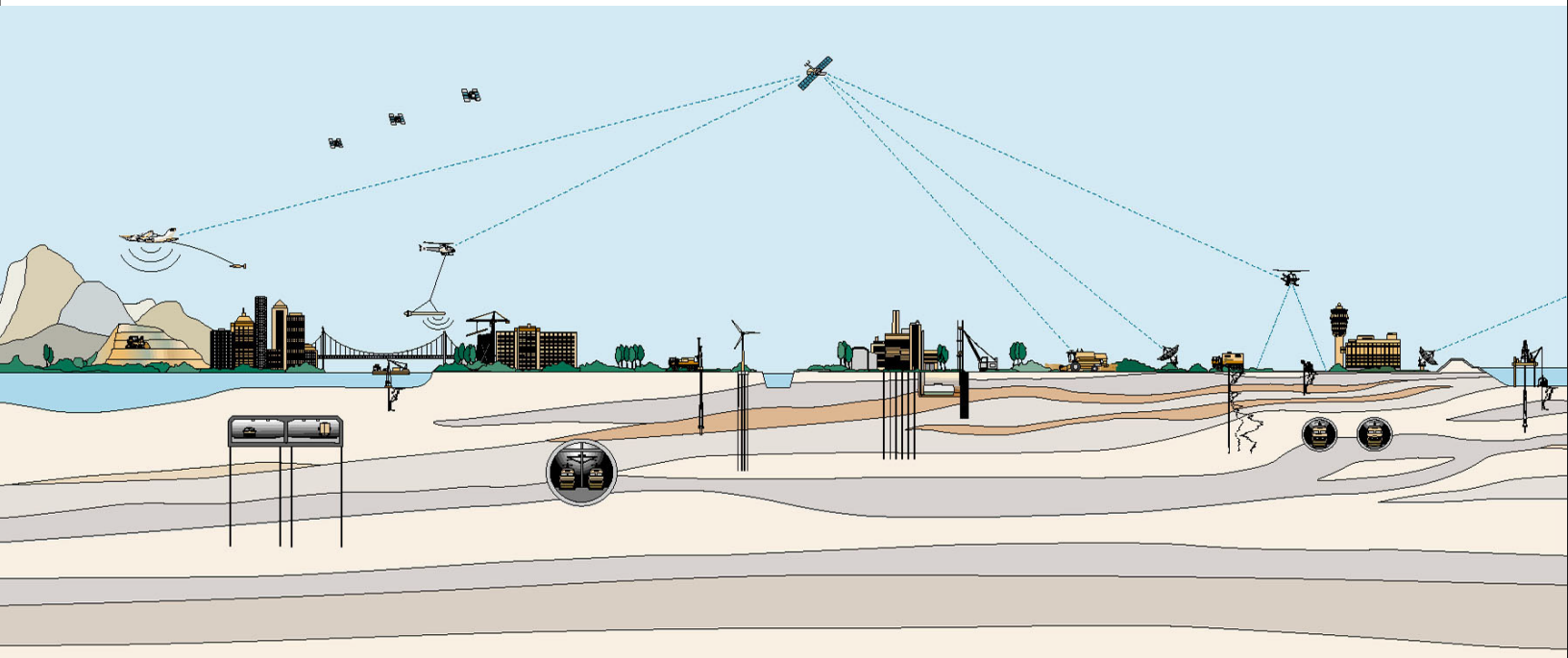
FOUNDATION REPORT
BRANCH MILL ROAD BRIDGE REPLACEMENT
OVER TAR SPRINGS CREEK
BRIDGE NO. 49C-0459
SAN LUIS OBISPO COUNTY, CALIFORNIA

Prepared for:

QUINCY ENGINEERING, INC.

September 8, 2014 (Rev. October 16, 2014)
Project No. 04.6111.0010



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September 8, 2014 (Rev. October 16, 2014)
Project No. 04.6111.0010

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Attention: Mr. Greg Young

Subject: Foundation Report, Branch Mill Road Bridge Replacement over Tar Springs Creek, Bridge No. 49C-0459, San Luis Obispo County, California

Dear Mr. Young:

Fugro is pleased to submit this Foundation Report for the design of the Branch Mill Road Bridge Replacement over Tar Springs Creek in San Luis Obispo County, California. This report was prepared in accordance with our Standard Agreement for Subconsultant Services with Quincy Engineering, Inc. dated February 3, 2011. This work generally consisted of conducting a program for field exploration, laboratory testing, and geotechnical analyses as a basis for providing the recommendations in the report. The recommendations are intended to be used with applicable Caltrans bridge design methods, standard specifications, and standard plans.

This Foundation Report provides geotechnical recommendations for the design of the bridge foundations, abutments, roadway approaches, and seismic data for use with Caltrans design methods. Field and laboratory data collected for this study, a log of test borings, and supporting graphics are presented in this report.

Please contact the undersigned if you have questions or we can be of additional service.

Sincerely,

FUGRO CONSULTANTS, INC.

Loree A. Berry

Loree Berry, P.E.
Senior Project Engineer

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CONTENTS

Page

1.0 PROJECT DESCRIPTION 1
1.1 General 1
1.2 Existing Facility 1
1.3 Proposed Improvements 2
2.0 WORK PERFORMED 2
2.1 Purpose and Scope 2
2.2 Scope 2
2.3 Field Exploration 4
2.4 Laboratory Testing 4
2.5 General Conditions 5
3.0 SITE CONDITIONS 5
3.1 Geologic Setting 5
3.2 Subsurface conditions 6
3.3 Groundwater 8
3.4 Seismic Setting 8
3.5 Ground Surface Fault Rupture 9
4.0 GEOTECHNICAL EVALUATION 10
4.1 Geologic Profiles and Engineering Parameters 10
4.2 Seismic Design Criteria 11
4.3 Liquefaction 11
4.4 Seismic Settlement 12
5.0 CONCLUSIONS AND RECOMMENDATIONS 13
5.1 General Summary of Findings 13
5.2 Seismic Design Criteria 14
5.2.1 ARS Curves 14
5.2.2 Approach Slab 14
5.3 Foundation Type 14
5.4 Axial Capacity of Driven Piles 14
5.5 Foundation Settlement 16

5.6 Lateral Capacity 16
5.7 Lateral Earth Pressures 17
5.8 Corrosion Considerations 18
5.8.1 Reinforced Concrete 18
5.8.2 Culverts and Drainage Pipes 18
5.9 Roadway Design 19
5.9.1 Naturally Occurring Asbestos (NOA) 19
5.9.2 Earthwork Factors 19
5.9.3 Approach Embankments 19
5.9.4 Structure Approach Embankment 19
5.9.5 Approach Fill Settlements 20
5.9.6 Pavement Structural Section 20
5.10 Suggested Material Specifications 20
5.11 Construction Considerations 21
5.11.1 Notes to Designer 21
5.11.2 Temporary Slopes and Shoring 21
5.11.3 Excavation Potential 21
5.11.4 Use of Onsite Soil 22
5.11.5 Groundwater and Dewatering 22
5.11.6 Pile Driving 22
6.0 CONTINUATION OF SERVICES 23
6.1 Review of Plans and Specifications 23
6.2 Geotechnical Observation and Testing 23
7.0 REFERENCES 24

LIST OF PLATES

No.

Vicinity Map 1

Field Exploration Plan 2

Regional Geologic Map 3

Subsurface Profile A-A' 4

Design Acceleration Response Spectrum 5

Summary of Liquefaction Potential 6

Lateral Load Resistance ..... 7a and 7b

Corrosion Test Results for Culverts 8

APPENDIX A – FIELD EXPLORATION

Key to Terms & Symbols Used On Logs A-1

Terms and Definitions Used for Rock A-2

Log of Boring No. B-1 to B-4 ..... A-3 to 6

APPENDIX B – LABORATORY TEST RESULTS

Summary of Laboratory Test Results B-1

Plasticity Chart B-2

Compaction Test Results B-3

Consolidation Test Results ..... B-4a and 4b

Consolidated Undrained (CU) Triaxial Test ..... B-5a to 5e

R-Value Test Results ..... B-6a and 6b

Expansion Index Test Results B-7

Corrosion Test Results ..... B-8a and 8b

1.0 PROJECT DESCRIPTION

1.1 GENERAL

The project will consist of replacing the existing Branch Mill Road Bridge over Tar Springs Creek (County Bridge No. 49C-0143) in San Luis Obispo County, California. The location of the site relative to nearby streets and landmarks is shown on Plate 1, Vicinity Map. The general layout of the site and proposed bridge are shown on Plate 2, Field Exploration Plan.

1.2 EXISTING FACILITY

Branch Mill Road runs generally northeast-southwest along the southerly boundary of the Arroyo Grande Valley and crosses Tar Springs Creek approximately ¼ mile southwest of the intersection with Huasna Road. The site is approximately 2 miles northeast of the City of Arroyo Grande. The terrain along the northerly side of Tar Springs Creek is generally flat cultivated land that is mainly used for irrigated row crops. The southerly approach to the bridge borders the base of a relatively steep (~1h:1v to 2h:1v) north-facing slope of Newsome Ridge. Tar Springs Creek flows to the west beneath the bridge and confluences with Arroyo Grande Creek approximately ½ mile downstream of the site. High voltage overhead transmission lines cross over the existing roadway near the north abutment of the bridge. The specific location and height of those lines above the existing roadway are not known at this time, but appear to be at least 40 feet above the roadway.

Details and the condition of the existing bridge were described in bridge inspection reports prepared by Caltrans (1952-2009). The existing bridge is an approximately 90-foot-long, three-span, steel-truss with a concrete deck constructed in 1949. The bridge foundations are likely reinforced concrete pile caps supported on driven timber piles, although some inspection reports state the bridge is supported by spread footing foundations. Branch Mill Road is a two-lane road (one in each direction) with minimal shoulders and is paved with asphaltic concrete. Unpaved roads providing access to adjacent properties and agricultural areas intersect with Branch Mill Road at several locations within the project limits.

The channel of Tar Springs Creek is steeply incised below the bridge. Natural lowering of the streambed has occurred at the bridge crossing due to erosion and scour and is evident by a concrete rubble check dam that has been constructed just downstream of the bridge. It also appears that a former secondary drainage channel flowed into Tar Springs Creek from the northeast, and intersected with the creek near the existing bridge. At this time, the secondary drainage channel appears to be disconnected from any source of surface water (see Fugro (2011) and EDR (2011)), likely because of re-grading and partial filling or blocking of the channel associated with local farming operations. No water was observed entering the creek from this area, although a depression, likely associated with the secondary drainage, is located upstream along the north approach to the bridge.

The end slopes below the abutments are also steep with evidence of erosion. There is evidence of ongoing instability and erosion of the stream banks in the form of slides within the bank, bare soils exposed by erosion, and soil deposits along the base of the slope in the vicinity of the bridge. Rock slope protection is present below the bridge and may have been added to help repair erosion that had undermined the slopes beneath the existing bridge.

1.3 PROPOSED IMPROVEMENTS

The current plan is to replace the existing bridge with a new clear-span structure. The new bridge will be constructed at generally the same location as the existing bridge. The vertical alignment will be raised approximately 2 feet at the bridge and the horizontal alignment will be modified to reduce the curve radius on the south approach to the bridge. The new bridge will be longer than the existing bridge with a clear span of approximately 105 feet across Tar Springs Creek (Quincy 2011). The bridge will be a precast, prestressed Bulb Teegirder with seat-type abutments and provide for two lanes (one in each direction). The bridge deck will be located at approximate elevation (el.) 193 feet and be 34'-10" wide.

Approximately 500 feet of new roadway will be reconstructed on either side of the new bridge to accommodate the improvements. Finish grades associated with the roadway construction are expected to be near or up to 2 feet above existing road grades for this project. The channel slopes below the new bridge will be graded to 1.5h:1v and covered with rock slope protection.

2.0 WORK PERFORMED

2.1 PURPOSE AND SCOPE

The purpose of this report is to provide geotechnical recommendations for the design of the Branch Mill Road Bridge Replacement project. The main geotechnical considerations that we evaluated for the project are 1) characterization of the soil and groundwater conditions encountered; 2) design of pile foundations to support lateral and vertical loads; 3) seismic considerations; 4) pavement design; and 5) construction considerations for the bridge replacement.

2.2 SCOPE

We performed the following scope of work to evaluate the geotechnical considerations for the project:

- Consulted with Quincy and the County of San Luis Obispo (County) to review our approach to providing geotechnical services for design, and visited the site to observe the general site conditions and mark the locations of our borings;
- Submitted a Preliminary Foundation Report (Fugro 2011) based on our review of existing documents and for use as input to the bridge type selection process;
- Obtained an encroachment permit from the County, prepared a health and safety plan for our work, contacted Underground Service Alert and Pacific Gas & Electric to review our proposed boring locations to stay clear of existing utilities, and coordinated the field exploration program with our drilling and traffic control subcontractors;
- Performed field exploration and sampling by drilling four hollow-stem auger borings to depths of approximately 10 feet along the roadway approaches, and to depths of up to approximately 100 feet at the bridge location;
- Performed a site reconnaissance of the existing bridge substructures for visual evidence of corrosion and obtained a water sample from Tar Springs Creek for corrosion testing;

- Performed laboratory tests on selected samples recovered from the borings;
- Prepared this report and the attached Log of Test Borings sheet summarizing the data collected and providing our conclusions and recommendations regarding:
  - Soil and groundwater conditions encountered;
  - Site geology, faulting, and seismicity;
  - Slope inclinations for the design of cut and embankment slopes;
  - Cuts and excavations including subdrain requirements, geologic structure, excavation characteristics of on-site soil and rock materials, and estimated rippability;
  - Embankments including suitability of excavated materials for use as fill, moisture control, erosion control, and drainage considerations, expansive soils, estimated settlement and special treatments to reduce settlement, if needed;
  - Corrosion considerations for culverts and buried concrete structures (cement factors, pipe thickness, and need for coatings);
  - Earthwork factors for onsite materials that are excavated and replaced as compacted fill;
  - Potential for naturally occurring asbestos (NOA) or radon gasses to impact the site in accordance with San Luis Obispo County Air Pollution Control District requirements;
  - Structural sections for asphaltic concrete pavements based traffic indices provided to us.
  - Seismic design criteria for use with Caltrans design methods;
  - Foundation considerations for scour;
  - Suitable foundations types for the conditions encountered (such as spread footings, driven concrete or steel piles, CISS, or CIDH piles);
  - Specified tip elevation, settlement, and size for Standard cast-in-steel shell pile foundations and Class 200 loading;
  - Lateral capacity of single pile foundations for free-head and fixed-head conditions based on p-y analysis;
  - Pile spacing and group reduction factors for vertical and lateral loads;
  - Lateral earth pressures, spring constants, and passive pressure resistance for abutment design;
  - Special considerations for approach fill settlements: allowable slope inclinations, waiting periods, and need for monitoring; and
  - Construction considerations regarding dewatering, pile driving, temporary excavations, and shoring.

2.3 FIELD EXPLORATION

Field exploration consisted of drilling and sampling 4 hollow-stem-auger borings at the site. Two borings were drilled along the roadway approaches to depths of approximately 10 feet, and two borings were drilled near the proposed bridge abutment locations to depths of approximately 80 and 100 feet. The field exploration program was performed July 18 and 19, 2012. The logs for the borings are presented in Appendix A. The locations of the borings are shown on Plate 2, Field Exploration Plan. The drilling subcontractor for the project was S/G Drilling Company of Lompoc, California. S/G used a CME85 truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers to advance the borings.

Standard penetration test (SPT) and modified California split-spoon samplers were used at selected depths to recover driven samples of soil and rock materials. Driven samples were taken at selected intervals using a 2-inch outside diameter SPT split-spoon sampler and a 3-inch outside diameter modified California split-barrel sampler. The SPT sampler was used without liners. The modified California sampler was used with brass liners. The samplers were driven into the materials at the bottom of the drill hole using a 140-pound automatic trip hammer with a 30-inch drop. The blow count (N-value) is the number of blows from the hammer that were needed to drive the sampler 1 foot after the sampler had been seated at least 6 inches into the material at the bottom of the hole. The recovered samples were sealed in plastic bags, set in protective containers, labeled and transported to the laboratory.

Thin-walled (Shelby) tube samples were taken at selected intervals by pushing the sampler into the soil at the bottom of the drill hole using the drill rig's hydraulics. The samplers were 3-inch diameter by 30-inch long steel tubes with a 1/16-inch wall thickness. The end of the tube is mandrelled to form a slightly smaller diameter cutting head. The tubes were typically pushed approximately 24 inches into the soil at the bottom of the hole. Once retrieved the ends of the samples were sealed with wax, capped, labeled, and set vertically in a cushioned transport box for transport to the laboratory.

Handheld pocket penetrometer or torvane tests were performed on the trimmed ends of selected samples in the field to estimate the undrained shear strength of cohesive soils. Bulk samples were collected from the drill cuttings retrieved from the auger flights. The sample intervals, N-values, a description of the subsurface conditions encountered and other field and laboratory data are presented on the logs of the borings in Appendix A.

2.4 LABORATORY TESTING

Laboratory testing was performed on selected samples obtained during the field exploration program. Laboratory tests for unit weight, moisture content, fines content, plasticity (Atterberg limits), laboratory compaction (modified Proctor), consolidation, consolidated undrained (CU) triaxial shear strength with pore pressure measurements, and expansion index (EI) tests were performed as part of this program in our laboratory in San Luis Obispo, California. Corrosion tests were performed on selected samples by Cooper Testing Laboratory of Palo Alto, California. The tests were performed in general accordance with the applicable standards of ASTM. Results of laboratory testing are presented in Appendix B.

2.5 GENERAL CONDITIONS

Fugro prepared the conclusions, recommendations, and professional opinions of this report in accordance with the generally accepted geotechnical principles and practices at this time and location. This warranty is in lieu of all other warranties, either expressed or implied. This report was prepared for the exclusive use of Quincy Engineering, Inc. and their authorized agents only. It may not contain sufficient information for the purposes of other parties or other uses. If any changes are made in the project as described in this report, the conclusions and recommendations contained in this report should not be considered valid unless Fugro reviews the changes and modifies and approves, in writing, the conclusions and recommendations of this report. This report and the drawings contained in this report are intended for design-input purposes; they are not intended to act as construction drawings or specifications.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements, or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

Soil and rock deposits can vary in type, strength, and other geotechnical properties between points of observations and exploration. Additionally, groundwater and soil moisture conditions can vary seasonally or for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation. These findings are subject to confirmation based on the conditions revealed by construction.

3.0 SITE CONDITIONS

3.1 GEOLOGIC SETTING

The project is located in the Arroyo Grande Valley and within the Coast Ranges geologic and geomorphic province. That province consists of north-northwest-trending sedimentary, volcanic, and igneous rocks extending from the Transverse ranges at the south into northern California. Rock formations of the Coast Ranges province are predominantly of Jurassic and Cretaceous age; however, some pre-Jurassic, along with Paleocene-age to Recent rock formations are present. The surficial geology in the project vicinity, as mapped by Hall et al. (1973), is shown on Plate 3, Regional Geologic Map.

The Arroyo Grande Valley and adjacent hills are the dominant geomorphic features within the project vicinity. The valleys were formed during a period of low sea level (the Wisconsin glacial stage), as coastal streams adjusted to the drop in sea level by carving into the landscape. A subsequent rise in sea level produced a dynamic depositional environment reflected in the discontinuous and variable subsurface stratigraphy. Up to approximately 800 feet of interlayered and unconsolidated sediments have been deposited within the valleys that typically dip gently to the west. The alluvium encountered at the site was underlain by bedrock of the Monterey Formation.

The Monterey Formation is generally composed of shale consisting of units of chalky white claystone, diatomite, and chert that are mapped along the ridges and hillsides that border the plain along Tar Springs Creek in the site vicinity. Hall (1973) maps the shale along the south approach to the bridge, where the rock has been exposed by instability and erosion, as dipping northwest at inclinations ranging from approximately 60 degrees to vertical. The shale encountered beneath the alluvium in our borings differed from the rock exposed along the roadside, and was composed of reddish brown chert and porcelaneous shale.

3.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site generally consisted of artificial fill material (Af), alluvium (Qa), and Monterey Formation bedrock. Logs of the explorations are presented in Appendix A. The approximate locations of the borings are shown on Plate 2. A generalized profile of the subsurface conditions along the bridge centerline is presented on Plate 4. Descriptions of the predominant geologic units and soil conditions encountered in our explorations is presented below.

Artificial Fill (Af): Artificial fill generally consists of the existing pavements, embankment fill, and fill placement behind the existing abutments. Rock slope protection to several feet in diameter and check dams are also present in the stream channel beneath the existing bridge. The pavement thicknesses encountered along the westbound shoulder are summarized below:

Approximate Pavement Thicknesses Encountered

Drill Hole No.	Pavement Section Encountered
B-1	5 inches asphalt concrete over 6 inches of aggregate base
B-2	6 inches of asphalt concrete over 7 inches of aggregate base
B-3	4 inches asphalt concrete over 4 inches of black clayey or stabilized base material
B-4	4 inches asphalt concrete over 6 inches of aggregate base

Artificial fill was encountered below the pavement in each of the drill holes (B-1 through B-4) to depths ranging from approximately 3 to 8 feet below the road surface. The artificial fill that was encountered below the north approach consisted predominantly of loose clayey sand and gravel and fill beneath the south approach consisted of stiff to very stiff clay with varying amounts of sand and gravel. The existing fill soils are associated with construction of the existing bridge and roadway.

Alluvium Deposits (Qal). The alluvium that was encountered was composed of sediments deposited along Tar Springs Creek. The alluvium was encountered below the artificial fill materials and was underlain by bedrock materials at depths of approximately 65 and 89 feet below the road surface in Borings B-2 and B-3, respectively. The alluvium is characterized as the four predominant units (Qal1 through Qal4) described below.

Qal1. This unit consisted predominantly of soft to medium stiff fat clay (CH) with interbeds of silt and sand that was encountered to approximately 38 feet below the road surface. These soils generally have low strength and are compressible. The banks of the creek are likely composed of this unit and have been locally displaced by landsliding and instability of the creek banks. Small roots (1 to 3 millimeters in diameter), minor amounts of organics, and secondary porosity are common within this unit.

Qal2. This unit consisted of an approximately 10-to-12-foot thick layer of stiff to very stiff fat clay that was encountered immediately below the Qal1 unit to depths of approximately 50 feet below the existing road surface.

Qal3. This unit consisted of relatively loose to medium dense silty to clayey sand (SM and SC) with extensive interbedding and intermixing with silt, clay, and gravel. Flowing sand and unstable borehole conditions were encountered within the sandy soils within this unit in B-2. These heterogeneous soils were encountered to approximately 65 and 77 feet below the road surface and were underlain by Monterey Formation bedrock and the Qal4 alluvial unit in B-2 and B-3, respectively.

Qal4. This unit was encountered in B-3 at approximately 77 feet below the road surface. The unit consisted of medium dense silty sand with gravel (SM) and was interbedded with layers of gravel. The alluvium was underlain by Monterey Formation bedrock at a depth of approximately 89 feet in that boring.

Monterey Formation (Tm). Monterey Formation was encountered below the alluvium in B-2 and B-3 at depths of approximately 65 and 89 feet below the road surface, respectively. The Monterey Formation consisted of intensely to moderately weathered, intensely fractured, and moderately hard cherty shale. Drilling with the hollow stem auger was relatively difficult with rates of drilling ranging from approximately 0.1 to 1 foot per minute.

Monterey shale is the predominate rock mapped along the hillsides in the site vicinity (see Plate 3). The shale is exposed along the eastbound shoulder of the south approach to the bridge. The shale is composed of moderately weathered, moderately soft shale that is dipping along the slope and is adversely oriented to the alignment of the roadway.

Geotechnical Properties. The results from selected laboratory tests and field data measured during the drilling are summarized below.

