TESTS

APPENDIX B



Sample No.	Classification	% Gravel	% Sand	% Fines	% Moist.	LL	PL	PI	Project	Englehart Avenue Bridge
B1 @ 5	Silty Sand (SM)	16.5	36.0	47.5	8.2					Fresno County, CA
B1 @ 11	Poorly Graded Sand w/ silt (SP-SM)	3.7	90.1	6.2	18.4				TES No.	160599
									Date	8/3/2016

TECHNICON
ENGINEERING SERVICES, INC.

Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Englehart Avenue Bridge	Technician	K.W.
	Fresno County, CA	Date	8/3/2016
TES No.	160599	Sample No.	B1 @ 5
Lab No.		Remarks	Silty Sand (SM)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	184.9	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash	99.47	1 1/2"	33.0 (15.0)
		2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	30.5	16.5	16.5	83.5	
#8	48.8	9.9	26.4	73.6	
#16	63.6	8.0	34.4	65.6	
#30	77.0	7.3	41.7	58.3	
#50	87.7	5.8	47.4	52.6	
#100	94.2	3.5	51.0	49.0	
#200	97.1	1.6	52.5	47.5	
Pan	99.12				

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TECHNICON
ENGINEERING SERVICES, INC.

Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Englehart Avenue Bridge	Technician	K.W.
	Fresno County, CA	Date	8/2/2016
TES No.	160599	Sample No.	B1 @ 11
Lab No.		Remarks	Poorly Graded Sand w/ silt (SP-SM)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	84.5	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash	79.8	1 1/2"	33.0 (15.0)
		2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	3.2	3.7	3.7	96.3	
#8	8.6	6.5	10.2	89.8	
#16	15.0	7.6	17.7	82.3	
#30	21.8	8.1	25.8	74.2	
#50	40.4	22.0	47.8	52.2	
#100	67.5	32.1	79.9	20.1	
#200	79.2	13.9	93.8	6.2	
Pan	79.8				

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TECHNICON
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Method for Estimating the Service Life of Steel Culverts
Caltrans California Test 643

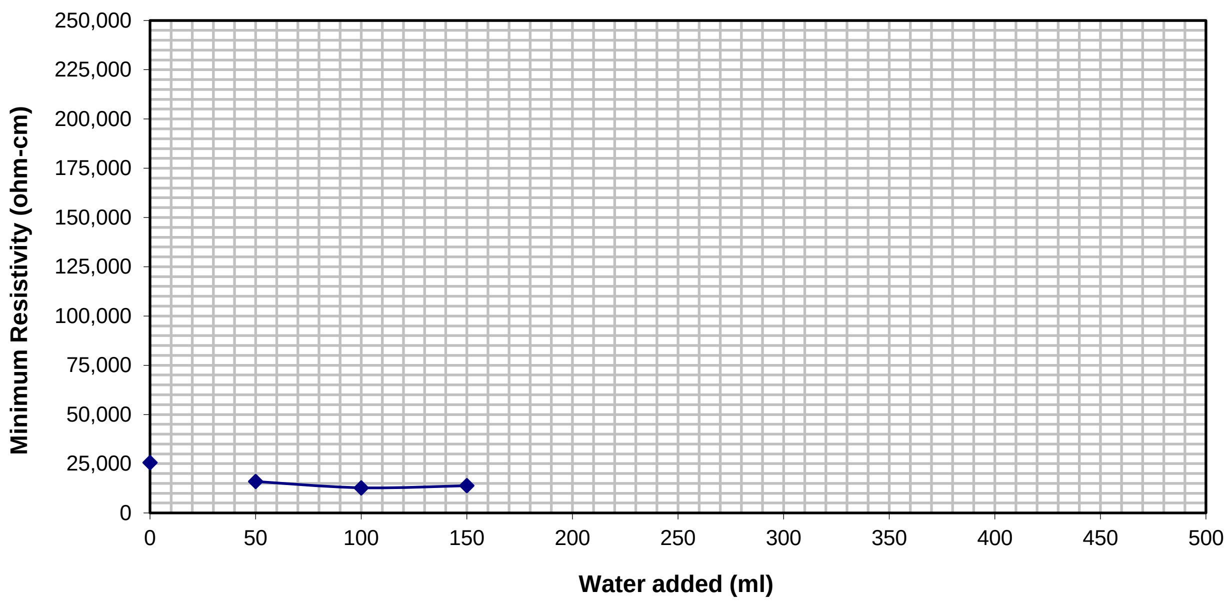
Project Name	Englehart Avenue Bridge	Sample Location	B-1 @ 0'-3'
Project Number	160599	Test Date	8/2/2016
Sample Date	7/15/2016	Tested By	K.W.
Sampled By	S. Athwal	Material Description	Silty Sand (SM)

Sample Condition	**As Received**	**Minimum Resistivity**					
Water Added (ml)	0	50	100	150			
Resistance (ohm)	24,000	15,000	12,000	13,000			
Resistivity (ohm-cm)	25,560	15,975	12,780	13,845			

Minimum Resistivity (ohm-cm)	**12,780**	**Field Resistivity (ohm-cm)**

pH = 7.38 **EC =**

Box Constant=1.065



Years to perforation*	70

* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts

Method for Estimating the Service Life of Steel Culverts
Caltrans California Test 643

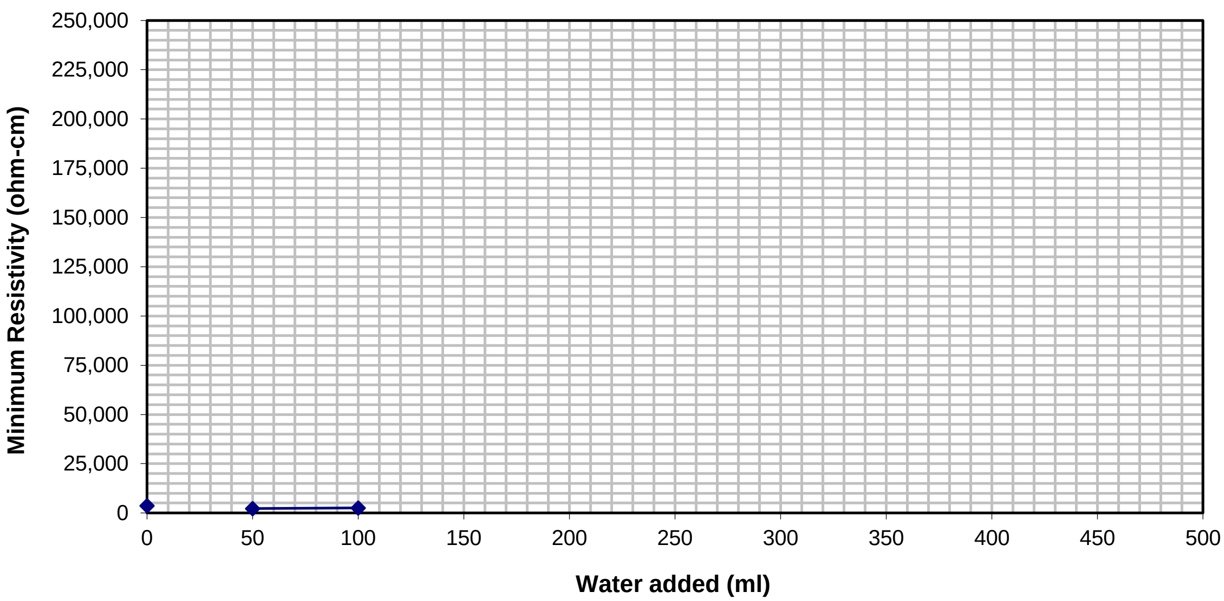
Project Name	Englehart Avenue Bridge	Sample Location	B-1 @ 16'
Project Number	160599	Test Date	8/2/2016
Sample Date	7/15/2016	Tested By	K.W.
Sampled By	S. Athwal	Material Description	Clayey Sand (SC)

Sample Condition	**As Received**	**Minimum Resistivity**					
Water Added (ml)	0	50	100				
Resistance (ohm)	3,400	2,100	2,400				
Resistivity (ohm-cm)	3,621	2,237	2,556				

Minimum Resistivity (ohm-cm)	**2,237**	**Field Resistivity (ohm-cm)**

pH = 7.82 **EC =**

Box Constant=1.065



Years to perforation*	34

* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts

TECHNICON ENGINEERING SERVICES, INC.

Chemical Analysis

SO_4 - Modified Caltrans 417 & CL - Modified Caltrans 417/422

Project	Englehart Avenue Bridge	Technician	K. W
	Fresno County, CA	Date	7/22/2016
TES No.	160599	Remarks	Silty Sand (SM)

Sample Location	Soluble Sulfate SO_4-S		Soluble Chloride Cl	
B-1 @ 0'-3'	0.4	mg/Kg	1.8	mg/Kg
B-1 @ 0'-3'	0.9	mg/Kg	1.8	mg/Kg
B-1 @ 0'-3'	0.4	mg/Kg	1.8	mg/Kg
Average	**5.00**	**mg/Kg**	**5.00**	**mg/Kg**

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TECHNICON
ENGINEERING SERVICES, INC.

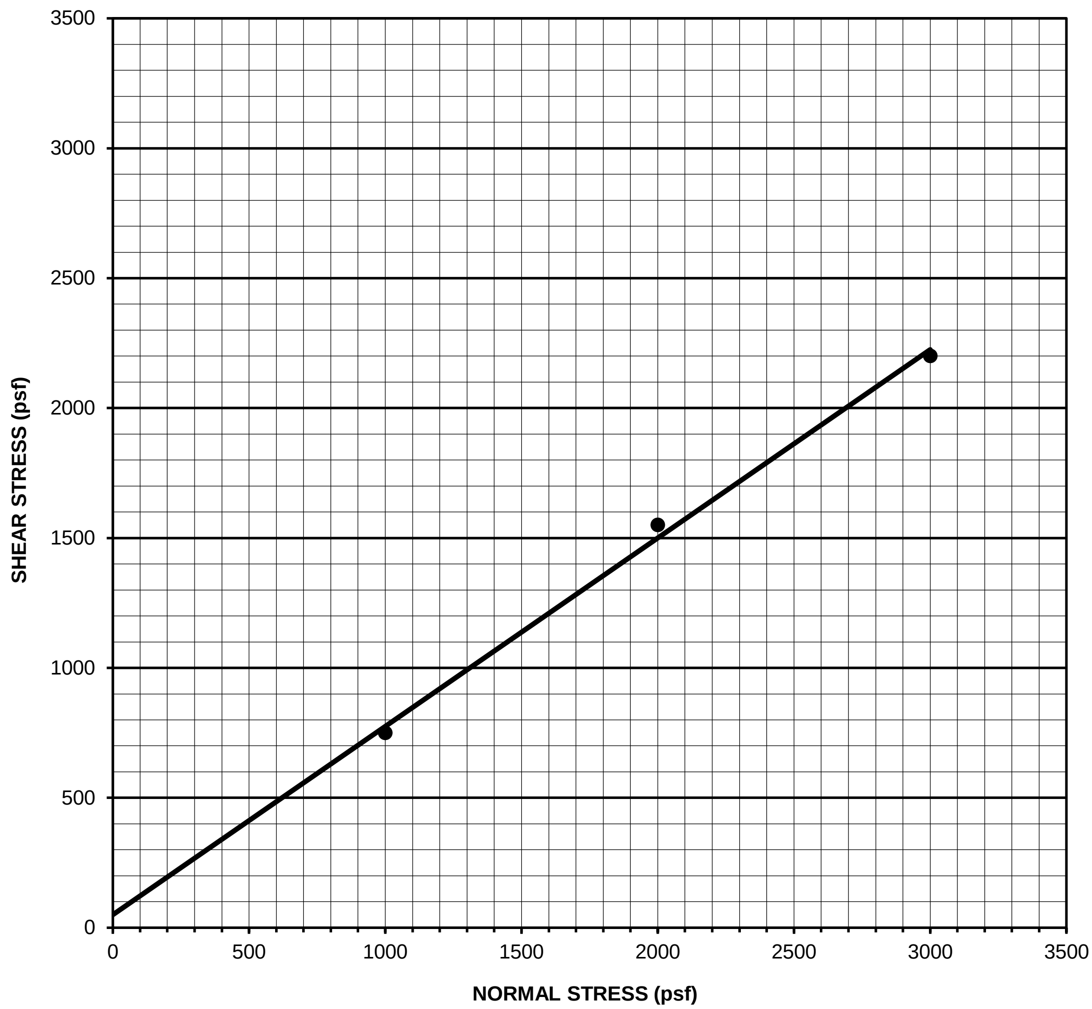
Chemical Analysis
SO_4 - Modified Caltrans 417 & CL - Modified Caltrans 417/422

Project	Englehart Avenue Bridge	Technician	K. W
	Fresno County, CA	Date	8/5/2016
TES No.	160599	Remarks	Clayey Sand (SC)

Sample Location		**Soluble Sulfate SO_4-S**		**Soluble Chloride Cl**	
B-1 @ 16'		1.8	mg/Kg	1.8	mg/Kg
B-1 @ 16'		1.5	mg/Kg	1.8	mg/Kg
B-1 @ 16'		1.9	mg/Kg	1.8	mg/Kg
	Average	**5.00**	**mg/Kg**	**5.00**	**mg/Kg**

Direct Shear Test

ASTM D3080



Project	Englehart Avenue Bridge
TES No.	160599
Sample Date	7/15/2016
Sample No.	B-1 @ 5'
Description	Silty SAND (SM)

Cohesion (psf)	**50**
Internal Friction Angle (ϕ)	**36**

Specimen	A	B	C	D	E
Dry Density (pcf)	106.8	106.8	106.8	---	---
Initial Water Content (%)	8.2	8.2	8.2	---	---
Final Water Content (%)	17.0	15.7	20.7	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	750	1550	2200	---	---

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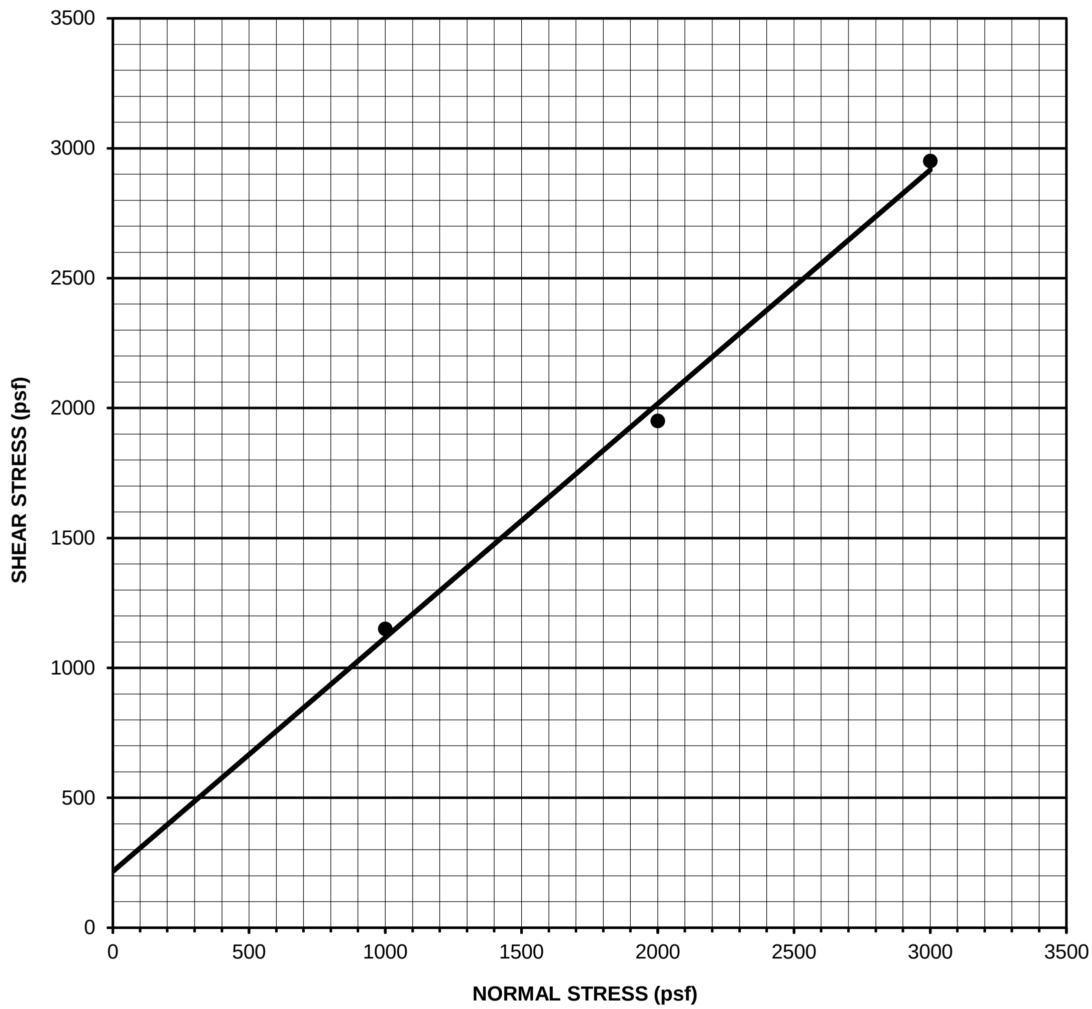
Direct Shear Test

ASTM D3080



Project	Englehart Avenue Bridge
TES No.	160599
Sample Date	7/15/2016
Sample No.	B-1 @ 11'
Description	Poorly Graded Sand /w Silt (SP-SM)

Cohesion (psf)	**220**
Internal Friction Angle (ϕ)	**42**

Specimen	A	B	C	D	E
Dry Density (pcf)	113.1	113.1	113.1	---	---
Initial Water Content (%)	18.4	18.4	18.4	---	---
Final Water Content (%)	20.2	22.5	22.4	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	1150	1950	2950	---	---

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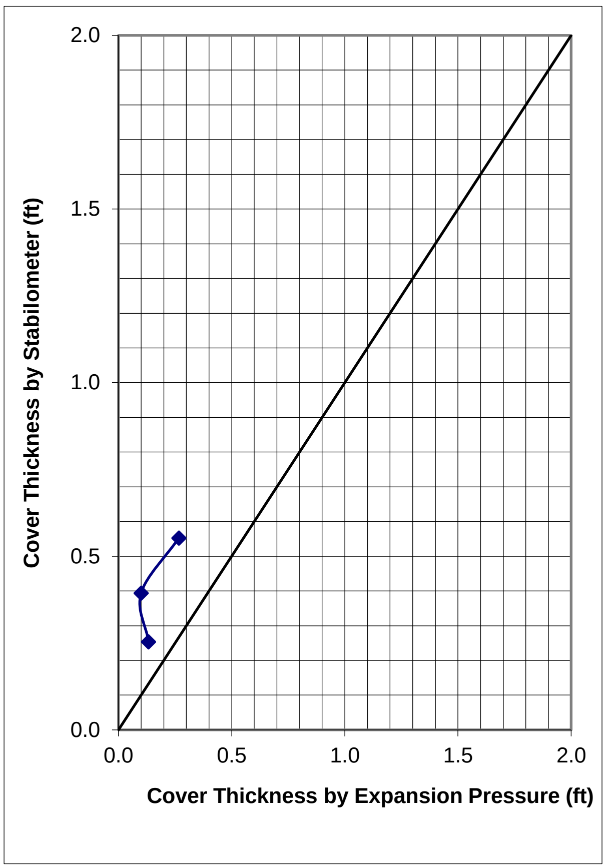
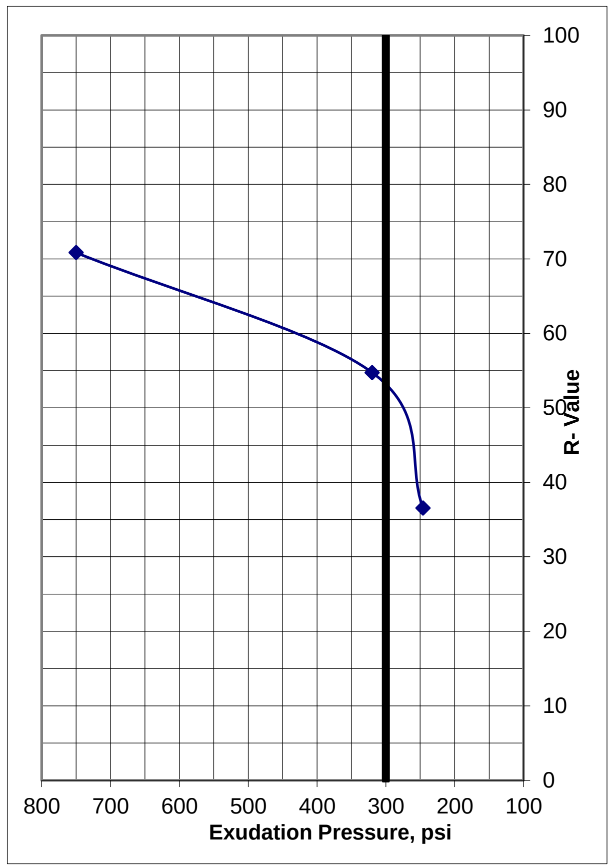
ENGINEERING SERVICES, INC.

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Englehart Avenue Bridge	Lab ID Number	16-353
Project Number	160599	Sample Location	RV-1 @ 0'-2'
Sample Date	7/15/16	Tested By	J.A.
Sampled By	S. Athwal	Date Tested	7/28/2016
Material Description	Silty Sand (SM)		





Specimen	1	2	3
Exudation Pressure, psi	246	320	750
Moisture at Test, %	11.1	10.1	8.9
Dry Density, pcf	122.3	124.0	124.0
Expansion Pressure, psf	35	13	17
Thickness by Stabilometer, ft.	0.6	0.4	0.3
Thickness by Expansion Pressure, ft.	0.3	0.1	0.1
R-Value by Stabilometer	37	55	71
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	53		

Controlling R-Value	**53**

Engineering Materials Laboratory

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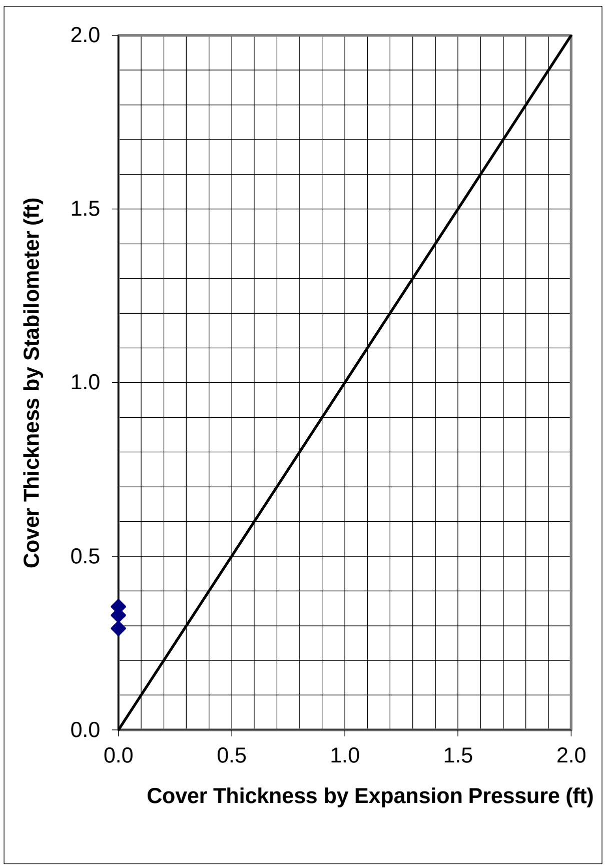
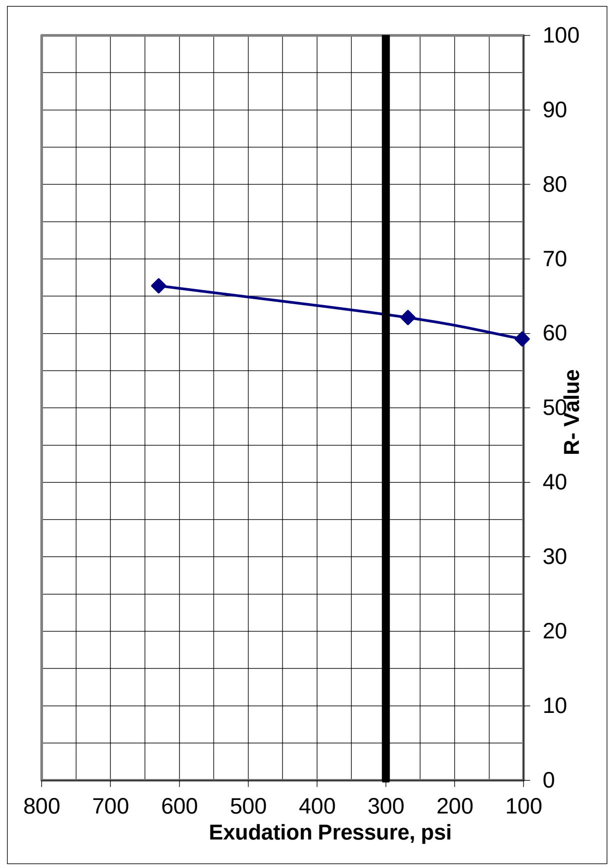
ENGINEERING SERVICES, INC.

Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Englehart Avenue Bridge	Lab ID Number	16.353
Project Number	160599	Sample Location	RV-2 @ 0'-1.5'
Sample Date	7/15/16	Tested By	J.A.
Sampled By	S. Athwal	Date Tested	7/27/2016
Material Description	Silty Sand (SM)		





Specimen	1	2	3
Exudation Pressure, psi	102	268	630
Moisture at Test, %	11.2	10.7	10.3
Dry Density, pcf	115.8	115.0	117.3
Expansion Pressure, psf	0	0	0
Thickness by Stabilometer, ft.	0.4	0.3	0.3
Thickness by Expansion Pressure, ft.	0.0	0.0	0.0
R-Value by Stabilometer	59	62	66
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	62		

Controlling R-Value	**62**

Engineering Materials Laboratory
4539 N. Brawley #108, Fresno, CA, 93722
WWW.TECHNICON.NET

DYNAMIC CONE PENETRATION TEST

APPENDIX C

TECHNICON
ENGINEERING SERVICES, INC.

Project Name:	Englehart Avenue Bridge	Date:	8/2/2016
Project #	160599	Hammer Weight:	15 lbs
Location:	Fresno County, CA	Field Engineer:	Sarbjit Athwal

Depth (in)	Depth (ft)	No. of Blows
1.75	0.15	3
3.5	0.29	5
5.25	0.44	5
7	0.58	10
8.75	0.73	17
10.5	0.88	22
12.25	1.02	24
14	1.17	25
15.75	1.31	29

**Note: Depth Measured from the Bottom of the Canal

DEASIGN ARS CURVE AND

SEISMIC ANALYSIS

APPENDIX D

Project: **Englehart Avenue Bridge**
Location: **Fresno County**
TES #: **160599**

TECHNICON ENGINEERING SERVICES, INC.

