FOR CONTRACT NO: 10-0G4704

ADDITIONAL INFORMATION HANDOUT

PERMITS

US ARMY CORP OF ENGINEERS PERMIT (404) AMENDMENT

REGIONAL WATER QUALITY CONTROL BOARD PERMIT (401) AMENDMENT

STATE DEPARTMENT FISH AND GAME PERMIT (1602) AMENDMENT

U.S. FISH AND WILDLIFE SERVICE BIOLOGICAL OPINION AND AMENDMENT

NATIONAL MARINE FISHERIES SERVICE LETTER AND AMENDMENT

MATERIALS INFORMATION

FOUNDATION REPORTS BY PARIKH CONSULTANTS, INC.
FOR THE FOLLOWING STRUCTURES:

COUNTRY CLUB BLVD (Br. No. 29-0189)
MICHIGAN AVE. UC (Br. No. 29-0190)
ALPINE AVE. UC (Br. No. 29-0191)
TELEGRAGH AVE. UC (Br. No. 29-0192)
CALVERAS RIVER BRIDGE (Br. No. 29-0174)
EBMUD AQUEDUCT UC (Br. No. 29-0226)
MARCH LANE UC (Br. No. 29-0193)
FOURTEEN MILE SLOUGH (Br. No. 29-0175)
SWAIN ROAD UC (Br. No. 29-0215)
BENJAMIN HOLT DRIVE UC (Br. No. 29-0194)
FIVE MILE SLOUGH CULVERT

ROUTE: 10-SJ-5-25.0/32.7

DEPARTMENT OF THE ARMY
U.S. ARMY ENGINEER DISTRICT, SACRAMENTO
CORPS OF ENGINEERS
1325 J STREET
SACRAMENTO CA 95814-2922

REPLY TO
ATTENTION OF

February 10, 2011

Regulatory Division (SPK-2009-00460)

State of California
Department of Transportation
Attn: Zachary K. Parker
2015 E. Shields Avenue, Suite 100
Fresno, California 93726-5428

Dear Mr. Parker:

We are responding to your request for a Department of the Army, Nationwide Permit (NWP) verification for the I-5 North Stockton Roadway Widening project, and request for an additional season of in-water work to widen structures at Calaveras River, Five Mile Slough, and Fourteen Mile Slough. This verification supersedes your May 14, 2010 authorization letter. The project involves activities, including discharges of dredged or fill material, in waters of the United States to improve traffic operations and congestion along I-5. The project is located near Stockton, in Section 10, Township 2 North, Range 6 East, Latitude 38.029191°, Longitude -121.312112°, MDB&M Survey, San Joaquin County, California.

California Department of Transportation (Caltrans) originally determined that one season of in-water work beginning July 1, 2011 and ending November 1, 2011 was adequate to complete the authorized work. Upon further review, Caltrans' bridge engineers determined that the number of working days required to complete the authorized work in-waters would exceed the original estimated number of days in a single four-month construction season. As a result, Caltrans is requesting that the work be completed within two, five-month, in-water construction seasons; the first beginning July 1, 2011 ending December 1, 2011 and the second beginning July 1, 2012 and ending December 1, 2012. Nighttime work is still anticipated for the project and all falsework would be removed prior to December 1, as originally described in the work plan.

On October 26, 2010, Caltrans requested, in a letter, an additional season of in-water work to widen the bridge structures at Calaveras River, Five Mile Slough, and Fourteen Mile Slough. The Corps responded by electronic mail on November 8, 2010, requesting Caltrans submit a final amended Biological Opinion issued by the U.S. Fish and Wildlife Service and concurrence letter issued by the U. S. Department of Commerce so that we could process a new NWP verification. The additional information needed to process the request was received by our office in an electronic mail on January 4, 2011.

Based on the information provided, the proposed activity, resulting in the permanent loss of approximately 0.0127 acres and temporary impacts to approximately 1.148 acres of waters of the United States, including wetlands, is authorized by Nationwide Permit Number (NWP) 14, Linear Transportation Projects. To ensure you and your contractors are aware of the new verification,

please provide the enclosed documents along with a copy of this verification letter to the employees and contractor s involved with this project. Your work must comply with the general terms and conditions listed on the enclosed Nationwide Permit information sheets and the following special conditions:

Special Conditions

1. All terms and conditions of the January 18, 2011 Section 401 Water Quality Certification (WDID#5A39CR00184) are expressly incorporated as conditions of this permit.

2. To mitigate at a 1:1 ratio for the permanent loss of 0.0127 acres and 0.5:1 ratio for temporary loss of 1.148 acres of waters of the United States, including indirect effects, you shall purchase 0.048 acres of wetland credits and 0.56 acres of open water credits at a Corps approved mitigation bank. The selected mitigation bank shall include the area of the permitted project within its service area. Evidence of this purchase shall be provided to this office prior to proceeding with any activity otherwise authorized by this permit.