Summary of Selected Geotechnical Properties

Geologic Unit	Soil	Range of Dry Unit Weights (pcf)	Range of Moisture Contents (%)	Plasticity (Atterberg Limits) LL = Liquid Limit PI = Plasticity Index	SPT N_{60}-Value	Undrained Shear Strength (ksf)[1]
Af	Fat clay (CH) and clayey sand (SC) with gravel	61-94	17-37	LL 42-67, PI 21-43	3-15	--
Qal1	Fat clay (CH) with interbeds of silt and sand	69-91	28-48	LL 57, PI 30	4-5	0.3 – 0.6 t 0.9 - 1.0 p
Qal2	Fat clay (CH)	75-82	38-42	LL 67-69, PI 46-48	--	1.5 – 1.8 p
Qal3	Silty sand (SM) and sand with clay (SC-SM) with interbeds of silt, clay and gravel	90-103	22-29	--	4-30	0.9p
Qal4	Silty sand (SM) with interbeds of gravel	--	--	--	21-40	--
Tm	Moderately hard and intensely fractured cherty shale (Rx)	--	--	--	Refusal – 50/5"	2,875 psi ua

3.3 GROUNDWATER

Groundwater and groundwater seepage was encountered at approximately 15 feet (el. 174 to 175 feet) below the road surface and above the stream bed. Tar Springs Creek was flowing at the time of our July 2012 field exploration program (stream bed el. 160 to 163 feet). Flowing sand and caving ground conditions were encountered when deeper sand layers were encountered in the borings. Groundwater and soil moisture conditions will vary seasonally, with storm runoff, and irrigation of the adjacent field.

3.4 SEISMIC SETTING

The project site is within a seismically active region of California. We performed a fault search of the project area using current Caltrans Seismic Design Criteria and utilizing the Caltrans ARS Online (Caltrans 2013) and the 2008 Caltrans Deterministic PGA (peak ground acceleration) Map. ARS Online is updated periodically and was accessed on September 4, 2014 through the Caltrans website. ARS Online displays information for faults included in the Caltrans 2008 Fault Database and calculates both deterministic and probabilistic acceleration response spectra (ARS) for locations in California as described in Appendix B of the Caltrans Seismic Design Criteria (Caltrans 2013). ARS Online was first used to identify faults in the site vicinity considered to be potential seismic sources. The table below presents a list of faults closest to the site identified using ARS Online and site coordinates corresponding to Latitude 35.1319 and Longitude -120.5428. ARS Online was also used to estimate strong ground motion and develop the design ARS curve for the site discussed in subsequent sections of this report.

[1] Denoted by the test used to estimate undrained shear strength: p = field pocket penetrometer; t = field torvane, uu = Unconsolidated, Undrained triaxial compression testing; cu – Consolidated, Undrained triaxial compression testing, ua = uniaxial compressive strength estimated by crushing small (1/4" x 1/8" ±) trimmed shale particles

Nearby Faults

Fault Name	Fault Type[1]	R_x Distance (mi)[2]	Maximum Magnitude (Mmax)[3]
Los Osos fault zone (Newsome Ridge) (ID 232)	Reverse with dip of 45 degrees SE	3.5	6.9
Southern San Luis Range fault zone (ID 238)	Reverse with dip of 45 degrees NE	2.2	7.1

[1)] Fault type per Caltrans 2008 Fault Database. [2)] Horizontal perpendicular distance to the fault trace (*fictitious fault trace for sites offset from the mapped fault location) or surface projection of the top of rupture plane measured perpendicular to the fault from the site per ARS Online and Caltrans Geotechnical Services Design Manual. [3)] Mmax values per ARS Online and Caltrans 2008 Database.

A brief description of faults identified by ARS Online that are closest to the site and are considered to be most capable of producing strong ground motion at the site is provided below.

Los Osos Fault. The Los Osos fault zone is mapped along the base of the Irish Hills near San Luis Obispo and along Newsome Ridge in the site vicinity. The fault is considered active, and a portion of the fault is a zoned active Alquist-Priolo fault hazard zone west of San Luis Obispo.

Southern San Luis Range Fault. The Southern San Luis Range Fault identified on ARS Online is mapped northeast of the project site and generally trends along Highway 101 near Nipomo Mesa. The fault dips to the northeast at approximately 45 degrees. The fault zone includes splays and other locally mapped faults such as the Willmar Avenue, Oceano, and Santa Maria River faults.

3.5 GROUND SURFACE FAULT RUPTURE

The site is not located in an Alquist-Priolo Earthquake Fault Zone as defined by the State of California. Geologic structure near the project site is controlled by broad folding associated with the northwest trending Pismo Syncline (Hall 1973). The trend of the Tar Springs Creek drainage generally follows the axis of synclinal folds trending through the project vicinity. The closest significant mapped faults to the project site are the Los Osos (Edna) and San Luis Range Faults discussed in the previous section of this report, and mapped approximately 3 miles northeast and 2 miles southwest of the site, respectively by Hall (1973) and ARS Online. On the basis of that information ground surface rupture from faulting does not need to be considered for the design of this project.

4.0 GEOTECHNICAL EVALUATION

4.1 GEOLOGIC PROFILES AND ENGINEERING PARAMETERS

Plate 4 presents a summary of the subsurface profile used in our geotechnical analyses. Geotechnical engineering properties were assigned to various geologic units within the profile for use in evaluating the axial and lateral load capacity of pile foundations for the new bridge. The following table summarizes engineering parameters assigned to the layers shown on Plate 4. Soil unit weights and shear strength parameters have been estimated based on measured blow counts and the results of laboratory tests. When analyzing deep foundations for down-drag due to liquefaction, friction angles for the liquefiable soil layers were set equal to zero degrees. When analyzing deep foundations for lateral load capacity and liquefaction, the soil resistance model was estimated to be one half of the static value.

Engineering Parameters for Northeast Abutment (Boring B-2)
Existing Ground el. 189 feet. Abutment Footing Cutoff Elevation = el. 176.6 feet

Geologic Unit	Elevation Interval (feet)	Predominant Material Type with Unit (see Plate 4)	Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle (deg.)	ε_{50}* (in/in)	k_s or k_{ir}* (pci)	Uniaxial Strength* σ_a (psi)	RQD* (%)	Eir* (ksi)
Af	189 180	Sand above water. Not used in analysis. Otherwise Structure Backfill	120	--	33	--	200	--	--	--
Qa1	180 151.5	Soft to medium stiff Fat CLAY	106	500	--	0.01	--	--	--	--
Qa2	151.5 139	Stiff to very stiff Fat CLAY below water	115	1500	--	0.006	500	--	--	--
Qa3	139 125	Sand below water with interbedded clay, clayey sand and silt*	120	--	30	--	40	--	--	--
Tm	125 --	Monterey Shale, moderately hard, intensely fractured (Weak Rock)*	135	--	--	--	0.0002	2000	0	400

*Parameters needed for Lpile input and defined in user's manual

Engineering Parameters for Southwest Abutment (Boring B-3)
Existing Ground el. 189 feet. Abutment Footing Cutoff Elevation = el. 177.0 feet

Geologic Unit	Elevation Interval (feet)	Predominant Material Type with Unit (see Plate 4)	Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle (deg.)	e50* (in/in)	k_s or k_{ir}* (pci)	Uniaxial Strength * σ_a (psi)	RQD* (%)	Eir* (ksi)
Af	189 180	Sand above water. Not used in analysis. Otherwise Structure Backfill	120	--	33	--	200	--	--	--
Qa1	180 154	Soft to firm Fat CLAY	106	700	--	0.01	--	--	--	--
Qa2	154 139	Stiff Fat CLAY below water	115	1500	--	0.006	500	--	--	--
Qa3/Qa4	139 100	Sand below water with interbedded clay, clayey sand and silt*	120	--	30	--	40	--	--	--
Tm	100 --	Monterey Shale, moderately hard, intensely fractured (Weak Rock)*	135	--	--	--	0.0002	2000	0	400
*Parameters needed for Lpile input and defined in user's manual										

4.2 SEISMIC DESIGN CRITERIA

A design acceleration response spectrum (ARS) curve for the site was estimated using ARS Online and guidelines set forth in Appendix B of the Caltrans Seismic Design Criteria. The deterministic and probabilistic spectra estimated from the ARS Online analysis for 5 percent damping is shown on Plate 5. In accordance with Caltrans guidelines, the design ARS curve is taken as the upper envelope of the deterministic and probabilistic ARS curves as shown on Plate 5. The design deterministic ARS curve was controlled by the San Luis Range fault zone for a M7.1 earthquake and an estimated peak ground acceleration of 0.52g. A near-fault factor was applied within ARS Online to the ARS curve because the site is located less than 15 miles (25km) from a potentially controlling fault.

We estimated the shear wave velocity for materials encountered in the borings using correlations to blowcounts and undrained shear strengths presented in the Caltrans Geotechnical Services Design Manual. An average shear wave velocity of approximately 175 meters/sec was estimated for the top 100 feet of soil at the site, which corresponds to a Soil Profile Type E. The Caltrans criteria uses the 2008 USGS Interactive Deaggregation calculator for the development of the design probabilistic ARS curve when the estimated shear wave velocity V_{s30} for the site is less than 300 meters/second (1,000 feet/second).

4.3 LIQUEFACTION

Liquefaction is the loss of soil strength due to an increase in soil pore-water pressure resulting from seismic ground shaking. Liquefaction typically occurs in loose to medium dense granular soil that is below the groundwater table. The extent and severity of liquefaction is

dependent upon the intensity and duration of the strong ground motion. We performed a liquefaction analysis for the controlling earthquake to check the liquefaction potential of materials encountered in the borings. Analyses were performed using the 1997 NCEER procedures (Youd et al. 2001), a M7.1 earthquake, and a peak ground acceleration of 0.52g.

For the liquefaction evaluation, field blow counts were converted to equivalent clean sand, corrected blow counts ($(N_1)_{60\text{-}cs}$). The $(N_1)_{60\text{-}cs}$ values were then compared with the corresponding critical N-values that would be needed to resist liquefaction using the NCEER procedures. The results of our comparison are summarized on Plate 6. Plate 6 illustrates the estimated factor of safety against liquefaction from the ratio of $(N_1)_{60\text{-}cs}$ to N_{crit} for various borings. A ratio equal to 1.0 or more indicates a relatively low potential for liquefaction.

The results shown on Plate 6 show potentially liquefiable sand layers were encountered within the deeper alluvium units Qal3 and Qal4 (see Plate 4). Ratios of $(N_1)_{60\text{-}cs}$ to N_{crit} values were less than 1.0 below approximately el. 138 feet below to bedrock.

4.4 SEISMIC SETTLEMENT

Seismic settlement analyses were performed in association with the liquefaction evaluation described in the previous section of this report. Seismically induced settlement or collapse can occur in soil that is loose or soft, or that is moderately dense. Seismic settlement can occur as excess pore-water pressure dissipates following liquefaction, or as a result of collapse or consolidation of the soil structure above the ground water table in response to earthquake vibrations. We used corrected SPT N-values and procedures outlined by Tokimatsu and Seed (1987) and Pradel (1998) to estimate seismically induced settlement.

We estimate that approximately 2 to 6 inches of seismic settlement could occur in response to the design earthquake. Seismic settlement within layers of the alluvium could result in down-drag on deep foundations. As the soil settles negative skin friction associated with the settlement can result in down forces to be mobilized along the soil-pile interface. Negative skin friction and down drag will mobilize on a piles within only about ½ –inch of settlement. Estimates of negative skin friction and foundation design considerations to address negative skin friction are presented in this report.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL SUMMARY OF FINDINGS

The following provides a brief summary of our findings and main geotechnical considerations for the design of the bridge. Detailed recommendations needed for the design of bridge and roadway improvements are provided in the following sections of this report.

- The site is underlain by a variable thickness of artificial fill pavement and embankment materials that overly alluvium and Monterey Formation. The alluvium materials were encountered to depths of approximately 65 to 89 feet below Branch Mill Road in the vicinity of the proposed bridge abutments and were underlain by moderately hard shale of the Monterey Formation. The alluvium is composed of layers of soft clay, stiff clay and sandy soils denoted Qal1 through Qal4 on the subsurface profile shown on Plate 4. The deeper alluvium (Qal3 and Qal4) encountered below a depth of approximately 50 feet is interbedded with loose and medium dense sand that is potentially liquefiable under the design earthquake. The bridge can be supported on deep pile foundations bearing in the Monterey Formation.
- Groundwater and seepage was encountered during field exploration at approximately 15 feet below the road surface, and above the streambed elevation of Tar Springs Creek. The creek appears to drain and draw the groundwater down from the adjacent area. The bottom of the existing creek channel is located at an elevation of approximately 160 feet (approximately 30 feet below the elevation of Branch Mill Road).
- There is a relatively high potential of interbedded sand layers within the alluvium to liquefy under a seismic event. Liquefaction in layers of the alluvium could result in down-drag on deep foundations and has been considered in the evaluation of specified tip elevations for the deep foundations. We estimated that approximately 2 to 6 inches of liquefaction-induced seismic settlement could occur within the alluvium at the bridge site for the seismic conditions considered. There is a low potential for liquefaction to occur within the recommended bearing material: moderately hard Monterey Formation.
- The new bridge can be supported on standard PP16 driven steel pipe piles, and can be converted to cast-in-steel shell (CISS) piles, if desired. Pipe piles should be driven open-ended and were selected for the bridge after considering the relatively deep depth to a bearing layer, the presence of soft and saturated soil within the alluvium, and the ability to add on or cut off steel piles to accommodate a potentially variable bedrock surface, as compared to a precast concrete pile. Pile driving operations will also need to consider the presence of overhead transmission lines that cross the north approach to the bridge. Because of a relatively high groundwater table, potential caving sand, and soft soil, cast-in-drilled hole (CIDH) piles and shallow foundations are not considered suitable for support of the bridge.

- The end slopes beneath the bridge abutments should be designed to 1.5h:1v or flatter. The placement of rock slope protection on those slopes is needed to help improve the surficial instability of the slopes.

5.2 SEISMIC DESIGN CRITERIA

5.2.1 ARS Curves

The seismic conditions for this project are discussed in Section 4.2 of this report. The design earthquake for this project is a M7.2 earthquake on the San Luis Range fault zone, mapped approximately 2 miles southwest of the site, with an estimated peak ground acceleration of 0.52g at the site. The subsurface conditions can be considered to be a Soil Profile Type E. The recommended ARS curve for design is shown on Plate 5, Design Acceleration Response Spectrum. The ARS curve generated by ARS Online includes an increase to account for the near source faults.

5.2.2 Approach Slab

Soil layers within the alluvium are prone to liquefaction and seismic settlement as discussed in Sections 4.3 and 4.4 of this report. A summary of the liquefaction potential of the soils encountered is presented on Plate 6. Because there is a potential for seismic settlement to impact the roadway and bridge foundations and result in abrupt differential settlement between the bridge and the approaches, we recommend that a seismic ramp be provided.

5.3 FOUNDATION TYPE

Preliminary geotechnical considerations for the design of the bridge foundations were presented in our Preliminary Foundation Report (Fugro 2011). Open-ended standard steel pipe piles are considered suitable for support of the bridge, and are the recommended preferred pile type for the subsurface conditions encountered. The anticipated bearing material for piles is moderately hard Monterey Formation that was encountered at depths of approximately 65 and 89 feet below surface of Branch Mill Road. Driving the piles open-ended should help to advance the piles through the overburden soil to reach the bedrock, and we anticipate that the piles would take-up relatively quickly (in less than 5 feet) once bedrock is encountered. Steel piles are preferred over precast driven piles because we anticipate the depth to bedrock between specific pile locations will vary, and steel piles can be cutoff or spliced more easily in the field than concrete piles.

Driven piles were selected over CIDH piles because of the relatively deep depth to bearing layer, the presence of high groundwater table, potential caving sand, and soft soil within the alluvium that would make construction of cast-in-drilled hole (CIDH) piles difficult and poorly suited for the site conditions. Shallow foundations are not considered suitable for support of the bridge due to the relatively high foundation loads, soft ground, the potential for liquefaction and seismic settlement, and considerations for scour.

5.4 AXIAL CAPACITY OF DRIVEN PILES

The recommendations for the design of the deep foundations considers that the bridge deck will be located near the existing deck elevation (approximately el. 190 feet) and the cutoff elevation for the piles at approximately el. 176.6 feet based on discussions with Quincy. The scour depth recommended by WRECO (2012 draft) is approximately 16.5 feet below the

existing streambed, approximately el. 143.2 feet. The 95% project plans include rock slope protection (RSP) along the creek embankment below the bridge abutments and extending both upstream and downstream of the bridge. The RSP is expected to mitigate the scour condition. Even without RSP, the scour case does not control the design of the bridge foundations as the piles will be bearing in bedrock well below the anticipated scour elevation. The recommended specified pile tip elevation (SPTE) was estimated for allowable stress design for the various piles considering the following:

- The SPTE for static axial loads is based on the service load of 200 kips per pile and a factor of safety of 2;
- The SPTE at the estimated maximum scour depth is based on a nominal resistance of 400 kips (by applying a factor of safety of 1);
- The SPTE considering downdrag forces due to liquefaction was evaluated by setting the SPTE for the static loading conditions, and then checking that the estimated settlement of the pile would not exceed a tolerance of 2 inches and that the SPTE was below the potentially liquefiable soil;
- The SPTE for the lateral load condition is estimated as 1.3 times the embedment needed to essentially fix the pile.

The bridge can be supported on Caltrans Standard Class 200 PP16x0.5 piles (Alternative W) driven into the underlying Monterey Formation. The piles will need to be advanced to depths of at least 65 to 90 feet below the ground surface, the depth in which Monterey Formation was encountered in the borings. The recommended SPTE for the pipe piles was estimated assuming the piles would be driven into moderately hard rock at depths encountered in our borings, and that the piles would achieve the necessary driving resistance within 5 feet of driving into the bedrock. The depth to the bedrock differed by 24 feet between the two borings, and we expect that the final tip elevation for piles bearing in rock may be different (more or less) at each specific pile location. To potentially reduce the need for splicing, the Contractor may choose to order piles with additional length. However, the use of longer piles does not preclude the potential for splicing and may result in excess pile waste if the additional length is not needed.

The nominal driving resistance and negative skin friction resulting from settlement of the overburden soils were estimated using the procedure using the Nordlund and alpha (α) methods for coarse and fine-grained (clay) units, respectively. The design capacity of the piles was estimated assuming the piles will be driven open-ended and will be end bearing in moderately hard shale bedrock. We estimate the driving resistance will likely exceed the downdrag force associated with seismic settlement with the alluvium, and that the driven piles will not likely be prone to significant settlement due to negative skin friction or would the additional downdrag force control the pile design. Similarly, the nominal axial resistance of the pile should not be exceeded if the soils are removed or neglected to the estimated scour elevation. The recommended pile data table for CIDH piles is presented below:

RECOMMENDED PILE DATA TABLE
for PP16x0.5 Piles

Support	Pile	Cut-off Elevation (ft)	Service I Limit State Total Load (kips) per Support		Service I Limit State Total Load (kips) per Pile Compression	Nominal Resistance (kips)	Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
			Total	Perma-nent					
Abut. 1 (south)	Class 200 PP16x0.5	176.6	1369	936	200	400	95 (a, c) 145 (d)	95	400
Abut. 2 (north)	Class 200 PP16x0.5	177.0	1369	936	200	400	120 (a, c) 145 (d)	120	400

Notes:

1. *Design tip elevation are controlled by (a-I) Compression (static), (a-II) Compression (Scour to el. 143.5 ft.), (c) Settlement, (d) Lateral Load. The piles will not resist tensile forces.*
2. *The specified tip elevation can be raised or lowered as-needed to develop suitable bearing in bedrock. Driving can be terminated where either a) piles have met the specified tip elevation and required nominal driving resistance or b) piles have met refusal to driving per the Standard Specifications within bedrock above the specified tip elevation.*
3. *Overburden soil layers (softer clay, liquefiable, scourable, etc.) that do not contribute to the design nominal resistance were encountered at Abutment 1 to about el. 100 ft. and at Abutment 2 to about el. 124 ft.*
4. *NA = "not applicable"*

5.5 FOUNDATION SETTLEMENT

We estimate that settlement of the abutments supported on driven steel piles should not exceed ½-inch when subjected to static loads and installed as recommended. As previously discussed in this report, liquefiable or granular soil layers that could be subject to seismic settlement were encountered at various depths above the recommended pile tip elevations. The estimated settlement of the roadway approaches for the design earthquake is approximately 2 to 6 inches. Seismic settlement can also result in potentially liquefiable soil layers that drag down on the piles as the settlement occurs. Because the piles will be driven above the service load into rock, the design does not need to consider additional settlement of the piles due to downdrag.

5.6 LATERAL CAPACITY

P-Y Analyses. The lateral load capacity of piles was estimated using a soil resistance-pile deflection model (p-y analysis) and the computer program Lpile Plus 5.0 (Ensoft 2005). Lpile Plus was used to estimate the horizontal movement, maximum moment, and depth of fixity of the pile in response to lateral loading under fixed- and free-head conditions. The plotted results of the p-y analyses are presented on Plates 7a and 7b. No factor of safety has been applied to the estimated loads or deflections. The SPTE for the lateral load was estimated from Lpile as 1.3 times depth of fixity for the piles. The lateral pile capacities are for piles with the pile head at the cut off elevation and setback at least 10 feet horizontally behind the finished grade of the slope below the abutments.

Group Effects. The abutments will likely be supported on two rows of piles. Where the piles are spaced at least 5 pile diameters or more on-center, or the piles do not shadow other piles relative to the direction of loading, the pile group capacity can be estimated as the sum of the individual pile capacities in the direction of loading. However, where piles are spaced closer than 5 diameters or will shadow other piles, the lateral capacity of the group in the direction of loading can be estimated as: the sum of the individual pile capacities for a given deflection in the first row, 0.85 times the sum of the individual pile capacity in the second row, and 0.70 in the third and any subsequent rows.

Abutment Resistance. Caltrans design methods allow abutments to be designed to resist horizontal seismic forces using an estimated lateral passive force of 5 kips per foot. The value is likely appropriate provided analysis indicates that the abutments will move into the backfill materials such that the top of the abutment rotates over the base of the abutment or remains vertical.

5.7 LATERAL EARTH PRESSURES

We recommend the following equivalent fluid weights for use in estimating the lateral earth pressures that will act on bridge abutments designed as retaining walls. The recommended lateral earth pressures are for level backfill conditions and retaining wall backfill material that conforms to the Caltrans Standard Specifications.

Lateral Earth Pressures

Condition	Lateral Earth Pressure Condition	Equivalent Fluid Weight (pcf)
Unbraced Wall – Drained	Active	35
Braced Wall – Drained	At-rest	60
Footing – above water and >10 feet from slopes	Passive	300
Footing – above water and <10 feet from slopes	Passive	125
Footing – below water and >10 feet from slopes	Passive	150
Footing – below water and <10 feet from slopes	Passive	62

The recommended equivalent fluid weights are provided for drained or undrained backfill conditions. The undrained equivalent fluid weights should be used when considering hydrostatic forces resulting from submerged conditions (i.e. standing water in the backfill material). The equivalent fluid weights do not provide for surcharge loads acting on the backfill. Surcharge traffic loads can be represented by the lateral earth pressures that result from an equivalent 2-foot soil surcharge. If surcharge loading from adjacent foundation loads are to be considered, then Fugro should be advised so that we can provide additional recommendations as needed.

5.8 CORROSION CONSIDERATIONS

Cooper Testing Laboratory of Palo Alto, California performed the corrosion tests, and the results are presented on Plates B-8a and 8b and summarized on Appendix Plate B-1 and in the table below. Corrosivity tests were performed on four selected samples obtained from the field exploration program as summarized below.