Site Information:

Latitude:	36.66253
Longitude:	-119.41258
V_{s30} (m/s)	311
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Distance (km)[1] =	125

Governing Curve:

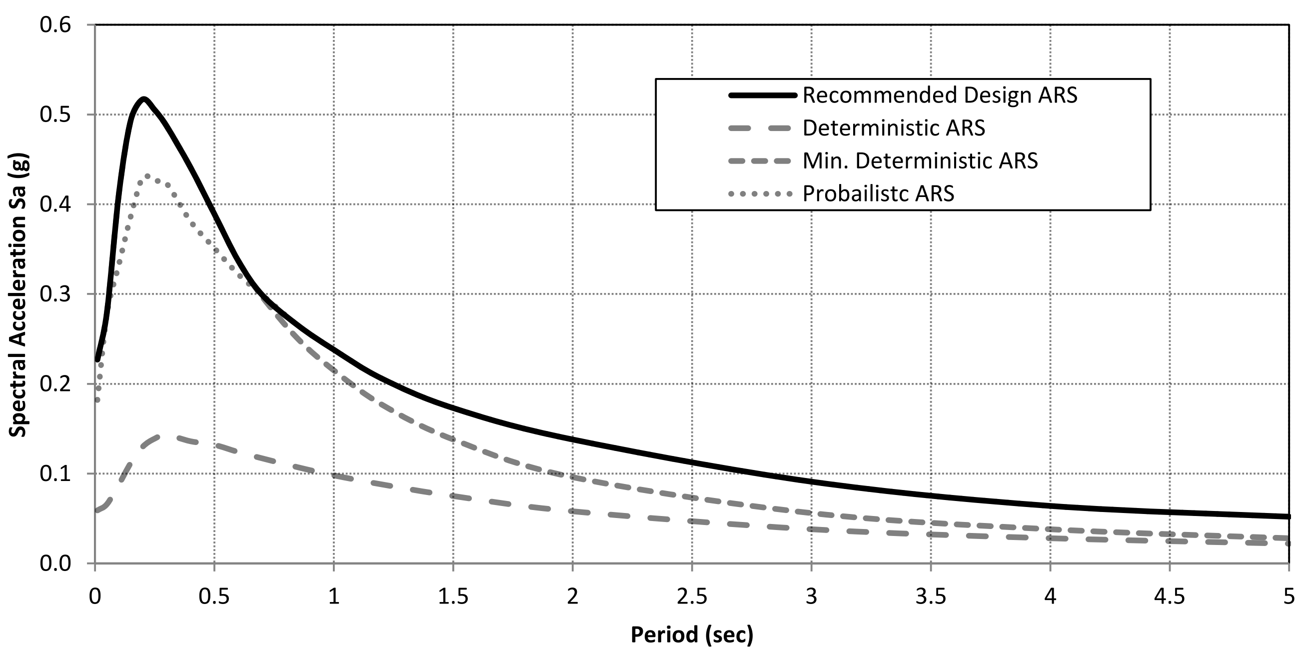
- ☐ Caltrans ARS OnLine Deterministic
- ☐ Minimum Deterministic
- ☐ Caltrans ARS OnLine Probabilistic
- ☑ Envelope of:
  - ☐ Caltrans ARS OnLine Deterministic
  - ☑ Caltrans Minimum Deterministic
  - ☑ Caltrans ARS OnLine Probabilistic

Recommended Response Spectrum

Period (sec)	SA Base Spectrum (g)	Adjusted for Basin Effect	Adjusted for Neaf Fault Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.227	-	-	0.227
0.1	0.412	-	-	0.412
0.2	0.517	-	-	0.517
0.3	0.487	-	-	0.487
0.5	0.389	1.000	1.000	0.389
1.0	0.238	1.000	1.000	0.238
2.0	0.138	1.000	1.000	0.138
3.0	0.091	1.000	1.000	0.091
4.0	0.064	1.000	1.000	0.064
5.0	0.052	1.000	1.000	0.052

RECOMMENDED ARS CURVE

Envelope of Deterministic and Probabilistic Curves (5% Damping)



Sources:

Caltrans Seismic Design Criteria, Version 1.7, April 2013
Caltrans Geotechnical Services Design Manual, August 2009
Caltrans ARS Online tool (v2.3.07, http://dap3.dot.ca.gov/shake_stable/)
USGS 2008 Interactive Daggregations (https://geohazards.usgs.gov/deaggint/2008/index.php)

May 2, 2024
Kleinfelder Project No.: 24005477.001A

Mr. Mark Weaver
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711
Phone: (559) 320-3200
Email: mweaver@cseg.com

Subject: **Final Design Memorandum**
Englehart Ave Bridge Replacement at Reedley Main Canal
Fresno County, California

Reference: **Foundation Report, Englehart Ave Bridge Replacement at Reedley Main Canal, Reedley, Fresno County, California, TECHNICON Engineering Services, Inc., File No 160599.001, dated September 9, 2016**

Dear Mr. Weaver:

In accordance with your request, Kleinfelder completed additional engineering analysis and prepared this final design memorandum to support the PS&E for the reinforced concrete box culvert (RCB) replacement on Englehart Avenue at the Reedley Main Canal in Fresno County, California. The memorandum serves to supplement the above referenced Foundation Report (FR) for the 100% submittal of the PS&E and construction phases of the project. In addition, the letter serves to maintain continuity of the Geotechnical Engineer of Record through the PS&E phase.

PROJECT UNDERSTANDING

An understanding of the project is based on telephone conversations and email correspondence with Regina Barton and Mark Weaver of Cornerstone Structural Engineering Group (CSEG) and Mr. Joseph Harrel of the County of Fresno. The above referenced Foundation Report (FR) was previously prepared to support the design of a bridge replacement located on Englehart Avenue at Reedley Main Canal. The replacement bridge is anticipated to consist of a reinforced concrete box culvert (RCB) with a closed bottom and utilizing retaining walls at the approaches.

Tables 1 through 3 present foundation design data and foundation design loads provided by CSEG and used for this geotechnical evaluation. Referenced elevations are based on elevations provided in General Layout and Foundation Plan Sheets, 100% Submittal, dated November 10, 2017.

Table 1
Box Culvert Foundation Data

Road Finished Grade Elev. (ft)	Bottom of Foundation Elev. (ft)	Foundation Size[1]		S_p[2]
		B	L	
399.1	391.46	58	22.1	1"

[1] B is measure perpendicular to the road and L is measured parallel to the road.
[2] Permissible settlement under service load

Table 2
Box Culvert Foundation Load Data

Maximum Service (Total) Bearing Pressure (ksf)	Maximum Service (Permanent) Bearing Pressure (ksf)	Maximum Strength Bearing Pressure (ksf)	Maximum Extreme Bearing Pressure (ksf)
1.11	0.502	1.80	0.502

Table 3
Retaining Wall Foundation Data

Design Height (ft)	Bottom of Footing Elev. (ft)	Min. Footing Embed. Depth (ft)	Effective Foundation Width, B' (ft)[1]		S_p[2]	Maximum Service (Total) Bearing Pressure (ksf)
			Strength 1A Limit State	Strength 1B Limit State		
4.88	392.3	2.03	2.48	2.82	1"	1.4
8.88	388.3	2.83	3.36	4.08	1"	1.4

[1] B is measure perpendicular to the wall.
[2] Permissible settlement under service load

PURPOSE AND SCOPE OF SERVICES

The purpose of this final design memorandum is to update the previous signed Foundation Report and address the following supplemental items:

- Perform a site visit to observe current site conditions.
- A summary of the updated project information and design details including loading information.
- Recommended gross and net permissible contract stress associated with tolerable settlements and bearing capacity and design footing elevations of spread footing foundation for the closed bottom area of the RCB.
- Recommended gross and net permissible contract stress associated with tolerable settlements and bearing capacity for retaining walls.
- Recommendations to stabilize soft or yielding subgrade soils with options for recompaction, replacement with aggregate base, and use of geotextile reinforcement.

SITE VISIT

Kleinfelder observed the site conditions on May 8th, 2023, at the Englehart Avenue and Reedley Main Canal crossing. The site conditions remained essentially unchanged from the previous field exploration completed on July 15, 2016. Englehart Avenue is a 2-lane asphalt paved road with unpaved shoulders and aligned north-south. The canal was unlined and flowed with a water depth of approximately 4 to 5 feet.

CONCLUSIONS AND SUPPLEMENTAL RECOMMENDATIONS

It is Kleinfelder's opinion that the recommendations presented in the FR may be used for PS&E and construction phases of the project along with the following supplemental geotechnical data and recommendations.

Box Culvert Bearing and Settlement

The nominal bearing capacity, which is based solely on soil strength, for a box culvert is extremely high (greater than 32 ksf). Table 4 "Foundation Data Table" provides the bearing resistance and settlement based on the design loads and dimensions provided.

Table 4
Footing Data Table
(Double Box Culvert)

Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth (ft)	Total Permissible Support Settlement (inches)	Service Limit State	Strength or Construction Limit State ϕb=0.45
L	B				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
58	22.1	391.46	1	1	4.5	15.5

Based on the Gross Maximum Bearing Stress (Service) of 1.11 ksf provided by CSEG, the total settlement of the RCB is approximately 0.25-inch. Differential settlement is anticipated to be reduced to half of the total settlement across the length/width of the RCB.

Retaining Wall Bearing and Settlement

Table 5 "Foundation Data Table" provides the bearing resistance and settlement of bridge approach retaining walls based on the design loads and dimensions provided by CSEG.

Table 5
Footing Data Table
(Retaining Walls)

Design Height (ft)	Bottom of Footing Elev. (ft)	Min. Footing Embed. Depth (ft)	Strength 1A Limit State			Strength 1B Limit State		
			Eff. Found. With (ft)	Gross Bearing Stress (ksf)	Factored Bearing Resist (ksf)	Eff Found. With (ft)	Gross Bearing Stress (ksf)	Factored Bearing Resist (ksf)
4.88	392.3	2.03	2.48	10.1	5.6	2.82	10.7	5.9
8.88	388.3	2.83	3.36	13.9	7.7	4.08	15.1	8.3

The estimated settlement based on the Gross Maximum Bearing Stress (Service) provided by CSEG for the walls is approximately 0.5-inch. Differential settlement is anticipated to be reduced to half of the total settlement across the length of the walls.

Unstable Foundation Recommendations

The design bearing stress/resistance given in Tables 4 and 5 requires that the RCB and walls will be placed on unyielding native soil or approved engineered fill. Any soft, unsuitable sediment in the canal bottom should be excavated to expose firm undisturbed soil and removed from the project site. If unstable foundation conditions are encountered it will be necessary to stabilize the area prior to foundation construction. Stabilization options include the following options:

Option 1 – Solar Drying, Mixing, and Blending of Dry Material

Unstable, shallow subgrade soils may be repeatedly disced to promote evaporation/natural drying and/or blended with dryer import fill soil to a compactable moisture range and recompacted in accordance with latest Caltrans Standard Specifications.

Option 2 – Mechanical Stabilization

Should the construction area experience moderate to severe instability, the foundation areas should be stabilized by removing a portion of the unstable subgrade followed by placement of Subgrade Enhancement Geotextile (SEG_T) or bi-axial Subgrade Enhancement Geogrid (SEG_G) that complies with Section 96 of the Caltrans Standards Specifications. SEG should be placed on the smooth subgrade followed by placement of 0.67-to-1.0-foot Caltrans Class 2 aggregate base (AB) and compacting to establish initial stability. The SEG should be smooth and taught and extend a minimum of 5 feet beyond unstable areas. Adjacent panels of SEG should be lapped a minimum of 2 feet.

AB should be front loaded onto SEG, spread with the equipment working on the AB, and densified with moderate to heavy compaction equipment. The equipment should not operate directly on the SEG. Aggregate base should be compacted to a minimum 95 percent relative compaction. If 95 percent compaction cannot be achieved with the initial 0.67- to 1.0-foot-thick layer of AB, subsequent, layers of SEG and 0.67- to 1.0-foot-thick layers of AB should be placed until stability is achieved. The final layer should be compacted to a minimum 95 percent relative compaction.

LIMITATIONS

Kleinfelder will perform its services in a manner consistent with the standards of care and skill ordinarily exercised by members of the profession practicing under similar conditions in the geographic vicinity and at the time the services will be performed. No warranty or guarantee, express or implied, is intended or provided.

CLOSING

Kleinfelder appreciates the opportunity to serve as geotechnical consultants to Cornerstone Structural Engineering Group and the County of Fresno during the PS&E phase of the project. If there are any questions concerning the information presented in this letter, please contact the undersigned at your convenience.

Respectfully submitted,
KLEINFELDER, INC.

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 09/30/2025
GEOTECHNICAL
STATE OF CALIFORNIA

Anthony Aquino	Stephen P. Plauson, PE, GE
Professional	Senior Principal Geotechnical Engineer

KLEINFELDER 3649 W. Holland Ave, Ste 105, Fresno, California 93722 p | 559.486.0750 f | 559.442.5081

September 9, 2016

TES No. 160598.001
Invoice No. 11

Mr. Jonathan P. Jensen
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

Project: Lincoln Avenue Bridge Replacement at
Travers Creek
Fresno County, California

Subject: Foundation Report

Dear Mr. Jensen:

The attached Foundation Report presents the results of a geotechnical investigation for the design and construction of a reinforced concrete box culvert (RCB) planned on Lincoln Avenue at Travers Creek near Reedley, in Fresno County, California. The report describes the study, findings, conclusions, and recommendations for use in project design and construction.

TECHNICON appreciates the opportunity to provide geotechnical engineering services to Cornerstone Structural Engineering Group during the design phase of this project. We trust this information meets your current needs. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully submitted,
TECHNICON Engineering Services, Inc.

Sarbjit Athwal, EIT
Project Engineer

Stephen P. Plauson, PE, GE
Geotechnical Engineering Manager

SS:SPP:mk

TECHNICON
ENGINEERING SERVICES, INC.

FOUNDATION REPORT
LINCOLN AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK
FRESNO COUNTY, CALIFORNIA

Prepared For:

Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

September 9, 2016

TES No. 160598.001

GEOTECHNICAL & ENVIRONMENTAL ENGINEERING – CONSTRUCTION TESTING & INSPECTION

TECHNICON
ENGINEERING SERVICES, INC.
1989 ♦ 2014 25 YEARS STRONG

GEOTECHNICAL & ENVIRONMENTAL ENGINEERING — CONSTRUCTION TESTING & INSPECTION

Prepared For:

Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

FOUNDATION REPORT
LINCOLN AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK
FRESNO COUNTY, CALIFORNIA

TECHNICON PROJECT
TES NO. 160598.001

Prepared by:

Sarbjit Athwal, EIT
Project Engineer

Stephen P. Plauson, PE, GE
Geotechnical Engineering Manager

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 9/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

TECHNICON Engineering Services, Inc.
4539 North Brawley Avenue, Suite 108
Fresno, California 93722
(559) 276-9311

September 9, 2016

TABLE OF CONTENTS

Page

1 INTRODUCTION

1.1 GENERAL

This Foundation Report presents the results of a geotechnical investigation for a reinforced concrete box culvert (RCB) planned on Lincoln Avenue at Travers Creek in Fresno County, California. The purpose of the investigation was to explore and evaluate the subsurface conditions at the site and prepare a Foundation Report containing recommendations to aid in project design and construction.

The Vicinity Map, presented on Figure 1, shows the location of the project and the Site Map, Figure 2, and Log of Test Boring drawing (LOTB) show the proposed bridge replacement and the approximate boring location for this study.

1.2 PROJECT DESCRIPTION

The project involves the replacement of an existing bridge located on Lincoln Avenue at Travers Creek. The existing bridge is a two-lane, timber stringer with asphalt concrete overlay, approximately 20 feet long by 24 feet wide. The replacement bridge is anticipated to consist of a double barrel RCB with a closed bottom. To accommodate the creek and roadway widths, the RCB will be approximately 24 feet in length and 38 feet in width. Based on preliminary information provided by Cornerstone Structural Engineering Group, the RCB will have an opening height of 6 feet and cover height equal to a typical asphalt concrete pavement section (e.g. less than 1.0 foot of cover) for a total height of approximately of 9 feet. The design will incorporate a concrete bottom slab and slab extensions up and down stream. Warped wing walls will form the transition of the bottom slab and side slopes.

It is anticipated that Caltrans Standards Plans will be utilized as the basis for design of the culvert and wingwalls.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to explore the site subsurface conditions to allow for development of recommendations and opinions to aid in project design. The report includes the following: A description of the proposed project including a vicinity map showing the location of the site and a site plan showing the locations of the exploration point for this study

- A description of the site surface and subsurface conditions encountered during the field investigation, including boring log
- A summary of the field exploration and laboratory testing program
- Discussion of regional and local geology including faults, seismicity, and liquefaction potential and associated effects
- Caltrans seismic design parameters
- Comments on the use of Caltrans Standard Plans for design of the box culvert and associated wingwalls
- Recommended Gross Nominal Bearing and Permissible Net Contact Stress for the box culvert foundation and anticipated settlement
- Recommended lateral earth pressures for design of the box culvert and wingwalls
- Comments on the corrosion potential of on-site soil
- Recommended pavement structural section for the design traffic index.
- Comments on site preparation and earthwork, including the use of on-site soils for engineered fill and recommended import fill specifications

The scope of services consisted of a field exploration program, laboratory testing, design analysis, and preparation of this written report as outlined in **TECHNICON'**s proposal dated April 13, 2016 (TES No. GP16-095A).

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

The field exploration, conducted on July 20, 2016 consisted of drilling one (1) exploratory test boring and site reconnaissance by a project engineer. The test boring was drilled with a CME 55 truck-mounted drill rig using hollow stem augers. The boring extended to a depth of 51.5 feet below the existing ground surface (bgs). The approximate location of the test boring is indicated on the Site Map, Figure 2, and the Log of Test Boring Drawing (LOTB), Sheet 2. In addition, a Dynamic Cone Penetration (DCP) Test was performed in the center of the canal to assess the depth of historic scour.

The soils encountered in the boring 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 as the number of blows per foot over the last 12 inches of sampler penetration on the LOTB. The blow counts listed in the LOTB have not been corrected for the effects of overburden pressure, rod length, boring diameter, sampler size, or hammer efficiency.

2.2 FIELD AND LABORATORY TESTING

Penetration rates, determined in general accordance with ASTM D-1586, were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

Laboratory tests were performed on selected near surface samples to evaluate their physical characteristics. The following laboratory tests were used to develop the design geotechnical parameters:

- ❑ Unit weight (ASTM D2937)
- ❑ Moisture content (ASTM D2216)
- ❑ Sieve Analysis (ASTM D422)
- ❑ Plasticity Index (ASTM D4318)
- ❑ Direct Shear (ASTM D3080)

- ❑ Soluble Sulfate, and Soluble Chloride Contents (California Test Method No's. 417 and 422)
- ❑ pH and Minimum Resistivity (California Test Method No. 643)
- ❑ Resistance Value (California Test Method No. 301)

The dry density and moisture content test results are shown on the LOTB in Appendix A. The soluble sulfate, soluble chloride, pH and minimum resistivity are discussed in the "Corrosion Potential" Section (Section 5.6). The remaining test results are provided in Appendix B.

3 SITE GEOLOGY AND CONDITIONS

3.1 SURFACE CONDITIONS

The subject bridge replacement is at the Lincoln Avenue and Travers Creek crossing. Lincoln Avenue is a 2 lane asphalt paved road with unpaved shoulders and aligned east-west. Travers Creek was unlined and at the time of the field investigation the creek was flowing with a water depth of approximately 3 to 4 feet. The slopes of Travers Creek were approximately 2:1 horizontal to vertical (H:V), with the creek crossing Lincoln Avenue in north to south direction. The bridge location is generally bounded by single family residence homes to the northwest and northeast and open fields to the southwest and southeast.

3.2 SUBSURFACE CONDITIONS

The natural site soil consists of nonmarine deposits with a geologic age of Pleistocene. The general earth material profile depicted by the subsurface exploration consisted primarily of clayey sand in the upper 12 feet, followed by poorly graded sand with silt to 17 feet and underlain by laterally discontinuous layers of clayey sand, silty sand, and sandy silty clay to the depth explored, 51.5 feet bgs. The granular soil generally had a relative consistency of loose to very dense while the fine grained soil generally had a relative consistency of hard.

The above is a general description of the earth material profile. A more detailed representation of the stratigraphy at the specific exploration location is provided on the LOTB included in Appendix A.

3.3 GROUNDWATER CONDITIONS

Groundwater was not encountered during the field exploration but at the time of the field investigation the creek supported water flow and could influence the localized groundwater. The State of California Department of Water Resources, “Lines of Equal Elevation of Water in Wells”, Spring 2011 indicates the regional depth to groundwater exceeds 50 feet. Additional research utilizing the California Department of Water Resources (DWR) website indicates the nearest monitored well to be approximately 1/8 of a mile to the east (Well No. 15S24E08A001M). Based on the groundwater elevation data collected at this well, the historic high groundwater depth was recorded at 13 feet bgs in the early 1980’s and the current recorded groundwater depth is below 50 feet bgs.

The groundwater elevation at the bridge site is likely is more likely influenced by flow or recency of flow within Travers Creek and could affect construction. Depending on the flow or recency of flow in Travers Creek at the time of construction, earthwork and construction may be impacted by soft/yielding subgrade and/or saturated conditions. It is assumed that construction may occur during the winter months shortly after closure of the creek. Therefore, it should be anticipated that the creek bottom and sides of the canal could be saturated and may not provide a stable bottom for construction activities.

4 SEISMIC RECOMMENDATIONS

4.1 SEISMIC SOURCES

The project site and its vicinity are located in an area traditionally characterized by relatively low seismic activity. The site is not located in an Alquist-Priolo Earthquake Fault Zone as established by the Alquist-Priolo Fault Zoning Act (Section 2622 of Chapter 7.5, Division 2 of the California Public Resources Code).

Review of the Caltrans Deterministic PGA Map (September 2007), indicates there are no existing major fault systems within 25 miles of the project vicinity. Based on review of published data and current understanding of the geologic framework and tectonic setting of the proposed improvements, the primary sources of seismic shaking at this site are listed in Table 4.1-1. A major seismic event on these or other nearby faults may cause ground shaking at the site. Based on the deterministic ground acceleration, the San Andreas Fault is considered the governing fault.

TABLE 4.1-1
LOCAL FAULTS AND ESTIMATED MOMENT MAGNITUDES

Fault	Approximate Distance from Site (km)	Maximum Credible Earthquake (Moment Magnitude, M_w)	Peak Ground Acceleration (g)
San Andreas Fault	129	8.0	0.091
Independence	97	7.1	0.085
Round Valley	96	7.0	0.081
Coast Ranges Sierran Block	89	6.5	0.067

4.2 SEISMIC DESIGN CRITERIA

Development of a site specific Acceleration Response Spectra (ARS) curve was undertaken in accordance Caltrans Geotechnical Design Manual (Ver. 2.3.07, March 2016) and the Caltrans Seismic Design Criteria (Ver. 1.7, November 2013).

The Wahtoke dated 1966, California 7½-minute Quadrangle Topographic Map indicates the proposed Lincoln Avenue Bridge Replacement lies on the south center of Section 5 and north center of Section 8, T15S, R24E. Furthermore, the average shear wave velocity for the upper 30m (100 feet) of the subsurface soil and rock at the bridge site was estimated by using established correlations and procedures presented in the Caltrans Geotechnical Design Manual. The estimated shear wave velocity is provided below.

Site Location: Latitude: 36.64729° N / Longitude: -119.38478° W

Shear Wave Velocity: Vs(30) = 344 m/s

ARS curves for the bridge site were determined based on the Caltrans Deterministic PGA Map (September 2007), Caltrans ARS Online (Ver. 2.3.07), the shear wave velocity of the soil, and the latitude/longitude at the bridge location. A Site Specific ARS curve was developed for the project and is included in Appendix D for use in the seismic analysis of the bridge. The recommended Design ARS curve consists of the envelope of the Caltrans Minimum Deterministic ARS and Caltrans Online Probabilistic ARS. The results of the 2008 USGS Deaggregation Tool (Beta) do not govern, since the shear wave velocity exceeds 300 m/s.

4.3 SEISMIC HAZARDS

Review of the Caltrans Deterministic PGA Map (September 2007) indicates that no mapped active faults cross or project toward the site. Additionally, no evidence of active faulting was visible on the site during our site reconnaissance. Therefore, it is our opinion that the potential for fault-related surface rupture at the proposed bridge site is very low. Furthermore, the Caltrans Deterministic PGA Map (September 2007) indicates the site is located relatively far from active faults, as such, the possibility for the site to experience strong ground shaking may be considered low.

4.4 SEISMICALLY INDUCED GROUND FAILURE

4.4.1 Design Ground Motion

For the purpose of evaluating liquefaction, a probabilistic seismic hazards analysis (PSHA) procedure was performed using the 2008 USGS Deaggregation Tool (Beta) to estimate the earthquake magnitude. The program allows user input of the project site coordinates and produces the expected peak ground motions for the site for selected probability of exceedance (e.g. return periods). The USGS Deaggregation Tool, based on a probability of exceedance of 2 percent in 50 years, determined a weighted magnitude of Mw = 6.08. The peak ground acceleration was assessed using ARS Online and found to be 0.226g.

4.4.2 Liquefaction

In order for liquefaction, and possible associated effects, 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 fine, granular, and uniform,
- ❑ Ground shaking of sufficient intensity should occur 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).

Saturated granular sediments can experience liquefaction if subject to seismically induced ground motion of sufficient intensity and duration. Based on the ground shaking which may be expected at this site, the relative density and geologic age of the sediments, analysis utilizing Youd (2001) indicates liquefaction, seismically induced settlement, or bearing loss is considered unlikely.

4.4.3 Dynamic Compaction

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Considering that problematic soils were not identified in the borings drilled for this study, seismically induced dry sand settlement is anticipated to be minimal. Calculations indicate that seismically induced dry sand settlement is negligible.

5 DESIGN RECOMMENDATIONS

5.1 GENERAL

Based on the laboratory data, field exploration, and geotechnical analyses conducted for this study, it is geotechnically feasible to construct the proposed RCB as currently envisioned. Provided that the recommendations presented in this report are incorporated into the project design and construction, use of a closed bottom RCB with bottom mat/slab bearing on recompacted native soil or approved engineered fill prepared in accordance with Caltrans Standard Specifications, Section 19 are considered appropriate for structure support. Recommendations regarding the geotechnical aspects of design are presented in subsequent sections.

5.2 SCOUR EVALUATION

TECHNICON performed a gradational analysis of the sediments within the test boring at the elevation of the Travers Creek bottom to aid in the hydraulic evaluation of the channel scour by others.

To evaluate the creek bottom for scour, **TECHNICON** performed Dynamic Cone Penetration (DCP) Test to determine the historic scour depth. The DCP test was performed by dropping a 15-lb slide hammer from a height of 20 inches driving a 1.5 inch cone pointed rod. Observations and hand exploration indicates the Travers Creek channel has undergone localized scour within isolated areas of the existing bridge. It is estimated that the scour depth has extended to a depths of approximately 18 to 24 inches below the current creek bottom elevation. A summary of the DCP Test results can be seen in Appendix C.

5.3 STABILITY OF SLOPES

Slope stability using dimensionless parameters by Janbu for permanent and temporary slopes was calculated for a canal and temporary slope height of 8 feet. It was determined that permanent slopes configured at 1½:1 H:V should be stable with regard to gross (deep seated) and surficial slope failure modes (factor of safety greater than 1.5, respectively). Temporary slopes configured at 3/4:1 H:V should be stable with regard to gross (deep seated) failure mode (factor of safety greater than 1.25).

5.4 BOX CULVERT DESIGN

5.4.1 Bearing and Settlement

Based on the field exploration, laboratory testing, and geotechnical analyses, the soils at the site are suitable for supporting the RCB. The General Plan indicates the proposed RCB length is approximately 24 feet and the width is approximately 38 feet. The opening height of the RCB is 6 feet and the overall structure height including pavement is estimated to be 9.0 feet.

Considering the base dimensions of the RCB and the shear strength of the on-site soils, the Gross Nominal Bearing Resistance is high. Table 5.4-1 “Footing Data Table” provides the bearing resistance and settlement.

TABLE 5.4-1
FOOTING DATA TABLE

Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth (ft)	Total Permissible Support Settlement (inches)	Service Limit State	Strength or Construction Limit State ϕ_b=0.45
L	B				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
38	24	373.76	1	1	5.0	4.8

Based on the Gross Maximum Bearing Stress of 3.0 ksf provided by the structural engineer for the RCB, the total settlement of the RCB is approximately 0.6-inch. Differential settlement is anticipated to be reduced to half of the total settlement across the length/width of the RCB.

The design bearing stress/resistance given in Table 5.4-1 requires that the RCB will be placed on unyielding native soil or approved engineered fill. Any soft, unsuitable sediment in the channel bottom should be excavated to expose firm undisturbed soil and removed from project site. Based on observations and DCP testing performed in the creek bottom, for preliminary planning it should be anticipated that a general excavation depth of 18 to 24 inches may be required to remove unsuitable soil. However, isolated deeper areas deemed unsuitable could exist, which may require deeper excavation.

If unstable foundation conditions are encountered it will be necessary to stabilize the area prior to foundation construction. Stabilization options include placing a minimum of 12 inches of either a lean concrete slurry or ¾-inch diameter crushed gravel. If the crushed gravel is utilized, an engineering fabric conforming to the requirements of Section 88 of the Caltrans Standard

Specifications should be placed on the subgrade prior to rock placement to prevent migration of fines into the rock. The fabric is necessary to add reinforcement and prevent migration of subgrade soil into the open spaces of the gravel. **TECHNICON** should be contacted to observe and approve the exposed subgrade prior to stabilizing the working/foundation area.

5.4.2 Lateral Earth Pressures

Caltrans Standard Plans, May 2010, for RCB's are based on the soil surrounding the planned RCB having minimum and maximum lateral earth pressures equal to 42 lb/ft^3 and 100 lb/ft^3. In addition, the maximum cover density is to be limited to 140 lb/ft^3. Based on the analysis of the native soil, the soil will exhibit an earth cover density of approximately 131 lb/ft^3. The minimum and maximum restrained lateral earth pressures of the native soil, backfilled in accordance with Caltrans Standard Specifications, Section 19 are 52.4 lb/ft^3 and 97 lb/ft^3. Consequently, the use of Caltrans Standard Plans for design of the RCB would be appropriate. Table 5.4-2 provides active and at-rest pressures and the dynamic incremental increase of the earth pressure against retaining walls considering earthquake loading. The pressures are based on the use of on-site soils for wall backfill.

TABLE 5.4-2
LATERAL EARTH PRESSURES

Loading Condition	Lateral Earth Pressure (psf/ft of Wall Height)		Earth Pressure Coefficient
	Drained	Undrained	
Active Pressure (psf/ft of depth)	39	22 + Hydrostatic	0.27
At-Rest Pressure (psf/ft of depth)	61	34.5 + Hydrostatic	0.43
Dynamic Active Incremental Increase (psf/ft of depth)	16.5		
Dynamic At-Rest Incremental Increase (psf/ft of depth)	8.5		

The Special Provisions requires that backfill placed within a 1:1 zone extending upward from the base of the RCB consist of low expansion granular fill (Expansion Index less than 10).