3. We understand the State of California, Department of Transportation (Caltrans) is the National Environmental Policy Act (NEPA) lead federal agency for this project, and as such, will ensure the authorized work complies with the National Environmental Policy Act, the Endangered Species Act, the National Historical Preservation Act and any other applicable federal laws. This authorization is contingent upon the permittee implementing all actions necessary to comply with these requirements.

4. If you discover any previously unknown historic or archeological remains while accomplishing the activity authorized by this permit, you must immediately notify the appropriate Caltrans and U.S. Army Corps of Engineers Regulatory offices of what you have found. Caltrans acting as the lead Federal agency for this project may consult as appropriate to determine if the remains warrant a recovery effort or if the site is eligible for listing in the National Register, pursuant to 36 CFR Part 800, as amended 8/05/2004.

5. This Corps permit does not authorize you to take an endangered species, in particular giant garter snake *(Thamnophis gigas)*, delta smelt *(Hypomesus transpacificus)*, or designated critical habitat. In order to legally take a listed species, you must have separate authorization under the Endangered Species Act (e.g., an Endangered Species Act Section 10 permit, or a Biological Opinion under Endangered Species Act Section 7, with "incidental take" provisions with which you must comply). To ensure your project complies with the Federal Endangered Species Act, you must implement all of the mitigating measures identified in the enclosed U.S. Fish and Wildlife Service letter of concurrence (81420-2009-F-0638-1, dated March 3, 2010, and as amended 81420-2009-F-0638-R001-1, dated January 3, 2011), including those ascribed to Caltrans therein. If you are unable to implement any of these measures, you must immediately notify the appropriate Caltrans office, the U.S. Army Corps of Engineers Regulatory office, and the appropriate U.S. Fish and Wildlife office so that Caltrans acting as the lead Federal agency for this project may consult as appropriate, prior to initiating the work, in accordance with Federal law.

6. To insure your project complies with the Federal Endangered Species Act, you must implement all of the mitigating measures identified in the enclosed National Marine Fisheries

Service letter of concurrence (2009/00440, dated April 15, 2009, and as amended December 29, 2010), including those ascribed to Caltrans therein. If you are unable to implement any of these measures, you must immediately notify the appropriate Caltrans office, the U.S. Army Corps of Engineers Regulatory office, and the appropriate National Marine Fisheries Service office so that Caltrans acting as the lead Federal agency for this project may consult as appropriate, prior to initiating the work, in accordance with Federal law.

7. All equipment staging, including Temporary Construction Areas (TCA's), shall take place within Caltrans approved areas within the project boundary identified on the enclosed Caltrans' *Pre-construction Notification, Figures/Maps/Design Plans*, dated October 26, 2010. Prior to construction implementation, you shall ensure all equipment staging, TCA's, demolition and excavation, off pavement detours, borrow and fill areas, and upland disposal areas have been evaluated under National Environmental Policy Act, Section 401 and 404 of the Clean Water Act, Section 7 of the Endangered Species Act and Section 106 of the National Historical Preservation Act and all required permits have been obtained.

8. You shall employ Best Management Practices (BMP's) identified in the Caltrans' *Pre-construction Notification Package*, dated October 26, 2010, to avoid and minimize environmental impacts. Temporary fills, dams, and water diversion structures identified on the enclosed Caltrans' *Pre-construction Notification, Figures/Maps/Design Plans*, dated October 26, 2010, must be removed in their entirety and the affected areas returned to pre-construction elevations, contours and conditions within 30 days of activity completion. Temporarily affected riparian areas must be revegetated with appropriate native trees and/or shrubs, using techniques or other methods approved by Caltrans.

9. Within 30 days prior to initiation of construction activities within waters of the U.S., you shall submit to the Corps pre-construction site photographs of the project site, which have been taken no more than 180 days prior to initiation of construction activities. Within 30 days following construction activities, you shall submit post-construction site photographs of the project site, showing the work conducted, to this office. The camera positions and view angles of post-construction photographs shall be identified on a map, aerial photo, or project drawing. Photo locations shall include all major project features (See condition 10) and areas of impacts to waters of the U.S.

10. Within 60 days following completion of the authorized work or at the expiration of the construction window of this permit, whichever occurs first, you shall submit as-built drawings and a description of the work conducted on the project site to this office for review. The drawings shall be signed and sealed by a registered professional engineer and include the following:

a. The Department of the Army Permit number.

b. A plan view drawing of the location of the authorized work footprint (as shown on the permit drawings) with an overlay of the work as constructed in the same scale as the attached permit drawings. The drawings shall contain, at a minimum, 5-foot topographic contours of the entire site.

c. Ground photographs of the completed work. The cameral positions and view-angles of the ground photographs shall be identified on a map, aerial photograph, or project drawing.

d. A description and list of all deviations between the work as authorized by this permit and the work as constructed. Clearly indicate on the as-built drawings the location of any deviations that have been listed.