Results of Corrosion Analyses

Boring	Depth (feet)	Material	pH	Chloride Content (ppm)	Sulfate Content (ppm)	Minimum Resistivity (ohm-cm)
B-2	2	Existing Artificial Fill Clayey SAND with gravel (SC)	7.4	13	304	1564
B-3	25	Alluvium Fat CLAY (CH)	7.6	51	296	711
B-3	40	Alluvium Fat CLAY (CH)	8.0	39	278	653
--	--	Tar Springs Creek Water Sampled on July 26, 2012	7.8	137	429	654

5.8.1 Reinforced Concrete

According to the Caltrans Corrosion Guidelines, a corrosive area is defined where "…the soil and/or water contains more than 500 ppm of chlorides, more than 2,000 ppm of sulfates, or has a pH less than 5.5." The concrete for the existing substructure appears to be in relative good condition and did not appear to contain swelling rebar or cracking that would be associated with corrosion. The results are “non-corrosive” for the samples tested based on Caltrans guidelines. Therefore, reinforced concrete piles and substructures may be designed using the minimum cement types and contents of the Standard Specifications.

5.8.2 Culverts and Drainage Pipes

Drainage structures should be designed according to the latest edition of the Highway Design Manual considering the presence of a corrosive soil having a pH greater than 7 and resistivity of 650 ohm-cm. Near surface soils encountered during the field exploration program contain varying amounts of gravel. Culverts should be considered to be subject to abrasion and appropriate flow velocity conditions.

Reinforced Concrete Pipe (RCP). The Caltrans corrosion requirements for reinforced concrete pipe are similar to those for reinforced concrete structures. The cement types and contents for reinforced concrete structures are applicable to the design of RCP.

Corrugated Metal Pipe (CMP). CMP will be vulnerable to corrosion and should be sized for appropriate losses. A summary of the corrosion test data for use is in estimating the thickness of CMP is presented on Plate 8. Galvanized steel metal pipe should have a minimum 10 gage thickness. Smaller gage pipe should have protective coatings to limit the necessary thickness of the pipe.

Plastic Pipe. According to the Highway Design Manual, plastic pipe is virtually corrosion free, permitting a 50-year maintenance free service life under most conditions. Plastic pipe, if used, should be designed according to the minimum requirements of the Highway Design Manual.

5.9 ROADWAY DESIGN

5.9.1 Naturally Occurring Asbestos (NOA)

The site is located within San Luis Obispo County's designated zone for evaluating the presence of NOA. Serpentine rocks found throughout San Luis Obispo County commonly contain NOA. Based on the conditions encountered in our field explorations no serpentine rocks were encountered or would be expected to be encountered based on the site geology. We do not expect that rocks containing NOA will be encountered during construction, and no special mitigation for NOA will need to be considered in the design of the project.

5.9.2 Earthwork Factors

We estimated earthwork factors based on a comparison of unit weight measurements of drive samples recovered from our borings, and maximum density tests performed on bulk samples recovered from our explorations. We expect that the actual earthwork factors will vary from those estimated, and the factors do not account for losses due to stripping during clearing and grubbing. Soils within 1 to 2 feet of the subgrade are likely denser, and should have limited shrinkage of about 5 to 10 percent when replaced as compacted fill. Soils below 2 feet appear to be relatively loose, and a relatively shrinkage of 15 to 20 percent should be anticipated for fill obtained from excavation more than about 2 feet deep. The dry units of the soils tested also appear to be influence of the presence of diatomaceous gravel clasts (that can have a dry unit weight near that of water) and therefore shrinkage factors will vary significant depending on the gravel content of the soil. Earthwork factors should be evaluated as the earthwork progresses, and be adjusted based on observations during grading to help balance quantities.

5.9.3 Approach Embankments

Site preparation and fill placement for the approach embankments should be performed according to Section 16, Clearing and Grubbing, and Section 19, Earthwork, of the Caltrans Standard Specifications. The recommendations presented below are intended to provide a relatively uniform transition between the approach fill and the proposed structure. In general, approach fill embankments should be designed using to a slope inclination of 2h:1v or flatter to help minimize erosion. The end slopes below the proposed bridge can be designed to a slope inclination of 1.5h:1v or flatter and should be covered with appropriately sized rock slope protection.

5.9.4 Structure Approach Embankment

The structure approach embankments should be designed according to Topic 610 of the Highway Design Manual. Fill materials placed within the structure approach embankment area should be compacted to at least 95 percent relative compaction, except where 90 percent relative compaction is permitted in the outer 5 feet of the embankment.

5.9.5 Approach Fill Settlements

We anticipate the approaches will be constructed near existing site grades, without appreciable addition of embankment fill. Therefore, we expect that the static settlement of the approach fill should be less than ½-inch, and would be associated with post-construction compression of the fill materials themselves. As discussed in Section 4.4 of this report, we estimate that up to about 2 to 6 inches of seismic settlement could occur in association with liquefaction and the design earthquake.

5.9.6 Pavement Structural Section

Structural section recommendations were calculated based on Caltrans design procedures, traffic indices provided to us, and R-Value testing. The results of two R-value tests performed on samples of subgrade soils recovered from the borings were 22. An R-value of 20 was selected for design of the structural section. The structural section was calculated for a traffic index of 7.0. The recommended thicknesses for a 2-layer structural section composed of asphalt concrete (AC) and aggregate base (AB) is provided below.

Summary of Structural Section Recommendations for Flexible Pavement
(TI=7.0, $R_{subgrade}$ = 20)

Section	Pavement Thicknesses	
	Asphalt Concrete Thickness	Aggregate Base Thickness
2-layer	0.45 ft	0.75 ft

AC = Asphalt Concrete, AB = Aggregate Base

Suggested materials specifications for pavement materials are provided in the next section.

5.10 SUGGESTED MATERIAL SPECIFICATIONS

Materials discussed or used in this report, as they relate to the Caltrans Highway Design Manual and Standard Specifications are discussed as follows:

Aggregate Base: Aggregate base (AB) shall be Class 2 conforming to Section 26-1.02A, "Class 2 Aggregate Base," of the Standard Specifications. Class 3 aggregate base that incorporates reclaimed material can be used as aggregate base, provided the Class 3 material meets the gradation and quality requirements for the Class 2 material.

Asphalt Concrete: Asphalt concrete (AC) shall be Type A conforming to Section 39, “Asphalt Concrete,” of the Standard Specifications. Asphalt binder shall be grade PG 64-10.

Imported Borrow: Imported borrow shall conform to Section 19-7.02, "Imported Borrow," of the Standard Specifications. Imported borrow shall have a resistance (R-value) of at least 25, as determined by California Test 301.

5.11 CONSTRUCTION CONSIDERATIONS

5.11.1 Notes to Designer

Driven pipe piles will extend to depths of approximately 60 to 85 feet below the cutoff elevation to penetrate bedrock and may require splicing and welding of short sections to achieve the specified tip elevation. In an effort to potentially reduce the need for splicing, the Contractor may choose to order piles with additional length. However, the use of longer piles does not preclude the potential for splicing and may result in excess pile waste if the additional length is not needed.

Overhead transmission lines cross Branch Mill Road just east of the north abutment. The specific height and location of those lines is not known. The location of the power lines should be reviewed relative to the pile driving operation to define adequate setbacks and clearances from the power lines.

The bedrock elevation between the two bridge borings differed by approximately 24 feet. The bedrock surface is likely irregular or sloping relative to the specific points of our exploration. The tip elevation of the piles will likely need to be varied (up or down) during construction based on the elevation that bedrock is encountered at each pile location.

5.11.2 Temporary Slopes and Shoring

Temporary slopes should be braced or sloped according to the requirements of OSHA. In accordance with OSHA requirements, the contractor should be responsible for job site safety, reviewing the soil conditions encountered during construction, and for the design of temporary slopes and shoring systems. Within the expected depth of excavation, the subsurface conditions are likely to consist of embankment fill over alluvium. The alluvium contains zones of clay soil having a shear strength of less than 500 psf. Based on OSHA guidelines, temporary slopes should be excavated to 1.5h:1v or be shored to support Type C soil conditions.

Slopes should not be considered stable if the slopes are to be excavated below the groundwater table or there is a potential for seepage to daylight on the slope face. Dewatering should be performed in advance of the excavation if temporary slopes are expected to extend below the water table, and the presence of groundwater should be considered in the design of temporary slopes and shoring.

5.11.3 Excavation Potential

The soils encountered within the anticipated depths of excavation consist of clayey sand and clay with varying amounts of gravel. We anticipate that the soils encountered can be excavated with conventional heavy-duty excavation-type equipment typically used for highway construction, such as suitably sized backhoes and dozers. Clay soils will be sensitive to changes in moisture content. Areas that become wet due to ponding water, surface runoff, rainfall or irrigation are likely to become soft and muddy and may be difficult to traverse and may not support construction equipment without stabilization.

5.11.4 Use of Onsite Soil

The onsite soils encountered within the anticipated depths of excavation are generally clay or clayey soils. The soils when cleansed of any organics or deleterious materials should be suitable for general embankment and backfill material. The onsite soils should not be considered suitable for use as select material such as for structure backfill, non-expansive soil to be placed in the vicinity of the bridge, or aggregate base or subbase material.

Clay soils will be sensitive to changes in moisture content and will be relatively difficult to compact. If wet, clay soils can be difficult to dry with typical methods such aeration and disking unless the weather is particularly well suited. Control of moisture content, compaction layer thickness, and selecting suitable compaction equipment will be necessary to achieve the specified compaction.

5.11.5 Groundwater and Dewatering

Groundwater conditions are discussed in Section 3.3 of this report. Groundwater and groundwater seepage was encountered at depths of approximately 15 feet below the existing ground surface (el. 175 feet), about 5 feet below the pile cut off elevation. Groundwater was recorded in borings B-1 and B-3 at approximately 24 to 43 feet below the ground surface (el. 121 feet to el. 140 feet), respectively. The groundwater level was near or above the existing streambed elevation (see Plate 3) at the time of our exploration. Groundwater levels are likely influenced by flows in the creek, and could be higher than the groundwater levels encountered for this study depending on storm runoff, the time of year, and other factors. The pile cutoff elevation is above the groundwater level encountered. The SPTE considers driven pipe piles will be extended below the groundwater level encountered.

The contractor should be responsible for both designing and maintaining the dewatering system for construction. Dewatering facilities, such as sump pits, wells, and well points should be designed by a qualified registered professional and with filters such that sand and fine-grained materials are not removed from the soil during dewatering operations. Dewatering facilities should be installed prior to beginning excavation, and time should be allowed for lowering of the groundwater table before beginning excavation. Shoring systems, such as sheet piling, should be embedded adequately below the base of the excavation to cutoff groundwater and help stabilize the base of the excavation.

5.11.6 Pile Driving

Pile driving will likely be difficult due to the soft ground conditions and deep depths to bedrock encountered at the site. The piles will be relatively long (up to 85 feet or more), and if additional length is needed, driving will need to stop and be restarted to allow for piles to be spliced and for welds to be inspected. In an effort to minimize the need for splicing, the Contractor may choose to order piles with additional length. However, the use of longer piles does not preclude the potential for splicing and may result in excess pile waste if the additional length is not needed.

Layers of relatively soft clay are common to depths of at least 40 feet below the surface of Branch Mill Road. Soft ground will have a tendency to allow piles to plunge and can make it difficult to maintain the proper location, alignment and to drive the piles vertically. Fixed leads or

templates should be provided as-needed to maintain the location and alignment of the pile during driving.

Piles will be designed to be end bearing in moderately hard shale of the Monterey Formation. The depth to bedrock differed by 24 feet between the two bridge borings. We expect that the bedrock elevation and resulting pile tip elevation will be different for each pile location and that piles need to be built-up or cut-off based on the elevation that rock is encountered at each pile location.

Hard driving conditions should be anticipated once bedrock is encountered. In no case should piles be driven beyond the refusal criteria allowed by the Standard Specifications. Proper sizing of the hammer, cushioning, and driving criteria should be provided such that piles are not damaged during driving. Hammer information should be provided for review by the geotechnical professional prior to mobilizing equipment to check that the hammer and proposed driving procedures are appropriate for the anticipated driving conditions and to provide driving and refusal criteria for the pile installation.

We recommend that the geotechnical professional observe the pile driving. The purpose of this observation is to evaluate if the soil conditions encountered and methods of construction are consistent with those assumed for this report. The project specifications should provide for expected variations in the driving conditions and materials encountered, and for deepening the driven pile foundations, if needed.

6.0 CONTINUATION OF SERVICES

The geotechnical evaluation consists of an ongoing process involving the planning, design and construction phases of the project. To provide this continued service, we recommend that the geotechnical engineer be provided the opportunity to review the project plans and specifications, and observe portions of the construction.

6.1 REVIEW OF PLANS AND SPECIFICATIONS

The geotechnical engineer should review the foundation and grading plans for the project. The purpose of the review is to evaluate if the plans and specifications were prepared in general accordance with the recommendations of this report.

6.2 GEOTECHNICAL OBSERVATION AND TESTING

Field exploration and site reconnaissance provides only a limited view of the geotechnical conditions of the site. Substantially more information will be revealed during the excavation and grading phases of the construction. Subsurface conditions, excavations and fill placement should be reviewed by the geotechnical professional during construction to evaluate if the materials encountered during construction are consistent with those assumed for this report.

7.0 REFERENCES

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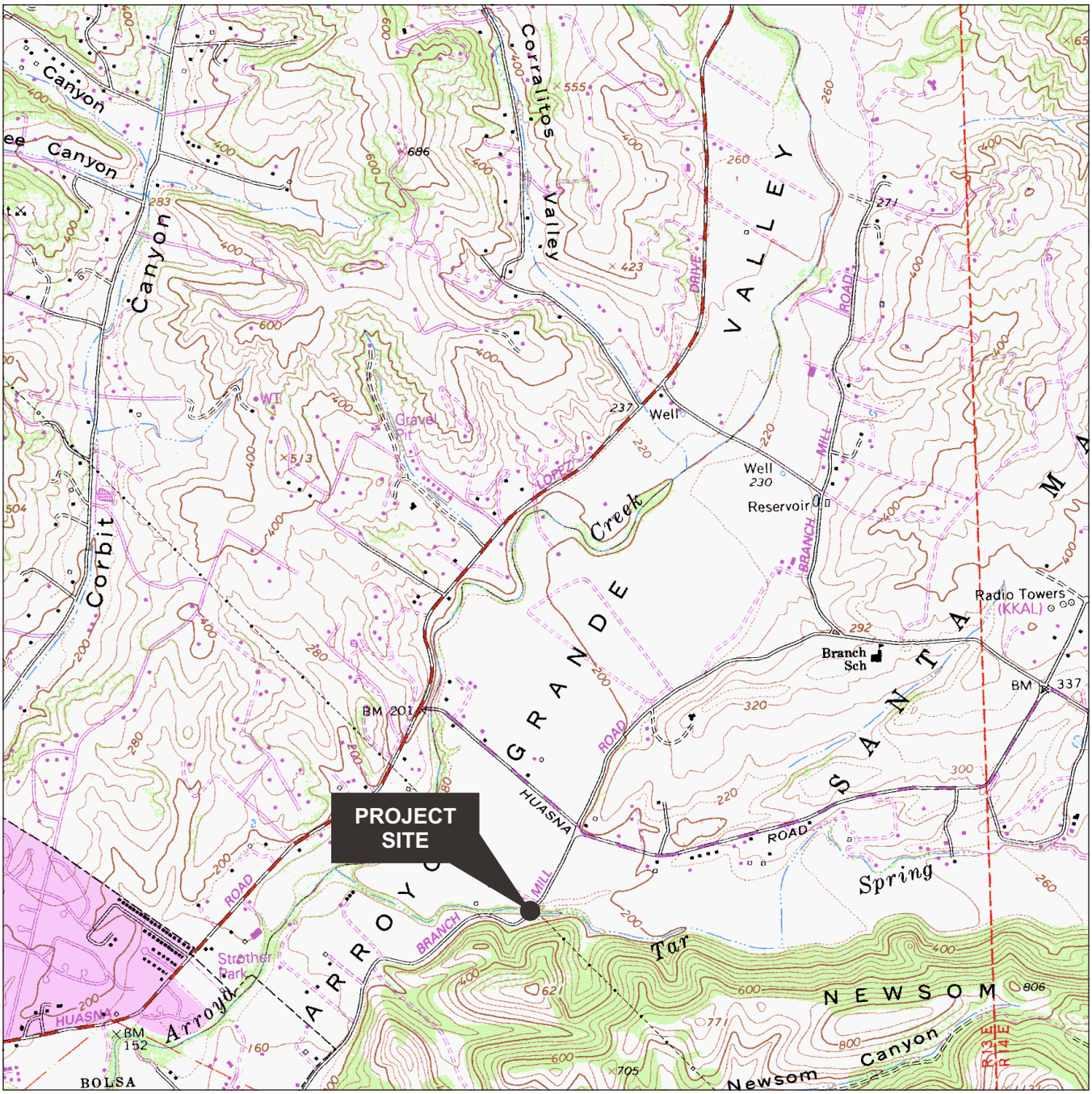
PLATES

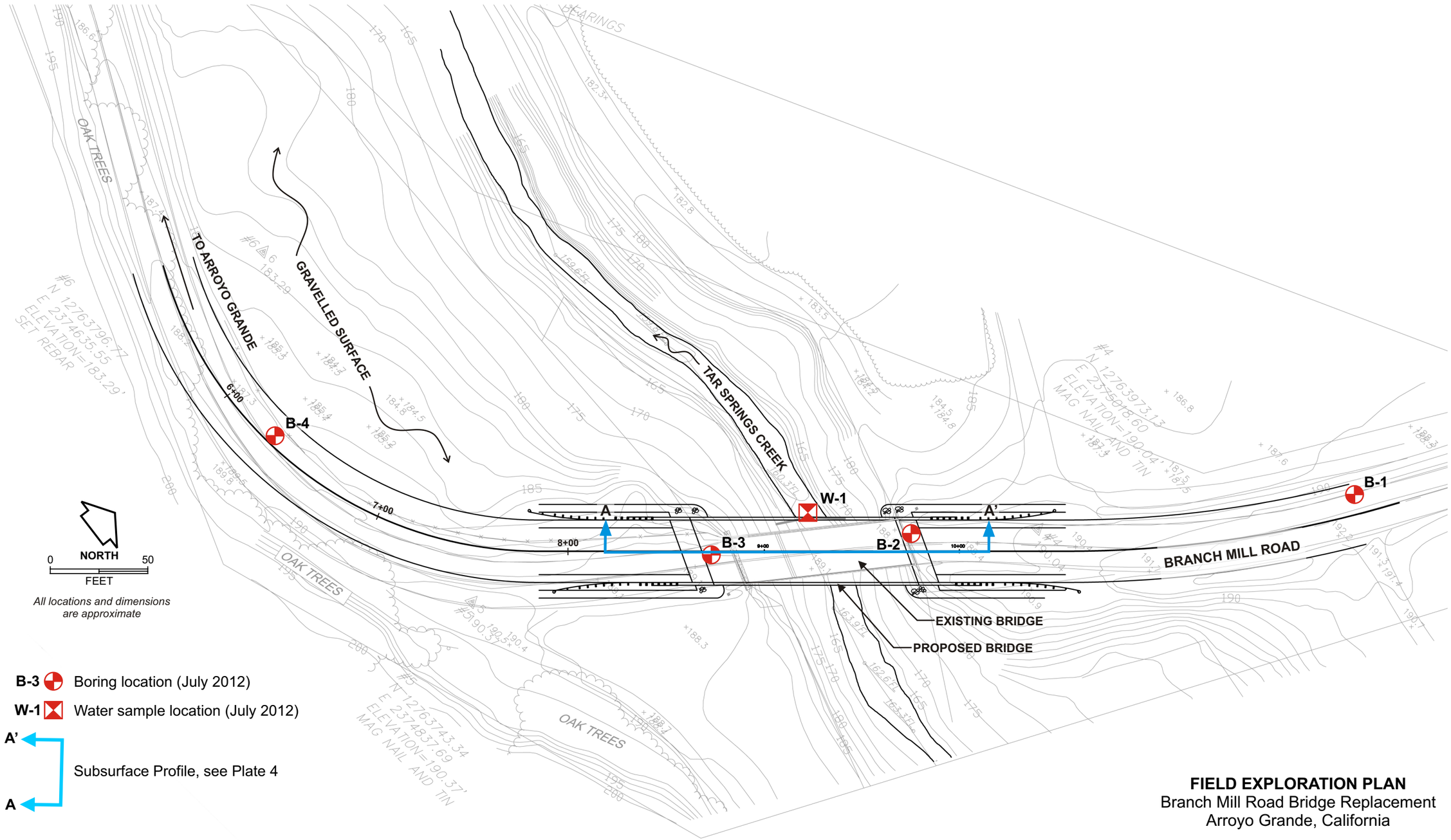


BASE MAP SOURCE: USGS 7.5' Arroyo Grande NE Quadrangle (Revised 1993).

NORTH

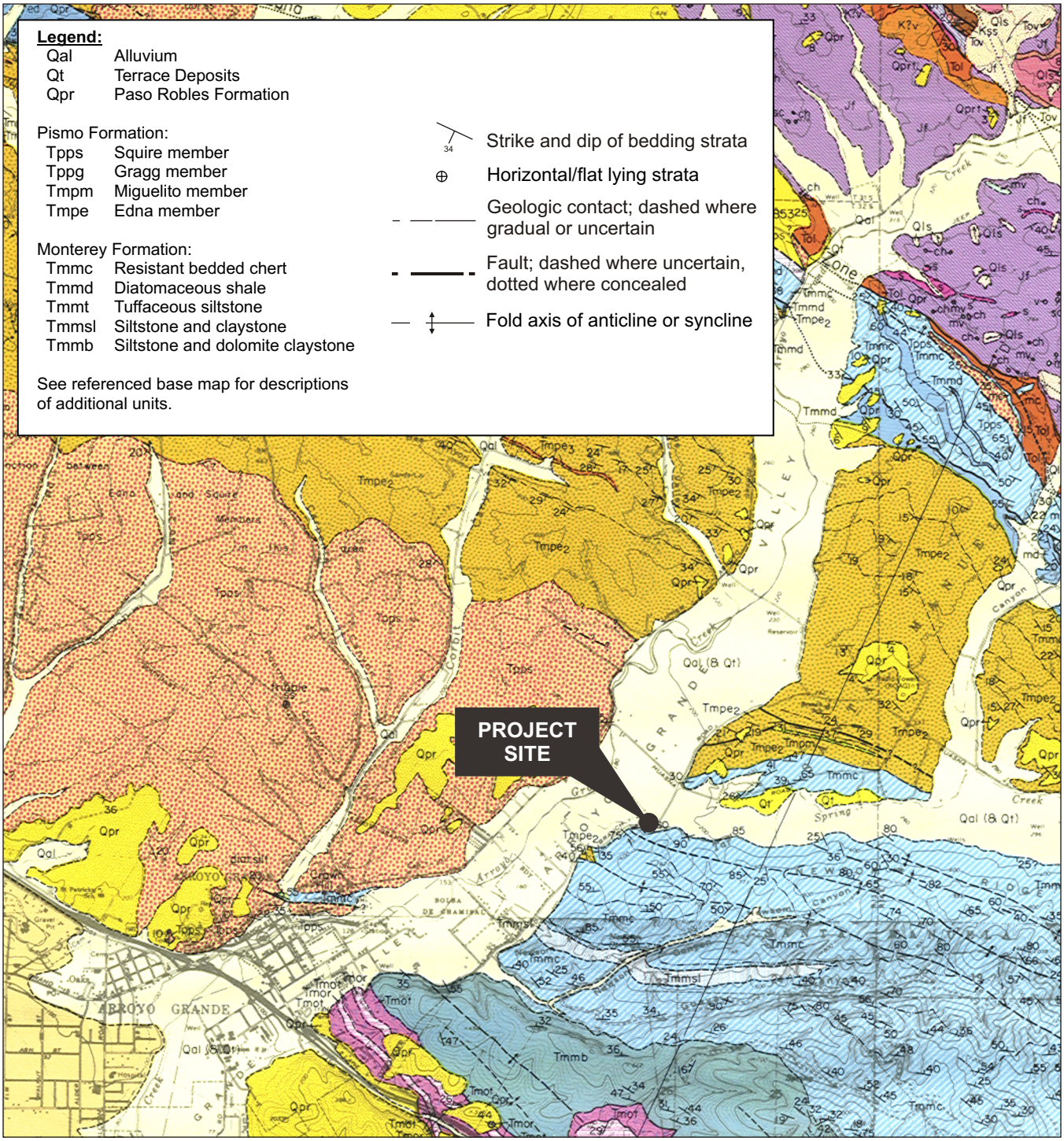
0 1000 2000
FEET

VICINITY MAP
Branch Mill Road Bridge Replacement
Arroyo Grande, California



FIELD EXPLORATION PLAN
Branch Mill Road Bridge Replacement
Arroyo Grande, California

Base Map: Quincy Engineering Topographical Map (2012).