Should retaining walls be influenced by surcharge loads, the surcharge against the walls can be evaluated by multiplying the surcharge pressure by the earth pressure coefficient. Surcharge loads should be modeled as a uniform pressure against the wall by multiplying the surcharge load by the earth pressure coefficient.

5.4.3 Resistance to Lateral Loading

Lateral loads applied to RCB can be resisted by a combination of passive lateral bearing and sliding resistance. The allowable and ultimate passive pressures and frictional resistance for the RCB are presented in Table 5.4-3.

TABLE 5.4-3
PASSIVE BEARING AND SLIDING RESISTANCE

	WSD		LRFD	
	Static	**Total Combined**	**Nominal**	**Strength Limit**
Frictional Coefficient (Sliding)	0.47	0.56	0.70	0.56
Passive Pressure (psf/ft of depth)	290	390	580	290
Lateral Translation Needed to Develop Passive Pressure	0.008D	0.015D	0.035D	0.008D

Note: D is the depth of the zone providing resistance.
WSD = Working Stress Design, LRFD = Load/Resistance Factor Design

5.4.4 Bottom Slab Cutoff Wall

Extensions of the culvert bottom slab are planned up and down stream of the proposed RCB. Based on the granular nature of the anticipated bottom sediments and presence of flowing water, it is recommended that a cutoff wall be constructed at the ends of the concrete channel lining. The cutoff wall could be designed in accordance with Caltrans Standard Plans and have a minimum embedment of 4 feet below the bottom of the RCB. The final embedment of the cutoff wall should be extended as dictated by the scour conditions.

5.4.5 Warped Wingwalls

Proposed warped wingwalls shall be supported on approved undisturbed native soil channel slopes or properly engineered fill as well as the bottom slab extension. The native soils have strength characteristics that result in design earth pressures compatible with Caltrans Standard Plans. Provided that the Special Provisions specify that imported backfill consist of soil similar to the native soil or soil having a ϕ angle of at least 35 degrees, Caltrans Standard Plans design could be used.

5.4.6 Construction Observations

The culvert excavation should be observed by a representative of the Geotechnical Engineer. The purpose of these observations is to check that the bearing soils exposed in the excavation are similar to those on which the recommendations are based.

5.5 PAVEMENT DESIGN

Bulk soil samples were tested at two locations for R-value for pavement design. The test results are presented in Table 5.5-1. Pavement recommendations will be provided in the "Final" Foundation Report for the design Traffic Index (TI) to be provided by Mark Thomas & Company.

TABLE 5.5-1
SUMMARY OF R-VALUE TESTS

Sample Location	Depth (ft)	Soil Type	R-Value by Exudation
RV-1	0-2	Clayey Sand (SC)	5
RV-2	0-2	Clayey Sand (SC)	25

5.6 CORROSION POTENTIAL

Two (2) soil samples obtained from the site were tested to evaluate pH, minimum electrical resistivity, and soluble sulfate and chloride content. Provided in Table 5.6-1 are the pH, minimum electrical resistivity and soluble sulfate and chloride content.

TABLE 5.6-1
CORROSION POTENTIAL

Depth (ft)	Location	Soil Type	pH	Minimum Resistivity (ohm-cm)	Soluble Sulfate (ppm)	Soluble Chloride (ppm)
0 to 3	B-1	Clayey Sand (SC)	7.23	4,526	5	9
10 to 16	B-1	Clayey Sand (SC)	7.65	3,716	5	5

These values are all outside the Caltrans threshold limits. Consequently, the site would be considered to be a non-corrosive environment with respect to foundations.

These values are generally representative of an environment that would be mildly corrosive to buried unprotected metals. An example of the potential soil corrosion is provided by utilizing methods provided in Caltrans California Test 643, "Method for Estimating the Service Life of Steel Culverts". The method indicates a 1-gauge steel zinc-coated culvert is estimated to have a maintenance-free service life (years to perforation) provided in Table 5.6-2. Therefore, if project improvements will involve metal that comes into contact with the on-site soil (e.g. steel barriers etc.), the design should consider the potential soil corrosiveness described.

TABLE 5.6-2
ESTIMATED SERVICE LIFE OF BURIED STEEL
"UTILIZING CALIFORNIA TEST METHOD 643"

Depth	Location	Maintenance-Free Service Life (Years to Perforation)
0 to 3	B-1	32
10 to 16	B-1	42

5.7 EARTHWORK

5.7.1 Grading

All grading operations should be performed in accordance with the project specifications and within the intent of applicable items of Section 19 of the Caltrans Standard Specifications, 2010. It is recommended that relative compaction be based on dry weight methodology for Caltrans 216 and 231. Where culvert and wingwall fill is place against the existing Travers Creek creek slopes, benches having horizontal dimensions of 2 vertical should be excavated to remove unsuitable/disturbed soil and expose competent subgrade.

5.7.2 Engineered Fill

All engineered fill soils should be non-expansive, relatively granular soil that is nearly free of, rubble, organics or other deleterious debris, and less than 3 inches in maximum dimension. Excavated on-site soil may be used as engineered fill, provided they meet the above criteria. Any imported soil shall meet also meet these criteria. Imported fill materials to be used for engineered fill should be sampled and tested by a representative of the project Geotechnical Engineer prior to being transported to the site.

6 ADDITIONAL SERVICES

6.1 DESIGN REVIEW AND CONSULTATION

It is recommended that **TECHNICON** be retained to review those portions of the contract drawings and specifications that pertain to earthwork, foundations, and pavements prior to finalization to determine whether they are consistent with our recommendations.

6.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that a representative of **TECHNICON** observe the excavation, earthwork, foundation, and pavement phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design. **TECHNICON** can conduct the necessary field testing and provide results on a timely basis so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of the observations, field testing, and conclusions regarding the conformance of the completed work to the intent of the plans and specifications will be provided. This additional service is not part of this current contractual agreement. **TECHNICON** firm will not be responsible for establishing or confirming building or foundations depths or locations unless retained to do so.

7 LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of our field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of the variations between borings may not become evident until construction. If variations or undesirable conditions are encountered during construction, our firm should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. The unexpected conditions frequently require additional expenditures for proper construction of the project. **TECHNICON Engineering Services, Inc.** will not assume any responsibility for errors or omissions if the final extent and depth of earthwork is not determined by our firm at the time of construction due to said variations or undesirable conditions encountered.

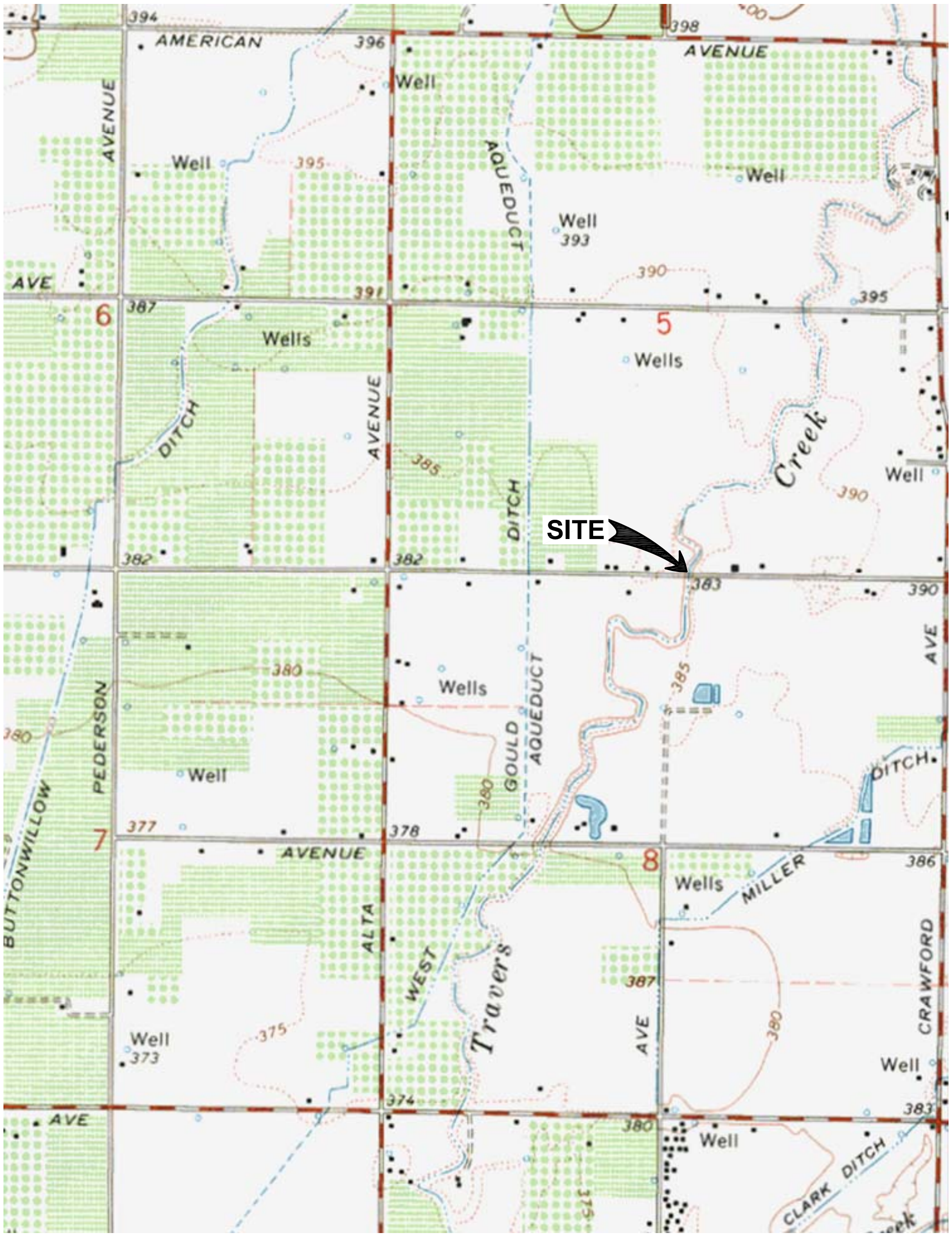
If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes, or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing. Such conditions may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. This report does not relieve the contractors of responsibility for temporary excavation construction, bracing and shoring in accordance with CAL OSHA requirements.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. This report should not be construed as an environmental audit or study.

This report has been prepared for the sole use by Cornerstone Structural Engineering Group and their designated consultants for the Lincoln Avenue Bridge Replacement at Travers Creek near Reedley, in Fresno County, California. Recommendations presented herein should not be extrapolated to other areas or used for other projects without prior review. This report has been prepared with the intent that the firm of **TECHNICON** will be performing the construction testing and observation for the complete project. If, however, another firm or individual(s) should be retained or employed to use this Foundation Report for the purpose of construction testing and observation, notice is hereby given that **TECHNICON** will not assume any responsibility for errors or omissions, if any, which may occur and which could have been avoided, corrected, or mitigated if **TECHNICON**, had performed the work. This notice also applies to the misuse or misinterpretation of the conclusions and recommendations outlined in this report. Furthermore, the other firm or individual(s) performing construction testing and observation should accept transfer of responsibility of the work, as required by the California Building Code, in writing to the project owner and **TECHNICON**. The firm accepting transfer of responsibility should perform additional investigation(s) as may be necessary to develop their own conclusions, evaluations, and recommendations for design and construction.

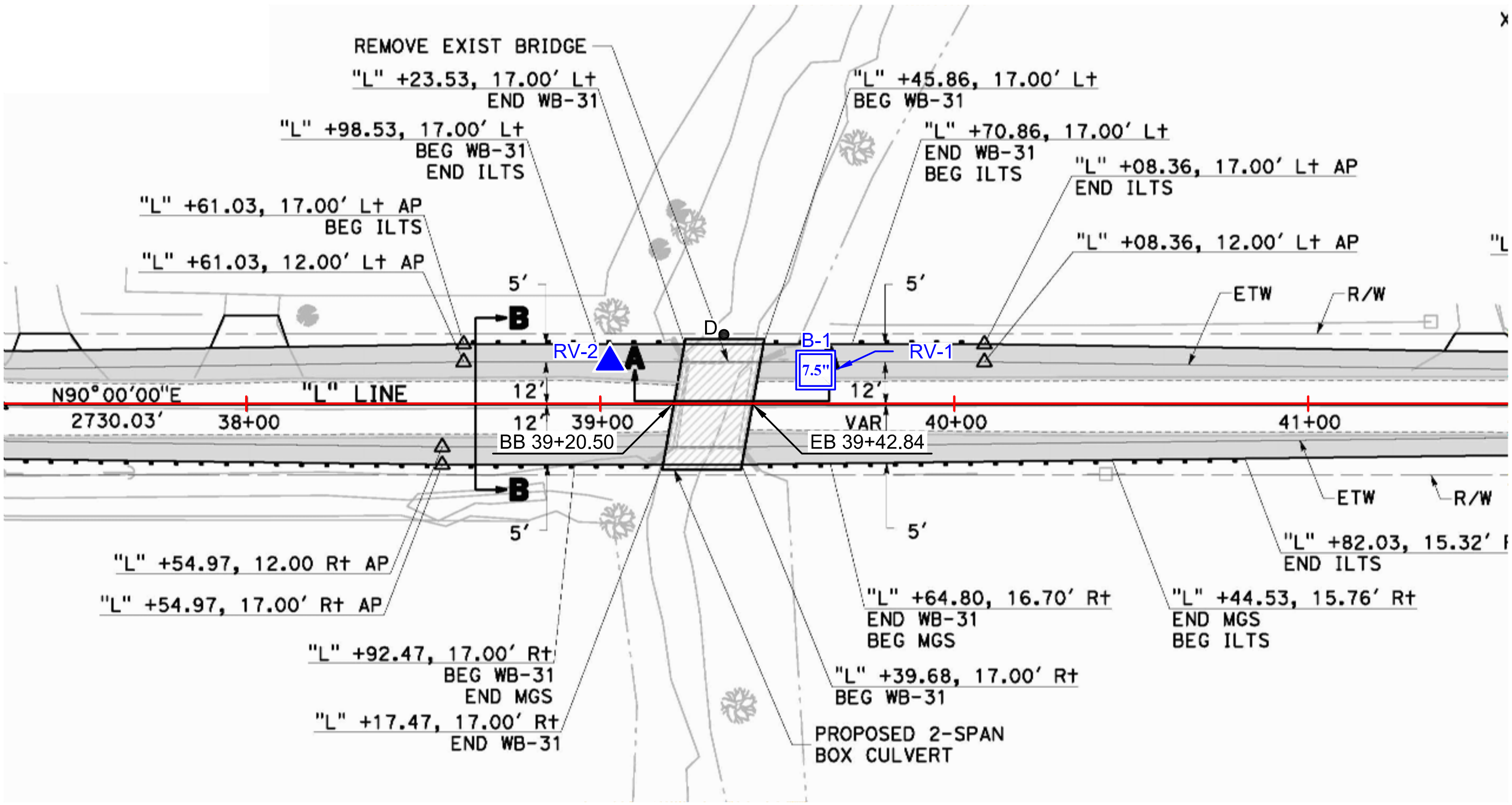
FIGURE 1 & 2



LAT.: 36.6473°N, LONG.: 119.3848°W, 5&8-T15S-R24E, MDB&M, USGS MAP: WAHTOKE, DATE: 1966

NORTH

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 160598	VICINITY MAP LINCOLN AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK COUNTY OF FRESNO, CALIFORNIA	FIGURE 1 NTS
	SOURCE: USGS TOPOGRAPHIC MAPS		



B-1 7.5" =SOIL BORING LOCATION

RV-2 =R-VALUE LOCATIONS

D =DYNAMIC CONE PENETRATION TEST

NORTH

SCALE: 1"=50'

0' 25' 50'

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 160598	DATE: 8/31/16	SITE MAP LINCOLN AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK COUNTY OF FRESNO, CALIFORNIA
	SOURCE: CORNERSTONE	APPROVED BY: SA	FIGURE 2

LOG TEST BORINGS

APPENDIX A

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL 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
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY)
GP-GC	Poorly-graded GRAVEL with SILT Poorly-graded GRAVEL with SILT and 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
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL
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)
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)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic/Symbol	Group Names
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
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
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT 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
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
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
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
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
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
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

FIELD AND LABORATORY TESTING

- C — Consolidation (ASTM D 2435)
- CL — Collapse Potential (ASTM D 5333)
- CP — Compaction Curve (CTM 216)
- CR — Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- CU — Consolidated Undrained Triaxial (ASTM D 4767)
- DS — Direct Shear (ASTM D 3080)
- EI — Expansion Index (ASTM D 4829)
- M — Moisture Content (ASTM D 2216)
- OC — Organic Content-% (ASTM D 2974)
- P — Permeability (CTM 220)
- PA — Particle Size Analysis (ASTM D 422)
- PI — Plasticity Index (AASHTO T 90)
Liquid Limit (AASHTO T 89)
- PL — Point Load Index (ASTM D 5731)
- PM — Pressure Meter
- R — R-Value (CTM 301)
- SA — Sieve Analysis
- SE — Sand Equivalent (CTM 217)
- SL — Shrinkage Limit (ASTM D 427)
- SW — Swell Potential (ASTM D 4546)
- UC — Unconfined Compression-Soil (ASTM D 2166)
Unconfined Compression-Rock (ASTM D 2938)
- UU — Unconsolidated Undrained Triaxial (ASTM D 2850)
- UW — Unit Weight (ASTM D 4767)

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

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 with finger pressure.

BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A R RW RC P	Auger Boring (hollow or solid stem bucket) Rotary drilled boring (conventional) Rotary drilled with self-casing wire-line Rotary core with continuously-sampled, self-casing wire-line Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hard driven(1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778)
	O	Other (note on LOTB)

Note: Size in inches

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 in.)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Greater than 50

MOISTURE

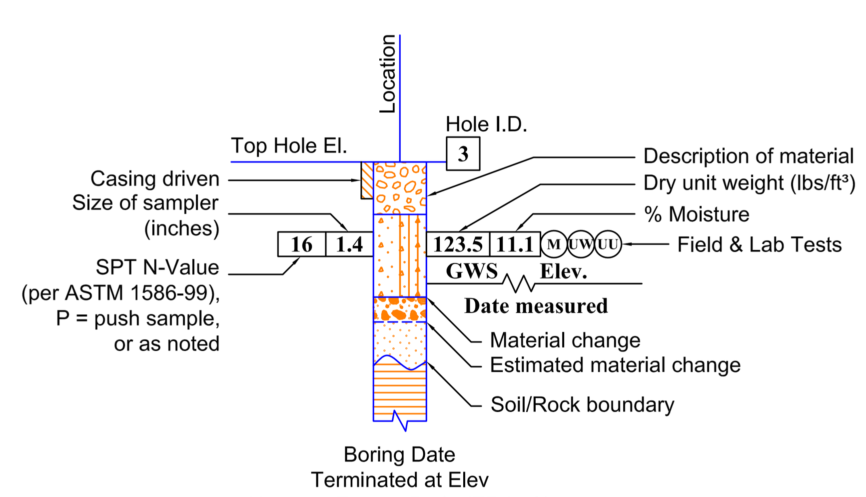
Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

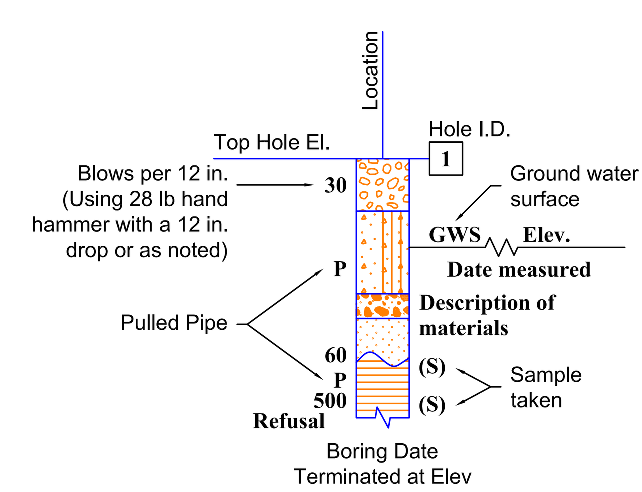
Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5% - 10%
Little	15% - 25%
Some	30% - 45%
Mostly	50% - 100%

PARTICLE SIZE

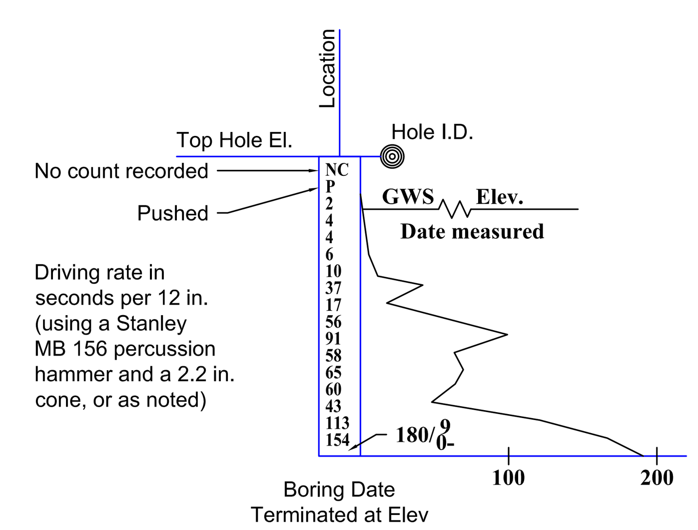
Description		Size (in.)
Boulder		Greater than 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		Less than 1/300



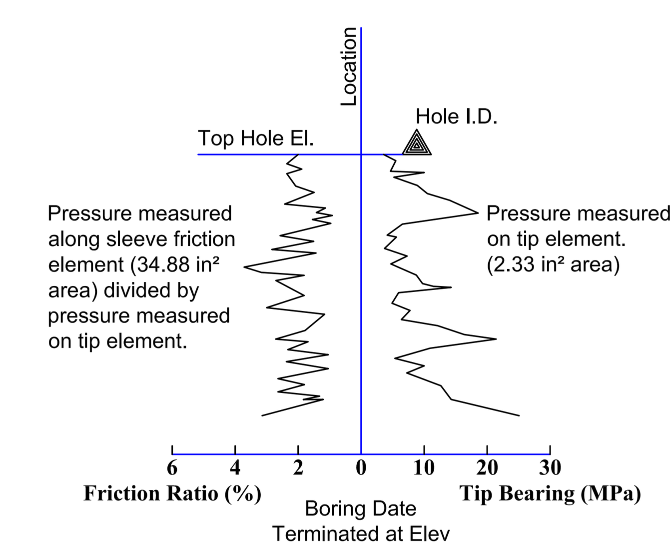
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) BORING

DESIGNED	S. Athwal	7/20/16
DRAWN	M. Heraz	8/31/16
CHECKED	S. Plauson	

DATE

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

TECHNICON ENGINEERING SERVICES, INC.
4539 N. BRAWLEY AVENUE - SUITE 108
FRESNO, CALIFORNIA 93722

PROJECT
LINCOLN AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK
COUNTY OF FRESNO, CA

Road No. Bridge No.

THE COUNTY OF FRESNO 1856

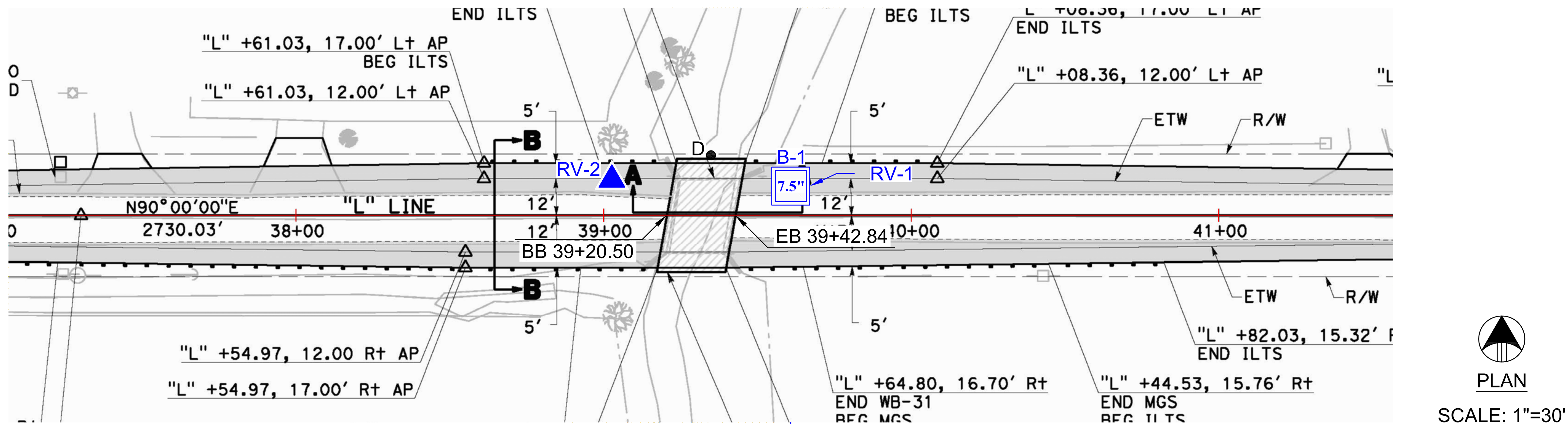
DEPARTMENT OF PUBLIC WORKS & PLANNING

LOG OF TEST BORINGS

Drawing No. 160598 Sheet No. 1 Total 2

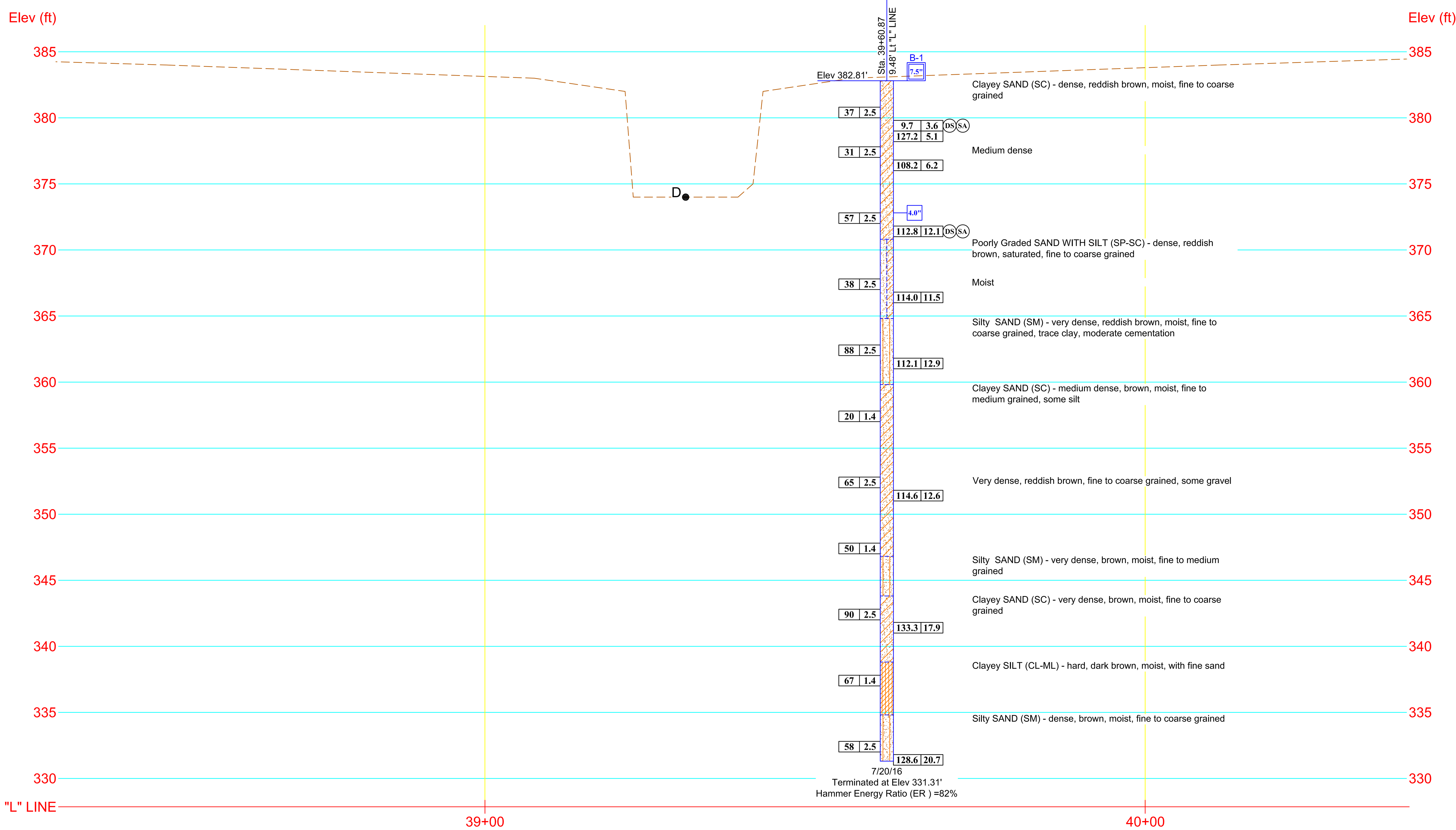
NOTES:

1. Ground water was not encountered.
2. Hammer type - CME Automatic 140 pound with 30-inch drop for all samples.
3. All dimensions are in feet unless otherwise noted.
4. This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Preparation Manual (June 2010).