BASE MAP SOURCE: Geology of the Arroyo Grande 15 minute Quadrangle, C.A. Hall, 1973.

NORTH

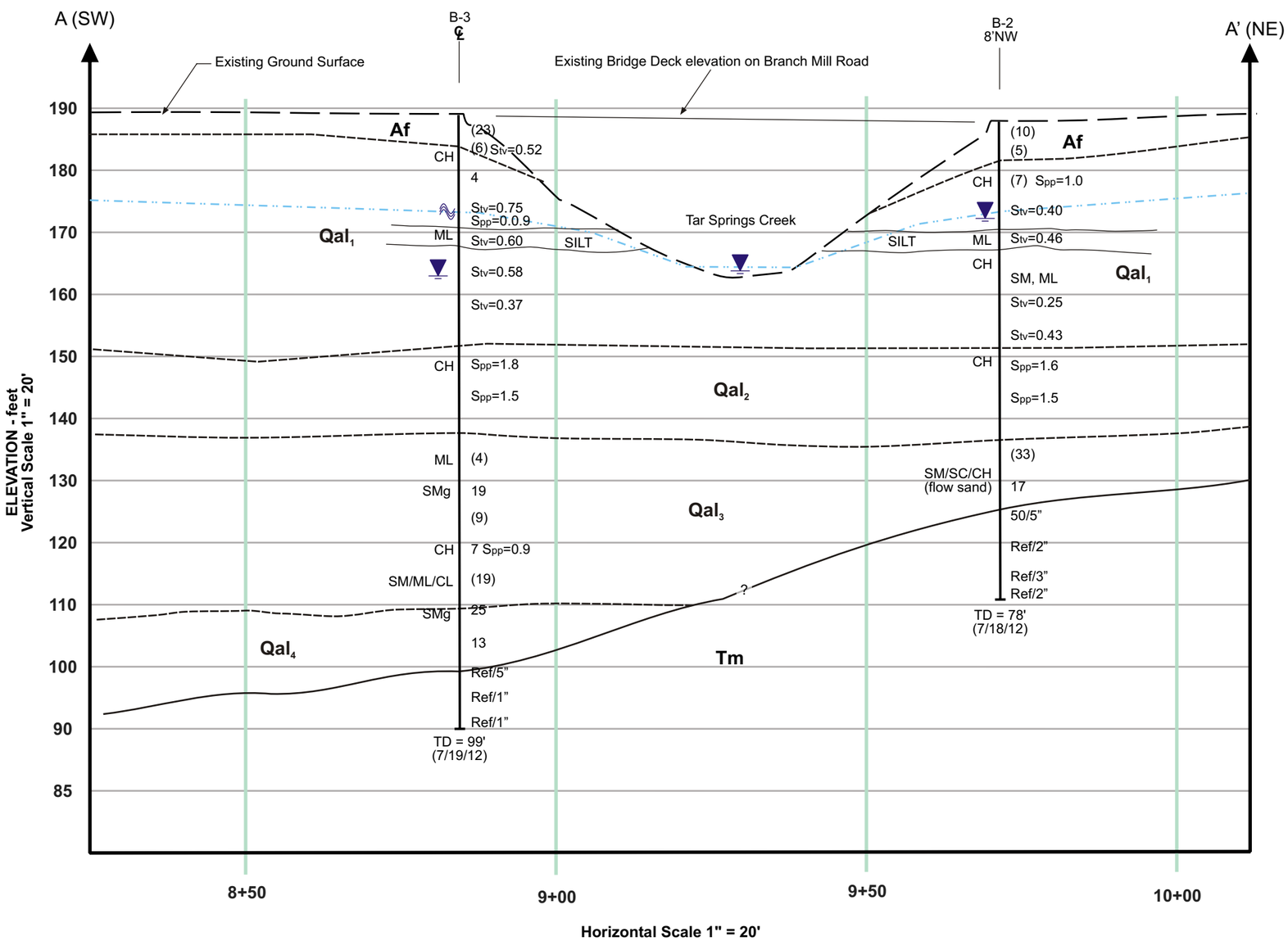
0 2000 4000

FEET

REGIONAL GEOLOGIC MAP

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE 3



Legend

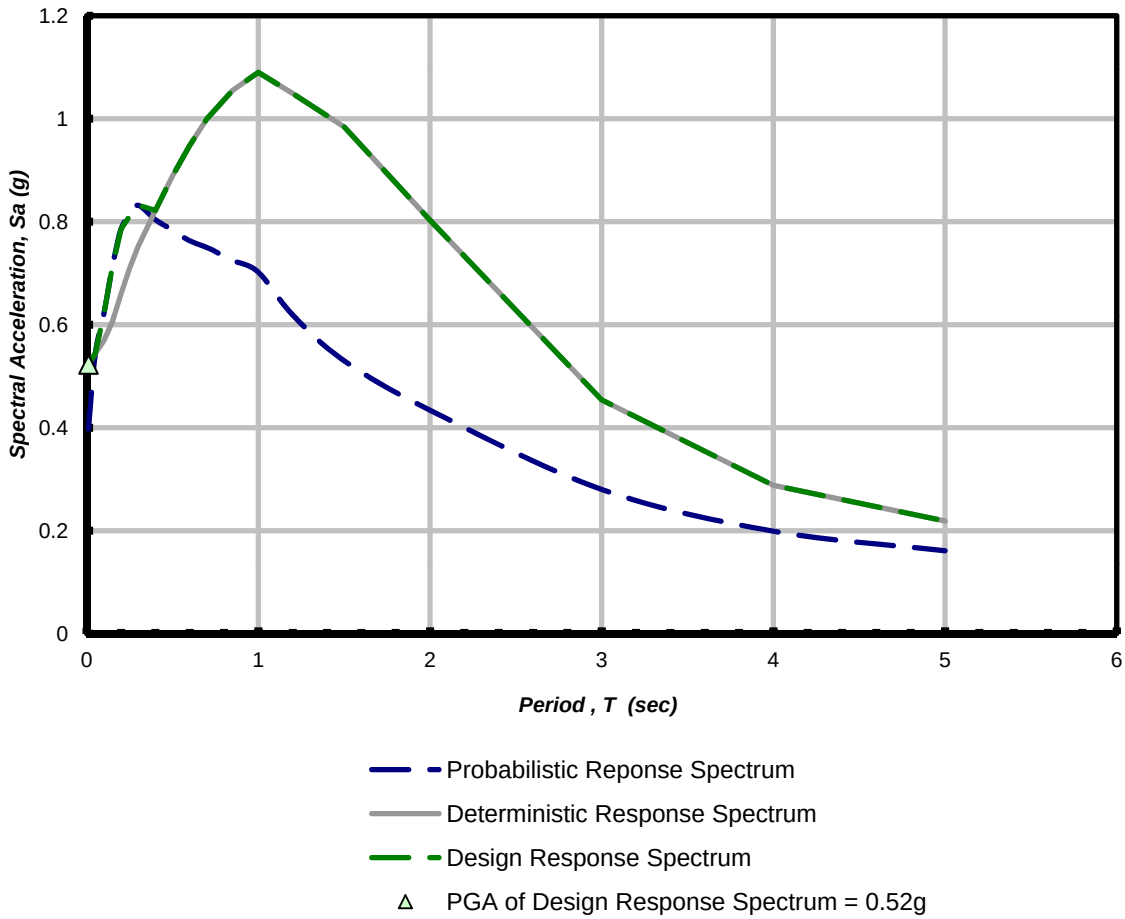
af	**Artificial fill:** medium to stiff clay and clayey sand with gravel
Qal_1	**Alluvium:** soft to medium stiff fat clay with interbeds of silt and sand
Qal_2	**Alluvium:** stiff to very stiff fat clay
Qal_3	**Alluvium:** loose to medium dense silty sand and sand with clay and interbeds of silt, clay, and gravel
Qal_4	**Alluvium:** medium dense silty sand with gravel
Tm	**Monterey Formation:** moderately hard and fractured cherty shale
B-1 130' SE	Boring number with offset and direction from section line when applicable
▼	Approximate groundwater elevation encountered in boring with assumed elevation between boring locations
≈	Zone of wet soil and groundwater seepage
S	Undrained shear strength tv is test result from torvane pp is test result from pocket penetrometer
┃	Approximate depth invertval of boring with sample blow counts (N-values) noted per Appendix A. N-value in parentheses were taken using the modfied California sampler
TD = 25' (6/17/10)	Total depth and completion date of boring

See text and logs for additional description and data concerning subsurface conditions.

Subsurface profile is interpolated between points of exploration and will vary from that shown.

All information and locations are approximate.

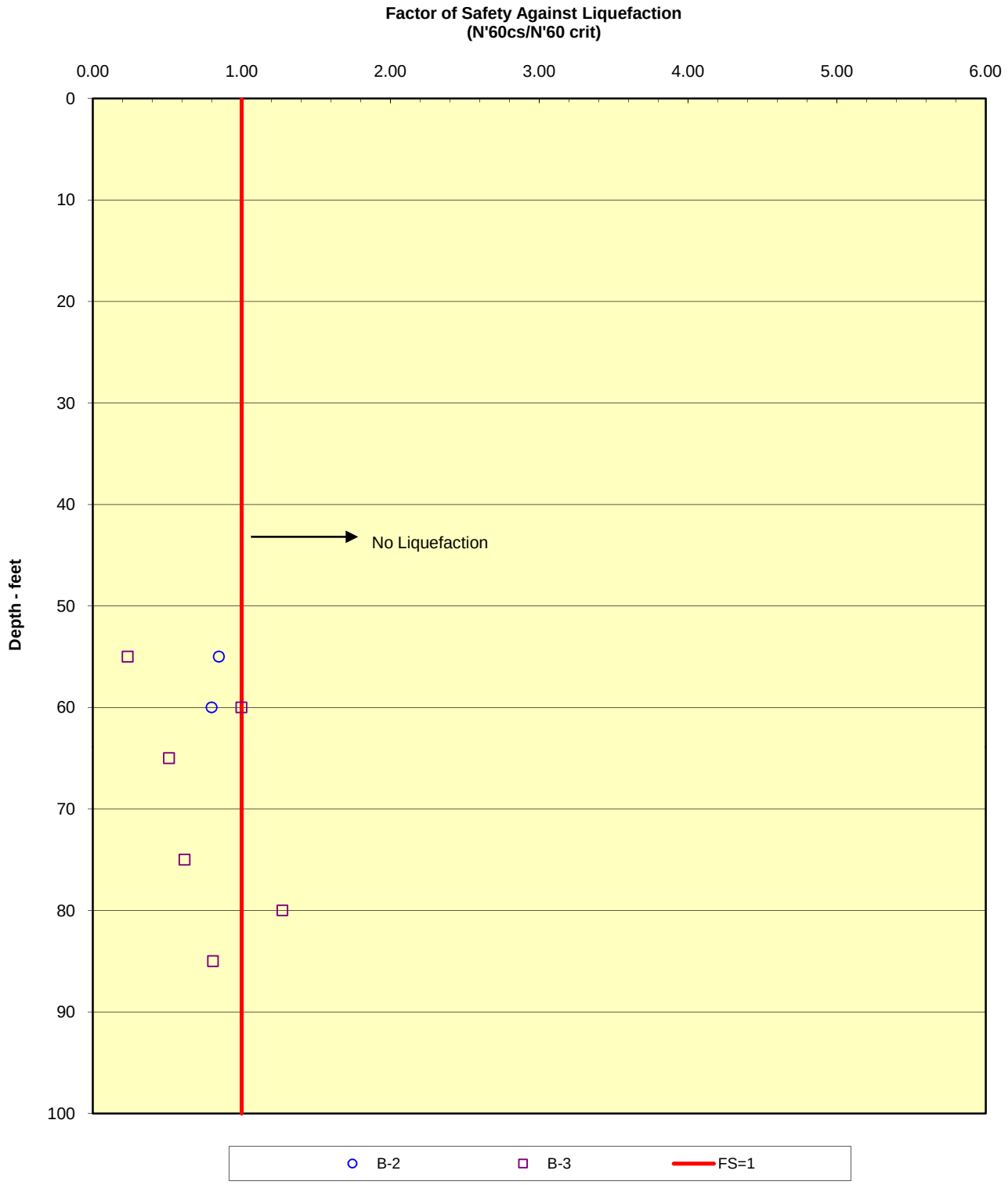
SUBSURFACE PROFILE A - A'
Branch Mill Road Bridge Replacement
Arroyo Grande, California



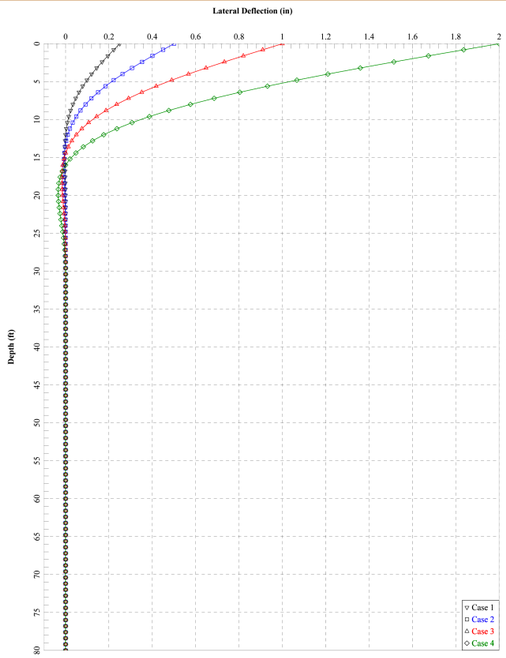
Soil Profile Type E
Estimated shear wave velocity of 175 meter/second
Site Coordinates: 35.1319 lat, -120.5428 long
Earthquake Magnitude: 7.2
Spectral Damping = 5 percent

DESIGN ACCELERATION RESPONSE SPECTRUM
Branch Mill Road Bridge Replacement
Arroyo Grande, California

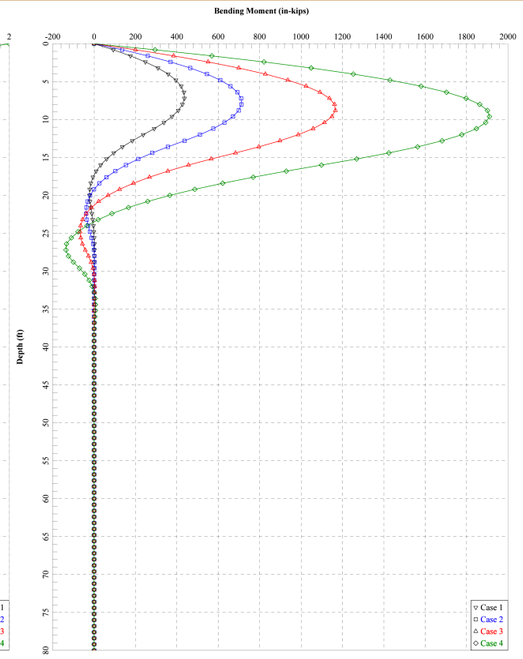
PLATE 5



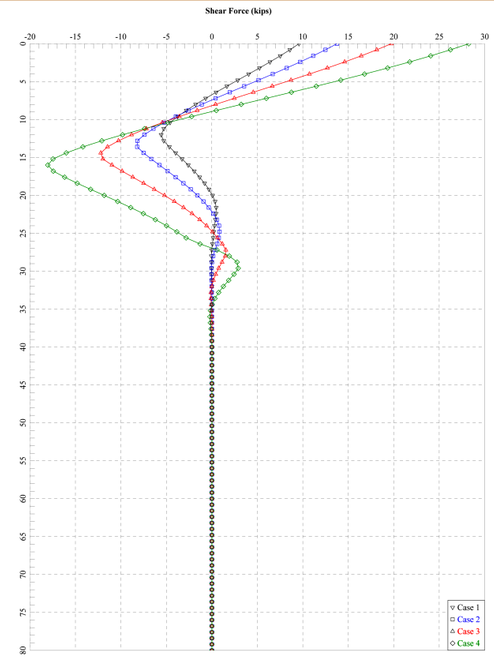
SUMMARY OF LIQUEFACTION POTENTIAL
Branch Mill Road Bridge Replacement
Arroyo Grande, California



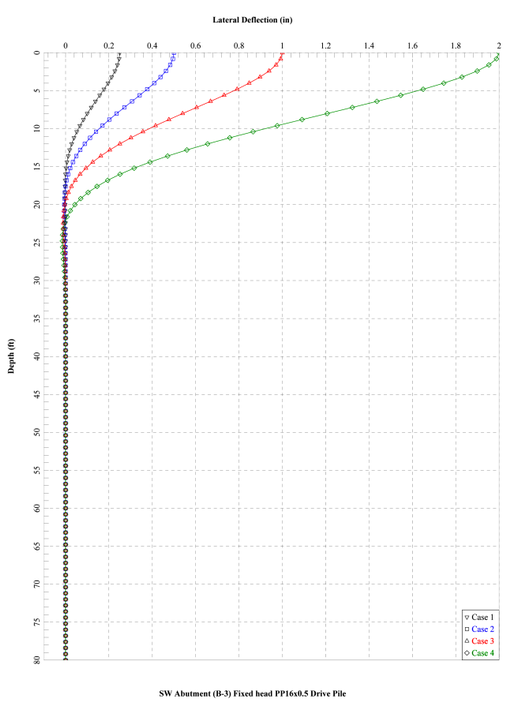
SW Abutment (B-3) Freehead PP16x0.5 Driven Pile



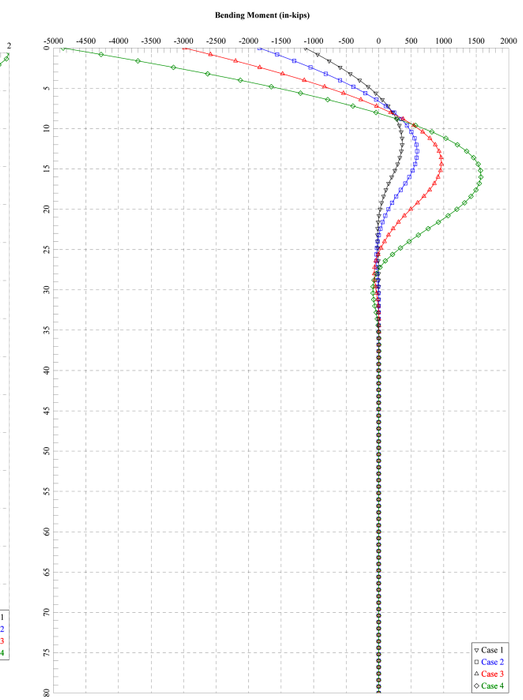
SW Abutment (B-3) Freehead PP16x0.5 Driven Pile

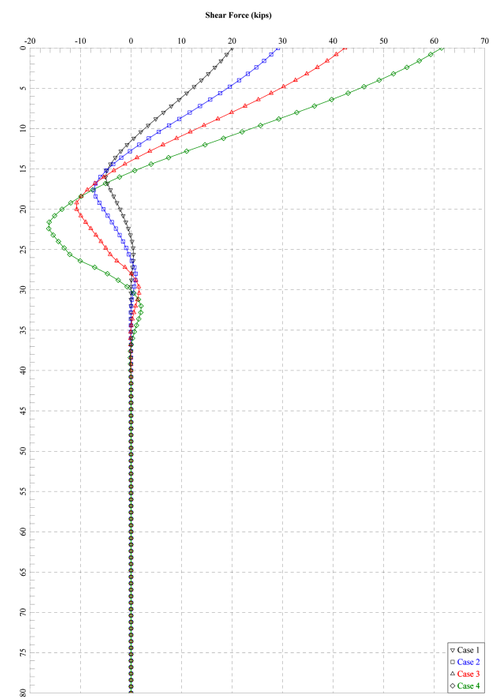


SW Abutment (B-3) Freehead PP16x0.5 Driven Pile



SW Abutment (B-3) Fixed head PP16x0.5 Drive Pile





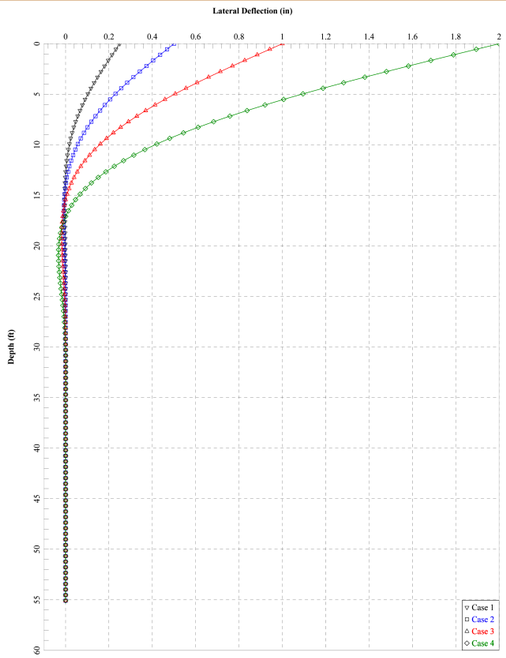
Free Head Condition

Pile Diameter (in.)	Deflection (in.)	Max. Shear (kips)	Max. Moment (in.-kips)
16	0.25	9.6	436
	0.50	13.7	712
	1.0	19.7	1165
	2.0	28.2	1910

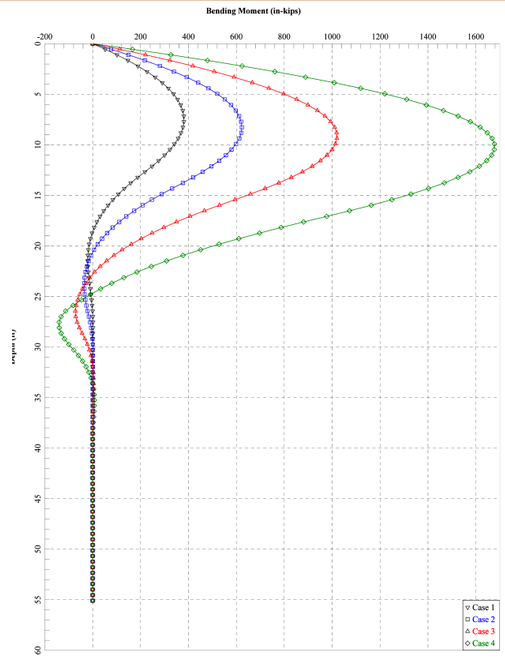
Fixed Head Condition

Pile Diameter (in.)	Deflection (in.)	Max. Shear (kips)	Max. Moment (in.-kips)
16	0.25	20.0	1128
	0.50	29.1	1834
	1.0	42.3	2985
	2.0	61.4	4850

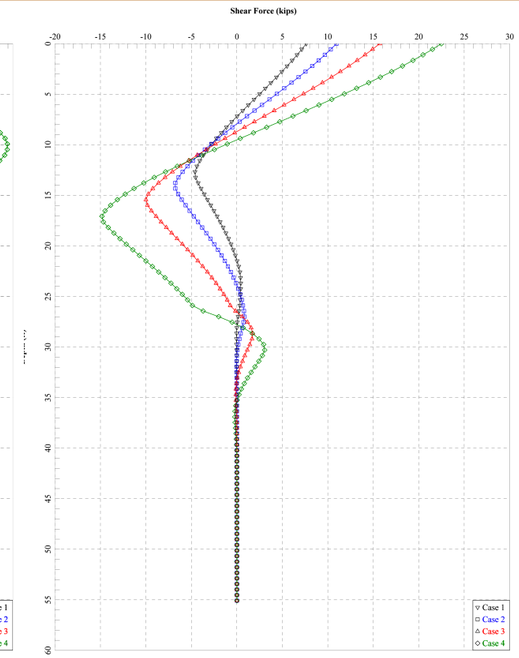
LATERAL LOAD RESISTANCE
SOUTHWEST ABUTMENT
Branch Mill Road Bridge Replacement
Arroyo Grande, California



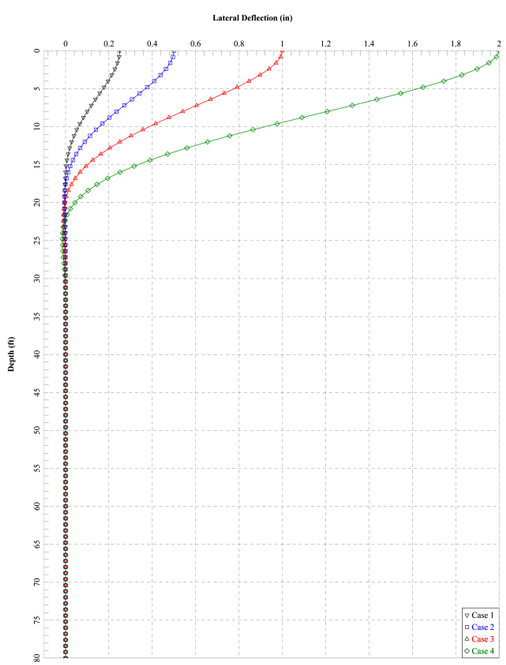
NE Abutment (B-2) Free Head PP16x0.5 Driven Pile



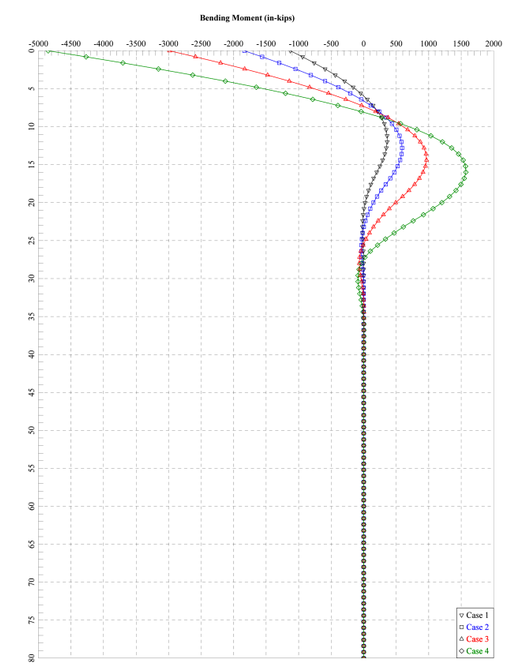
NE Abutment (B-2) Free Head PP16x0.5 Driven Pile



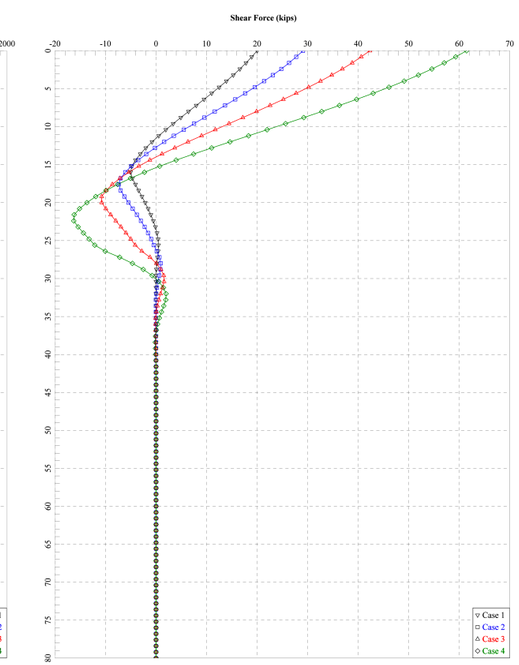
NE Abutment (B-2) Free Head PP16x0.5 Driven Pile



NE Abutment (B-2) Fixed Head PP16x0.5 Driven Pile



NE Abutment (B-2) Fixed Head PP16x0.5 Driven Pile



NE Abutment (B-2) Fixed Head PP16x0.5 Driven Pile

Free Head Condition

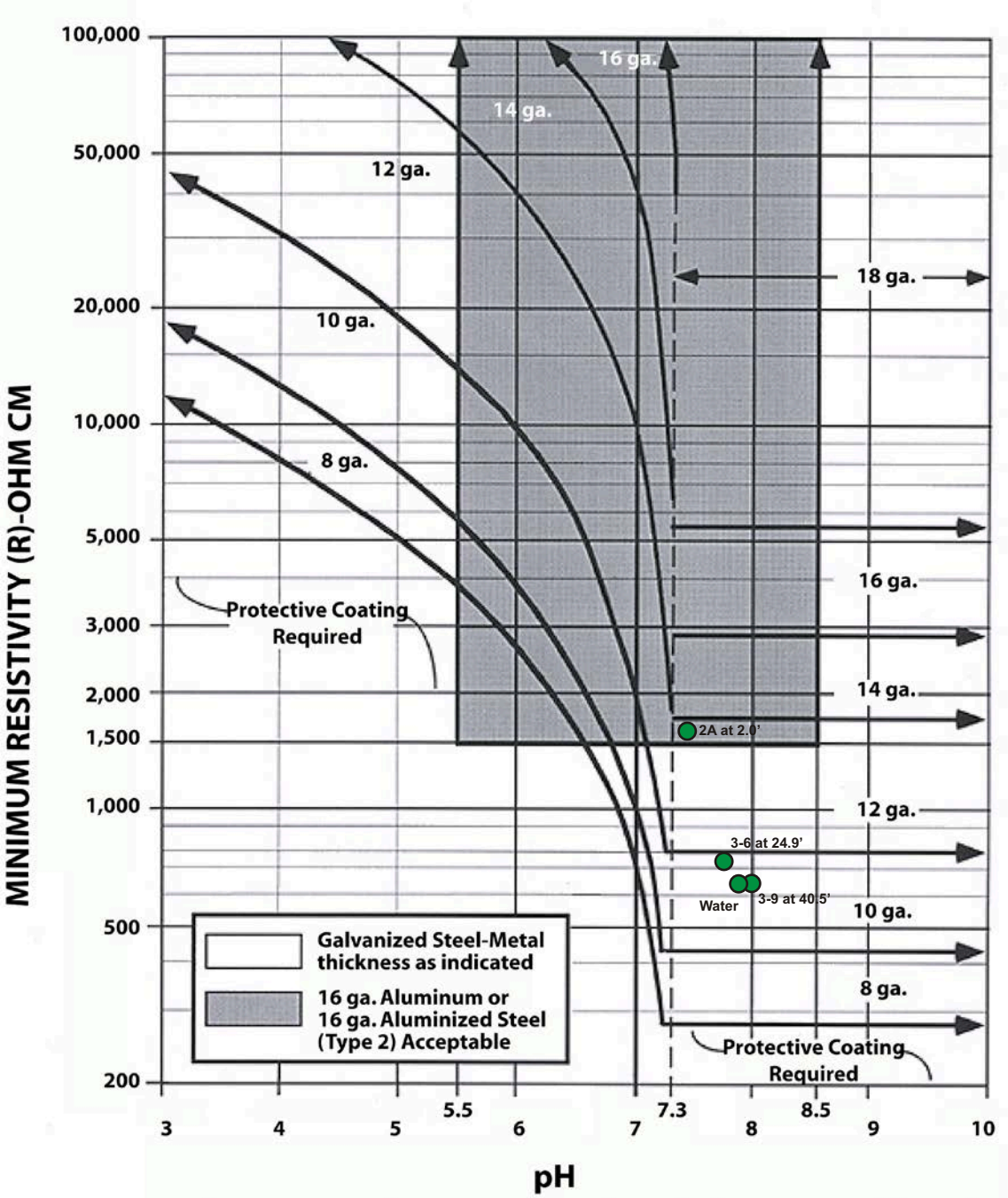
Pile Diameter (in.)	Deflection (in.)	Max. Shear (kips)	Max. Moment (in.-kips)
16	0.25	7.6	381
	0.50	10.9	622
	1.0	15.7	1020
	2.0	22.4	1677

Fixed Head Condition

Pile Diameter (in.)	Deflection (in.)	Max. Shear (kips)	Max. Moment (in.-kips)
16	0.25	16.1	982
	0.50	23.4	1599
	1.0	34.0	2597
	2.0	49.3	4207

LATERAL LOAD RESISTANCE
NORTHEAST ABUTMENT
Branch Mill Road Bridge Replacement
Arroyo Grande, California

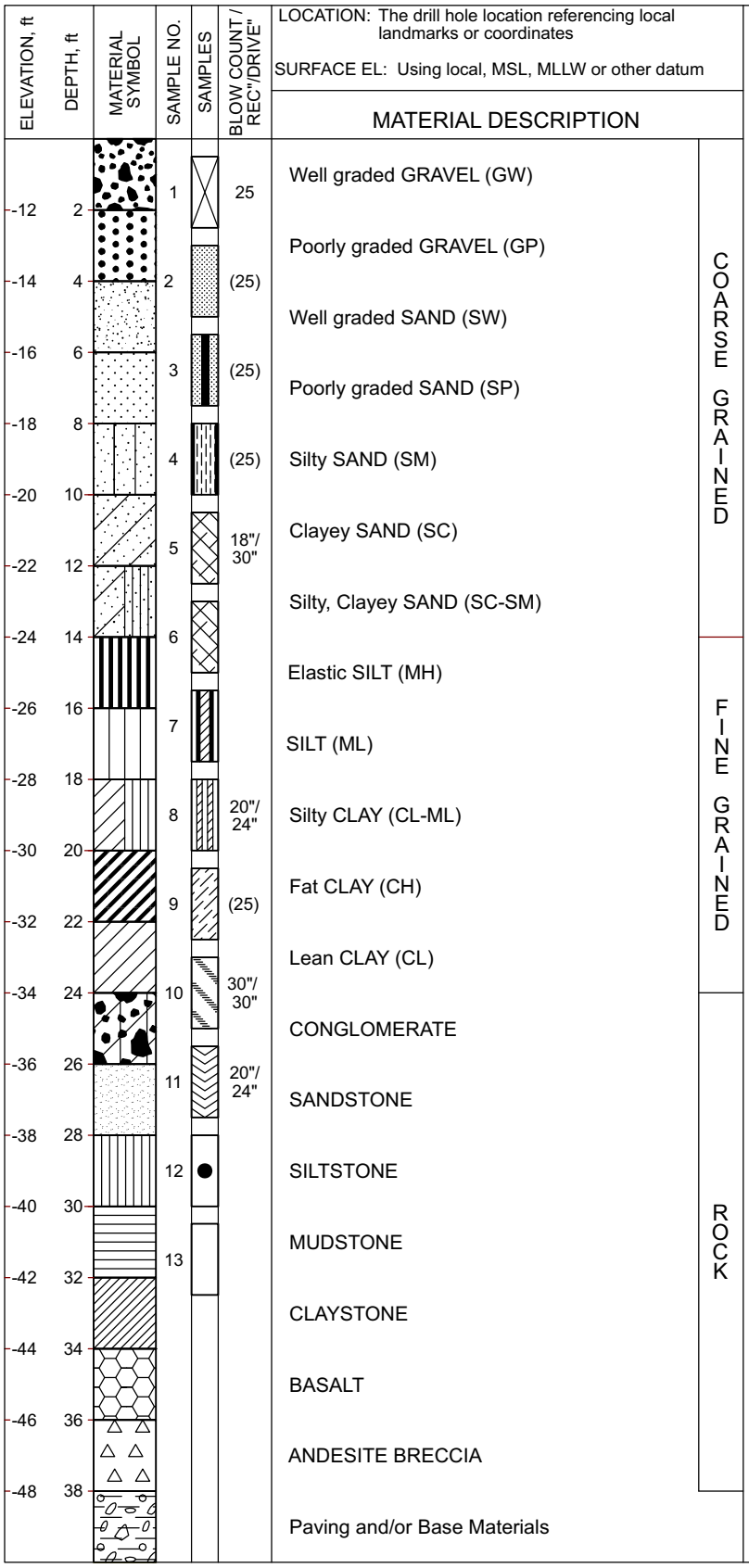
Figure 855.3A
Minimum Thickness of Metal Pipe for 50 Year Maintenance-Free Service Life (2)



Notes:
1. For pH and minimum resistivity levels not shown refer to Fig. 854.3C steel pipes. (California Test 643)
2. Service life estimate are for various corrosive conditions only.
3. Refer to index 854.3(2) and 854.4(2) for appropriate selection of metal thickness and protection coating to achieve service life requirements.

CORROSION TEST RESULTS FOR CULVERTS
Branch Mill Road Bridge Replacement
Arroyo Grande, California

APPENDIX A



LOCATION: The drill hole location referencing local landmarks or coordinates

SURFACE EL: Using local, MSL, MLLW or other datum

MATERIAL DESCRIPTION

COARSE GRAINED	Well graded GRAVEL (GW)
	Poorly graded GRAVEL (GP)
	Well graded SAND (SW)
	Poorly graded SAND (SP)
	Silty SAND (SM)
	Clayey SAND (SC)
	Silty, Clayey SAND (SC-SM)
FINE GRAINED	Elastic SILT (MH)
	SILT (ML)
	Silty CLAY (CL-ML)
	Fat CLAY (CH)
	Lean CLAY (CL)
ROCK	CONGLOMERATE
	SANDSTONE
	SILTSTONE
	MUDSTONE
	CLAYSTONE
	BASALT
	ANDESITE BRECCIA
	Paving and/or Base Materials

General Notes

Soil Texture Symbol

Sloped line in symbol column indicates transitional boundary

Samplers and sampler dimensions (unless otherwise noted in report text) are as follows:

Symbol for:

1	SPT Sampler, driven 1-3/8" ID, 2" OD
2	CA Liner Sampler, driven 2-3/8" ID, 3" OD
3	CA Liner Sampler, disturbed 2-3/8" ID, 3" OD
4	Thin-walled Tube, pushed 2-7/8" ID, 3" OD
5	Bulk Bag Sample (from cuttings)
6	CA Liner Sampler, Bagged
7	Hand Auger Sample
8	CME Core Sample
9	Pitcher Sample
10	Lexan Sample
11	Vibracore Sample
12	No Sample Recovered
13	Sonic Soil Core Sample

Sampler Driving Resistance

Number of blows with 140 lb. hammer, falling 30" to drive sampler 1 ft. after seating sampler 6"; for example,

Blows/ft	Description
25	25 blows drove sampler 12" after initial 6" of seating
86/11"	After driving sampler the initial 6" of seating, 36 blows drove sampler through the second 6" interval, and 50 blows drove the sampler 5" into the third interval
50/6"	50 blows drove sampler 6" after initial 6" of seating
Ref/3"	50 blows drove sampler 3" during initial 6" seating interval

Blow counts for California Liner Sampler shown in ( )

Length of sample symbol approximates recovery length

Classification of Soils per ASTM D2487 or D2488

Geologic Formation noted in bold font at the top of interpreted interval

Strength Legend

Q = Unconfined Compression
u = Unconsolidated Undrained Triaxial
t = Torvane
p = Pocket Penetrometer
m = Miniature Vane
c = Consolidated Undrained Triaxial

Water Level Symbols

Initial or perched water level
Final ground water level
Seepages encountered

Rock Quality Designation (RQD) is the sum of recovered core pieces greater than 4 inches divided by the length of the cored interval.

KEY TO TERMS & SYMBOLS USED ON LOGS

PLATE A-1

BEDDING SPACING	
Description	**Thickness/Spacing**
Massive	Greater than 10 ft
Very Thickly Bedded	3 ft - 10 ft
Thickly Bedded	1 ft - 3 ft
Moderately Bedded	4 in - 1 ft
Thinly Bedded	1 in - 4 in
Very Thinly Bedded	1/4 in - 1 in
Laminated	Less than 1/4 in

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Leaching		
Description	**Body of Rock**	**Fracture Surfaces**		**Texture**	**Leaching**	**General Characteristics**
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No leaching	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, grain boundary conditions	All fracture surfaces are discolored or oxidized; surfaces friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Texture altered by chemical disintegration (hydration, argillation)	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

ROCK HARDNESS

Description	Criteria
Extremely Hard	Cannot be scratched with a pocketknife or sharp pick. Can only be chipped with repeated heavy hammer blows
Very Hard	Cannot be scratched with a pocketknife or sharp pick. Breaks with repeated heavy hammer blows.
Hard	Can be scratched with a pocketknife or sharp pick with difficulty (heavy pressure). Breaks with heavy hammer blows.
Moderately Hard	Can be scratched with a pocketknife or sharp pick with light or moderate pressure. Breaks with moderate hammer blows
Moderately Soft	Can be grooved 1/16 in. deep with a pocketknife or sharp pick with moderate or heavy pressure. Breaks with light hammer blow or heavy manual pressure.
Soft	Can be grooved or gouged easily with a pocketknife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	Can be readily indented, grooved or gouged with fingernail, or carved with a pocketknife. Breaks with light manual pressure.

PERCENT CORE RECOVERY (REC)

$$\frac{\Sigma \text{ Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

ROCK QUALITY DESIGNATION (RQD)

$$\frac{\Sigma \text{ Length of intact core pieces } \geq 4 \text{ in.}}{\text{Total length of core run (in.)}} \times 100$$

RQD* indicates soundness criteria not met.

FRACTURE DENSITY

Description	Observed Fracture Density
Unfractured	No fractures
Very Slightly Fractured	Core lengths greater than 3 ft.
Slightly Fractured	Core lengths mostly from 1 to 3 ft.
Moderately Fractured	Core lengths mostly 4 in. to 1 ft.
Intensely Fractured	Core lengths mostly from 1 to 4 in.
Very Intensely Fractured	Mostly chips and fragments.

Reference: Caltrans (2010) Soil and Rock Logging and Presentation Manual, Figure 5-18.

TERMS AND DEFINITIONS USED FOR ROCK

Branch Mill Road Bridge
Arroyo Grande, California

LOCATION: Westbound lane, 231' east of existing bridge

SURFACE EL: 191 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	CORE/SAMPLE NO.	SAMPLERS	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ARTIFICIAL FILL (af) 5" asphalt concrete pavement over 6" aggregate base							
190						Clayey SAND with gravel (SC): loose, grayish brown, moist							
	2		1		(10)		109	94	17				
			B							42	38	21	
188			2		7								
	4					**ALLUVIUM (Qal)** Gravelly lean CLAY (CL): firm, brown, moist							
186			3		(8)		95	78	19 21				
	6					Silty SAND (SM): loose, light brown, moist							
184													
	8					Fat CLAY (CH): soft, dark gray, moist, with organics, interbedded light gray clayey SAND (SC)							
182			4		2								
	10												t 0.3
180													
	12												
178													
	14												
176													
	16												
174													
	18												
172													
	20												
170													
	22												
168													
	24												
166													
	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 10.5 ft
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 18, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-1

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE A-3

LOCATION: Westbound lane, 5' east of existing bridge

SURFACE EL: 189.5 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	CORE/SAMPLE NO.	SAMPLERS	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ARTIFICIAL FILL (af) 6" asphalt concrete over 7" aggregate base							
188	2		A			Clayey GRAVEL (GC) to Silty CLAY with gravel (CL-ML): firm, dark brown, moist, shale gravel to 3" diameter							
			1		(10)		93	77	20				
186	4												
			2		(5)								
184	6												
182	8					**ALLUVIUM (Qal)** Fat CLAY (CH): firm, dark gray to black, moist, with roots and organic pockets, secondary porosity and roots to 0.5mm common							
			3		(7)								p 1.0
180	10												
178	12												
176	14		4										
						- soft, black, wet	106	75	43				t 0.4
174	16												
172	18					Clayey SAND (SC): soft, gray, wet, lenses of silt, brown clay and sandy SILT							
			5										
170	20												t 0.4
						Fat CLAY (CH): soft, dark brown, wet, extensive roots and secondary porosity to 0.5mm					57	30	t 0.5
168	22												
166	24		6			- layer of silty SAND to sandy SILT (SM/ML): medium dense, gray, wet, weakly cemented, trace of clay	116	91	28	31			
164	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 78.0 ft
DEPTH TO WATER: 18.0 ft
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 18, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-2

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE A-4a

LOCATION: Westbound lane, 5' east of existing bridge

SURFACE EL: 189.5 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	CORE/SAMPLE NO.	SAMPLERS	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
162	28												
160	30		7			- moderate gray with pockets of gray sand							t 0.3
158	32												
156	34		8										
154	36					- dark gray/black							t 0.4
152	38					Fat CLAY (CH): stiff, black, moist							
150	40		9				112	82	38		67	46	p 1.6
148	42												
146	44		10										
144	46												p 1.5
142	48												
140	50		11										
138	52					Interbedded layers of clayey SAND with gravel (SC) and fat CLAY (CH): very stiff, olive, wet							

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 78.0 ft
DEPTH TO WATER: 18.0 ft
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 18, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-2

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE A-4b

LOCATION: Westbound lane, 5' east of existing bridge

SURFACE EL: 189.5 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	CORE/SAMPLE NO.	SAMPLERS	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
136						Poorly graded SAND with clay (SP-SC): interbed of fine to medium grained sand							
	54		12		(33)		125	103	22				
134													
	56												
132													
	58												
			13		17								
130	60					- approximately 6' of sand flowed into augers, redrill prior to sampling							
128													
	62												
126													
	64		14		50/5"								
124						**MONTEREY FORMATION (Tm)** SHALE: intensely weathered, olive to tan, moderately hard							
	66												
122													
	68												
120					(Ref/2")								
	70												
118													
	72					- hard drilling, approximately 1'/minute							
116													
	74		15		Ref/3"								
114						- dark gray							
	76												
112						- harder drilling, approximately 0.15'/minute							
	78				Ref/2"	- Practical refusal to drilling at 78'							
110													

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 78.0 ft
DEPTH TO WATER: 18.0 ft
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 18, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-2

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE A-4c

LOCATION: Westbound lane, 6' west of existing bridge

SURFACE EL: 189 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	CORE/SAMPLE NO.	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
				ARTIFICIAL FILL (af)							
				4" asphalt concrete pavement over 4" aggregate base							
188				Lean CLAY with gravel (CL): very stiff, brown, moist							
	2										
186		1	(23)								
	4	2	(6)								
				ALLUVIUM (Qal)							
184				Sandy SILT (ML): loose, light brown, moist							
	6			Fat CLAY (CH): medium stiff, dark gray, moist, organics							
182											t 0.5
	8										
180		3	4								
	10			- roots to 3mm, secondary porosity common							
178											
	12										
176											
	14	4									
174				- organic pockets, extensive iron oxide staining							t 0.8
	16				102	69	48				p 0.9
172											
	18			SILT with sand (ML): medium stiff, laminated gray and dark gray, wet, roots and secondary porosity common							
170		5									
	20										
168				Fat CLAY (CH): medium stiff, gray to dark gray, wet, slightly blocky structure, roots and secondary porosity common							t 0.6
	22										
166											
	24	6									
164				- slightly to moderately well-developed prismatic structure	108	75	44				t 0.6
	26										t 0.4

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 99.0 ft
DEPTH TO WATER: 26.0 ft
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 19, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-3