PLAN

SCALE: 1"=30'



PROFILE

HOR. SCALE: 1"=10'

VERT. SCALE: 1"=5'

DESIGNED	S. Athwal 7/20/16
DRAWN	M. Heraz 8/31/16
CHECKED	S. Plauson
REVISION	FOR R/W DATA AND ACCURATE ACCESS DETERMINATION SEE R/W RECORDS AT PUBLIC WORKS

Scale in Feet 0 30 60

TECHNICON ENGINEERING SERVICES, INC.
4539 N. BRAWLEY AVENUE - SUITE 108
FRESNO, CALIFORNIA 93722

PROJECT

LINCOLN AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK
COUNTY OF FRESNO, CA

Road No. Bridge No.

THE COUNTY OF FRESNO 1856

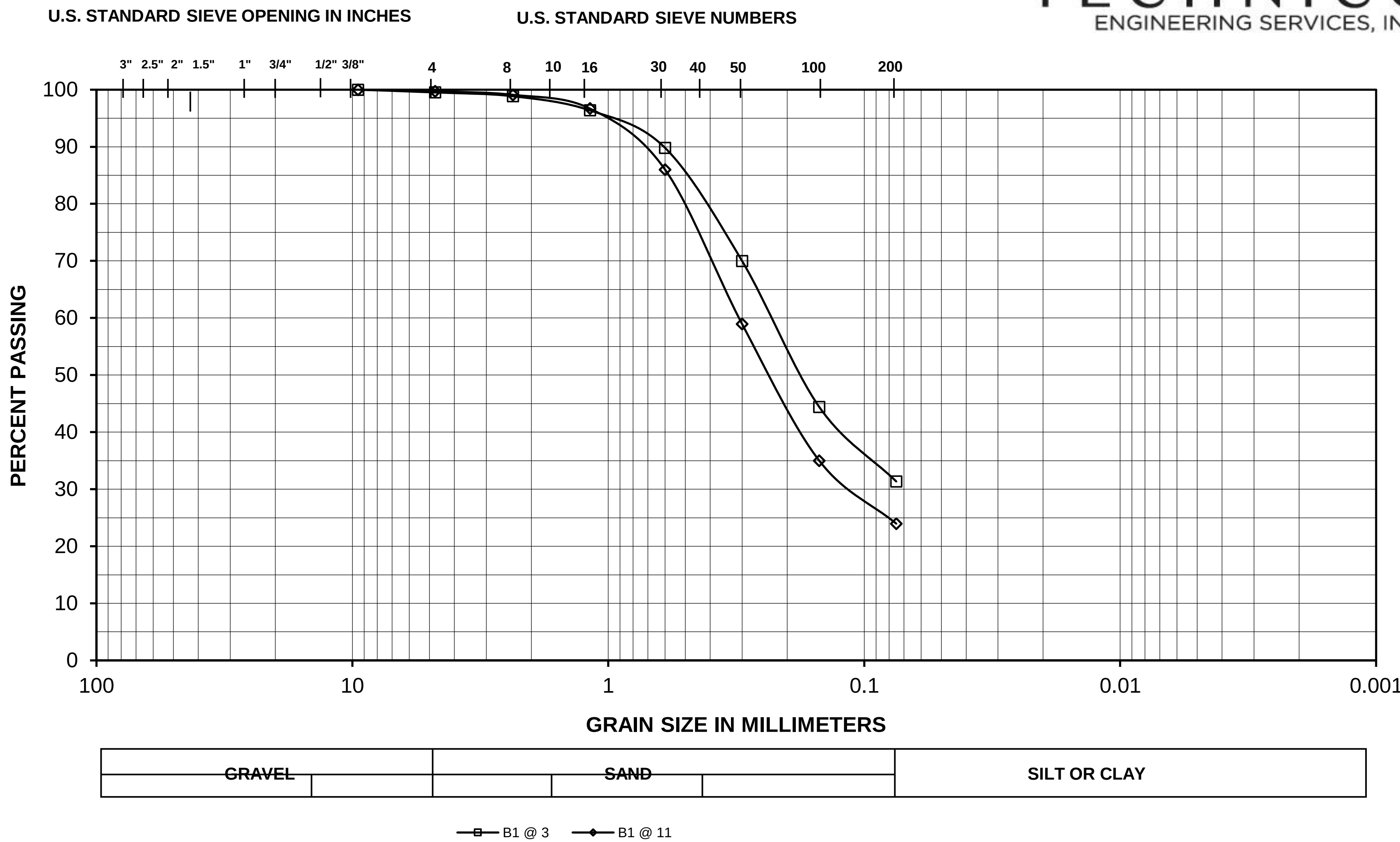
DEPARTMENT OF PUBLIC WORKS & PLANNING

LOG OF TEST BORINGS

Drawing No. 160598 Sheet No. 2 Total 2

LABORATORY TESTS

APPENDIX B



Sample No.	Classification	% Gravel	% Sand	% Fines	% Moist.	LL	PL	PI	Project	Lincoln Avenue Bridge
B1 @ 3	Clayey Sand (SC)	0.5	68.2	31.4	2.4					Fresno County, CA
B1 @ 11	Clayey Sand (SC)	0.3	75.8	24.0	12.1				TES No.	160598
									Date	8/3/2016

TECHNICON
ENGINEERING SERVICES, INC.

Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Lincoln Avenue Bridge	Technician	JW
	Fresno County, CA	Date	8/3/2016
TES No.	160598	Sample No.	B1 @ 3
Lab No.		Remarks	Clayey Sand (SC)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	104.9	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash	75.1	1 1/2"	33.0 (15.0)
		2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	0.5	0.5	0.5	99.5	
#8	1.2	0.7	1.1	98.9	
#16	3.8	2.5	3.6	96.4	
#30	10.7	6.6	10.2	89.8	
#50	31.5	19.8	30.0	70.0	
#100	58.3	25.5	55.6	44.4	
#200	72.0	13.1	68.6	31.4	
Pan	74.7				

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Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Lincoln Avenue Bridge	Technician	JW
	Fresno County, CA	Date	8/3/2016
TES No.	160598	Sample No.	B1 @ 11
Lab No.		Remarks	Clayey Sand (SC)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	178.5	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash	138.3	1 1/2"	33.0 (15.0)
		2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	0.5	0.3	0.3	99.7	
#8	1.6	0.6	0.9	99.1	
#16	5.9	2.4	3.3	96.7	
#30	25.0	10.7	14.0	86.0	
#50	73.3	27.1	41.1	58.9	
#100	116.0	23.9	65.0	35.0	
#200	135.7	11.0	76.0	24.0	
Pan	138.7				

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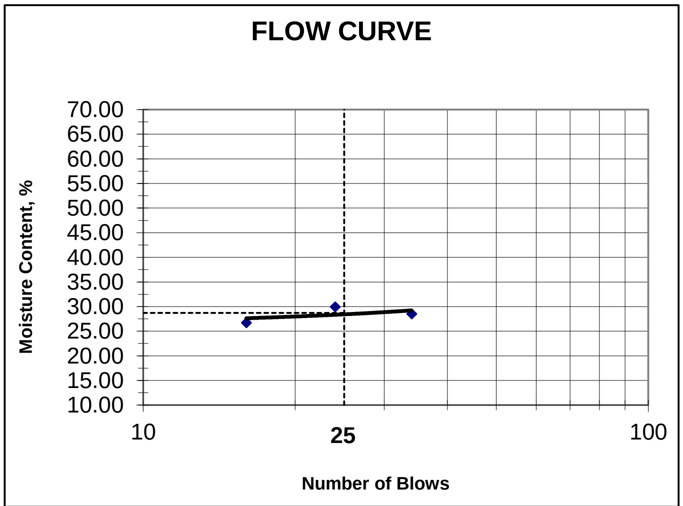
Determination of Atterberg Limits

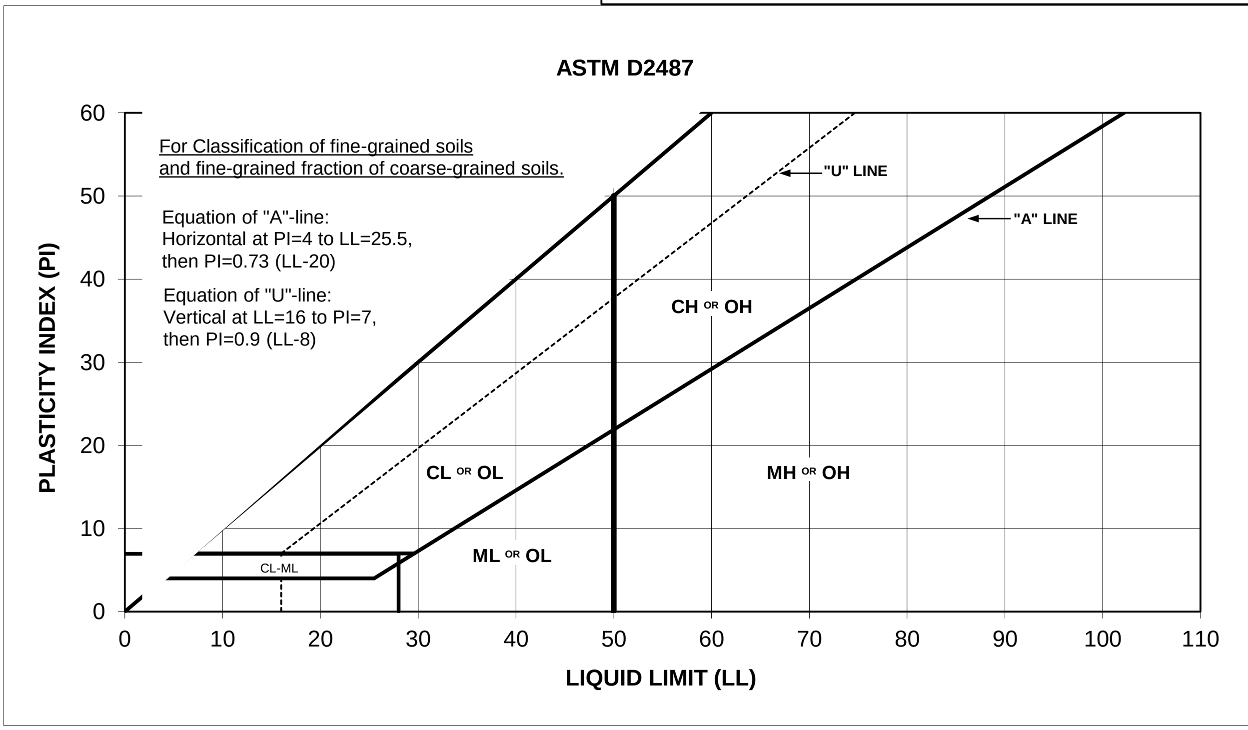
ASTM D 4318, CTM 204

Project Name	Lincoln Avenue Bridge	Project No.	160598
Sample Location	B-1 @ 11'	Tested By	JS
Soil Classification	Clayey Sand (SC)	Date	8/30/16

		PLASTIC LIMIT				LIQUID LIMIT			
A	Tes No.	1	2	3	No. of Blows	16	24	34	
B	Tare No.								
C	Mass of Pan + Dry Soil, g	25.44	35.10	25.77		43.91	41.14	37.39	
D	Mass of Pan + Wet Soil, g	26.35	36.43	26.67		47.76	44.70	39.89	
E	Mass of Pan, g	21.22	29.23	21.01		29.51	29.25	28.62	
F	Mass of Water, g	0.91	1.33	0.90		3.85	3.56	2.50	
G	Mass of Dry Soil, g	4.22	5.87	4.76		14.40	11.89	8.77	
H	Moisture Content, %	21.56	22.66	18.91		26.74	29.94	28.51	
I	Average Moisture Content, % (PL)		21.04						

Liquid Limit: Read from graph	**28.0**
Plastic Limit: Line I	**21.0**
Plasticity Index: PI = LL - PL	**7.0**





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Method for Estimating the Service Life of Steel Culverts
Caltrans California Test 643

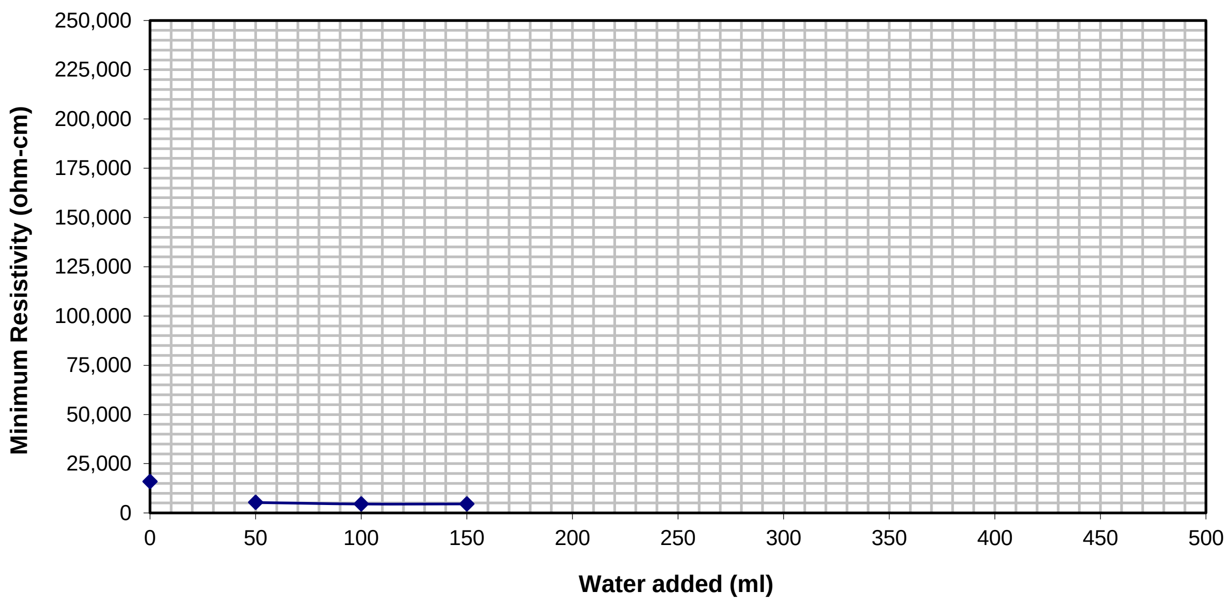
Project Name	Lincoln Avenue Bridge	Sample Location	B-1 @ 0'-3'
Project Number	160598	Test Date	8/10/2016
Sample Date	7/20/2016	Tested By	K.W.
Sampled By	S. Athwal	Material Description	Clayey Sand (SC)

Sample Condition	**As Received**	**Minimum Resistivity**					
Water Added (ml)	0	50	100	150			
Resistance (ohm)	15,000	5,000	4,250	4,300			
Resistivity (ohm-cm)	15,975	5,325	4,526	4,580			

Minimum Resistivity (ohm-cm)	**4,526**	**Field Resistivity (ohm-cm)**

pH = 7.23 **EC =**

Box Constant=1.065



Years to perforation*	32

* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts

TECHNICON ENGINEERING SERVICES, INC.

Method for Estimating the Service Life of Steel Culverts
Caltrans California Test 643

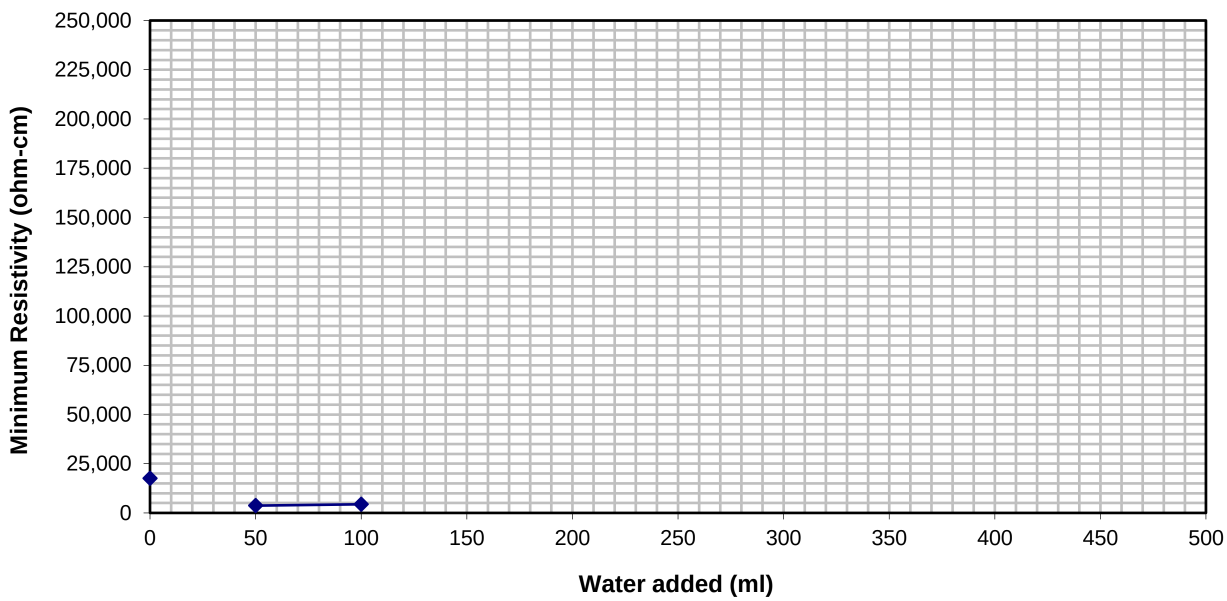
Project Name	Lincoln Avenue Bridge	Sample Location	B-1 @ 16'
Project Number	160598	Test Date	8/8/2016
Sample Date	7/20/2016	Tested By	K.W.
Sampled By	S. Athwal	Material Description	Clayey Sand (SC)

Sample Condition	**As Received**	**Minimum Resistivity**					
Water Added (ml)	0	50	100				
Resistance (ohm)	16,500	3,489	4,119				
Resistivity (ohm-cm)	17,573	3,716	4,387				

Minimum Resistivity (ohm-cm)	**3,716**	**Field Resistivity (ohm-cm)**

pH = 7.65 **EC =**

Box Constant=1.065



Years to perforation*	42

* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts

TECHNICON
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Chemical Analysis

SO_4 - Modified Caltrans 417 & CL - Modified Caltrans 417/422

Project	Lincoln Avenue Bridge	Technician	K. W
	Fresno County, CA	Date	7/22/2016
TES No.	160598	Remarks	Silty Sand (SM)

Sample Location	Soluble Sulfate SO_4-S		Soluble Chloride Cl	
B-1 @ 0'-3'	1.2	mg/Kg	8.9	mg/Kg
B-1 @ 0'-3'	1.7	mg/Kg	7.1	mg/Kg
B-1 @ 0'-3'	1.9	mg/Kg	10.6	mg/Kg
Average	**5.00**	**mg/Kg**	**9.00**	**mg/Kg**

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TECHNICON ENGINEERING SERVICES, INC.

Chemical Analysis

SO_4 - Modified Caltrans 417 & CL - Modified Caltrans 417/422

Project	Lincoln Avenue Bridge	Technician	K. W
	Fresno County, CA	Date	8/5/2016
TES No.	160598	Remarks	Clayey Sand (SC)

Sample Location	**Soluble Sulfate SO_4-S**		**Soluble Chloride Cl**	
B-1 @ 16'	2.3	mg/Kg	3.6	mg/Kg
B-1 @ 16'	2.9	mg/Kg	4.4	mg/Kg
B-1 @ 16'	1.9	mg/Kg	1.8	mg/Kg
Average	**5.00**	**mg/Kg**	**5.00**	**mg/Kg**

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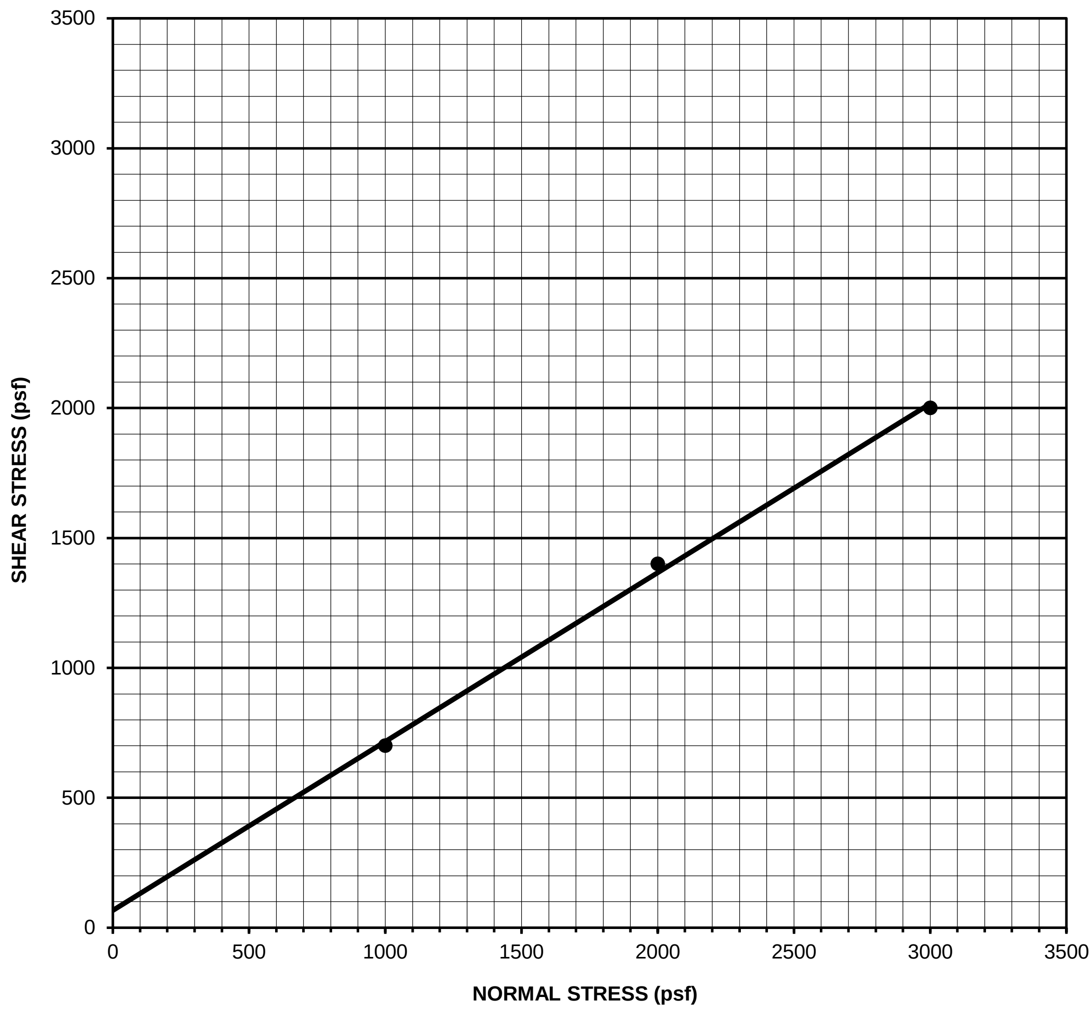
Direct Shear Test

ASTM D3080



Project	Lincoln Avenue Bridge
TES No.	160598
Sample Date	7/20/2016
Sample No.	B-1 @ 3'
Description	Clayeye Sand (SC)

Cohesion (psf)	**70**
Internal Friction Angle (ϕ)	**33**

Specimen	A	B	C	D	E
Dry Density (pcf)	127.5	127.5	127.5	---	---
Initial Water Content (%)	5.1	5.1	5.1	---	---
Final Water Content (%)	24.6	14.6	13.6	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	700	1400	2000	---	---

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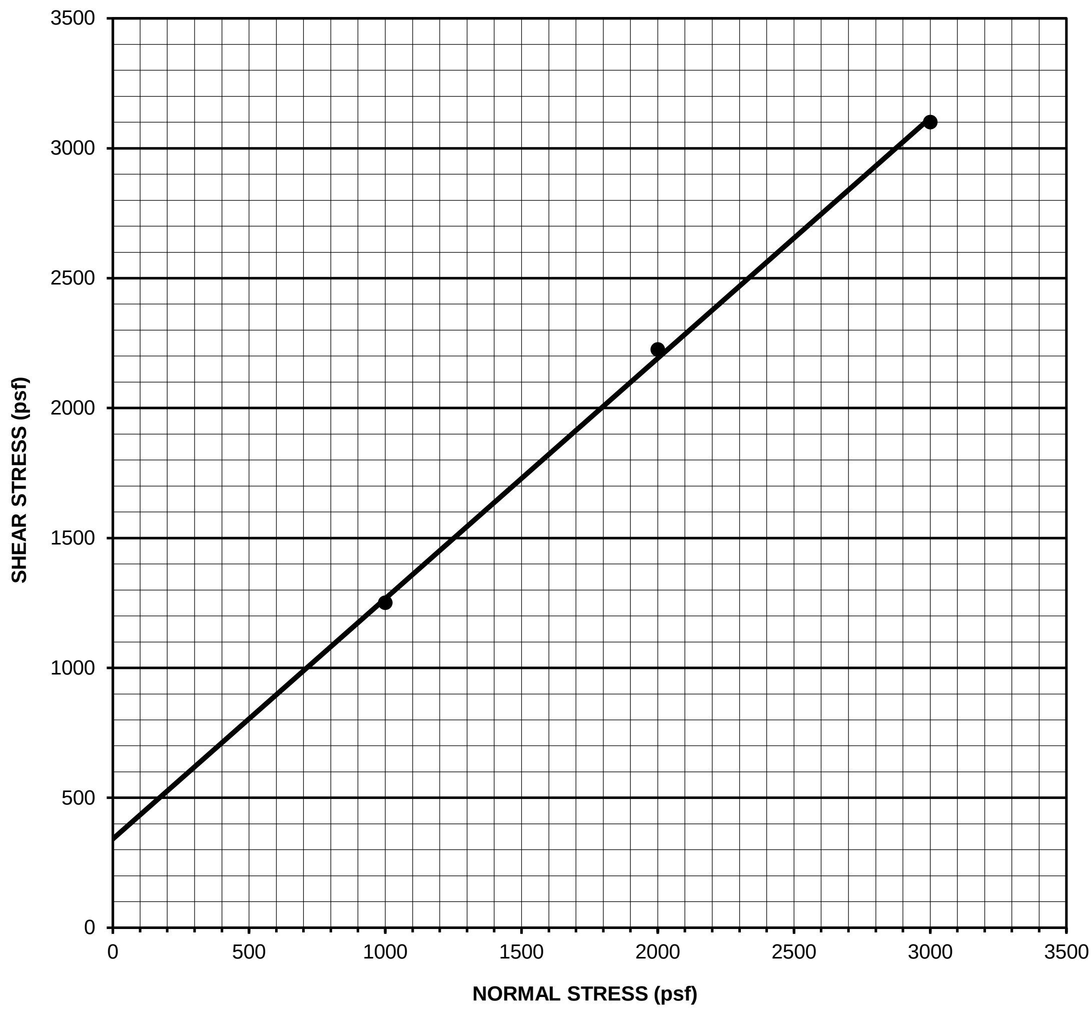
Direct Shear Test

ASTM D3080



Project	Lincoln Avenue Bridge
TES No.	160598
Sample Date	7/20/2016
Sample No.	B-1 @ 11'
Description	Clayey Sand (SC)

Cohesion (psf)	**340**
Internal Friction Angle (ϕ)	**43**

Specimen	A	B	C	D	E
Dry Density (pcf)	112.8	112.8	112.8	---	---
Initial Water Content (%)	12.1	12.1	12.1	---	---
Final Water Content (%)	17.2	16.1	17.2	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	1250	2225	3100	---	---