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE A-5a

LOCATION: Westbound lane, 6' west of existing bridge

SURFACE EL: 189 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	CORE/SAMPLE NO.	SAMPLERS	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
162													
	28												
160			7										
	30					- soft, wet							t 0.4
158													
	32												
156													
	34		8										
154						Clayey SAND (SC): medium dense, light olive gray, moist							
	36												
152													
	38												
150			9			Fat CLAY (CH): stiff to very stiff, dark gray/black, moist							
	40												p 1.8
148													
	42												
146													
	44		10										
144													p 1.5
	46						107	75	42		69	48	
142													
	48					Sandy lean CLAY (CL): firm, very dark greenish gray, wet, interbed of clayey sand							
140			11										
	50												
138						Silty SAND (SM): very loose, gray, wet, with interbedded sandy SILT (ML) and silty CLAY with sand (CL-ML)							
	52												
136													

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 99.0 ft
DEPTH TO WATER: 26.0 ft
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 19, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-3

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE A-5b

LOCATION: Westbound lane, 6' west of existing bridge

SURFACE EL: 189 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	CORE/SAMPLE NO.	SAMPLERS	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
	54		12		(4)		117	90	29	20			
134													
	56												
132													
	58					Silty SAND with gravel (SM): medium dense, olive gray, wet, angular clasts of green chert and shale							
130			13		19								
	60												
128													
	62												
126													
	64				9								
124													
	66												
122						Fat CLAY (CH): firm, dark olive gray, moist							
	68												
120			14		7								p 0.9
	70												
118													
	72					Interbedded silty SAND (SM): loose; sandy SILT (ML) and lean CLAY with sand (CL): dark olive, wet							
116													
	74		15		(19)		121	94	29	15			
114													
	76												
112													
	78					Silty SAND with gravel (SM): medium dense, dark olive gray, wet, with angular to subangular gravel							
110			16		25								

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 99.0 ft
DEPTH TO WATER: 26.0 ft
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 19, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-3

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE A-5c

LOCATION: Westbound lane, 6' west of existing bridge

SURFACE EL: 189 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	CORE/SAMPLE NO.	SAMPLERS	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
108													
	82												
106													
	84		17		13								
104													
	86												
102						- interbedded layers of gravel							
	88												
100			18		Ref/5"	**MONTEREY FORMATION (Tm)** SHALE: intensely weathered, dark brown, moderately hard							
	90												
98													
	92												
96						- drilling rate approximately 0.3'/minute							
	94		19		Ref/1"								
94													
	96												
92													
	98					- drilling rate approximately 0.1'/minute							
90			20		Ref/1"								
	100												
88													
	102												
86													
	104												
84													
	106												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 99.0 ft
DEPTH TO WATER: 26.0 ft
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 19, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-3

Branch Mill Road Bridge Replacement
Arroyo Grande, California

PLATE A-5d

LOCATION: Westbound lane, 250' west of existing bridge

SURFACE EL: 187.5 ft +/- (rel. datum)

ELEVATION, ft	DEPTH, ft	MATERIAL SYMBOL	CORE/SAMPLE NO.	SAMPLERS	BLOW COUNT OR RECOVERY/RQD %	MATERIAL DESCRIPTION	UNIT WET WEIGHT, pcf	UNIT DRY WEIGHT, pcf	WATER CONTENT, %	% PASSING #200 SIEVE	LIQUID LIMIT, %	PLASTICITY INDEX, %	UNDRAINED SHEAR STRENGTH, S_u, ksf
						ARTIFICIAL FILL (af)							
						4" asphalt concrete pavement over 6" aggregate base							
						Gravelly fat CLAY with sand (CH): stiff, dark brown, dry							
186	2		1		(17)		81	61	31				
			C			- moist				25	67	43	
184			2		11								
	4					- medium stiff							
			3		(12)		87	63	37				
182	6												
180						**ALLUVIUM (Qal)**							
	8					Fat CLAY with gravel (CH): soft, dark brown, moist							
178			4		4								
	10												
176	12												
174	14												
172	16												
170	18												
168	20												
166	22												
164	24												
162	26												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

COMPLETION DEPTH: 10.5 ft
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Sand Cement Slurry
DRILLING DATE: July 19, 2012

DRILLING METHOD: 8-inch-dia. Hollow Stem Auger
HAMMER TYPE: Automatic Trip
DRILLED BY: S/G Drilling Company
LOGGED BY: C Lovato
CHECKED BY: J Blanchard

LOG OF BORING NO. B-4

Branch Mill Road Bridge Replacement
Arroyo Grande, California

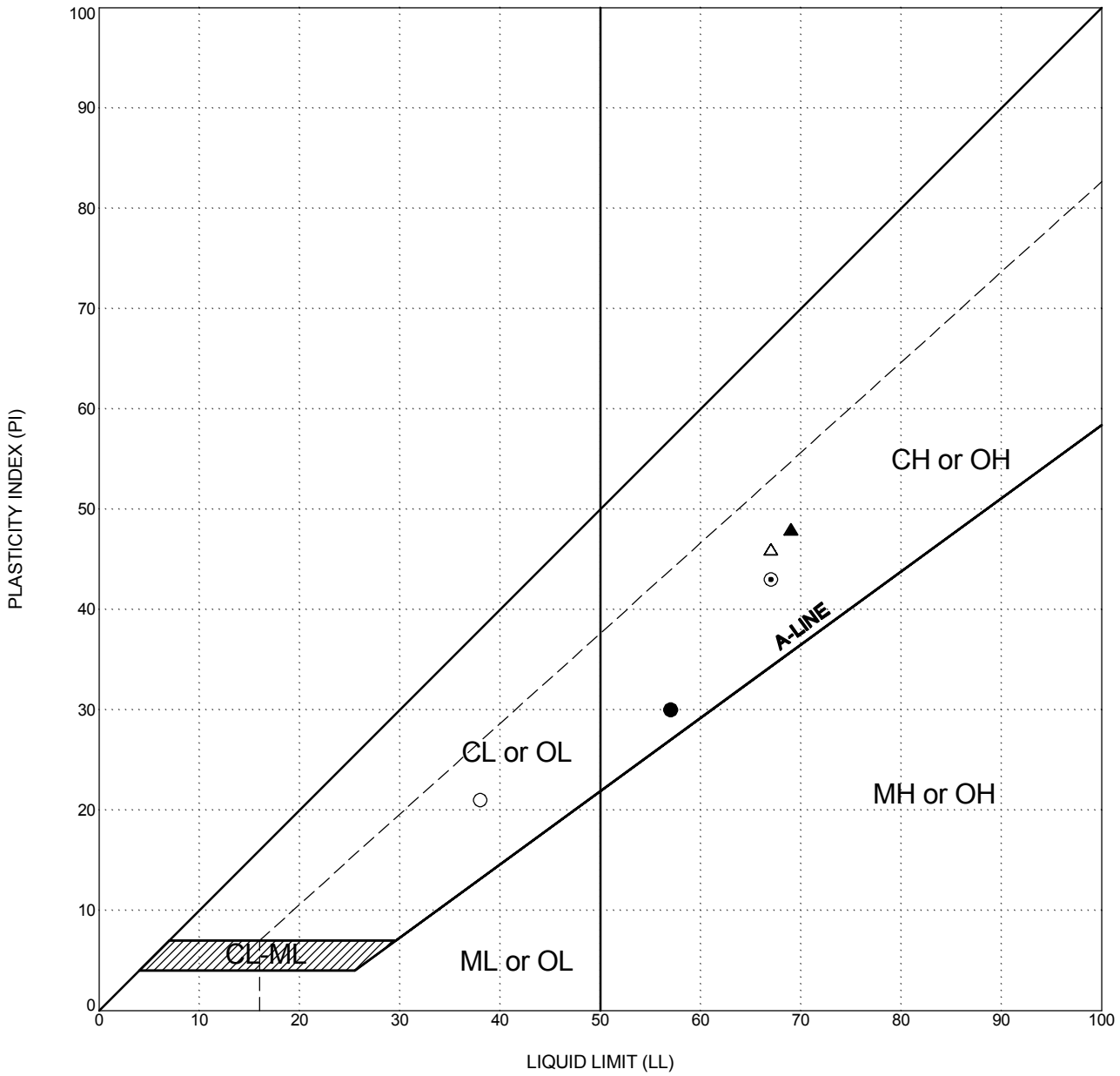
APPENDIX B

DRILL HOLE	DEPTH, ft	SAMPLE NUMBER	MATERIAL DESCRIPTION	UWW pcf	UDW pcf	MC %	FINES %	ATTERBERG LIMITS		COMPACTION TEST		DIRECT SHEAR		COMPRESSIVE STRENGTH TESTS		CORROSIVITY TESTS				R-VALUE	EXPANSION INDEX	SAND EQUIVALENT (SE)	SPECIFIC GRAVITY
								LL	PI	MAX DD pcf	OPT MC %	C ksf	PHI deg	Qu, ksf	S_u (Cell Prs.) ksf	R	pH	Cl	So_4 (ppm)				
B-1	1.5	1	Clayey SAND with gravel (SC)	109	94	17																	
B-1	2.0	B	Clayey SAND (SC)				42	38	21											22			
B-1	4.5	3	Lean CLAY with sand (CL)			19																	
B-1	4.7	3	Poorly graded SAND with silt (SP-SM)	95	78	21																	
B-2	2.0	A	Clayey SAND with gravel (SC)							102.3	20.6					1564	7.40	13	304.0		28		
B-2	2.5	1	Poorly graded GRAVEL with clay and sand (GP-GC)	93	77	20																	
B-2	15.0	4	Fat CLAY (CH)	106	75	43																	
B-2	20.8	5	Fat CLAY (CH)					57	30														
B-2	24.0	6	Clayey SAND (SC)	116	91	28	31																
B-2	40.0	9	Fat CLAY (CH)	112	82	38		67	46														
B-2	54.8	12	Poorly graded SAND with clay (SP-SC)	125	103	22																	
B-3	15.9	4	Fat CLAY (CH)	102	69	48																	
B-3	24.9	6	Fat CLAY (CH)													711	7.60	51	296.0				
B-3	25.4	6	Fat CLAY (CH)	108	75	44																	
B-3	40.5	9	Fat CLAY (CH)													653	8.00	39	278.0				
B-3	45.4	10	Fat CLAY (CH)	107	75	42		69	48														
B-3	54.0	12	Silty SAND (SM)	117	90	29	20																
B-3	74.0	15	Silty SAND (SM)	121	94	29	15																
B-4	1.5	1	Silty SAND with gravel (SM)	81	61	31																	
B-4	2.0	C	Clayey SAND with gravel (SC)				25	67	43											4			
B-4	4.5	3	Clayey SAND with gravel (SC)	87	63	37																	
W-1	0.0	W-1	Water													654	7.80	137	429.0				

SUMMARY OF LABORATORY TEST RESULTS

Branch Mill Road Bridge Replacement
Arroyo Grande, California

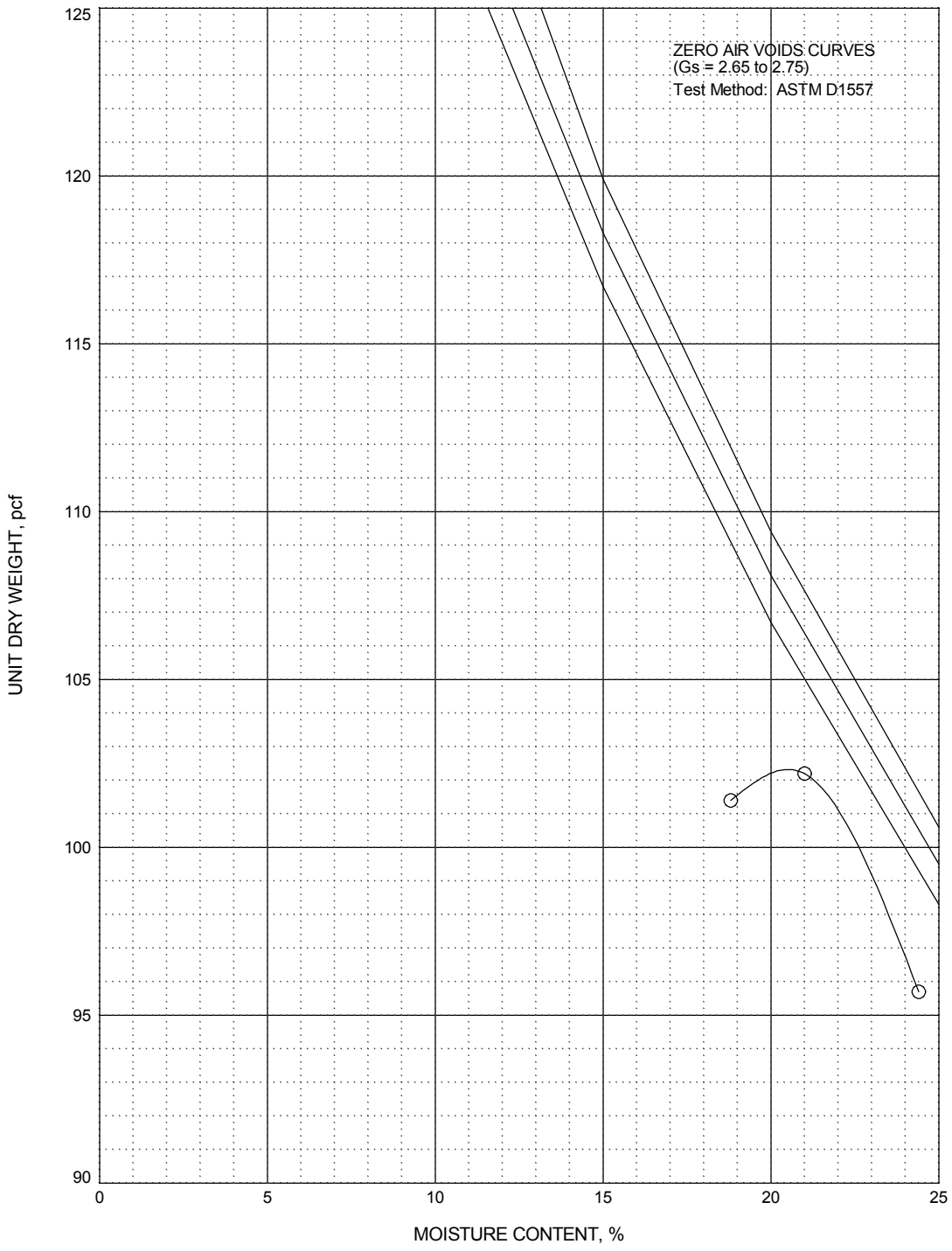
LAB SUMMARY TABLE VENTURA _F:\FUGRO SLO GEOTECH DOCUMENTS\GINT\GINT PROJECTS\04.6111.0010.GPJ_ 8/28/12 04:29 PM-cab



LEGEND	location	depth, ft	CLASSIFICATION	ATTERBERG LIMITS TEST RESULTS LIQUID LIMIT(LL)	PLASTIC LIMIT(PL)	PLASTICITY INDEX (PI)
○	B-1	2.0	Clayey SAND (SC)	38	17	21
●	B-2	20.8	Fat CLAY (CH)	57	27	30
△	B-2	40.0	Fat CLAY (CH)	67	21	46
▲	B-3	45.4	Fat CLAY (CH)	69	21	48
⦿	B-4	2.0	Clayey SAND with gravel (SC)	67	24	43

PLASTICITY CHART
Branch Mill Road Bridge Replacement
Arroyo Grande, California

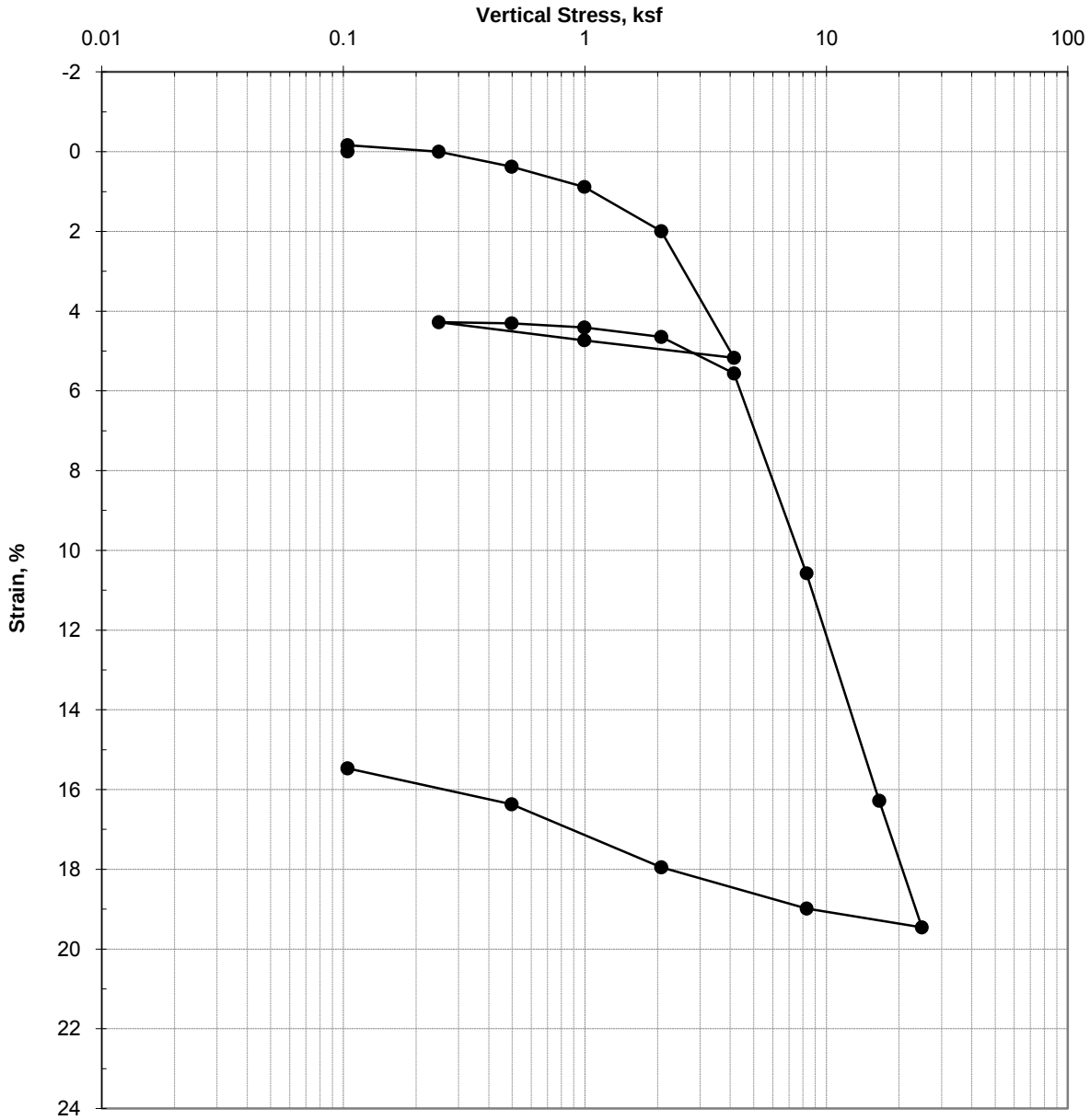
PLATE B-2



LEGEND	(location)	depth, ft	CLASSIFICATION	MAXIMUM UNIT DRY WEIGHT, pcf	OPTIMUM WATER CONTENT, %
○	B-2	2.0	Clayey SAND with gravel (SC)	102.3	20.6

COMPACTION TEST RESULTS
Branch Mill Road Bridge Replacement
Arroyo Grande, California

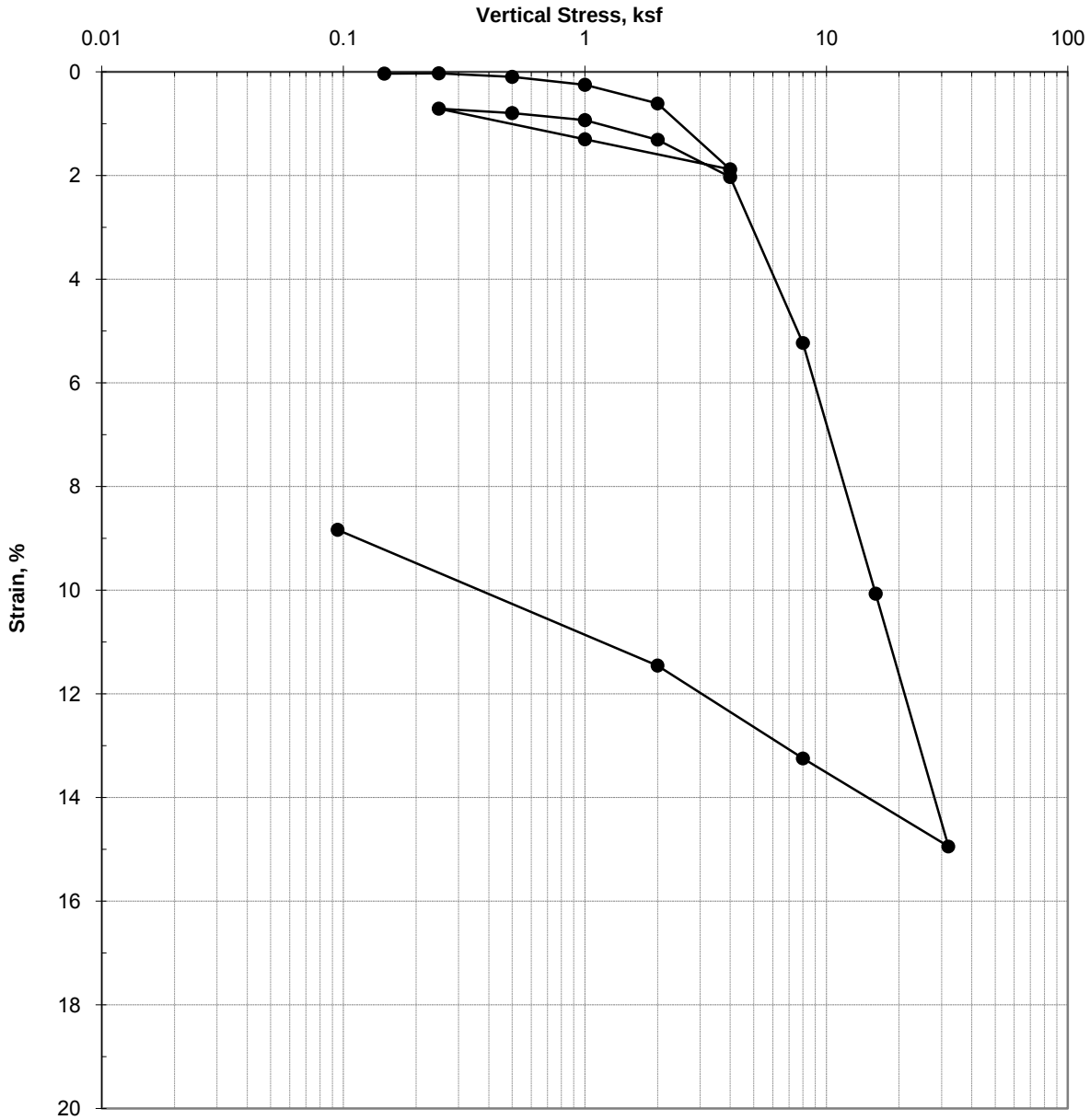
PLATE B-3



SAMPLE ID	Boring, Sample #, Depth	B-02 , #8 , 36.2 ft	
	USCS Classification:	Lean CLAY (CL): very dark grayish brown, moist	
PROPERTIES		Initial	Final
	Water Content, %	52.2%	39.6%
	Dry Unit Weight, pcf	68.1	80.6
	Saturation, %	97%	100%
	Voids Ratio	1.43	1.05
	Diameter, in	2.42	2.42
	Height, in	0.83	0.70

SUMMARY	Preconsolidation Pressure, ksf	---
	Inundation Increment, ksf	0.10
	Liquid Limit	---
	Plastic Limit	---
	Plasticity Index	---
	Passing #200	---
	Estimated Gs	2.65
REMARKS	Test Method: D2435	

CONSOLIDATION TEST RESULTS
Branch Mill Road Bridge Replacement
Arroyo Grande, California



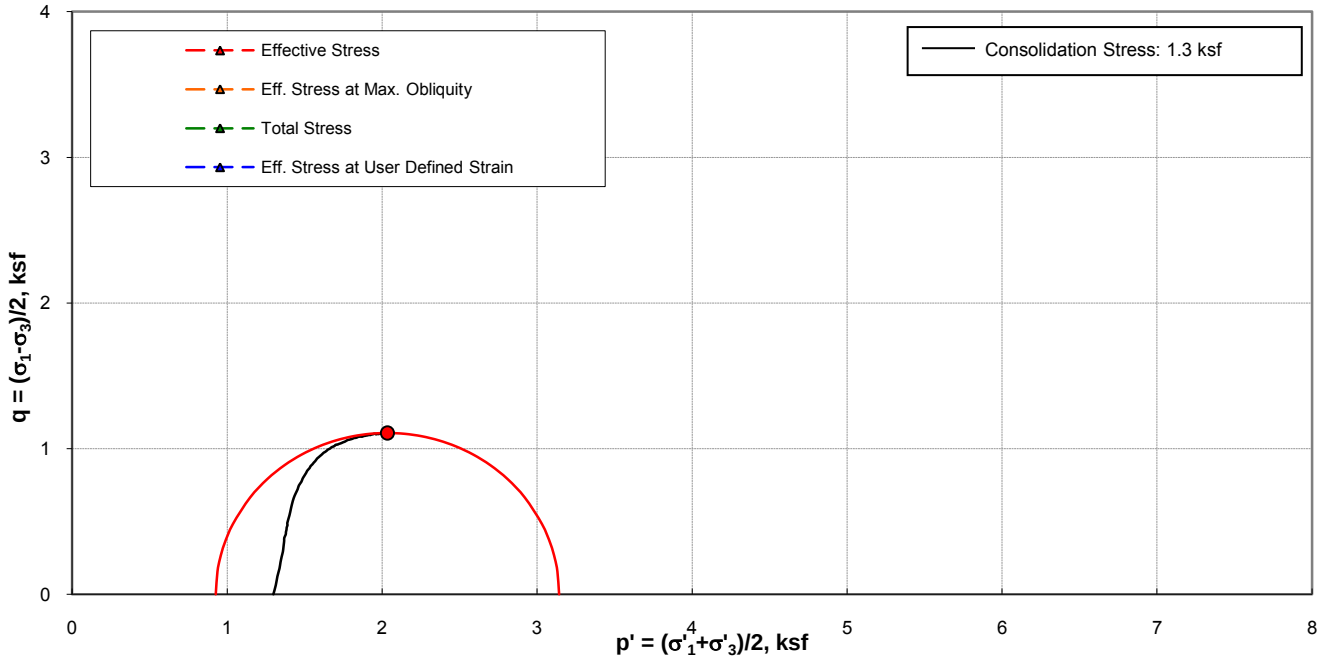
SAMPLE ID	Boring, Sample #, Depth	B-03 , #9 , 40.3 ft		SUMMARY	Preconsolidation Pressure, ksf	---
	USCS Classification:	Fat CLAY (CH): very dark grayish brown, moist			Inundation Increment, ksf	0.10
					Liquid Limit	---
PROPERTIES		Initial	Final		Plastic Limit	---
					Plasticity Index	---
	Water Content, %	44.2%	40.1%		Passing #200	---
	Dry Unit Weight, pcf	71.6	78.6		Estimated Gs	2.65
	Saturation, %	89%	96%	REMARKS	Test Method: D2435	
	Voids Ratio	1.31	1.11			
	Diameter, in	2.52	2.52			
	Height, in	0.92	0.84			

CONSOLIDATION TEST RESULTS

Branch Mill Road Bridge Replacement

Arroyo Grande, California

PLATE B-4b



SAMPLE ID				
Boring Number.:	B-02	---	---
Sample Number.:	4	---	---
Sample Depth:	15.9 ft	---	---
USCS Classification:	Fat CLAY (CH): very dark grayish brown, moist	---	---

INITIAL			
Sample No.	A	B	C
Water Content, %	42.5%	---	---
Dry Unit Weight, pcf	74.5	---	---
Saturation, %	92%	---	---
Void Ratio	1.22	---	---
Diameter, in	2.81	---	---
Height, in	5.80	---	---

PRE-SHEAR			
Water Content, %	44.2%	---	---
Dry Unit Weight, pcf	76.2	---	---
Saturation, %	100%	---	---
Void Ratio	1.17	---	---

REMARKS
Test Method: ASTM 4767

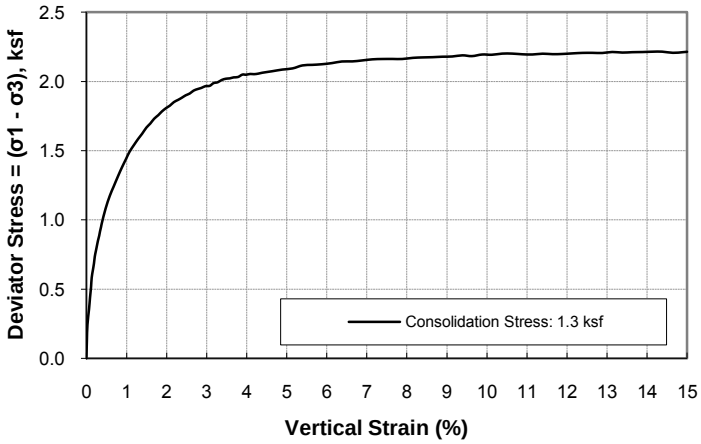
CLASSIFICATION			
Sample No.	A	B	C
Liquid Limit	---	---	---
Plastic Limit	---	---	---
Plastic Index	---	---	---
Passing #4 (4.75 mm)	---	---	---
Passing #200 (0.075 mm)	---	---	---
Estimated Gs	2.65	---	---

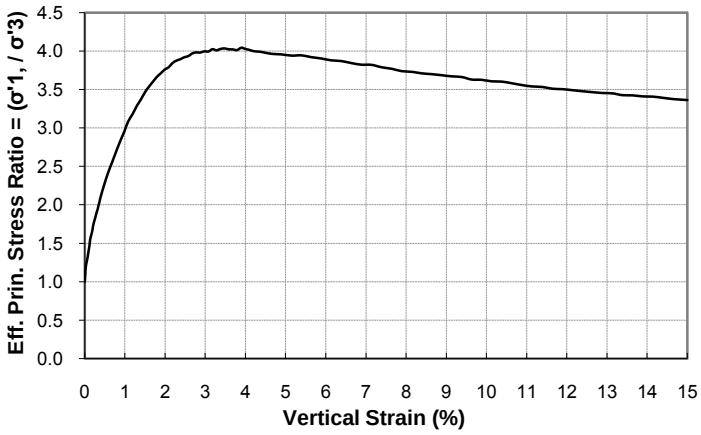
TEST SUMMARY			
Sample No.	A	B	C
B-Parameter	0.98	---	---
t_{50}, minutes	4.2	---	---
Strain Rate, %/min	0.0	---	---
Cell Pressure, ksf	8.6	---	---
Back Pressure, ksf	7.3	---	---
Consolidation Stress, ksf	1.3	---	---
Deviator Stress @ Failure, ksf	2.2	---	---
Axial Strain @ Failure, %	14.4	---	---
σ'_{1F}, ksf	3.1	---	---
σ'_{3F}, ksf	0.9	---	---
Tested By:	JC	---	---
Date Tested:	7/25/12	---	---

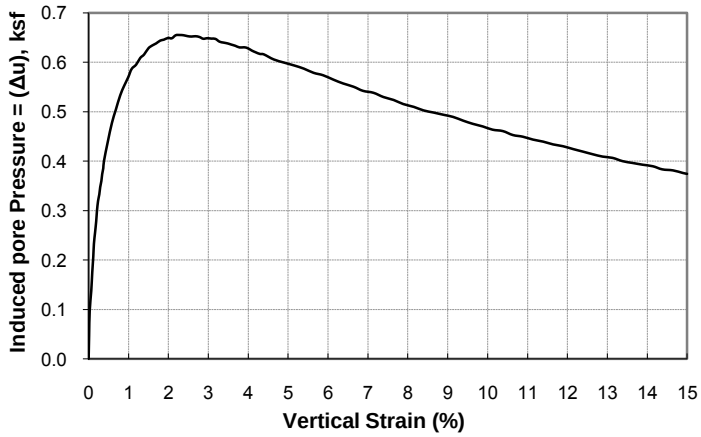
CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California







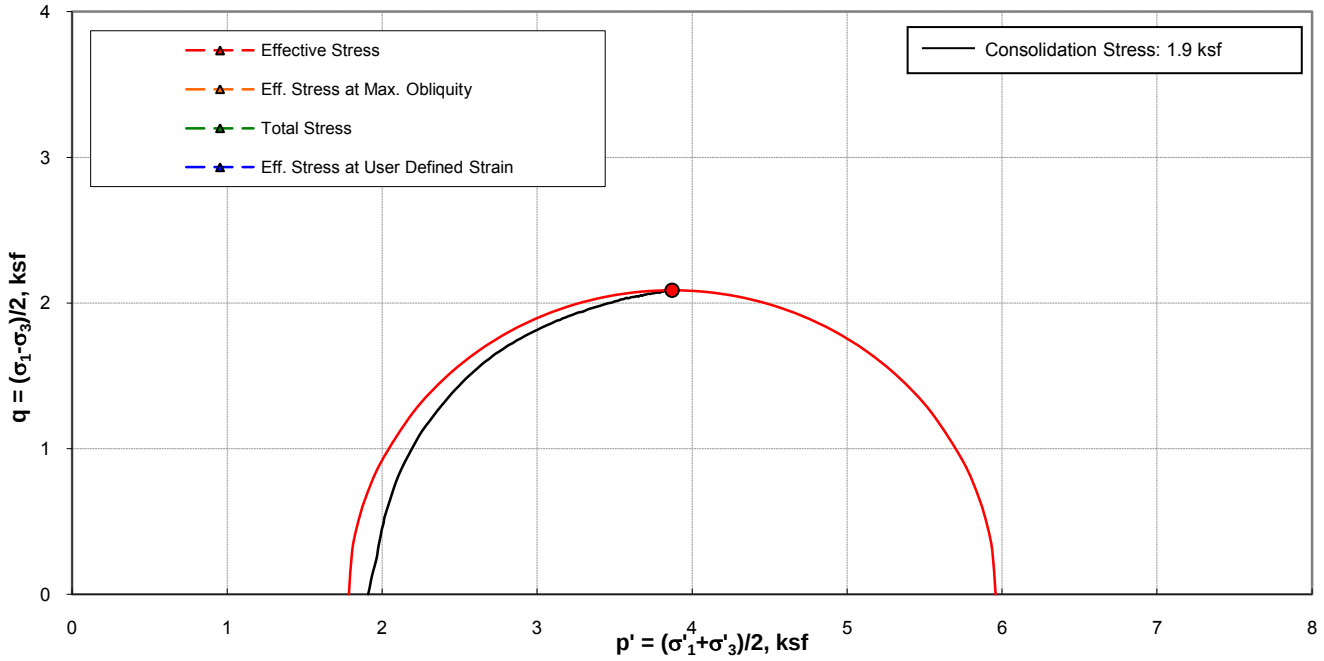
Consolidation Stress: 1.3 ksf

A	B-02	#4	15.9 ft	Fat CLAY (CH): very dark grayish brown, moist
B	---	---	---	---
C	---	---	---	---

CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California



SAMPLE ID				
Boring Number.:	B-02	---	---
Sample Number.:	9	---	---
Sample Depth:	40.8 ft	---	---
USCS Classification:	Fat CLAY (CH): very dark grayish brown, moist	---	---

INITIAL			
Sample No.	A	B	C
Water Content, %	37.7%	---	---
Dry Unit Weight, pcf	81.6	---	---
Saturation, %	96%	---	---
Void Ratio	1.06	---	---
Diameter, in	2.88	---	---
Height, in	5.80	---	---

PRE-SHEAR			
Water Content, %	38.0%	---	---
Dry Unit Weight, pcf	83.0	---	---
Saturation, %	100%	---	---
Void Ratio	1.03	---	---

REMARKS
Test Method: ASTM 4767

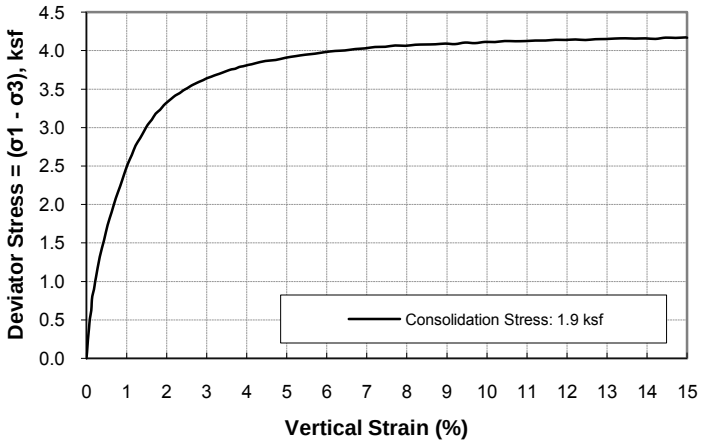
CLASSIFICATION			
Sample No.	A	B	C
Liquid Limit	67	---	---
Plastic Limit	21	---	---
Plastic Index	46	---	---
Passing #4 (4.75 mm)		---	---
Passing #200 (0.075 mm)	---	---	---
Estimated Gs	2.70	--	---

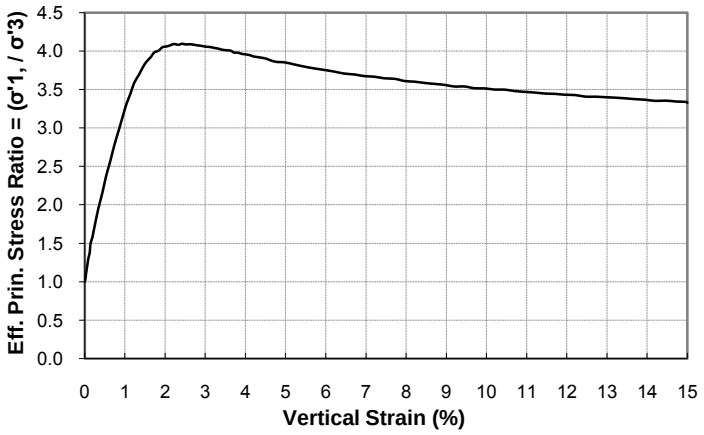
TEST SUMMARY			
Sample No.	A	B	C
B-Parameter	0.97	---	---
t_{50}, minutes	10.9	---	---
Strain Rate, %/min	0.0	---	---
Cell Pressure, ksf	9.2	---	---
Back Pressure, ksf	7.3	---	---
Consolidation Stress, ksf	1.9	---	---
Deviator Stress @ Failure, ksf	4.1	---	---
Axial Strain @ Failure, %	15.0	---	---
σ'_{1F}, ksf	5.9	---	---
σ'_{3F}, ksf	1.8	---	---
Tested By:	JC	---	---
Date Tested:	7/25/12	---	---

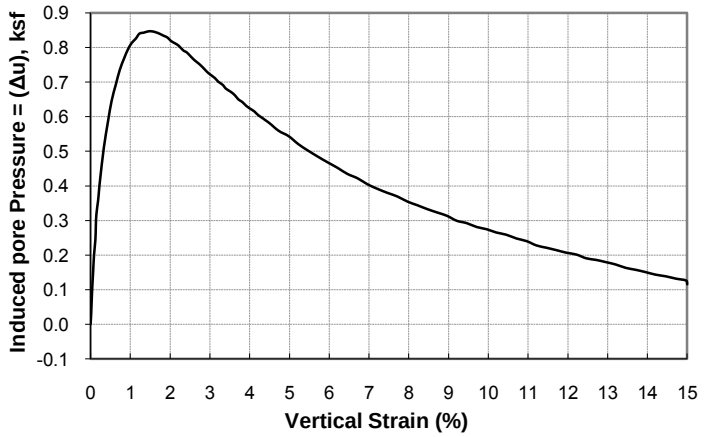
CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California







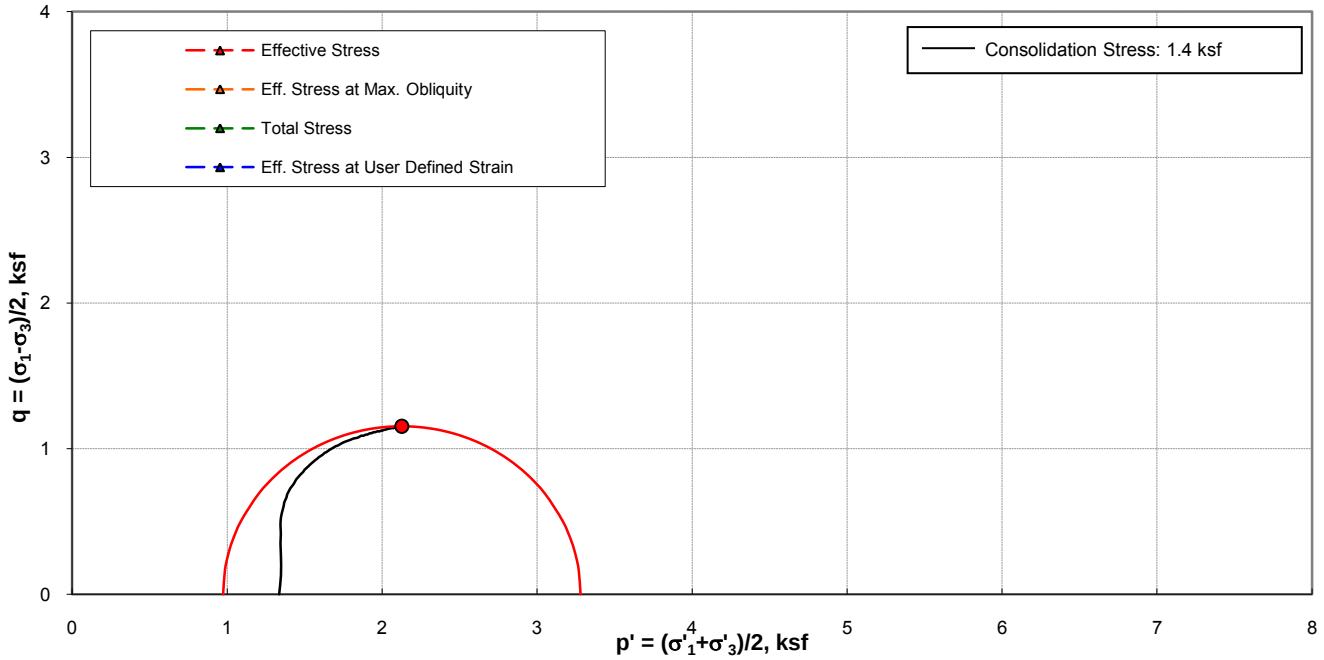
Consolidation Stress: 1.9 ksf

A	B-02	#9	40.8 ft	Fat CLAY (CH): very dark grayish brown, moist
B	---	---	---	---
C	---	---	---	---

CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California



SAMPLE ID				
Boring Number.:	B-03	---	---
Sample Number.:	4	---	---
Sample Depth:	15.9 ft	---	---
USCS Classification:	Fat CLAY (CH): very dark grayish brown, moist	---	---

INITIAL			
Sample No.	A	B	C
Water Content, %	47.9%	---	---
Dry Unit Weight, pcf	68.7	---	---
Saturation, %	90%	---	---
Void Ratio	1.41	---	---
Diameter, in	2.84	---	---
Height, in	5.80	---	---

PRE-SHEAR			
Water Content, %	48.9%	---	---
Dry Unit Weight, pcf	72.0	---	---
Saturation, %	100%	---	---
Void Ratio	1.30	---	---

REMARKS
Test Method: ASTM 4767

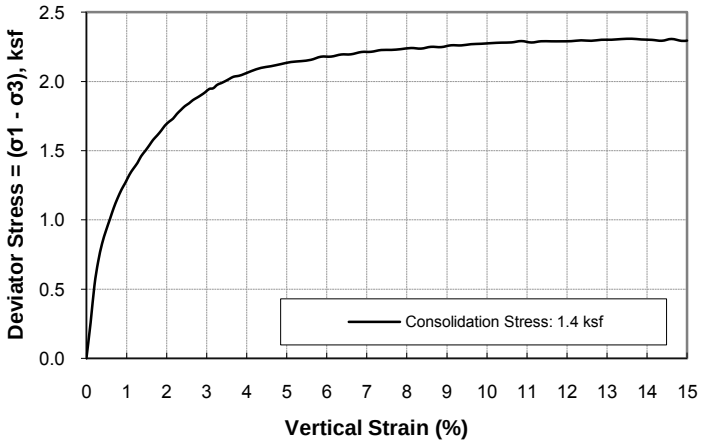
CLASSIFICATION			
Sample No.	A	B	C
Liquid Limit	---	---	---
Plastic Limit	---	---	---
Plastic Index	---	---	---
Passing #4 (4.75 mm)	---	---	---
Passing #200 (0.075 mm)	---	---	---
Estimated Gs	2.65	--	---

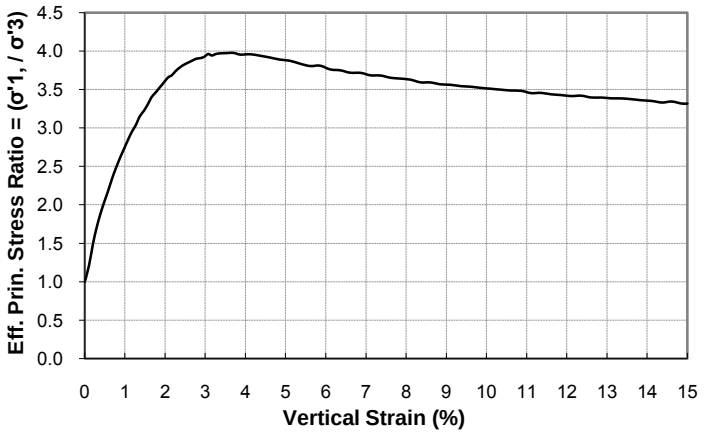
TEST SUMMARY			
Sample No.	A	B	C
B-Parameter	0.96	---	---
t_{50}, minutes	24.5	---	---
Strain Rate, %/min	0.0	---	---
Cell Pressure, ksf	11.3	---	---
Back Pressure, ksf	10.0	---	---
Consolidation Stress, ksf	1.4	---	---
Deviator Stress @ Failure, ksf	2.3	---	---
Axial Strain @ Failure, %	13.6	---	---
σ'_{1F}, ksf	3.3	---	---
σ'_{3F}, ksf	1.0	---	---
Tested By:	JC	---	---
Date Tested:	7/25/12	---	---

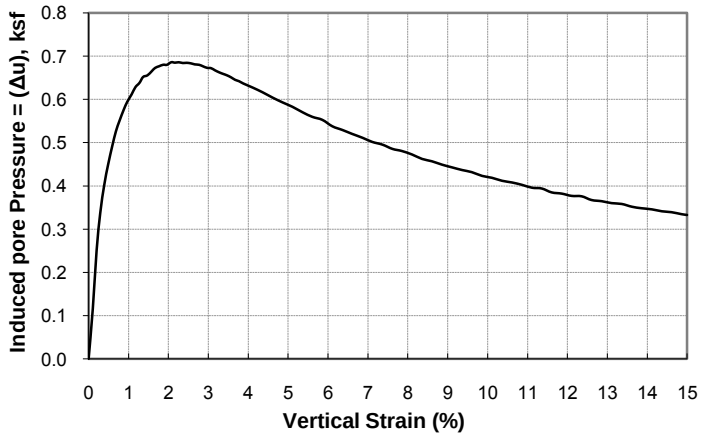
CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California