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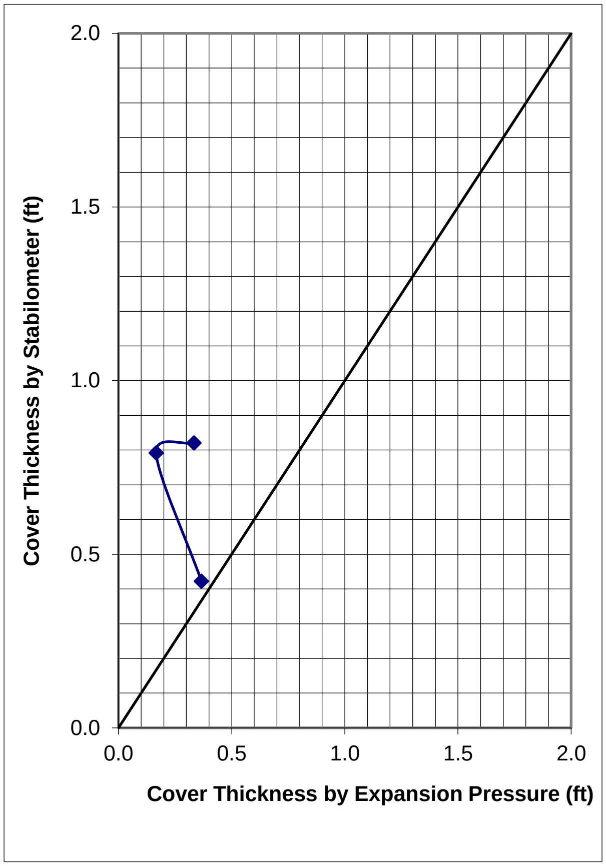
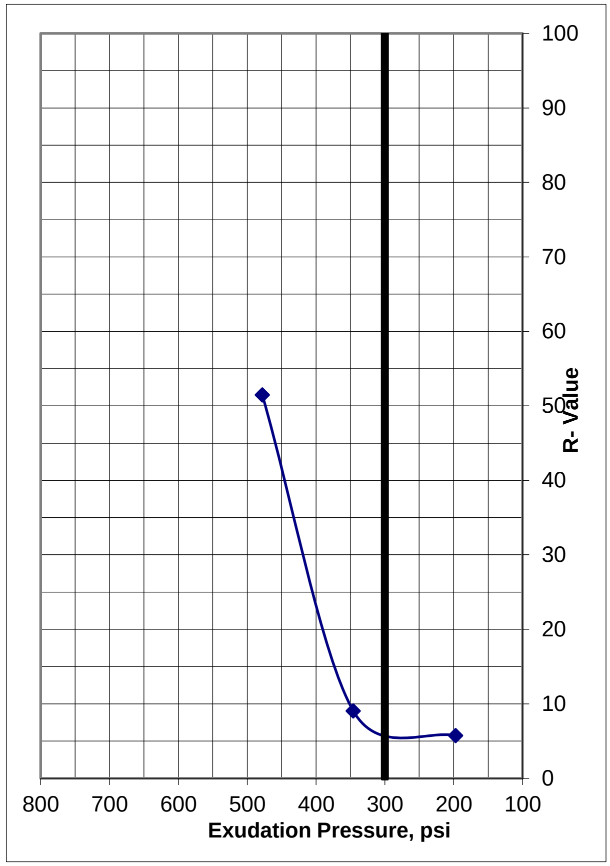
Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Lincoln Avenue Bridge	Lab ID Number	16-354
Project Number	160598	Sample Location	RV-1 @ 0'-2'
Sample Date	7/20/16	Tested By	J.A.
Sampled By	S. Athwal	Date Tested	8/1/2016
Material Description	Clayey Sand (SC)		





Specimen	1	2	3
Exudation Pressure, psi	197	346	478
Moisture at Test, %	11.7	11.0	8.8
Dry Density, pcf	123.3	125.5	129.7
Expansion Pressure, psf	43	22	48
Thickness by Stabilometer, ft.	0.8	0.8	0.4
Thickness by Expansion Pressure, ft.	0.3	0.2	0.4
R-Value by Stabilometer	6	9	51
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	5		

Controlling R-Value	**5**

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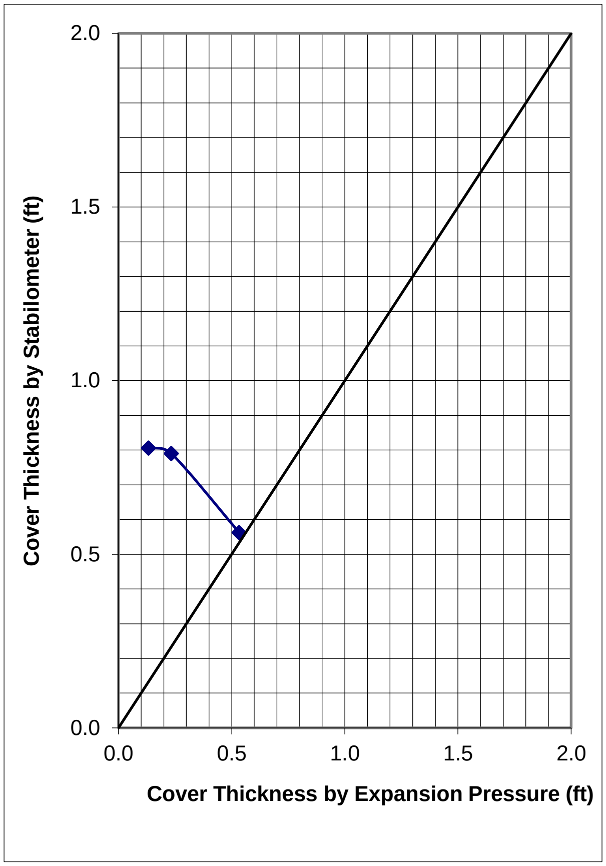
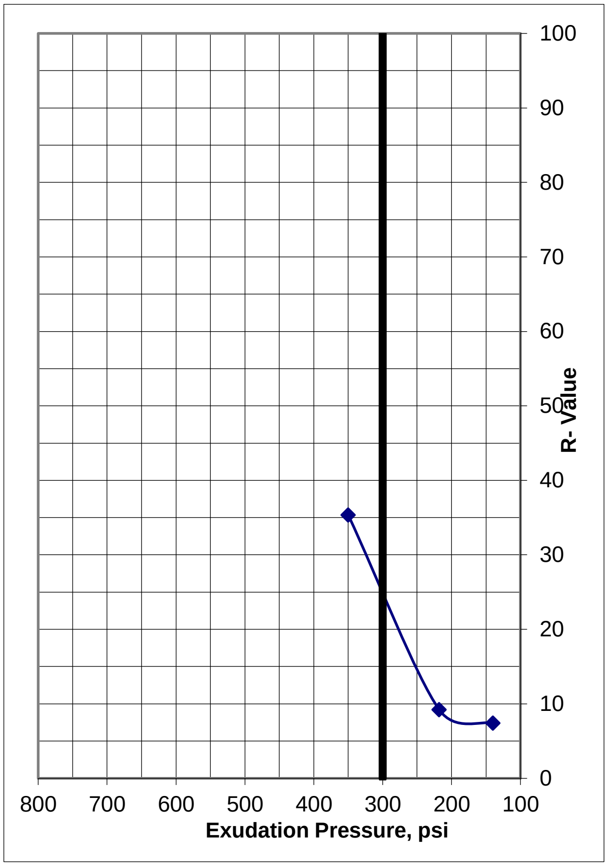
Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Lincoln Avenue Bridge	Lab ID Number	16-354
Project Number	160598	Sample Location	RV-2 @ 0'-1.5'
Sample Date	7/20/16	Tested By	J.A.
Sampled By	S. Athwal	Date Tested	8/1/2016
Material Description	Clayey Sand (SC)		





Specimen	1	2	3
Exudation Pressure, psi	140	218	350
Moisture at Test, %	13.2	12.2	10.6
Dry Density, pcf	117.5	119.9	122.8
Expansion Pressure, psf	17	30	69
Thickness by Stabilometer, ft.	0.8	0.8	0.6
Thickness by Expansion Pressure, ft.	0.1	0.2	0.5
R-Value by Stabilometer	7	9	35
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	25		

Controlling R-Value	**25**

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DYNAMIC CONE PENETRATION TEST

APPENDIX C

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	Lincoln Avenue Bridge			
Project Name:	Repacement at Travers Creek	Date:	8/29/2016	
Project #	160598	Hammer Weight:		15 lbs
Location:	Fresno County, CA	Field Engineer:	Sarbjit Athwal	

Depth (in)	Depth (ft)	No. of Blows
1.75	0.15	8
3.5	0.29	15
5.25	0.44	25
7	0.58	14
8.75	0.73	21
10.5	0.88	28
12.25	1.02	26
14	1.17	34
15.75	1.31	35

**Note: Depth Measured from the Bottom of the Canal

DEASIGN ARS CURVE AND

SEISMIC ANALYSIS

APPENDIX D

Project: **Lincoln Avenue Bridge Replacement at Travers Creeek**
Location: **Fresno County**
TES #: **160598**

TECHNICON ENGINEERING SERVICES, INC.

Site Information:

Latitude:	36.64729
Longitude:	-119.38478
V_{s30} (m/s)	344
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Distance (km)[1] =	126

Governing Curve:

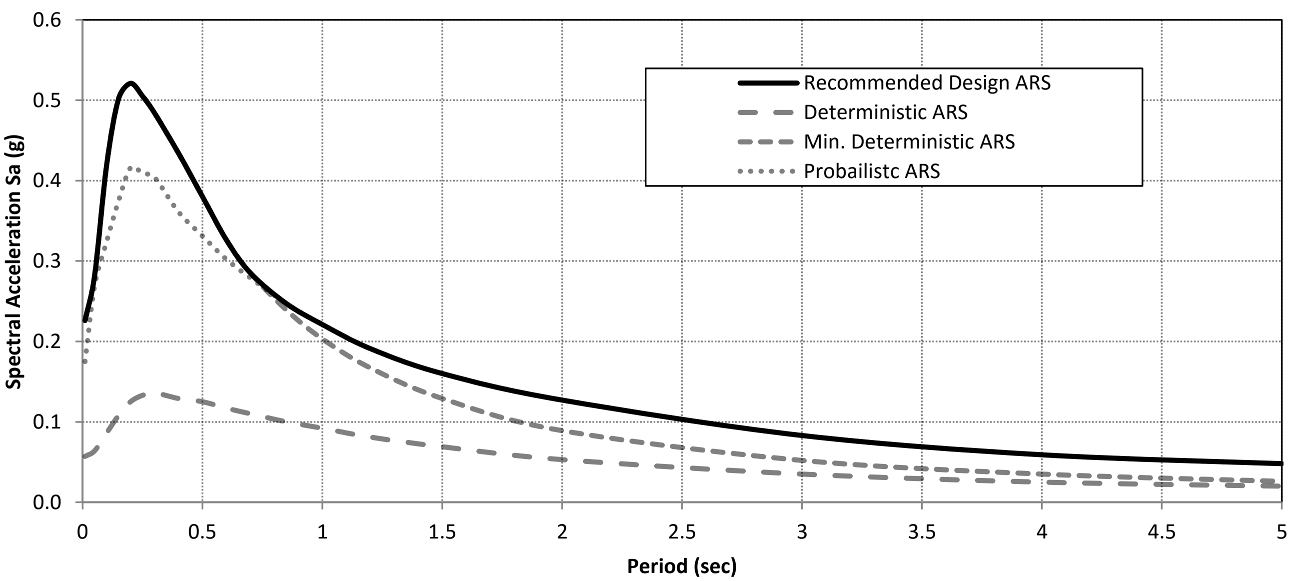
- ☐ Caltrans ARS OnLine Deterministic
- ☐ Minimum Deterministic
- ☐ Caltrans ARS OnLine Probabilistic
- ☑ Envelope of:
  - ☐ Caltrans ARS OnLine Deterministic
  - ☑ Caltrans Minimum Deterministic
  - ☑ Caltrans ARS OnLine Probabilistic

Recommended Response Spectrum

Period (sec)	SA Base Spectrum (g)	Adjusted for Basin Effect	Adjusted for Neaf Fault Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.226	-	-	0.226
0.1	0.419	-	-	0.419
0.2	0.521	-	-	0.521
0.3	0.484	-	-	0.484
0.5	0.38	-	-	0.380
1.0	0.221	1.000	1.000	0.221
2.0	0.127	1.000	1.000	0.127
3.0	0.083	1.000	1.000	0.083
4.0	0.059	1.000	1.000	0.059
5.0	0.048	1.000	1.000	0.048

RECOMMENDED ARS CURVE

Envelope of Deterministic and Probabilistic Curves (5% Damping)



Sources:

Caltrans Seismic Design Criteria, Version 1.7, April 2013
Caltrans Geotechnical Services Design Manual, August 2009
Caltrans ARS Online tool (v2.3.07, http://dap3.dot.ca.gov/shake_stable/)
USGS 2008 Interactive Daggregations (https://geohazards.usgs.gov/deaggint/2008/index.php)

KLEINFELDER
Bright People. Right Solutions.

May 2, 2024
Kleinfelder Project No.: 24005503.001A

Mr. Mark Weaver
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711
Phone: (559) 320-3200
Email: mweaver@cseg.com

Subject:	**Final Design Memorandum** **Lincoln Ave Bridge Replacement at Travers Creek** **Fresno County, California**
Reference:	**Foundation Report, Lincoln Ave Bridge Replacement at Travers Creek, Fresno County, California, TECHNICON Engineering Services, Inc., File No 160598.001, dated September 9, 2016**

Dear Mr. Weaver:

In accordance with your request, Kleinfelder completed additional engineering analysis and prepared this final design memorandum to support the PS&E for the reinforced concrete box culvert (RCB) replacement on Lincoln Avenue at the Travers Creek in Fresno County, California. The memorandum serves to supplement the above referenced Foundation Report (FR) for the 100% submittal of the PS&E and construction phases of the project. In addition, the letter serves to maintain continuity of the Geotechnical Engineer of Record through the PS&E phase.

PROJECT UNDERSTANDING

An understanding of the project is based on telephone conversations and email correspondence with Regina Barton and Mark Weaver of Cornerstone Structural Engineering Group (CSEG) and Mr. Joseph Harrel of the County of Fresno. The above referenced Foundation Report (FR) was previously prepared to support the design of a bridge replacement located on Lincoln Avenue at Travers Creek. The replacement bridge is anticipated to consist of a reinforced concrete box culvert (RCB) with an open bottom and utilizing retaining walls at the approaches.

Tables 1 through 3 present foundation design data and foundation design loads provided by CSEG and used for this geotechnical evaluation. Referenced elevations are based on elevations provided in General Layout and Foundation Plan Sheets, 100% Submittal, dated November 10, 2017.

Table 1
Box Culvert Foundation Data

Road Finished Grade Elev. (ft)	Bottom of Foundation Elev. (ft)	Foundation Size[1]		S_p[2]
		B	L	
383.0	366.43	6.0	66.0	1”

[1] B is measure perpendicular to the road and L is measured parallel to the road.
[2] Permissible settlement under service load

Table 2
Box Culvert Foundation Load Data

Maximum Service (Total) Bearing Pressure (ksf)	Maximum Service (Permanent) Bearing Pressure (ksf)	Maximum Strength Bearing Pressure (ksf)	Maximum Extreme Bearing Pressure (ksf)
4.76	1.29	6.89	1.29

Table 3
Retaining Wall Foundation Data

Design Height (ft)	Bottom of Footing Elev. (ft)	Min. Footing Embed. Depth (ft)	Effective Foundation Width, B' (ft)[1]		S_p[2]	Maximum Service (Total) Bearing Pressure (ksf)
			Strength 1A Limit State	Strength 1B Limit State		
7.08	374.4	2.46	4.08	4.52	1”	1.66
11.08	370.4	4.27	4.10	5.00	1”	2.41
14.75	366.4	6.24	4.94	6.20	1”	3.06

[1] B is measure perpendicular to the wall.
[2] Permissible settlement under service load

PURPOSE AND SCOPE OF SERVICES

The purpose of this final design memorandum is to update the previous signed Foundation Report and address the following supplemental items:

- Perform a site visit to observe current site conditions.
- A summary of the updated project information and design details including loading information.
- Recommended gross and net permissible contract stress associated with tolerable settlements and bearing capacity and design footing elevations of spread footing foundation for the open bottom area of the RCB.
- Recommended gross and net permissible contract stress associated with tolerable settlements and bearing capacity for retaining walls.

- Recommendations to stabilize soft or yielding subgrade soils with options for recompaction, replacement with aggregate base, and use of geotextile reinforcement.

SITE VISIT

Kleinfelder observed the site conditions on May 8th, 2023, at the Lincoln Avenue and Travers crossing. The site conditions remained essentially unchanged from the previous field exploration completed on July 20, 2016. Lincoln Avenue is a 2-lane bridge, timber stringer with asphalt concrete overlay, approximately 20 feet long by 24 feet wide. The canal was unlined and flowed with a water depth of approximately 4 to 5 feet.

CONCLUSIONS AND SUPPLEMENTAL RECOMMENDATIONS

It is Kleinfelder's opinion that the recommendations presented in the FR may be used for PS&E and construction phases of the project along with the following supplemental geotechnical data and recommendations.

Box Culvert Bearing and Settlement

Table 4 "Foundation Data Table" provides the bearing resistance and settlement based on the design loads and dimensions provided.

Table 4
Footing Data Table
(Double Box Culvert)

Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth (ft)	Total Permissible Support Settlement (inches)	Service Limit State	Strength or Construction Limit State ϕb=0.45
L	B				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
66	6.0	366.43	2	1	18.4	7.3

Based on the Gross Maximum Bearing Stress (Service) of 4.76 ksf provided by CSEG, the total settlement of the RCB is approximately 0.25-inch. Differential settlement is anticipated to be reduced to half of the total settlement across the length/width of the RCB.

Retaining Wall Bearing and Settlement

Table 5 "Foundation Data Table" provides the bearing resistance and settlement of bridge approach retaining walls based on the design loads and dimensions provided by CSEG.

Table 5
Footing Data Table
(Retaining Walls)

Design Height (ft)	Bottom of Footing Elev. (ft)	Min. Footing Embed. Depth (ft)	Strength 1A Limit State			Strength 1B Limit State		
			Eff. Found. With (ft)	Gross Bearing Stress (ksf)	Factored Bearing Resist (ksf)	Eff Found. With (ft)	Gross Bearing Stress (ksf)	Factored Bearing Resist (ksf)
7.08	374.4	2.46	4.08	14.4	7.9	4.52	15.1	8.3
11.08	370.4	4.27	4.10	20.0	11.0	5.00	21.5	11.8
14.75	366.4	6.24	4.94	27.4	15.1	6.20	29.6	16.3

The estimated settlement based on the Gross Maximum Bearing Stress (Service) provided by CSEG for the walls is approximately 0.5-inch. Differential settlement is anticipated to be reduced to half of the total settlement across the length of the walls.

Unstable Foundation Recommendations

The design bearing stress/resistance given in Tables 4 and 5 requires that the RCB and walls will be placed on unyielding native soil or approved engineered fill. Any soft, unsuitable sediment in the canal bottom should be excavated to expose firm undisturbed soil and removed from the project site. If unstable foundation conditions are encountered it will be necessary to stabilize the area prior to foundation construction. Stabilization options include the following:

Option 1 – Solar Drying, Mixing, and Blending of Dry Material

Unstable, shallow subgrade soils may be repeatedly disced to promote evaporation/natural drying and/or blended with dryer import fill soil to a compactable moisture range and recompacted in accordance with latest Caltrans Standard Specifications.

Option 2 – Mechanical Stabilization

Should the construction area experience moderate to severe instability, the foundation areas should be stabilized by removing a portion of the unstable subgrade followed by placement of Subgrade Enhancement Geotextile (SEG_T) or bi-axial Subgrade Enhancement Geogrid (SEG_G) that complies with Section 96 of the Caltrans Standards Specifications. SEG should be placed on the smooth subgrade followed by placement of 0.67-to-1.0-foot Caltrans Class 2 aggregate base (AB) and compacting to establish initial stability. The SEG should be smooth and taught and extend a minimum of 5 feet beyond unstable areas. Adjacent panels of SEG should be lapped a minimum of 2 feet.

AB should be front loaded onto SEG, spread with the equipment working on the AB, and densified with moderate to heavy compaction equipment. The equipment should not operate directly on the SEG. Aggregate base should be compacted to a minimum 95 percent relative compaction. If 95 percent compaction cannot be achieved with the initial 0.67- to 1.0-foot-thick layer of AB, subsequent, layers of

SEG and 0.67- to 1.0-foot-thick layers of AB should be placed until stability is achieved. The final layer should be compacted to a minimum 95 percent relative compaction.

LIMITATIONS

Kleinfelder will perform its services in a manner consistent with the standards of care and skill ordinarily exercised by members of the profession practicing under similar conditions in the geographic vicinity and at the time the services will be performed. No warranty or guarantee, express or implied, is intended or provided.

CLOSING

Kleinfelder appreciates the opportunity to serve as geotechnical consultants to Cornerstone Structural Engineering Group and the County of Fresno during the PS&E phase of the project. If there are any questions concerning the information presented in this letter, please contact the undersigned at your convenience.

Respectfully submitted,
KLEINFELDER, INC.

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 09/30/2025
GEOTECHNICAL
STATE OF CALIFORNIA

Anthony Aquino	Stephen P. Plauson, PE, GE
Professional	Senior Principal Geotechnical Engineer

September 21, 2016

TES No. 160597.001
Invoice No. 11964

Mr. Jonathan P. Jensen
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

Project: Parlier Avenue Bridge Replacement at Travers Creek
Fresno County, California

Subject: Foundation Report

Dear Mr. Jensen:

The attached Foundation Report presents the results of a geotechnical investigation for the design and construction of a reinforced concrete box culvert (RCB) planned on Parlier Avenue at Travers Creek near Reedley, in Fresno County, California. The report describes the study, findings, conclusions, and recommendations for use in project design and construction.

TECHNICON appreciates the opportunity to provide geotechnical engineering services to Cornerstone Structural Engineering Group during the design phase of this project. We trust this information meets your current needs. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully submitted,
TECHNICON Engineering Services, Inc.

Sarbjit Athwal, EIT
Project Engineer

Stephen P. Plauson, PE, GE
Geotechnical Engineering Manager

SS:SPP:mk

TECHNICON
ENGINEERING SERVICES, INC.

FOUNDATION REPORT
PARLIER AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK
FRESNO COUNTY, CALIFORNIA

Prepared For:

Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

September 21, 2016

TES No. 160597.001

GEOTECHNICAL & ENVIRONMENTAL ENGINEERING – CONSTRUCTION TESTING & INSPECTION

TECHNICON
ENGINEERING SERVICES, INC.

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YEARS STRONG

GEOTECHNICAL & ENVIRONMENTAL ENGINEERING — CONSTRUCTION TESTING & INSPECTION

Prepared For:

Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711

FOUNDATION REPORT
PARLIER AVENUE BRIDGE REPLACEMENT AT
TRAVERS CREEK
FRESNO COUNTY, CALIFORNIA

TECHNICON PROJECT
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Prepared by:

Sarbjit Athwal, EIT
Project Engineer

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Geotechnical Engineering Manager

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 9/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

TECHNICON Engineering Services, Inc.
4539 North Brawley Avenue, Suite 108
Fresno, California 93722
(559) 276-9311

September 21, 2016

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1 INTRODUCTION

1.1 GENERAL

This Foundation Report presents the results of a geotechnical investigation for a reinforced concrete box culvert (RCB) planned on Parlier Avenue at Travers Creek near Reedley, in Fresno County, California. The purpose of the investigation was to explore and evaluate the subsurface conditions at the site and prepare a Foundation Report containing recommendations to aid in project design and construction.

The Vicinity Map, presented on Figure 1, shows the location of the project and the Site Map, Figure 2, and Log of Test Boring drawing (LOTB) show the proposed bridge replacement and the approximate boring location for this study.

1.2 PROJECT DESCRIPTION

The project involves the replacement of an existing bridge located on Parlier Avenue at Travers Creek. The existing bridge is a two-lane, reinforced concrete bridge, approximately 28 feet long by 21.5 feet wide. The replacement bridge is anticipated to consist of a double barrel RCB utilizing either a closed or open bottom RCB utilizing strip and spread footings at the supports. To accommodate the Creek and roadway widths, the RCB will be approximately 36 feet in length and 34 feet in width. Based on preliminary information provided by Cornerstone Structural Engineering Group, it is reported the RCB will have an opening height of 6 feet and cover height equal to a typical asphalt concrete pavement section (e.g. less than 1.0 foot of cover) for a total height of approximately 9 feet. The design may incorporate an open bottom configuration or closed bottom with slab extensions up and down stream. Warped wing walls will form the transition of the bottom slab and side slopes. For the closed bottom option, it's assumed that Cornerstone Structural Engineering Group will make provisions to protect the RCB foundations from scour. A gradation of the creek sediments is provided for use in hydraulic analysis of the potential scour.

It is anticipated that Caltrans Standards Plans will be utilized as the basis for design of the culvert and wingwalls.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to explore the site subsurface conditions to allow for development of recommendations and opinions to aid in project design. The report includes the following: A description of the proposed project including a vicinity map showing the location of the site and a site plan showing the locations of the exploration point for this study

- ❑ A description of the site surface and subsurface conditions encountered during the field investigation, including boring log
- ❑ A summary of the field exploration and laboratory testing program
- ❑ Discussion of regional and local geology including faults, seismicity, and liquefaction potential and associated effects
- ❑ Caltrans seismic design parameters
- ❑ Comments on the use of Caltrans Standard Plans for design of the box culvert and associated wingwalls
- ❑ Recommended Gross Nominal Bearing and Permissible Net Contact Stress for the box culvert foundation and anticipated settlement
- ❑ Recommended lateral earth pressures for design of the box culvert and wingwalls
- ❑ Comments on the corrosion potential of on-site soil
- ❑ Recommended pavement structural section for the design traffic index.
- ❑ Comments on site preparation and earthwork, including the use of on-site soils for engineered fill and recommended import fill specifications.

The scope of services consisted of a field exploration program, laboratory testing, design analysis, and preparation of this written report as outlined in **TECHNICON'**s proposal dated April 20, 2016 (TES No. GP16-95B).

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

The field exploration, conducted on July 20, 2016 consisted of drilling one (1) exploratory test boring and site reconnaissance by a project engineer. The test boring was drilled with a CME 55 truck-mounted drill rig using hollow stem augers. The boring extended to a depth of 51.5 feet below the existing ground surface (bgs). The approximate location of the test boring is indicated on the Site Map, Figure 2, and the Log of Test Boring Drawing (LOTB), Sheet 2. In addition, a Dynamic Cone Penetration (DCP) Test was performed in the center of the creek to assess the depth of historic scour.

The soils encountered in the boring 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 as the number of blows per foot over the last 12 inches of sampler penetration on the LOTB. The blow counts listed in the LOTB have not been corrected for the effects of overburden pressure, rod length, boring diameter, sampler size, or hammer efficiency.

2.2 FIELD AND LABORATORY TESTING

Penetration rates, determined in general accordance with ASTM D-1586, were used to aid in evaluating the consistency, compression, and strength characteristics of the foundation soils.

Laboratory tests were performed on selected near surface samples to evaluate their physical characteristics. The following laboratory tests were used to develop the design geotechnical parameters:

- Unit weight (ASTM D2937)
- Moisture content (ASTM D2216)
- Sieve Analysis (ASTM D422)
- Direct Shear (ASTM D3080)
- Soluble Sulfate, and Soluble Chloride Contents (California Test Method No's. 417 and 422)
- pH and Minimum Resistivity (California Test Method No. 643)

- ❑ Resistance Value (California Test Method No. 301)

The dry density and moisture content test results are shown on the LOTB in Appendix A. The soluble sulfate, soluble chloride, pH and minimum resistivity are discussed in the "Corrosion Potential" Section (Section 5.5). The remaining test results are provided in Appendix B.

3 SITE GEOLOGY AND CONDITIONS

3.1 SURFACE CONDITIONS

The subject bridge replacement is at the Parlier Avenue and Travers Creek crossing. Parlier Avenue is a 2 lane asphalt paved road with unpaved shoulders and aligned east-west. Travers Creek was unlined and at the time of the field investigation the creek was flowing with a water depth of approximately 3 to 4 feet. The slopes of Travers Creek were approximately 1½ :1 horizontal to vertical (H:V), with the creek crossing Parlier Avenue in a north to south direction. The bridge location is generally bounded by open agricultural fields to the northwest and southwest, an old wooden barn building to the northeast and single family residence to the southeast.

3.2 SUBSURFACE CONDITIONS

The natural site soil consists of nonmarine deposits with a geologic age of Pleistocene. The general earth material profile depicted by the subsurface exploration consisted primarily of silty sand in the upper 7 feet, followed by poorly graded sand to 11 feet and underlain by laterally discontinuous layers of clayey sand, silty sand, poorly graded sand, poorly graded sand with silt, and sandy clay to the depth explored, 51.5 feet bgs. The granular soil generally had a relative consistency of medium dense to very dense while the fine grained soil generally had a relative consistency of hard.

The above is a general description of the earth material profile. A more detailed representation of the stratigraphy at the specific exploration location is provided on the LOTB included in Appendix A.

3.3 GROUNDWATER CONDITIONS

Purged groundwater from the creek was encountered at depth ranging from 7 to 11 feet bgs at the test boring location. The water encountered appears to be perched due to water flow in Travers Creek. The State of California Department of Water Resources, “Lines of Equal Elevation of Water in Wells”, Spring 2011 indicates the regional depth to groundwater exceeds 50 feet. Additional research utilizing the California Department of Water Resources (DWR) website indicates the nearest monitored well to be approximately ¼ of a mile to the northeast (Well No. 15S24E19H001M). Based on the groundwater elevation data collected at this well, the historic high groundwater depth was recorded at 12 feet bgs in the late 1969’s and the current recorded groundwater depth is approximately 55 feet bgs.