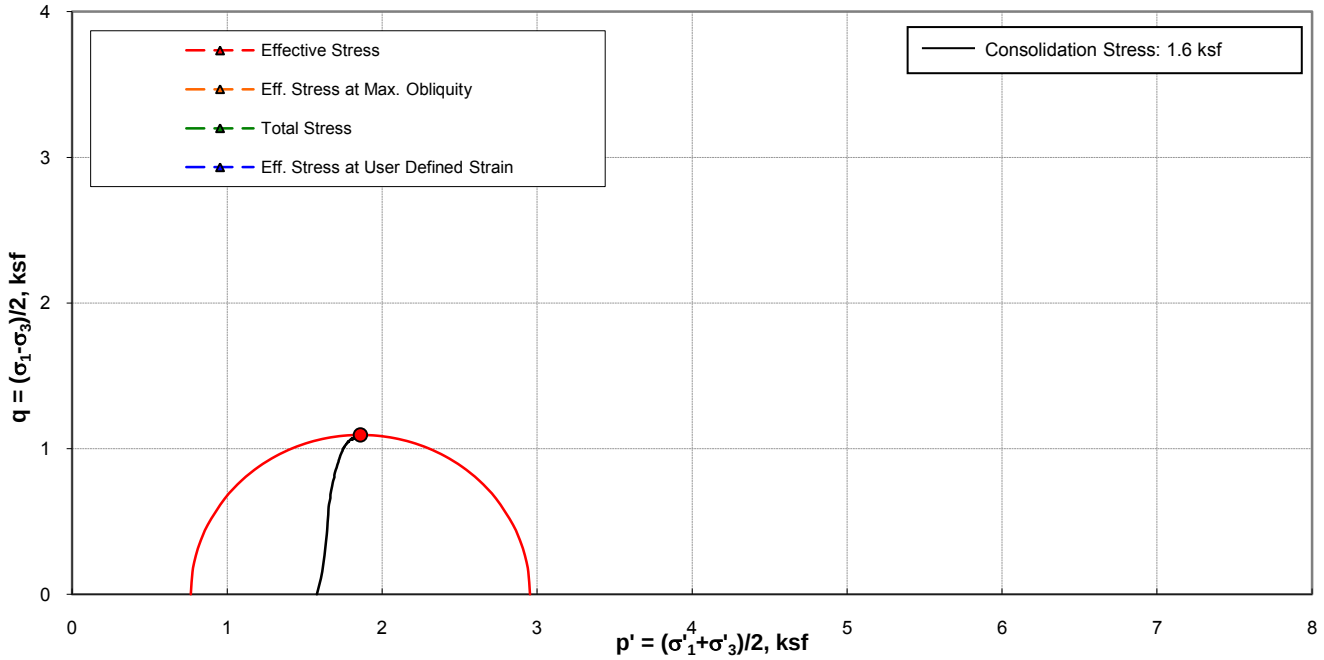
Consolidation Stress: 1.4 ksf

A	B-03	#4	15.9 ft	Fat CLAY (CH): very dark grayish brown, moist
B	---	---	---	---
C	---	---	---	---

CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California



SAMPLE ID				
Boring Number.:	B-03	---	---
Sample Number.:	6	---	---
Sample Depth:	25.4 ft	---	---
USCS Classification:	Fat CLAY (CH): very dark grayish brown, moist	---	---

INITIAL			
Sample No.	A	B	C
Water Content, %	44.4%	---	---
Dry Unit Weight, pcf	75.1	---	---
Saturation, %	98%	---	---
Void Ratio	1.20	---	---
Diameter, in	2.86	---	---
Height, in	5.80	---	---

PRE-SHEAR			
Water Content, %	41.7%	---	---
Dry Unit Weight, pcf	78.5	---	---
Saturation, %	100%	---	---
Void Ratio	1.11	---	---

REMARKS
Test Method: ASTM 4767

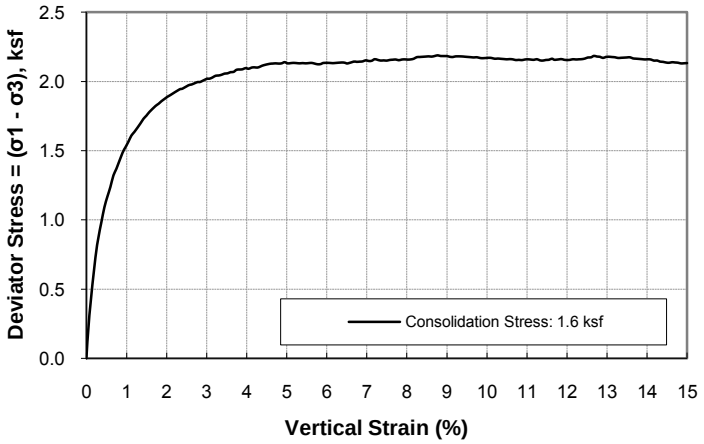
CLASSIFICATION			
Sample No.	A	B	C
Liquid Limit	---	---	---
Plastic Limit	---	---	---
Plastic Index	---	---	---
Passing #4 (4.75 mm)	---	---	---
Passing #200 (0.075 mm)	---	---	---
Estimated Gs	2.65	--	---

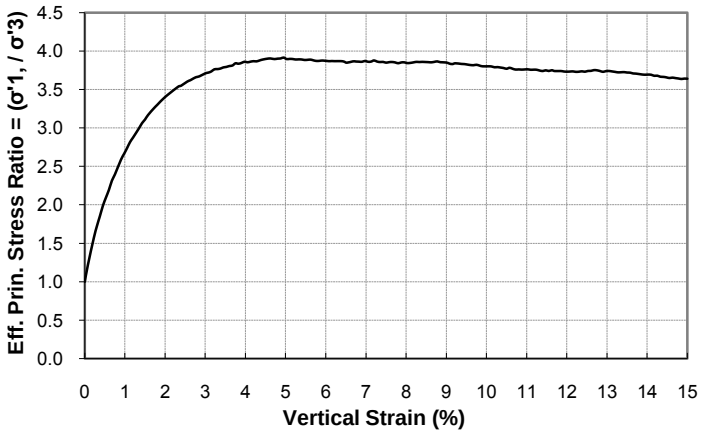
TEST SUMMARY			
Sample No.	A	B	C
B-Parameter	0.99	---	---
t_{50}, minutes	7.8	---	---
Strain Rate, %/min	0.0	---	---
Cell Pressure, ksf	11.6	---	---
Back Pressure, ksf	10.0	---	---
Consolidation Stress, ksf	1.6	---	---
Deviator Stress @ Failure, ksf	2.2	---	---
Axial Strain @ Failure, %	8.8	---	---
σ'_{1F}, ksf	2.9	---	---
σ'_{3F}, ksf	0.8	---	---
Tested By:	JC	---	---
Date Tested:	7/25/12	---	---

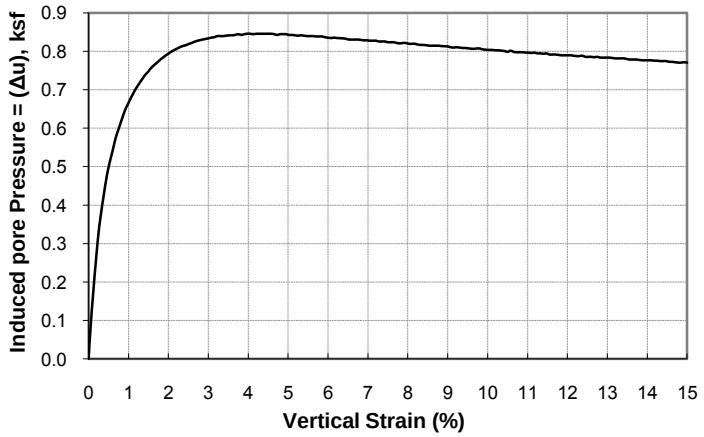
CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California







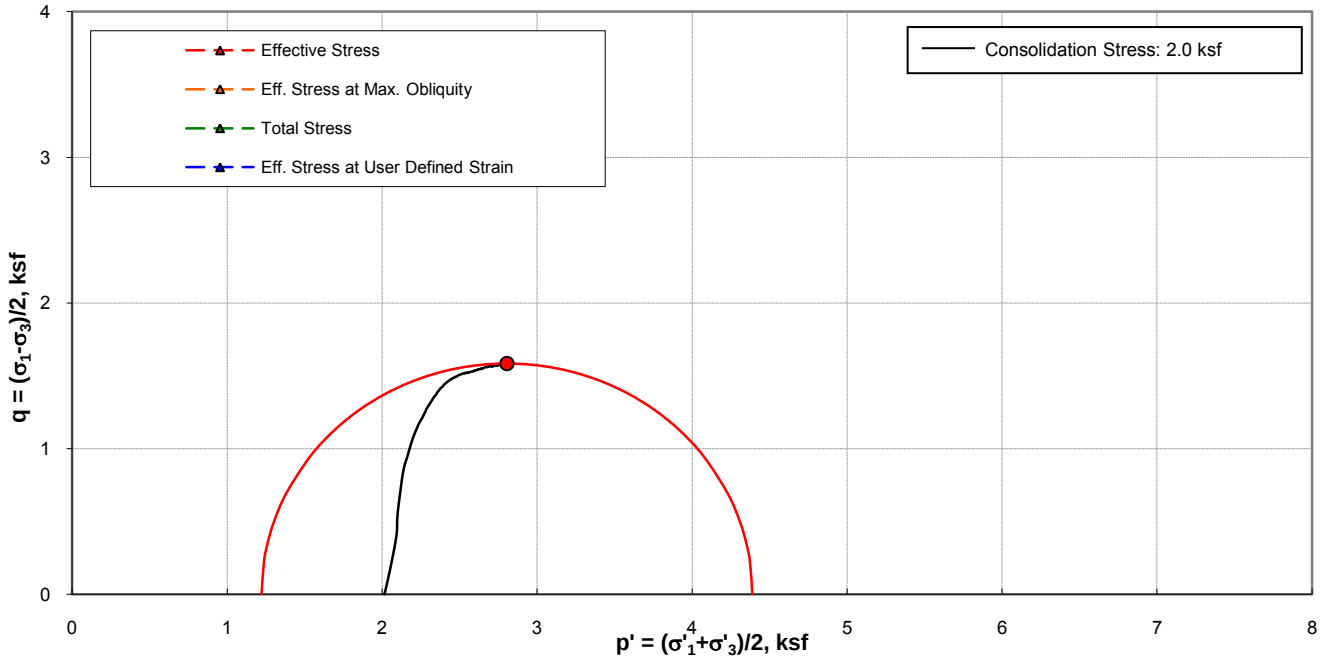
Consolidation Stress: 1.6 ksf

A	B-03	#6	25.4 ft	Fat CLAY (CH): very dark grayish brown, moist
B	---	---	---	---
C	---	---	---	---

CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California



SAMPLE ID				
Boring Number.:	B-03	---	---
Sample Number.:	10	---	---
Sample Depth:	45.4 ft	---	---
USCS Classification:	Fat CLAY (CH): very dark grayish brown, moist	---	---

INITIAL			
Sample No.	A	B	C
Water Content, %	42.3%	---	---
Dry Unit Weight, pcf	75.3	---	---
Saturation, %	94%	---	---
Void Ratio	1.20	---	---
Diameter, in	2.90	---	---
Height, in	5.80	---	---

PRE-SHEAR			
Water Content, %	43.2%	---	---
Dry Unit Weight, pcf	77.1	---	---
Saturation, %	100%	---	---
Void Ratio	1.15	---	---

REMARKS
Test Method: ASTM 4767

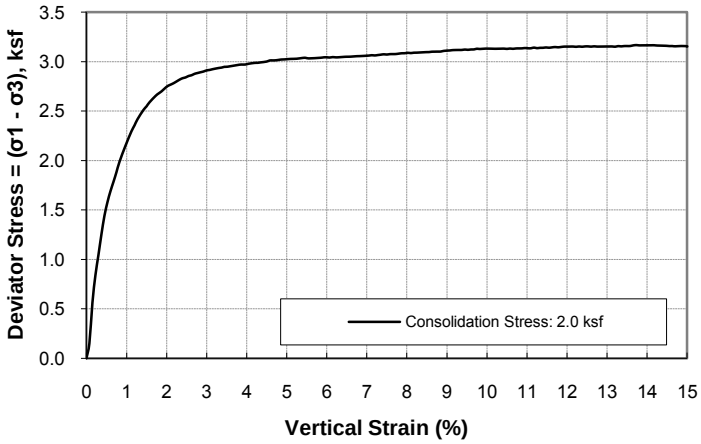
CLASSIFICATION			
Sample No.	A	B	C
Liquid Limit	69	---	---
Plastic Limit	21	---	---
Plastic Index	48	---	---
Passing #4 (4.75 mm)	---	---	---
Passing #200 (0.075 mm)	---	---	---
Estimated Gs	2.65	--	---

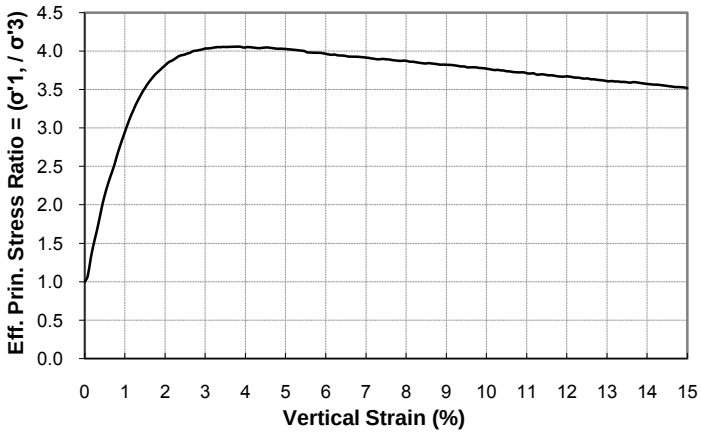
TEST SUMMARY			
Sample No.	A	B	C
B-Parameter	0.97	---	---
t_{50}, minutes	4.6	---	---
Strain Rate, %/min	0.0	---	---
Cell Pressure, ksf	12.0	---	---
Back Pressure, ksf	10.0	---	---
Consolidation Stress, ksf	2.0	---	---
Deviator Stress @ Failure, ksf	3.1	---	---
Axial Strain @ Failure, %	13.7	---	---
σ'_{1F}, ksf	4.4	---	---
σ'_{3F}, ksf	1.2	---	---
Tested By:	JC	---	---
Date Tested:	7/25/12	---	---

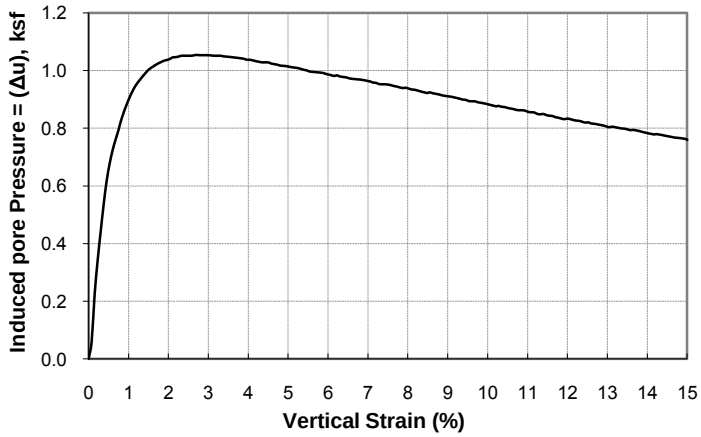
CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California







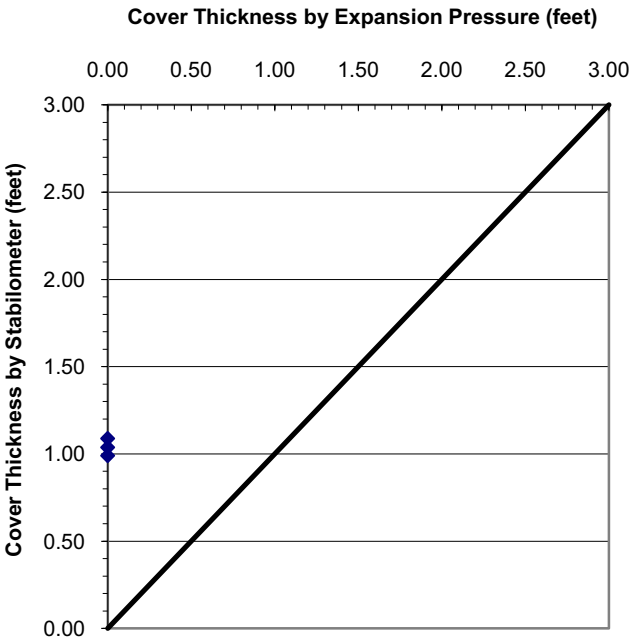
Consolidation Stress: 2.0 ksf

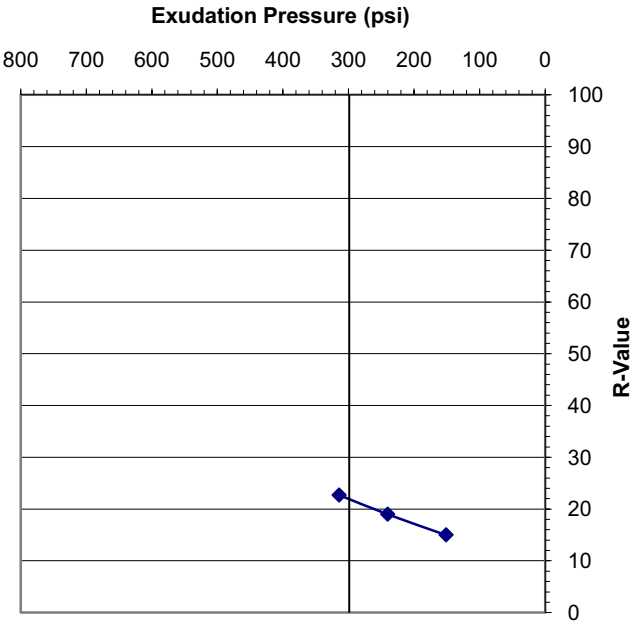
A	B-03	#10	45.4 ft	Fat CLAY (CH): very dark grayish brown, moist
B	---	---	---	---
C	---	---	---	---

CONSOLIDATED, UNDRAINED (CU) TRIAXIAL TEST

Branch Mill Road Bridge Replacement

San Luis Obispo County, California





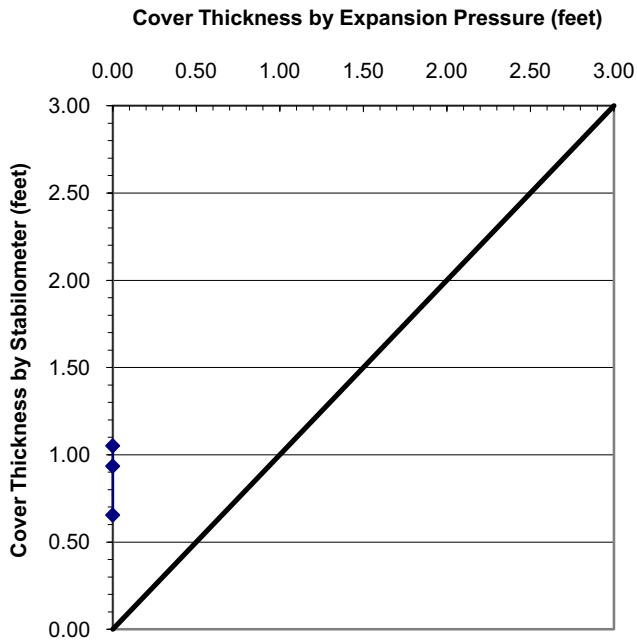
Sample No. B-1, #B **Depth:** 2.0' - 6.0'
Description: Dark Yellowish Brown Clayey SAND (SC)
Date Tested: 7/30/2012
Test Method: ASTM D2844, CT301
Initial Moisture Content: 20.6%

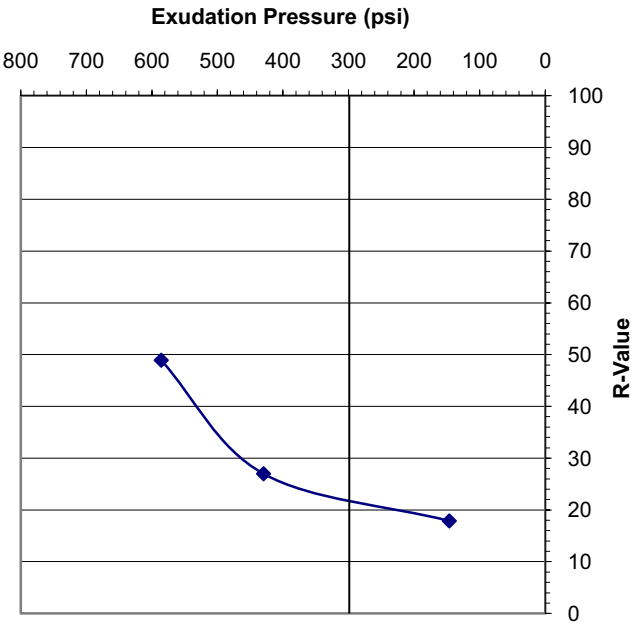
Dry Unit Weight (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
96.9	22.9	151	0	15
99.6	21.9	240	0	19
100.8	20.6	314	0	23

R-value at Exudation Pressure of 300 psi: **22**
R-value by Expansion Pressure: TI = 4 **N/A**

Remarks: R-value by stabilometer controls.

R-VALUE TEST RESULTS
Branch Mill Road Bridge Replacement
Arroyo Grande, California





Sample No. B-4, #C **Depth:** 2.0' - 6.0'
Description: Black Clayey SAND w/ Gravel (SC)
Date Tested: 7/30/2012
Test Method: ASTM D2844, CT301
Initial Moisture Content: 36.2%

Dry Unit Weight (pcf)	Water Content (%)	Exudation Pressure (psi)	Expansion Pressure (psf)	R-Value
73.0	40.7	146	0	18
75.4	38.1	430	0	27
76.7	36.2	586	0	49

R-value at Exudation Pressure of 300 psi: **22**
R-value by Expansion Pressure: TI = 4 **N/A**

Remarks: R-value by stabilometer controls.

R-VALUE TEST RESULTS
Branch Mill Road Bridge Replacement
Arroyo Grande, California

Boring No.	**B-2**
Sample No.	**A (Bulk)**
Depth (feet)	**2 to 6**
Initial Water Content, %	16.1
Initial Dry Unit Weight, pcf	88.4*
Degree of Saturation, %	48.4
Final Water Content, %	30.3
$EI_{measured}$	29
EI_{50}	**28**
ASTM Expansion Potential	**Low**
Description:	**Clayey SAND with gravel (SC)**

* Corresponds to approximately 86% relative compaction per ASTM D1557 curve

EXPANSION INDEX TEST RESULTS
Branch Mill Road Bridge Replacement
Arroyo Grande, California

Corrosivity Test Summary

CTL # 446-143 **Date:** 7/30/2012 **Tested By:** PJ **Checked:** PJ
Client: Fugro West, Inc **Project:** Branch Mill Road Bridge Replacement **Proj. No:** 04.6110.0010.03
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-2	A	2-6'	-	1564	-	13	304	0.0304	7.4	-	25.4	Brown Clayey SAND w/ Gravel
B-3	6	24.9'	-	711	-	51	296	0.0296	7.6	-	48.1	Very Dark Gray CLAY
B-3	9	40.5'	-	653	-	39	278	0.0278	8.0	-	44.7	Very Dark Gray CLAY

CORROSION TEST RESULTS
Branch Mill Road Bridge Replacement
Arroyo Grande, California

Corrosivity Test Summary

CTL # 446-144 **Date:** 7/30/2012 **Tested By:** PJ **Checked:** PJ
Client: Fugro **Project:** Branch Mill Road Bridge Replacement **Proj. No:** 04.6110.0010.03
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/L	mg/L	%		(Redox) mv	At Test %	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
W-1	-	-	-	654	-	137	429	0.0429	7.8	-	-	Water

CORROSION TEST RESULTS
Branch Mill Road Bridge Replacement
Arroyo Grande, California