The groundwater elevation at the bridge site is likely is more likely influenced by flow or recency of flow within Travers Creek and could affect construction. Depending on the flow or recency of flow in Travers Creek at the time of construction, earthwork and construction may be impacted by soft/yielding subgrade and/or saturated conditions. It is assumed that construction may occur during the winter months shortly after closure of the creek. Therefore, it should be anticipated that the creek bottom and sides of the creek could be saturated and may not provide a stable bottom for construction activities.

4 SEISMIC RECOMMENDATIONS

4.1 SEISMIC SOURCES

The project site and its vicinity are located in an area traditionally characterized by relatively low seismic activity. The site is not located in an Alquist-Priolo Earthquake Fault Zone as established by the Alquist-Priolo Fault Zoning Act (Section 2622 of Chapter 7.5, Division 2 of the California Public Resources Code).

Review of the Caltrans Deterministic PGA Map (September 2007), indicates there are no existing major fault systems within 25 miles of the project vicinity. Based on review of published data and current understanding of the geologic framework and tectonic setting of the proposed improvements, the primary sources of seismic shaking at this site are listed in Table 4.1-1. A major seismic event on these or other nearby faults may cause ground shaking at the site. Based on the deterministic ground acceleration, the San Andreas Fault is considered the governing fault.

TABLE 4.1-1
LOCAL FAULTS AND ESTIMATED MOMENT MAGNITUDES

Fault	Approximate Distance from Site (km)	Maximum Credible Earthquake (Moment Magnitude, M_W)	Peak Ground Acceleration (g)
San Andreas Fault	125	8.0	0.093
Independence	100	7.1	0.084
Round Valley	100	7.0	0.079
Coast Ranges Sierran Block	85	6.5	0.069

4.2 SEISMIC DESIGN CRITERIA

Development of a site specific Acceleration Response Spectra (ARS) curve was undertaken in accordance Caltrans Geotechnical Design Manual (Ver. 2.3.07, March 2016) and the Caltrans Seismic Design Criteria (Ver. 1.7, November 2013).

The Reedley California 7½-minute Quadrangle Topographic Map, dated 1966, indicates the proposed Parlier Avenue Bridge Replacement lies on the north edge of Section 19, T15S, R24E. Furthermore, the average shear wave velocity for the upper 30m (100 feet) of the subsurface soil and rock at the bridge site was estimated by using established correlations and procedures presented in the Caltrans Geotechnical Design Manual. The estimated shear wave velocity is provided below.

Site Location: Latitude: 36.611325° N / Longitude: -119.404130° W

Shear Wave Velocity: Vs(30) = 340 m/s

ARS curves for the bridge site were determined based on the Caltrans Deterministic PGA Map (September 2007), Caltrans ARS Online (Ver. 2.3.07), the shear wave velocity of the soil, and the latitude/longitude at the bridge location. A Site Specific ARS curve was developed for the project and is included in Appendix D for use in the seismic analysis of the bridge. The recommended Design ARS curve consists of the envelope of the Caltrans Minimum Deterministic ARS and Caltrans Online Probabilistic ARS. The results of the 2008 USGS Deaggregation Tool (Beta) do not govern, since the shear wave velocity exceeds 300 m/s.

4.3 SEISMIC HAZARDS

Review of the Caltrans Deterministic PGA Map (September 2007) indicates that no mapped active faults cross or project toward the site. Additionally, no evidence of active faulting was visible on the site during our site reconnaissance. Therefore, it is our opinion that the potential for fault-related surface rupture at the proposed bridge site is very low. Furthermore, the Caltrans Deterministic PGA Map (September 2007) indicates the site is located relatively far from active faults, as such, the possibility for the site to experience strong ground shaking may be considered low.

4.3.1 SEISMICALLY INDUCED GROUND FAILURE

4.3.2 Design Ground Motion

For the purpose of evaluating liquefaction, a probabilistic seismic hazards analysis (PSHA) procedure was performed using the 2008 USGS Deaggregation Tool (Beta) to estimate the earthquake magnitude. The program allows user input of the project site coordinates and produces the expected peak ground motions for the site for selected probability of exceedance (e.g. return periods). The USGS Deaggregation Tool, based on a probability of exceedance of 2 percent in 50 years, determined a weighted magnitude of Mw = 6.08. The peak ground acceleration was assessed using ARS Online and found to be 0.226g.

4.3.3 Liquefaction

In order for liquefaction, and possible associated effects, 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 fine, granular, and uniform,
- ❑ Ground shaking of sufficient intensity should occur 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).

Saturated granular sediments can experience liquefaction if subject to seismically induced ground motion of sufficient intensity and duration. Based on the ground shaking which may be expected at this site, the relative density and geologic age of the sediments, analysis utilizing Youd (2001) indicates liquefaction, seismically induced settlement, or bearing loss is considered unlikely.

4.3.4 Dynamic Compaction

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Considering that problematic soils were not identified in the borings drilled for this study, seismically induced dry sand settlement is anticipated to be minimal. Calculations indicate that seismically induced dry sand settlement is negligible.

5 DESIGN RECOMMENDATIONS

5.1 GENERAL

Based on the laboratory data, field exploration, and geotechnical analyses conducted for this study, it is geotechnically feasible to construct the proposed RCB as currently envisioned. Provided that the recommendations presented in this report are incorporated into the project design and construction, use of a closed bottom or open bottom RCB with mat or spread foundations bearing on recompacted native soil or approved engineered fill prepared in accordance with Caltrans Standard Specifications, Section 19 are considered appropriate for structure support. Recommendations regarding the geotechnical aspects of design are presented in subsequent sections.

5.2 SCOUR EVALUATION

TECHNICON performed a gradational analysis of the sediments within the test boring at the elevation of the Travers Creek bottom to aid in the hydraulic evaluation of the channel scour by others.

To evaluate the creek bottom for scour, **TECHNICON** performed Dynamic Cone Penetration (DCP) Test to determine the historic scour depth. The DCP test was performed by dropping a 15-lb slide hammer from a height of 20 inches driving a 1.5 inch cone pointed rod. Observations and hand exploration indicates the Travers Creek channel has undergone localized scour within isolated areas of the existing bridge. It is estimated that the scour depth has extended to a depths of approximately 18 to 24 inches below the current creek bottom elevation. A summary of the DCP Test results can be seen in Appendix C.

An open bottom RCB option may should consider potential scour effects on the RCB foundations. It's recommended that Cornerstone Structural Engineering Group through the hydraulic and scour analysis either embed the foundations below the design scour depth or protect the foundations with rip rap protection, canal lining, or other means.

5.3 STABILITY OF SLOPES

Slope stability using dimensionless parameters by Janbu for permanent and temporary slopes was calculated for a creek and temporary slope height of 8 feet. It was determined that permanent slopes configured at 1½:1 H:V should be stable with regard to gross (deep seated) and surficial slope failure modes (factor of safety greater than 1.5, respectively). Temporary slopes configured at 1¼:1 H:V should be stable with regard to gross (deep seated) failure mode (factor of safety greater than 1.25)..

5.4 BOX CULVERT DESIGN

5.4.1 Bearing and Settlement

Based on the field exploration, laboratory testing, and geotechnical analyses, the soils at the site are suitable for supporting the RCB. The General Plan indicates the proposed RCB considers options for both open bottom and closed bottom box culverts. For the closed bottom RCB option, the General Plan indicates the proposed RCB length is approximately 40 feet and the width is approximately 30 feet. For the open bottom RCB, Caltrans Bridge Standard Detail Sheet for CIP Bottomless Culvert (Sheet xs17-050-3, dated July 12, 2016) indicates the footing width for the end supports are 5 feet with an effective width of 3.19 feet. Cornerstone Structural Engineering Group indicates the center pier foundation is estimated to have a Soil Pressure (qu) of 9.0 ksf, which is estimated to result in a preliminary footing width of 5.0 feet. The opening height of the RCB is 6 feet and the overall structure height including pavement is estimated to be 9.0 feet.

Table 5.4-1 "Footing Data Table" provides the bearing resistance and Net Permissible Contact Stress for 1-inch of settlement.

TABLE 5.4-1
FOOTING DATA TABLE

Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth (ft)	Service Limit State	Strength or Construction Limit State ϕ_b=0.45	Extreme Event Limit State ϕ_b=1.0
L'	B'			Permissible Net Contact Stress (s = 1.0") (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
40	30$^{(1)}$	347.9	1	3.0	22.1	49.2
40	3.19$^{(2)}$	344.5	2	10.1	4.9	10.8
40	5.0$^{(2)}$	344.5	2	6.5	6.2	13.6

Note 1: Footing size for closed bottom RCB
Note 2: Footing sizes for open bottom RCB

For the open bottom RCB option, the foundation embedment depth to the bottom of the foundation shall be the greater depth of either 2 feet below scour depth unless the foundations are protected from scour, or a minimum of 3.5 feet below the flow line.

The design bearing stress/resistance given in Table 5.2-1 requires that the RCB will be placed on unyielding native soil or approved engineered fill. Any soft, unsuitable sediment in the channel bottom should be excavated to expose firm undisturbed soil and removed from project site. Based on observations and DCP testing performed in the Creek bottom, for preliminary planning it should be anticipated that a general excavation depth of 18 to 24 inches may be required to remove unsuitable soil. However, isolated deeper areas deemed unsuitable could exist, which may require deeper excavation.

If unstable foundation conditions are encountered it will be necessary to stabilize the area prior to foundation construction. Stabilization options include placing a minimum of 12 inches of either a lean concrete slurry or ¾-inch diameter crushed gravel. If the crushed gravel is utilized, an engineering fabric conforming to the requirements of Section 88 of the Caltrans Standard Specifications should be placed on the subgrade prior to rock placement to prevent migration of fines into the rock. The fabric is necessary to add reinforcement and prevent migration of subgrade soil into the open spaces of the gravel. **TECHNICON** should be contacted to observe and approve the exposed subgrade prior to stabilizing the working/foundation area.

5.4.2 Lateral Earth Pressures

Caltrans Standard Plans, May 2010, for RCB's are based on the soil surrounding the planned RCB having minimum and maximum lateral earth pressures equal to 42 lb/ft^3 and 100 lb/ft^3. In addition, the maximum cover density is to be limited to 140 lb/ft^3. Based on the analysis of the native soil, the soil will exhibit an earth cover density of approximately 131 lb/ft^3. The minimum and maximum restrained lateral earth pressures of the native soil, backfilled in accordance with Caltrans Standard Specifications, Section 19 are 54 lb/ft^3 and 98 lb/ft^3. Consequently, the use of Caltrans Standard Plans for design of the RCB would be appropriate. Table 5.4-3 provides active and at-rest pressures and the dynamic incremental increase of the earth pressure against retaining walls considering earthquake loading. The pressures are based on the use of on-site soils for wall backfill.

TABLE 5.4-3
LATERAL EARTH PRESSURES

Loading Condition	Lateral Earth Pressure (psf/ft of Wall Height)		Earth Pressure Coefficient
	Drained	Undrained	
Active Pressure (psf/ft of depth)	40.5	23 + Hydrostatic	0.28
At-Rest Pressure (psf/ft of depth)	63	35.5 + Hydrostatic	0.43
Dynamic Active Incremental Increase (psf/ft of depth)	17.5		
Dynamic At-Rest Incremental Increase (psf/ft of depth)	9.0		

The Special Provisions requires that backfill placed within a 1:1 zone extending upward from the base of the RCB consist of low expansion granular fill (Expansion Index less than 10).

Should retaining walls be influenced by surcharge loads, the surcharge against the walls can be evaluated by multiplying the surcharge pressure by the earth pressure coefficient. Surcharge loads should be modeled as a uniform pressure against the wall by multiplying the surcharge load by the earth pressure coefficient.

5.4.3 Resistance to Lateral Loading

Lateral loads applied to RCB can be resisted by a combination of passive lateral bearing and frictional resistance. The allowable and ultimate passive pressures and sliding resistance for the RCB are presented in Table 5.4-4.

TABLE 5.4-4
PASSIVE BEARING AND SLIDING RESISTANCE

	WSD		LRFD	
	Static	Total Combined	Nominal	Strength Limit
Frictional Coefficient (Sliding)	0.45	0.54	0.67	0.54
Passive Pressure (psf/ft of depth)	260	345	520	260
Lateral Translation Needed to Develop Passive Pressure	0.005D	0.01D	0.025D	0.005D

Note: D is the depth of the zone providing resistance.
WSD = Working Stress Design, LRFD = Load/Resistance Factor Design

5.4.4 Bottom Slab Cutoff Wall

Extensions of the culvert bottom slab are planned up and down stream of the proposed RCB. Based on the granular nature of the anticipated bottom sediments and presence of flowing water, it is recommended that a cutoff wall be constructed at the ends of the concrete channel lining. The cutoff wall could be designed in accordance with Caltrans Standard Plans and have a minimum embedment of 4 feet below the bottom of the RCB. The final embedment of the cutoff wall should be extended as dictated by the scour conditions.

5.4.5 Warped Wingwalls

Proposed warped wingwalls shall be supported on approved undisturbed native soil channel slopes or properly engineered fill as well as the bottom slab extension. The native soils have strength characteristics that result in design earth pressures compatible with Caltrans Standard Plans. Provided that the Special Provisions specify that imported backfill consist of soil similar to the native soil or soil having a ϕ angle of at least 35 degrees, Caltrans Standard Plans design could be used.

5.4.6 Construction Observations

The culvert excavation should be observed by a representative of the Geotechnical Engineer. The purpose of these observations is to check that the bearing soils exposed in the excavation are similar to those on which the recommendations are based.

5.5 PAVEMENT DESIGN

Bulk soil samples were tested at two locations for R-value for pavement design. The test results are presented in Table 5.5-1. Pavement recommendations will be provided in the “final” Foundation Report for the design Traffic Index (TI) to be provided by Mark Thomas & Company.

TABLE 5.5-1
SUMMARY OF R-VALUE TESTS

Sample Location	Depth (ft)	Soil Type	R-Value by Exudation
RV-1	0-2	Silty SAND (SM)	72
RV-2	0-2	Silty SAND (SM)	63

5.6 CORROSION POTENTIAL

Two (2) soil samples obtained from the site were tested to evaluate pH, minimum electrical resistivity, and soluble sulfate and chloride content. Provided in Table 5.6-1 are the pH, minimum electrical resistivity and soluble sulfate and chloride content.

TABLE 5.6-1
CORROSION POTENTIAL

Depth (ft)	Locatio n	Soil Type	pH	Minimum Resistivity (ohm-cm)	Soluble Sulfate (ppm)	Soluble Chloride (ppm)
0 to 3	B-1	Silty Sand (SM)	7.33	3,728	14	5
10 to16	B-1	Clayey Sand (SC)	7.91	3,195	5	9

These values are all outside the Caltrans threshold limits. Consequently, the site would be considered to be a non-corrosive environment with respect to foundations.

These values are generally representative of an environment that would be mildly corrosive to buried unprotected metals. An example of the potential soil corrosion is provided by utilizing methods provided in Caltrans California Test 643, "Method for Estimating the Service Life of Steel Culverts". The method indicates a 1-gauge steel zinc-coated culvert is estimated to have a maintenance-free service life (years to perforation) provided in Table 5.6-2. Therefore, if project improvements will involve metal that comes into contact with the on-site soil (e.g. steel barriers, etc.), the design should consider the potential soil corrosiveness described.

TABLE 5.6-2
ESTIMATED SERVICE LIFE OF BURIED STEEL
"UTILIZING CALIFORNIA TEST METHOD 643"

Depth (ft)	Location	Maintenance-Free Service Life (Years to Perforation)
0 to 3	B-1	42
10 to 16	B-1	40

5.7 EARTHWORK

5.7.1 Grading

All grading operations should be performed in accordance with the project specifications and within the intent of applicable items of Section 19 of the Caltrans Standard Specifications, 2010. It is recommended that relative compaction be based on dry weight methodology for Caltrans 216 and 231. Where culvert and wingwall fill is place against the existing Travers CreekCreek slopes, benches having horizontal dimensions of 2 vertical should be excavated to remove unsuitable/disturbed soil and expose competent subgrade.

5.7.2 Engineered Fill

All engineered fill soils should be non-expansive, relatively granular soil that is nearly free of, rubble, organics or other deleterious debris, and less than 3 inches in maximum dimension. Excavated on-site soil may be used as engineered fill, provided they meet the above criteria. Any imported soil shall meet also meet these criteria. Imported fill materials to be used for engineered fill should be sampled and tested by a representative of the project Geotechnical Engineer prior to being transported to the site.

6 ADDITIONAL SERVICES

6.1 DESIGN REVIEW AND CONSULTATION

It is recommended that **TECHNICON** be retained to review those portions of the contract drawings and specifications that pertain to earthwork, foundations, and pavements prior to finalization to determine whether they are consistent with our recommendations.

6.2 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that a representative of **TECHNICON** observe the excavation, earthwork, foundation, and pavement phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design. **TECHNICON** can conduct the necessary field testing and provide results on a timely basis so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of the observations, field testing, and conclusions regarding the conformance of the completed work to the intent of the plans and specifications will be provided. This additional service is not part of this current contractual agreement. **TECHNICON** firm will not be responsible for establishing or confirming building or foundations depths or locations unless retained to do so.

7 LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of our field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of the variations between borings may not become evident until construction. If variations or undesirable conditions are encountered during construction, our firm should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. The unexpected conditions frequently require additional expenditures for proper construction of the project. **TECHNICON Engineering Services, Inc.** will not assume any responsibility for errors or omissions if the final extent and depth of earthwork is not determined by our firm at the time of construction due to said variations or undesirable conditions encountered.

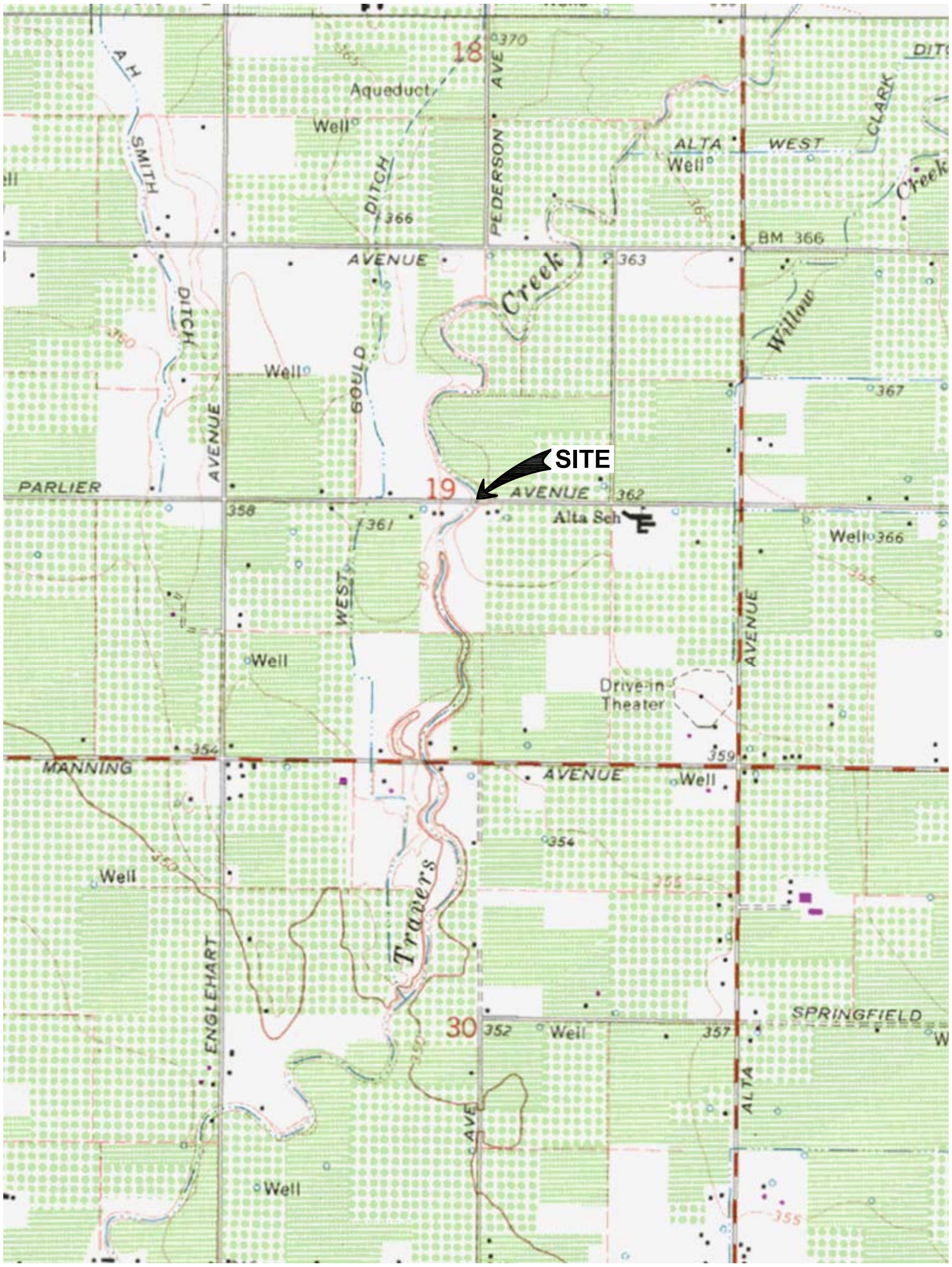
If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes, or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing. Such conditions may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. This report does not relieve the contractors of responsibility for temporary excavation construction, bracing and shoring in accordance with CAL OSHA requirements.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. This report should not be construed as an environmental audit or study.

This report has been prepared for the sole use by Cornerstone Structural Engineering Group and their designated consultants for the on Parlier Avenue Bridge Replacement at Travers Creek near Reedley, in Fresno County, California. Recommendations presented herein should not be extrapolated to other areas or used for other projects without prior review. This report has been prepared with the intent that the firm of **TECHNICON** will be performing the construction testing and observation for the complete project. If, however, another firm or individual(s) should be retained or employed to use this Foundation Report for the purpose of construction testing and observation, notice is hereby given that **TECHNICON** will not assume any responsibility for errors or omissions, if any, which may occur and which could have been avoided, corrected, or mitigated if **TECHNICON**, had performed the work. This notice also applies to the misuse or misinterpretation of the conclusions and recommendations outlined in this report. Furthermore, the other firm or individual(s) performing construction testing and observation should accept transfer of responsibility of the work, as required by the California Building Code, in writing to the project owner and **TECHNICON**. The firm accepting transfer of responsibility should perform additional investigation(s) as may be necessary to develop their own conclusions, evaluations, and recommendations for design and construction.

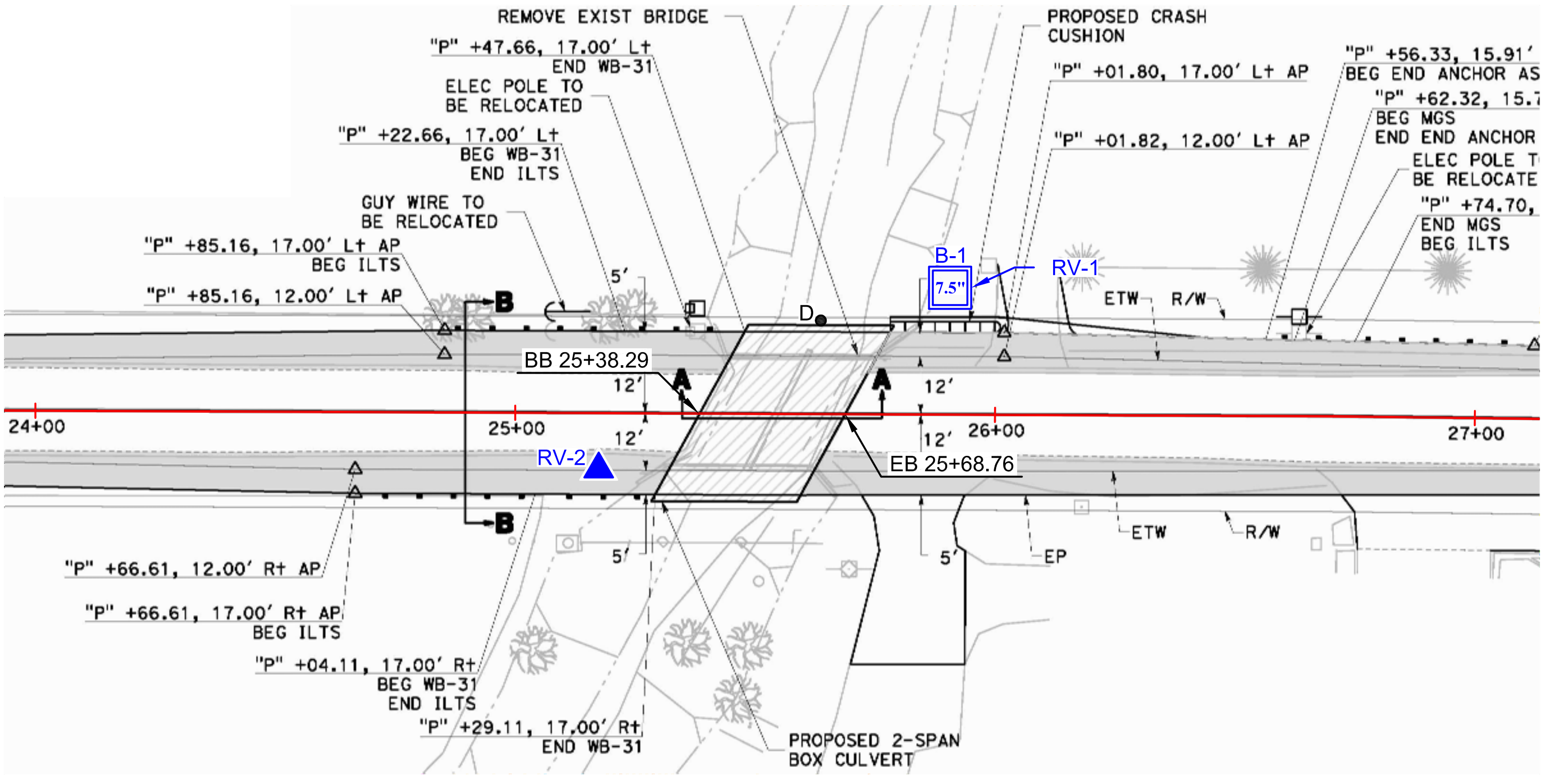
FIGURE 1



LAT.: 36.6113°N, LONG.: 119.4043°W, 19-T15S-R24E, MDB&M, USGS MAP: REEDLEY, DATE: 1966, PHOTO REV.: 1981

NORTH

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 160597	VICINITY MAP PARLIER AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK COUNTY OF FRESNO, CALIFORNIA	FIGURE 1 NTS
	SOURCE: USGS TOPOGRAPHIC MAPS		



B-1 7.5" =SOIL BORING LOCATION

RV-2 =R-VALUE LOCATIONS

D =DYNAMIC CONE PENETRATION TEST

NORTH

SCALE: 1"=40'

0' 20' 40'

TECHNICON ENGINEERING SERVICES, INC.	PROJECT: 160597	DATE: 9/1/16	SITE MAP PARLIER AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK COUNTY OF FRESNO, CALIFORNIA
	SOURCE: CORNERSTONE	APPROVED BY: SA	FIGURE 2

LOG TEST BORINGS

APPENDIX A

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

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)			
	GP-GC	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)		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)		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

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (R) R-Value (CTM 301)
- (SA) Sieve Analysis
- (SE) Sand Equivalent (CTM 217)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)

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

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 with finger pressure.

BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A R RW RC P	Auger Boring (hollow or solid stem bucket) Rotary drilled boring (conventional) Rotary drilled with self-casing wire-line Rotary core with continuously-sampled, self-casing wire-line Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hard driven(1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778)
	O	Other (note on LOTB)

Note: Size in inches

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 in.)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Greater than 50

MOISTURE

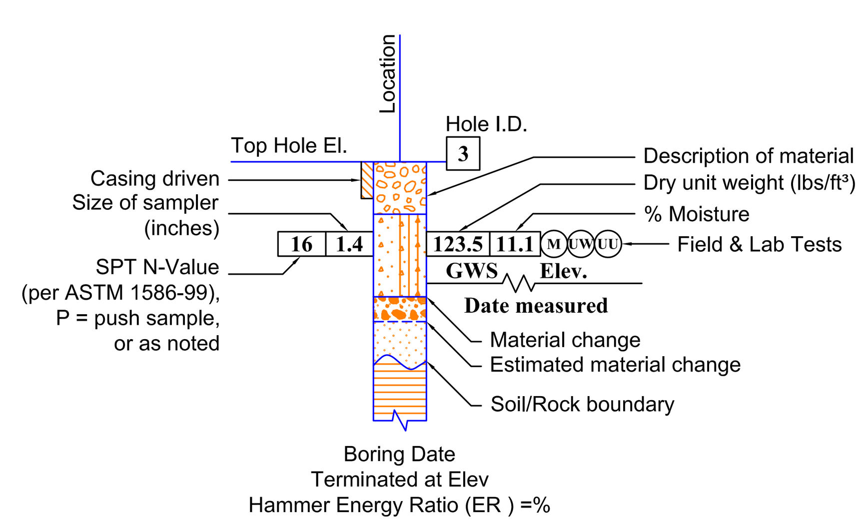
Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

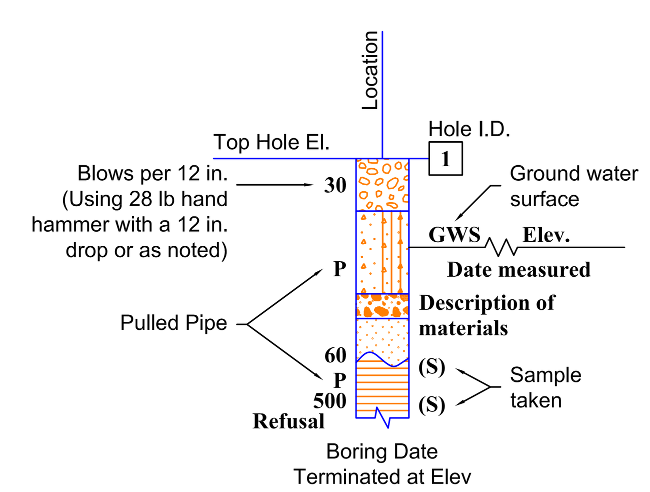
Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5% - 10%
Little	15% - 25%
Some	30% - 45%
Mostly	50% - 100%

PARTICLE SIZE

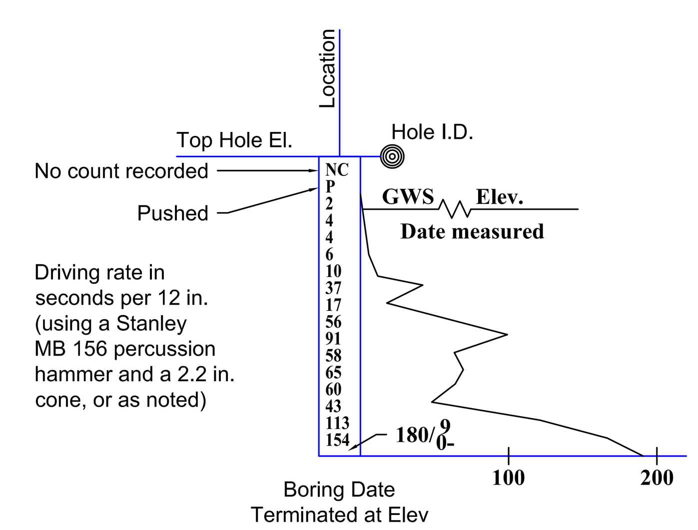
Description		Size (in.)
Boulder		Greater than 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		Less than 1/300



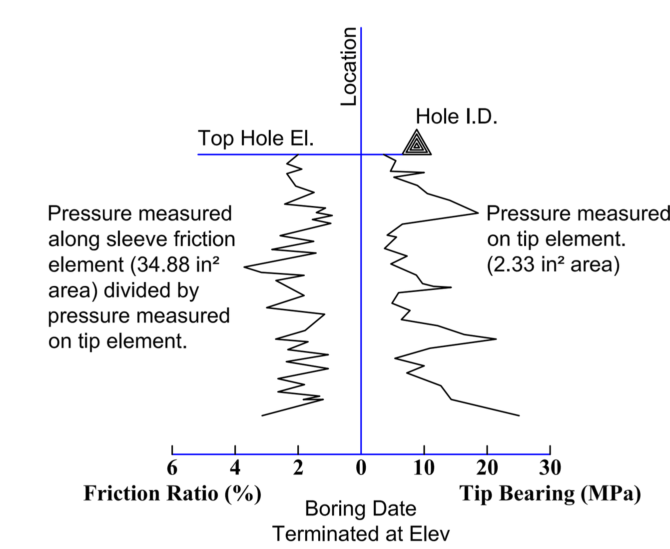
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) BORING

DESIGNED	S. Athwal	DATE 7/20/16
DRAWN	M. Heraz	9/1/16
CHECKED	S. Plauson	
REVISION		

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

TECHNICON ENGINEERING SERVICES, INC.
4539 N. BRAWLEY AVENUE - SUITE 108
FRESNO, CALIFORNIA 93722

PROJECT
PARLIER AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK
COUNTY OF FRESNO, CA

Road No. Bridge No.

THE COUNTY OF FRESNO 1856

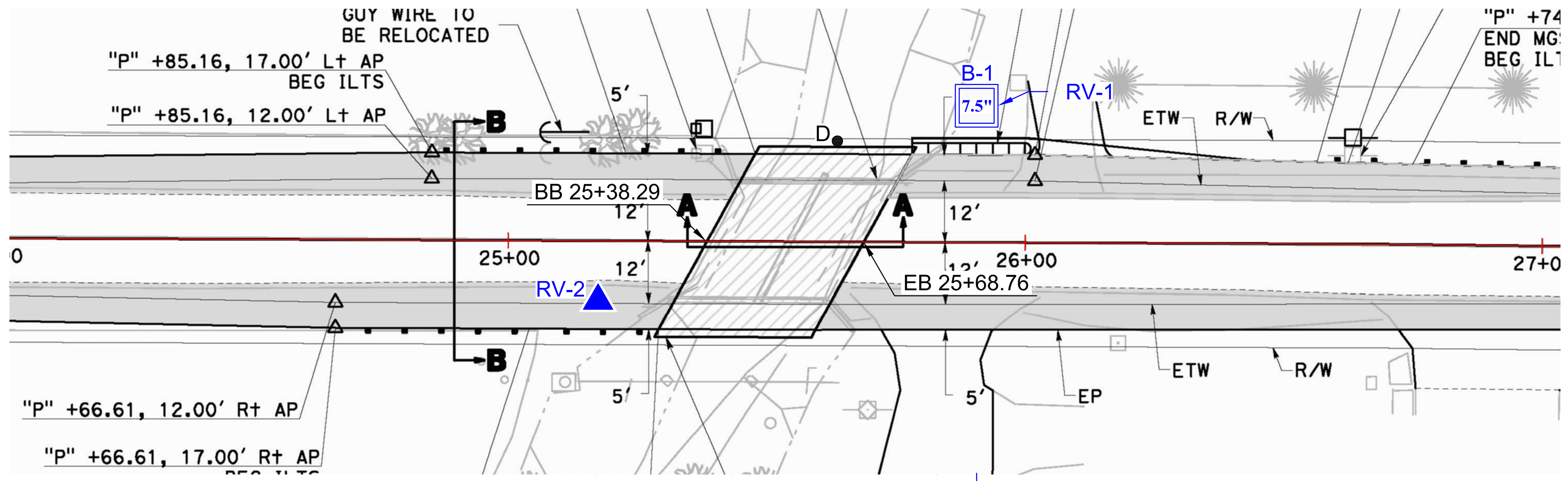
DEPARTMENT OF PUBLIC WORKS & PLANNING

LOG OF TEST BORINGS

Drawing No. 160597 Sheet No. 1 Total 2

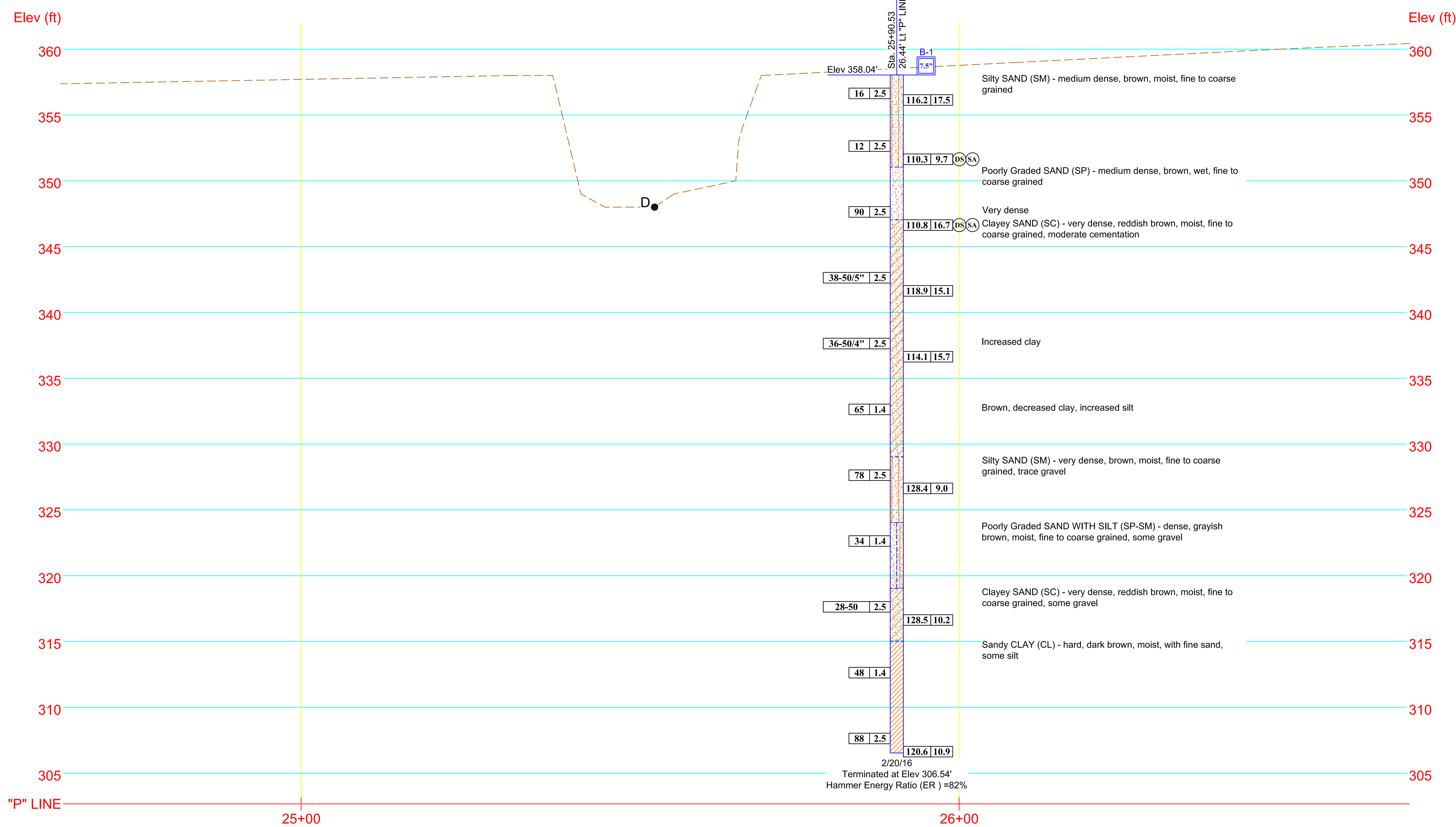
NOTES:

1. Purge water from the canal ranging at Elev 351.04' to Elev 347.04'.
2. Hammer type - CME Automatic 140 pound with 30-inch drop for all samples.
3. All dimensions are in feet unless otherwise noted.
4. This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Preparation Manual (June 2010).



PLAN

SCALE: 1"=20'



PROFILE

HOR. SCALE: 1"=10'

VERT. SCALE: 1"=5'

DESIGNED S. Athwal		7/20/16
DRAWN M. Heraz		9/1/16
CHECKED S. Plauson		

DATE

REVISION

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

Scale in Feet

0 20 40

TECHNICON ENGINEERING SERVICES, INC.
4539 N. BRAWLEY AVENUE - SUITE 108
FRESNO, CALIFORNIA 93722

PROJECT

PARLIER AVENUE BRIDGE REPLACEMENT AT TRAVERS CREEK

COUNTY OF FRESNO, CA

Road No. Bridge No.

THE COUNTY OF FRESNO 1856

DEPARTMENT OF PUBLIC WORKS & PLANNING

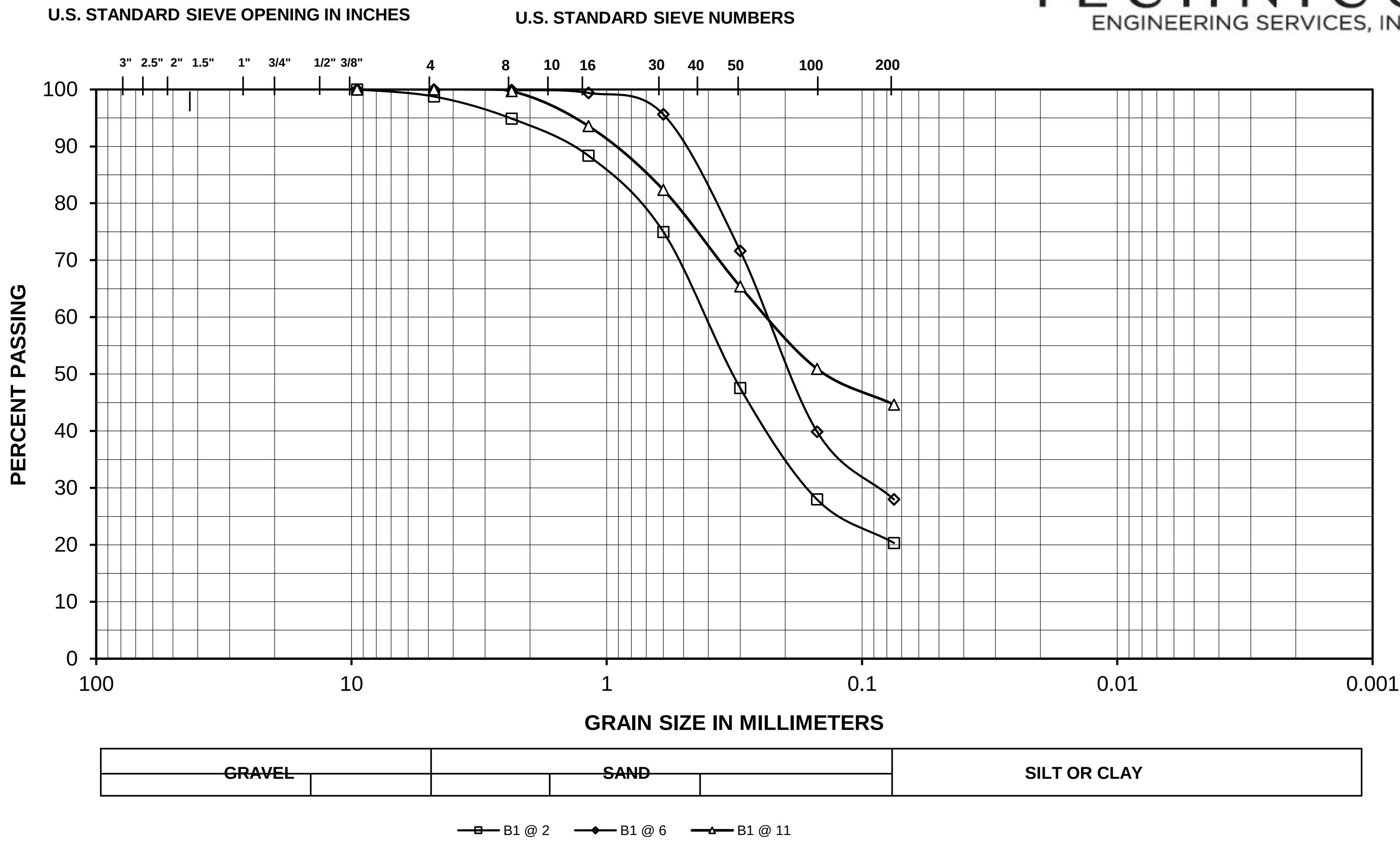
LOG OF TEST BORINGS

Drawing No. 160597 Sheet No. 2 Total 2

LABORATORY TESTS

APPENDIX B

TECHNICON
ENGINEERING SERVICES, INC.



Sample No.	Classification	% Gravel	% Sand	% Fines	% Moist.	LL	PL	PI	Project	Parlier Avenue Bridge
B1 @ 2	Silty Sand (SM)	1.2	78.4	20.3	17.5					Fresno County, CA
B1 @ 6	Silty Sand (SM)	0.0	72.0	28.0	19.4				TES No.	160597
B1 @ 11	Clayey Sand (SC)	0.1	55.2	44.6	10.2				Date	8/3/2016

TECHNICON
ENGINEERING SERVICES, INC.

Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Parlier Avenue Bridge	Technician	K.W.
	Fresno County, CA	Date	8/3/2016
TES No.	160597	Sample No.	B1 @ 2
Lab No.		Remarks	Silty Sand (SM)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	170.2	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash		1 1/2"	33.0 (15.0)
	136.28	2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	2.1	1.2	1.2	98.8	
#8	8.7	3.9	5.1	94.9	
#16	19.8	6.5	11.6	88.4	
#30	42.6	13.4	25.0	75.0	
#50	89.3	27.4	52.5	47.5	
#100	122.6	19.6	72.0	28.0	
#200	135.6	7.6	79.7	20.3	
Pan	136.1				

TECHNICON
ENGINEERING SERVICES, INC.

Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Parlier Avenue Bridge	Technician	K.W.
	Fresno County, CA	Date	8/4/2016
TES No.	160597	Sample No.	B1 @ 6
Lab No.		Remarks	Silty Sand (SM)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	182.3	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash	133.9	1 1/2"	33.0 (15.0)
		2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	0.0	0.0	0.0	100.0	
#8	0.2	0.1	0.1	99.9	
#16	1.1	0.5	0.6	99.4	
#30	8.0	3.8	4.4	95.6	
#50	51.7	24.0	28.4	71.6	
#100	109.6	31.8	60.1	39.9	
#200	131.3	11.9	72.0	28.0	
Pan	133.9				

4539 N. Brawley Avenue, #108, Fresno, CA 93722
Phone (559) 276-9311 Fax (559) 276-9344

TECHNICON
ENGINEERING SERVICES, INC.

Sieve Analysis for Coarse and Fine Aggregate
ASTM C 136

Project	Parlier Avenue Bridge	Technician	K.W.
	Fresno County, CA	Date	8/31/2016
TES No.	160597	Sample No.	B1 @ 11
Lab No.		Remarks	Clayey Sand (SC)

	Weight (lbs. or grams)	Maximum Sieve Size	Minimum Weight of Test Specimen, lbs. (kg)
Total Dry Sample + Tare Wt.		Sand	1.0 (0.5)
Tare Weight		3/8"	2.0 (1.0)
Total Dry Sample Wt.	133.2	1/2"	4.0 (2.0)
Initial Weight Fine Aggregate Before Wash		3/4"	11.0 (5.0)
		1"	22.0 (10.0)
Final Weight Fine Aggregate After Wash		1 1/2"	33.0 (15.0)
	75.03	2"	44.0 (20.0)

Sieve Size	Cumulative Weight Retained	Individual % Retained	Cumulative % Retained	Cumulative % Passing	Specs.
3 in.		0.0	0.0	100.0	
2 1/2 in.		0.0	0.0	100.0	
2 in.		0.0	0.0	100.0	
1 1/2 in.		0.0	0.0	100.0	
1 in.		0.0	0.0	100.0	
3/4 in.		0.0	0.0	100.0	
1/2 in.		0.0	0.0	100.0	
3/8 in.		0.0	0.0	100.0	
#4	0.0	0.0	0.0	100.0	
#8	0.4	0.3	0.3	99.7	
#16	8.6	6.1	6.4	93.6	
#30	23.5	11.2	17.6	82.4	
#50	46.1	17.0	34.6	65.4	
#100	65.4	14.5	49.1	50.9	
#200	73.7	6.3	55.4	44.6	
Pan	75.03				

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Phone (559) 276-9311 Fax (559) 276-9344

TECHNICON
ENGINEERING SERVICES, INC.

Method for Estimating the Service Life of Steel Culverts
Caltrans California Test 643

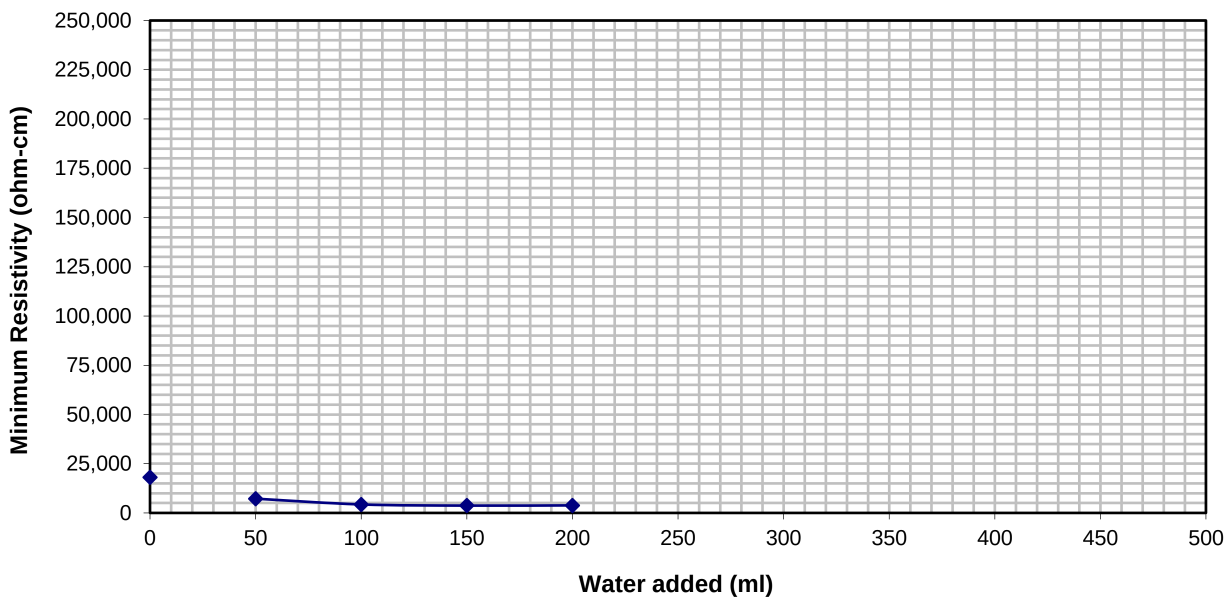
Project Name	Parlier Avenue Bridge	Sample Location	B-1 @ 0'-3'
Project Number	160597	Test Date	8/10/2016
Sample Date	7/20/2016	Tested By	K.W.
Sampled By	S. Athwal	Material Description	Silty Sand (SM)

Sample Condition	**As Received**	**Minimum Resistivity**					
Water Added (ml)	0	50	100	150	200		
Resistance (ohm)	17,000	6,800	4,050	3,500	3,600		
Resistivity (ohm-cm)	18,105	7,242	4,313	3,728	3,834		

Minimum Resistivity (ohm-cm)	**3,728**	**Field Resistivity (ohm-cm)**

pH = 7.33 EC =

Box Constant=1.065



Years to perforation*	42

* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts

Method for Estimating the Service Life of Steel Culverts
Caltrans California Test 643

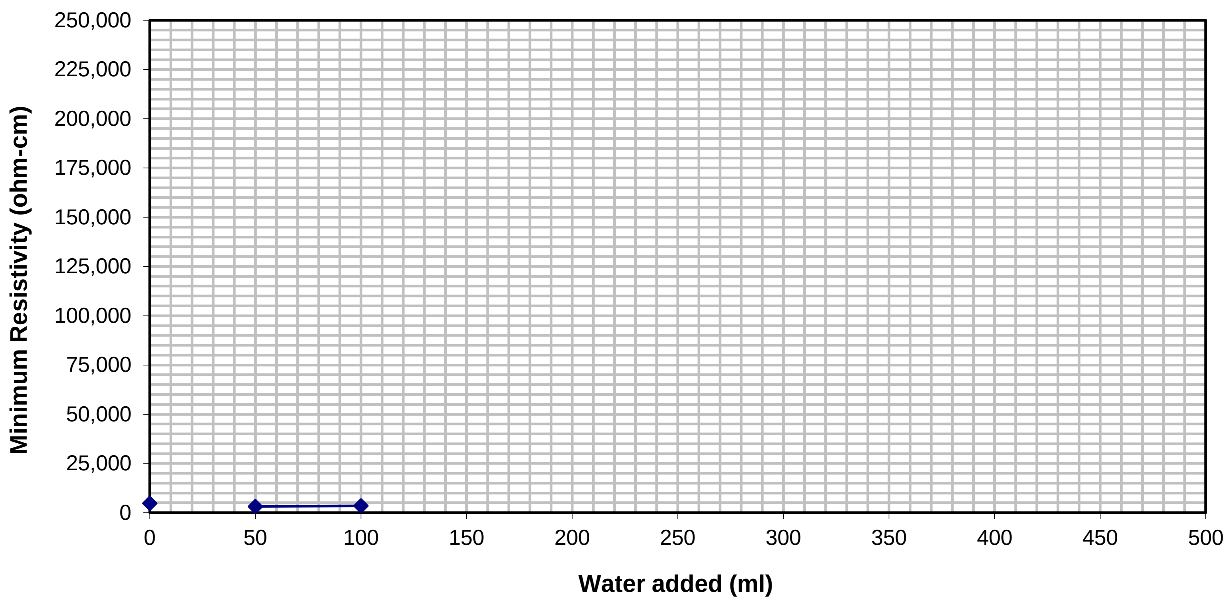
Project Name	Parlier Avenue Bridge	Sample Location	B-1 @ 16'
Project Number	160597	Test Date	8/2/2016
Sample Date	7/20/2016	Tested By	K.W.
Sampled By	S. Athwal	Material Description	Clayey Sand (SC)

Sample Condition	As Received	Minimum Resistivity					
Water Added (ml)	0	50	100				
Resistance (ohm)	4,450	3,000	3,250				
Resistivity (ohm-cm)	4,739	3,195	3,461				

Minimum Resistivity (ohm-cm)	**3,195**	**Field Resistivity (ohm-cm)**

pH = 7.91 **EC =**

Box Constant=1.065



Years to perforation*	40

* Caltrans California Test 643 - Method for Estimating the Service Life of Steel Culverts

TECHNICON
ENGINEERING SERVICES, INC.

Chemical Analysis
SO_4 - Modified Caltrans 417 & CL - Modified Caltrans 417/422

Project	Parlier Avenue Bridge	Technician	K. W
	Fresno County, CA	Date	8/5/2016
TES No.	160597	Remarks	Sitly Sand (SM)

Sample Location	Soluble Sulfate SO_4-S		Soluble Chloride Cl	
B-1 @ 0'-3'	12.3	mg/Kg	1.8	mg/Kg
B-1 @ 0'-3'	14.9	mg/Kg	1.8	mg/Kg
B-1 @ 0'-3'	15.8	mg/Kg	1.8	mg/Kg
Average	**14.00**	**mg/Kg**	**5.00**	**mg/Kg**

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Phone (559) 276-9311 Fax (559) 276-9344

TECHNICON
ENGINEERING SERVICES, INC.

Chemical Analysis
SO_4 - Modified Caltrans 417 & CL - Modified Caltrans 417/422

Project	Parlier Avenue Bridge	Technician	K. W
	Fresno County, CA	Date	8/5/2016
TES No.	160597	Remarks	Clayey Sand (SC)

Sample Location		Soluble Sulfate SO_4-S		Soluble Chloride Cl	
B-1 @ 16'		1.8	mg/Kg	8.9	mg/Kg
B-1 @ 16'		1.7	mg/Kg	8.9	mg/Kg
B-1 @ 16'		1.5	mg/Kg	8.9	mg/Kg
	Average	**5.00**	**mg/Kg**	**9.00**	**mg/Kg**

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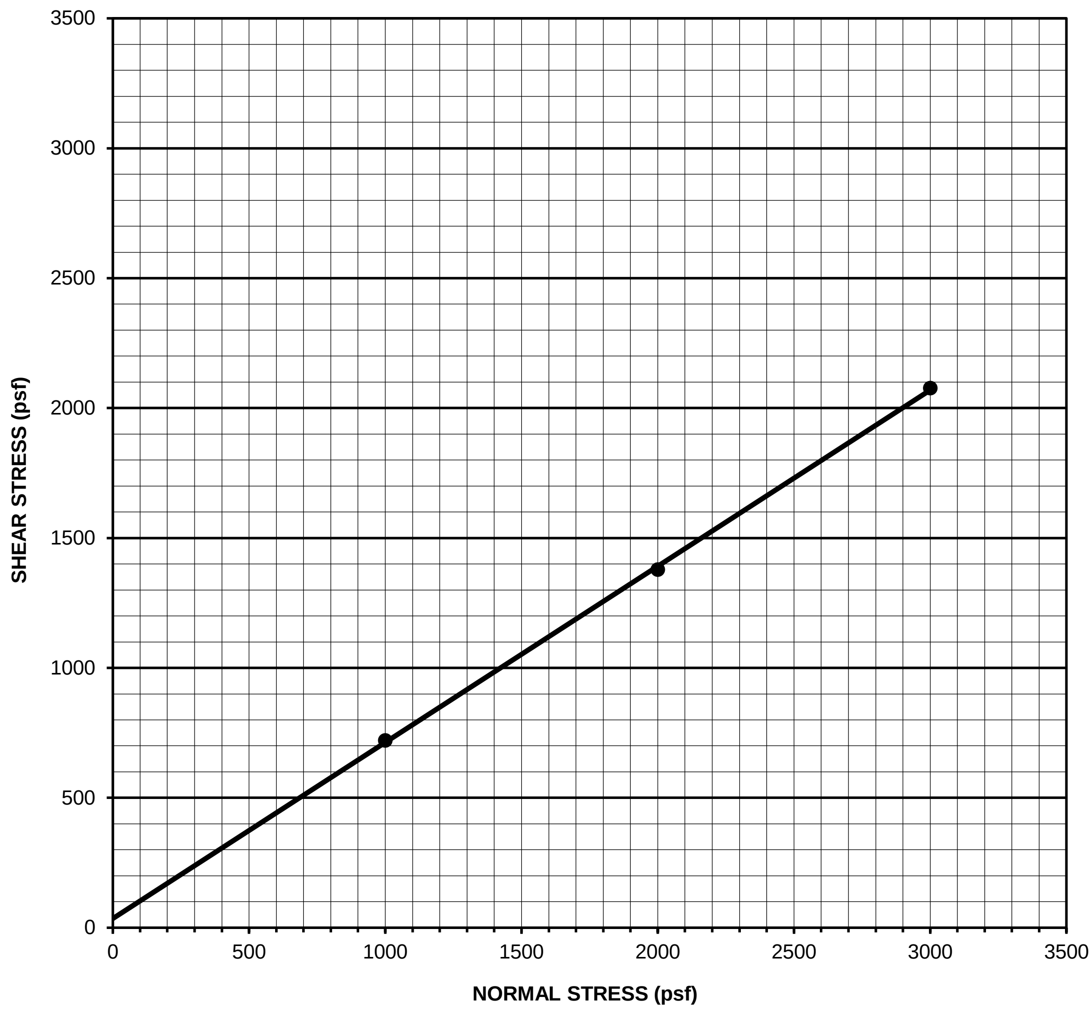
TECHNICON
ENGINEERING SERVICES, INC.

Direct Shear Test

ASTM D3080



Project	Parlier Avenue Bridge
TES No.	160597
Sample Date	7/15/2016
Sample No.	B-1 @ 6'
Description	Silty SAND (SM)

Cohesion (psf)	**40**
Internal Friction Angle (ϕ)	**34**

Specimen	A	B	C	D	E
Dry Density (pcf)	110.3	110.3	110.3	---	---
Initial Water Content (%)	9.7	9.7	9.7	---	---
Final Water Content (%)	18.1	17.1	17.4	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	720	1378	2076	---	---

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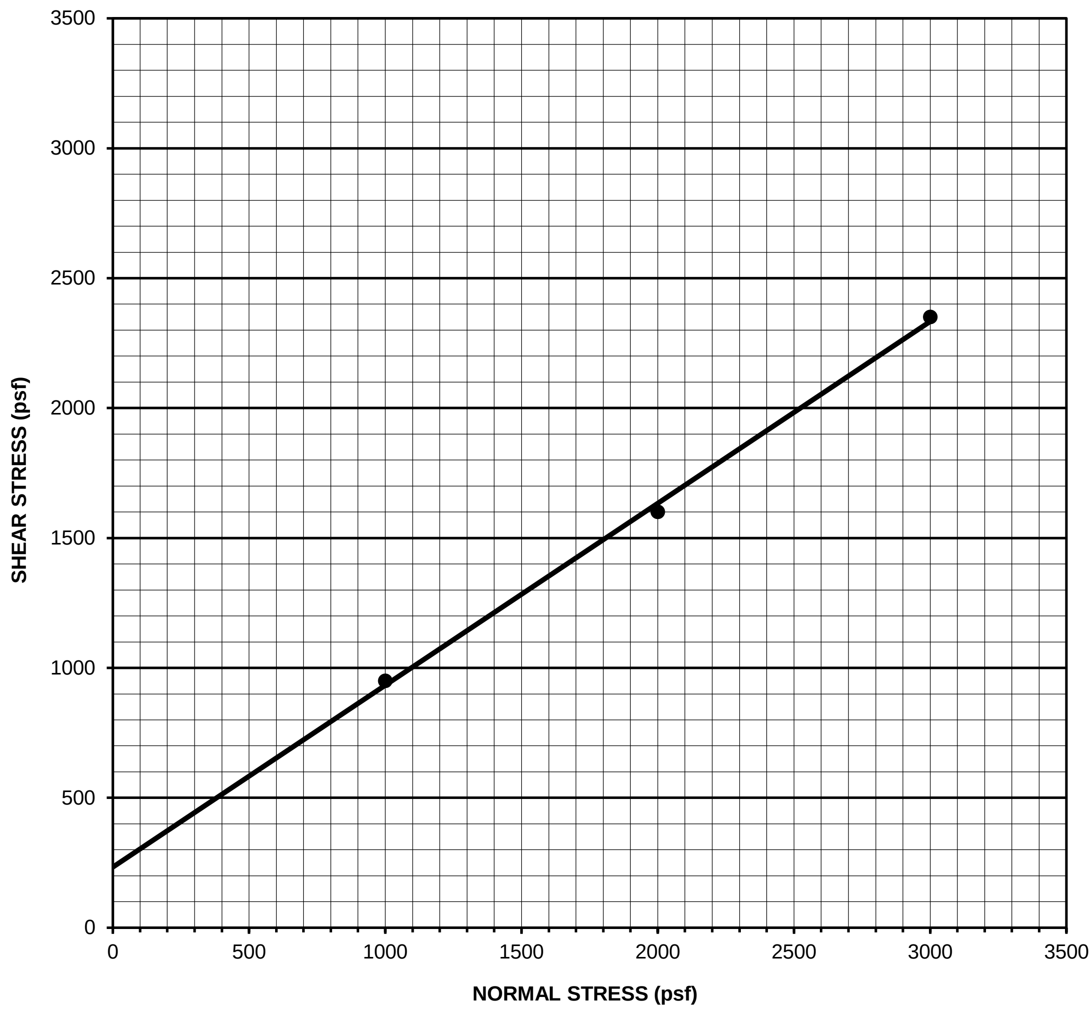
Direct Shear Test

ASTM D3080



Project	Parlier Avenue Bridge
TES No.	160597
Sample Date	7/20/2016
Sample No.	B-1 @ 11'
Description	Clayey Sand (SC)

Cohesion (psf)	**230**
Internal Friction Angle (ϕ)	**35**

Specimen	A	B	C	D	E
Dry Density (pcf)	113.1	113.1	113.1	---	---
Initial Water Content (%)	18.4	18.4	18.4	---	---
Final Water Content (%)	20.2	22.5	22.4	---	---
Normal Stress (pcf)	1000	2000	3000	---	---
Maximum Shear (pcf)	950	1600	2350	---	---

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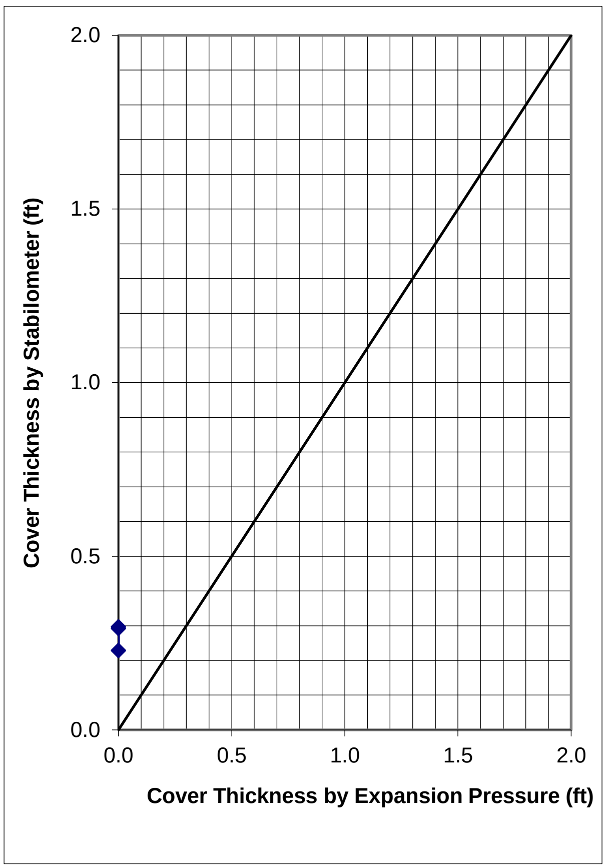
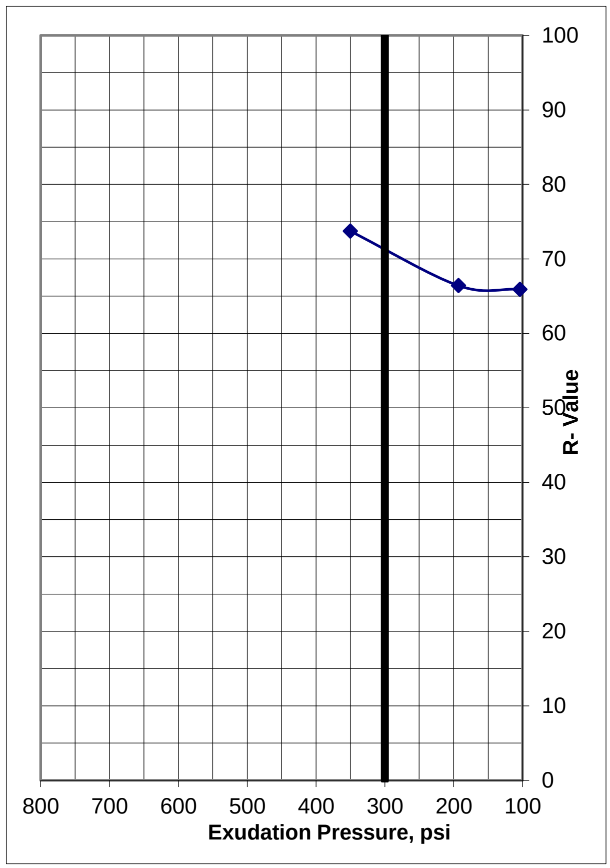
Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Parlier Avenue Bridge	Lab ID Number	16-354
Project Number	160597	Sample Location	RV-1 @ 0'-2'
Sample Date	7/20/16	Tested By	J.A.
Sampled By	S. Athwal	Date Tested	7/29/2016
Material Description	Silty Sand (SM)		





Specimen	1	2	3
Exudation Pressure, psi	104	193	350
Moisture at Test, %	9.8	9.3	8.7
Dry Density, pcf	125.0	122.4	125.6
Expansion Pressure, psf	0	0	0
Thickness by Stabilometer, ft.	0.3	0.3	0.2
Thickness by Expansion Pressure, ft.	0.0	0.0	0.0
R-Value by Stabilometer	66	66	74
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	72		

Controlling R-Value	**72**

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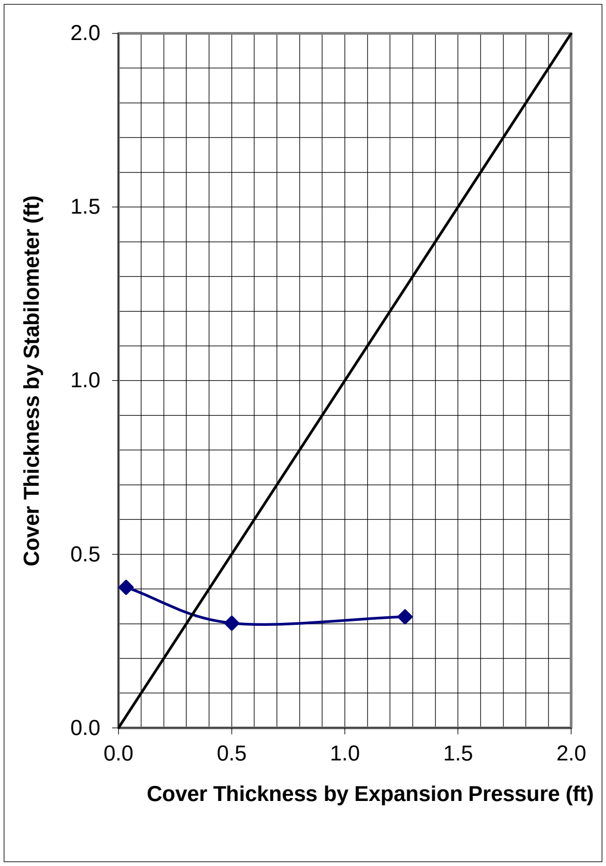
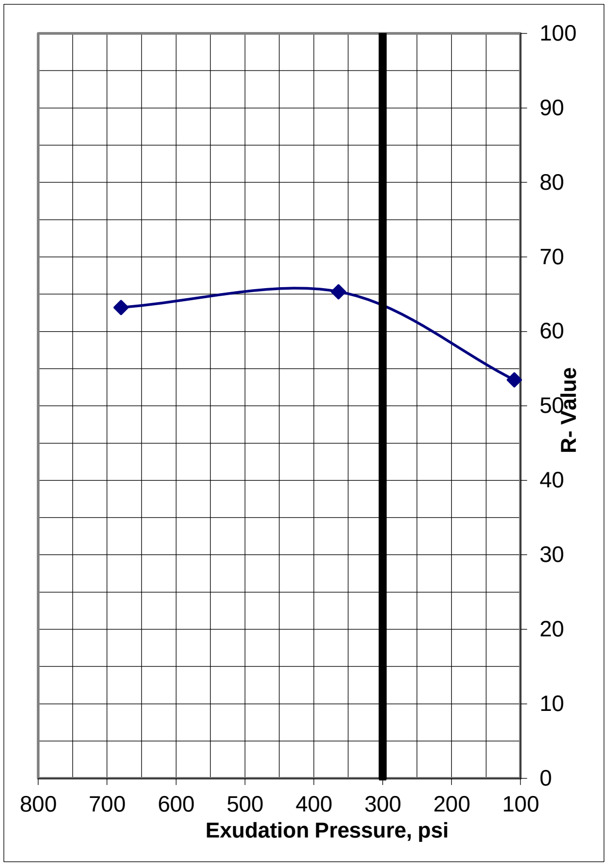
Resistance *R* - Value and Expansion Pressure of Compacted Soils

ASTM D2844-94, Cal 301

Project Name	Parlier Avenue Bridge	Lab ID Number	16-354
Project Number	160597	Sample Location	RV-2 @ 0'-1.5'
Sample Date	7/20/16	Tested By	J.A.
Sampled By	S. Athwal	Date Tested	7/29/2016
Material Description	Silty Sand (SM)		





Specimen	1	2	3
Exudation Pressure, psi	109	364	680
Moisture at Test, %	12.8	11.2	10.9
Dry Density, pcf	112.1	111.8	114.2
Expansion Pressure, psf	4	65	165
Thickness by Stabilometer, ft.	0.4	0.3	0.3
Thickness by Expansion Pressure, ft.	0.0	0.5	1.3
R-Value by Stabilometer	53	65	63
R-Value by Expansion Pressure (TI=4.5)	NA		
R-Value at 300 psi Exudation Pressure	63		

Controlling R-Value	**63**

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DYNAMIC CONE PENETRATION TEST

APPENDIX C

TECHNICON ENGINEERING SERVICES, INC.

Project Name:	Parlier Avenue Bridge Replacement at Travers Creek		Date:	8/29/2016
Project #	160597		Hammer Weight:	15 lbs
Location:	Fresno County, CA		Field Engineer:	Sarbjit Athwal

Depth (in)	Depth (ft)	No. of Blows
1.75	0.15	4
3.5	0.29	7
5.25	0.44	9
7	0.58	13
8.75	0.73	15
10.5	0.88	20
12.25	1.02	22
14	1.17	23
15.75	1.31	31

**Note: Depth Measured from the Bottom of the Canal

DEASIGN ARS CURVE AND

SEISMIC ANALYSIS

APPENDIX D

Project: Parlier Avenue Bridge Replacement at Travers Creek
Location: Fresno County
TES #: 160597

TECHNICON ENGINEERING SERVICES, INC.

Site Information:

Latitude:	36.61125
Longitude:	-119.4043
V_{s30} (m/s)	340
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Distance (km)[1] =	122

Governing Curve:

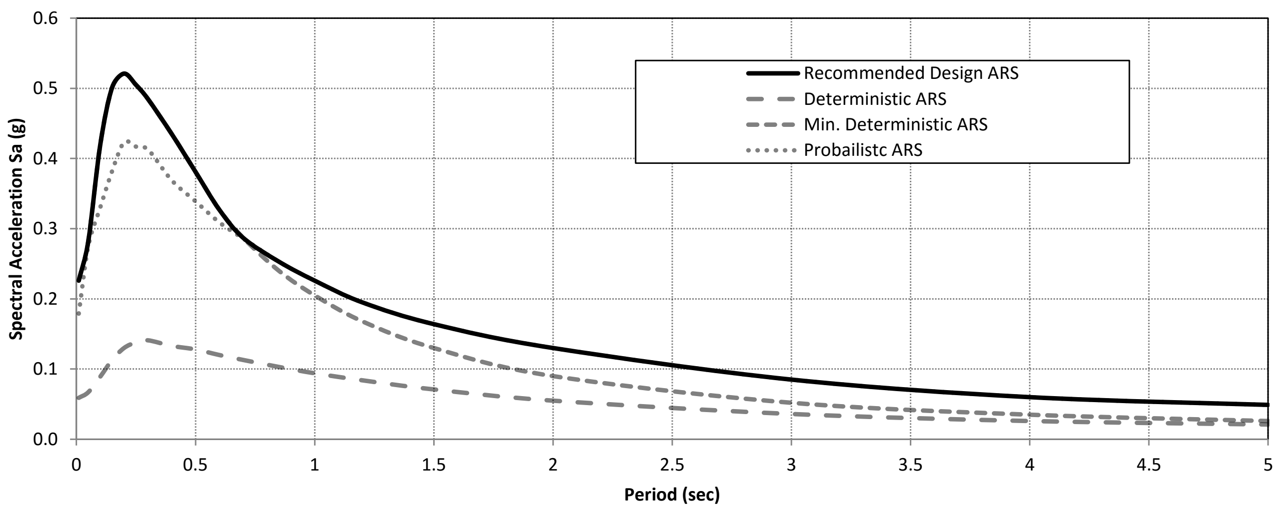
- ☐ Caltrans ARS OnLine Deterministic
- ☐ Minimum Deterministic
- ☐ Caltrans ARS OnLine Probabilistic
- ☑ Envelope of:
  - ☐ Caltrans ARS OnLine Deterministic
  - ☑ Caltrans Minimum Deterministic
  - ☑ Caltrans ARS OnLine Probabilistic

Recommended Response Spectrum

Period (sec)	SA Base Spectrum (g)	Adjusted for Basin Effect	Adjusted for Neaf Fault Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.226	-	-	0.226
0.1	0.419	-	-	0.419
0.2	0.521	-	-	0.521
0.3	0.485	-	-	0.485
0.5	0.381	-	-	0.381
1.0	0.226	1.000	1.000	0.226
2.0	0.13	1.000	1.000	0.130
3.0	0.085	1.000	1.000	0.085
4.0	0.06	1.000	1.000	0.060
5.0	0.049	1.000	1.000	0.049

RECOMMENDED ARS CURVE

Envelope of Deterministic and Probabilistic Curves (5% Damping)



Sources:

Caltrans Seismic Design Criteria, Version 1.7, April 2013
Caltrans Geotechnical Services Design Manual, August 2009
Caltrans ARS Online tool (v2.3.07, http://dap3.dot.ca.gov/shake_stable/)
USGS 2008 Interactive Daggregations (https://geohazards.usgs.gov/deaggint/2008/index.php)

May 2, 2024
Kleinfelder Project No.: 24005507.001A

Mr. Mark Weaver
Cornerstone Structural Engineering Group
986 W. Alluvial Avenue, Suite 201
Fresno, California 93711
Phone: (559) 320-3200
Email: mweaver@cseg.com

Subject: **Final Design Memorandum**
Parlier Ave Bridge Replacement at Traverse Creek
Fresno County, California

Reference: **Foundation Report, Parlier Ave Bridge Replacement at Traverse Creek, Reedley, Fresno County, California, TECHNICON Engineering Services, Inc., File No 160597.001, dated September 21, 2016**

Dear Mr. Weaver:

In accordance with your request, Kleinfelder completed additional engineering analysis and prepared this final design memorandum to support the PS&E for the reinforced concrete box culvert (RCB) replacement on Parlier Avenue at the Traverse Creek in Fresno County, California. The memorandum serves to supplement the above referenced Foundation Report (FR) for the 100% submittal of the PS&E and construction phases of the project. In addition, the letter serves to maintain continuity of the Geotechnical Engineer of Record through the PS&E phase.

PROJECT UNDERSTANDING

An understanding of the project is based on telephone conversations and email correspondence with Regina Barton and Mark Weaver of Cornerstone Structural Engineering Group (CSEG) and Mr. Joseph Harrel of the County of Fresno. The above referenced Foundation Report (FR) was previously prepared to support the design of a bridge replacement located on Parlier Avenue at Traverse Creek. The replacement bridge is anticipated to consist of a reinforced concrete box culvert (RCB) with a closed bottom and utilizing retaining walls at the approaches.

Tables 1 through 3 present foundation design data and foundation design loads provided by CSEG and used for this geotechnical evaluation. Referenced elevations are based on elevations provided in General Layout and Foundation Plan Sheets, 100% Submittal, dated November 10, 2017.

Table 1
Box Culvert Foundation Data

Road Finished Grade Elev. (ft)	Bottom of Foundation Elev. (ft)	Foundation Size[1]		S_p[2]
		B	L	
358.1	346.75	53	26.3	1”

[1] B is measure perpendicular to the road and L is measured parallel to the road.
[2] Permissible settlement under service load

Table 2
Box Culvert Foundation Load Data

Maximum Service (Total) Bearing Pressure (ksf)	Maximum Service (Permanent) Bearing Pressure (ksf)	Maximum Strength Bearing Pressure (ksf)	Maximum Extreme Bearing Pressure (ksf)
1.05	0.593	1.65	0.593

Table 3
Retaining Wall Foundation Data

Design Height (ft)	Bottom of Footing Elev. (ft)	Min. Footing Embed. Depth (ft)	Effective Foundation Width, B' (ft)[1]		S_p[2]	Maximum Service (Total) Bearing Pressure (ksf)
			Strength 1A Limit State	Strength 1B Limit State		
With Toe						
5.79	350.8	3.11	3.24	3.66	1”	1.50
9.79	346.8	4.60	3.98	4.74	1”	2.21
13.46	342.8	5.79	5.58	6.68	1”	2.66
Without Toe						
5.79	350.8	9.29	3.04	3.36	1”	1.81
9.79	346.8	9.79	3.72	4.38	1”	2.69
13.46	342.8	13.46	5.18	6.12	1”	3.40

[1] B is measure perpendicular to the wall.
[2] Permissible settlement under service load

PURPOSE AND SCOPE OF SERVICES

The purpose of this final design memorandum is to update the previous signed Foundation Report and address the following supplemental items:

- Perform a site visit to observe current site conditions.
- A summary of the updated project information and design details including loading information.
- Recommended gross and net permissible contract stress associated with tolerable settlements and bearing capacity and design footing elevations of spread footing foundation for the closed bottom area of the RCB.
- Recommended gross and net permissible contract stress associated with tolerable settlements and bearing capacity for retaining walls.
- Recommendations to stabilize soft or yielding subgrade soils with options for recompaction, replacement with aggregate base, and use of geotextile reinforcement.

SITE VISIT

Kleinfelder observed the site conditions on May 8th, 2023, at the Parlier Avenue and Traverse Creek crossing. The site conditions remained essentially unchanged from the previous field exploration completed on July 20, 2016. Parlier Avenue is a 2-lane existing reinforced bridge that is approximately 28 feet long by 21.5 feet wide. The canal was unlined and flowed with a water depth of approximately 4 to 5 feet.

CONCLUSIONS AND SUPPLEMENTAL RECOMMENDATIONS

It is Kleinfelder's opinion that the recommendations presented in the FR may be used for PS&E and construction phases of the project along with the following supplemental geotechnical data and recommendations.

<u>Box Culvert Bearing and Settlement</u>

The nominal bearing capacity, which is based solely on soil strength, for a box culvert is extremely high (greater than 32 ksf). Table 4 "Foundation Data Table" provides the bearing resistance and settlement based on the design loads and dimensions provided.

Table 4
Footing Data Table
(Double Box Culvert)

Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth (ft)	Total Permissible Support Settlement (inches)	Service Limit State	Strength or Construction Limit State ϕb=0.45
L	B				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
53	26.3	346.75	1	1	4.3	14.5

Based on the Gross Maximum Bearing Stress (Service) of 1.05 ksf provided by CSEG, the total settlement of the RCB is approximately 0.25-inch. Differential settlement is anticipated to be reduced to half of the total settlement across the length/width of the RCB.

Retaining Wall Bearing and Settlement

Table 5 "Foundation Data Table" provides the bearing resistance and settlement of bridge approach retaining walls based on the design loads and dimensions provided by CSEG.

Table 5
Footing Data Table
(Retaining Walls)

Design Height (ft)	Bottom of Footing Elev. (ft)	Min. Footing Embed. Depth (ft)	Strength 1A Limit State			Strength 1B Limit State		
			Eff. Found. With (ft)	Gross Bearing Stress (ksf)	Factored Bearing Resist (ksf)	Eff Found. With (ft)	Gross Bearing Stress (ksf)	Factored Bearing Resist (ksf)
With Toe								
5.79	350.8	3.11	3.24	13.0	7.1	3.66	13.6	7.5
9.79	346.8	4.60	3.98	18.0	9.9	4.74	19.1	10.5
13.46	342.8	5.79	5.58	23.5	12.9	6.68	25.0	13.8
Without Toe								
5.79	350.8	9.29	3.04	29.4	16.2	3.36	29.8	16.4
9.79	346.8	9.79	3.72	31.7	17.4	4.38	32.6	17.9
13.46	342.8	13.46	5.18	43.6	24.0	6.12	45.0	24.7

The estimated settlement based on the Gross Maximum Bearing Stress (Service) provided by CSEG for the walls is approximately 0.5-inch. Differential settlement is anticipated to be reduced to half of the total settlement across the length of the walls.

Unstable Foundation Recommendations

The design bearing stress/resistance given in Tables 4 and 5 requires that the RCB and walls will be placed on unyielding native soil or approved engineered fill. Any soft, unsuitable sediment in the canal bottom should be excavated to expose firm undisturbed soil and removed from the project site. If unstable foundation conditions are encountered it will be necessary to stabilize the area prior to foundation construction. Stabilization options include the following options:

Option 1 – Solar Drying, Mixing, and Blending of Dry Material

Unstable, shallow subgrade soils may be repeatedly disced to promote evaporation/natural drying and/or blended with dryer import fill soil to a compactable moisture range and recompacted in accordance with latest Caltrans Standard Specifications.

KLEINFELDER 3649 W. Holland Ave, Ste 105, Fresno, California 93722 p | 559.486.0750 f | 559.442.5081

Option 2 – Mechanical Stabilization

Should the construction area experience moderate to severe instability, the foundation areas should be stabilized by removing a portion of the unstable subgrade followed by placement of Subgrade Enhancement Geotextile (SEG_T) or bi-axial Subgrade Enhancement Geogrid (SEG_G) that complies with Section 96 of the Caltrans Standards Specifications. SEG should be placed on the smooth subgrade followed by placement of 0.67-to-1.0-foot Caltrans Class 2 aggregate base (AB) and compacting to establish initial stability. The SEG should be smooth and taught and extend a minimum of 5 feet beyond unstable areas. Adjacent panels of SEG should be lapped a minimum of 2 feet.

AB should be front loaded onto SEG, spread with the equipment working on the AB, and densified with moderate to heavy compaction equipment. The equipment should not operate directly on the SEG. Aggregate base should be compacted to a minimum 95 percent relative compaction. If 95 percent compaction cannot be achieved with the initial 0.67- to 1.0-foot-thick layer of AB, subsequent, layers of SEG and 0.67- to 1.0-foot-thick layers of AB should be placed until stability is achieved. The final layer should be compacted to a minimum 95 percent relative compaction.

LIMITATIONS

Kleinfelder will perform its services in a manner consistent with the standards of care and skill ordinarily exercised by members of the profession practicing under similar conditions in the geographic vicinity and at the time the services will be performed. No warranty or guarantee, express or implied, is intended or provided.

CLOSING

Kleinfelder appreciates the opportunity to serve as geotechnical consultants to Cornerstone Structural Engineering Group and the County of Fresno during the PS&E phase of the project. If there are any questions concerning the information presented in this letter, please contact the undersigned at your convenience.

Respectfully submitted,
KLEINFELDER, INC.

REGISTERED PROFESSIONAL ENGINEER
STEPHEN P. PLAUSON
No. 2731
Exp. 09/30/2025
GEOTECHNICAL
STATE OF CALIFORNIA

Anthony Aquino
Professional

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