11. You shall follow the specifications and standards described in the Storm Water Pollution Prevention Plan (SWPPP) and/or Water Pollution Control Plan (WPCP), to prevent erosion and sedimentation during and after construction. Construction work within waters of the United States shall be performed when the flows are at their seasonal low or when they have ceased and the areas are dry, typically April 15 - October 15 (outside the rainy season work window).

12. The Permittee is responsible for all work authorized herein. To ensure that involved contractors are aware of the terms, conditions and limitations of this authorization, the permittee shall post a copy of the permit authorization and associated drawings at the project site during all phases of construction to ensure that contractors are aware of the terms and conditions of this authorization.

13. No activity may cause more than a minimal adverse effect on navigation. Any safety lights and signals prescribed by the U.S. Coast Guard, through regulations or otherwise, must be installed and maintained at the permittees' expense on authorized facilities or structures in navigable waters of the United States.

14. You shall notify the Sacramento District, Regulatory Division Office of the **start** of the authorized work within seven (7) calendar days of initiating construction activities.

15. You and your authorized contractor shall notify this office of any proposed modifications to the project, including revisions to any of the work plans or documents cited in this authorization, for review and approval prior to construction work associated with the proposed modification.

16. You and your authorized contractor shall notify the Sacramento District, Regulatory Division Office immediately if any of the above conditions are violated or unauthorized activities occur, and shall provide a description of measures taken to remedy the violation.

17. You and your authorized contractor shall allow representatives from this office to inspect the authorized activity at any time deemed necessary to ensure that work is being or has been accomplished in accordance with the terms and conditions of this verification.

18. You must sign the enclosed *Compliance Certification Form* and return it to this office within 30 days **after** completion of the authorized work.

This verification is valid for two (2) years from the date of this letter or until the Nationwide Permit is modified, reissued, or revoked, whichever comes first. All of the existing NWPs are scheduled to be modified, reissued, or revoked prior to March 18, 2012. It is incumbent upon you to remain informed of changes to the NWPs. We will issue a public notice when the NWPs are reissued. Furthermore, if you commence or are under contract to commence this activity before the date that the relevant NWP is modified or revoked, you will have twelve (12) months from the date of the modification or revocation of the NWP to complete the activity under the present terms and conditions of this nationwide permit. Failure to comply with the General Conditions of this Nationwide Permit, or the project-specific Special Conditions of this authorization, may result in the suspension or revocation of your authorization.

We appreciate your feedback. At your earliest convenience, please tell us how we are doing by completing the customer survey on our website under *Customer Service Survey*.

Please refer to identification number SPK-2009-00460 in any correspondence concerning this project. If you have any questions, please contact Ms. Leah Fisher at our California South Branch Office, 1325 J Street, Room 1480, Sacramento, California 95814-2922, email *Leah.M.Fisher@usace.army.mil*, or telephone 916-557-6639. For more information regarding our program, please visit our website at *www.spk.usace.army.mil/regulatory.html*.

Sincerely,

Paul M. Maniccia
Chief, California South Branch

Enclosures:

Copies Furnished without enclosures:
California Regional Water Quality Control Board, Storm Water and Water Quality Certification Unit, Central Valley Region, 11020 Sun Center Drive #200, Rancho Cordova, California 95670-6114
U.S. Fish and Wildlife Service, Endangered Species Division, 2800 Cottage Way, Suite W2605, Sacramento, California 95825-3901
National Marine Fisheries Service, Regional Administrator, 650 Capitol Mall, Suite 8-300, Sacramento, California 95814-4706
California Department of Fish and Game, 1701 Nimbus Road, Rancho Cordova, California 95670-4504
Central Valley Flood Protection Board, 3310 El Camino Avenue, Room LL40, Sacramento, California 95821
U.S. Environmental Protection Agency, Region IX, Wetlands Regulatory Office (WTR-8), 75 Hawthorne Street, San Francisco, California, 94105-3901
U.S. Coast Guard, Eleventh Coast Guard District, Bridge Section, Building 50-2, C.G. Island, Alameda, California 94501-5100

COMPLIANCE CERTIFICATION

Permit File Number: SPK-2009-00460

Nationwide Permit Number: NWP 14 Linear Transportation Projects

Permittee: State of California
Department of Transportation
Attn: Zachary K. Parker
2015 E. Shields Avenue, Suite 100
Fresno, California 93726-5428

County: San Joaquin

Date of Verification: February 10, 2011

Within 30 days after completion of the activity(s) authorized by this permit, sign this certification form and return it; along with the items identified in Special Condition #14, to the following address:

U.S. Army Corps of Engineers
Regulatory Division
1325 J Street, Room 1480
Sacramento, California 95814-2922
DLL-CESPK-RD-Compliance@usace.army.mil
FAX: (916) 557-6877

Please note that your permitted activity is subject to a compliance inspection by a U.S. Army Corps of Engineers representative. If you fail to comply with the terms and conditions of the permit your authorization may be suspended, modified, or revoked. If you have any questions about this certification, please contact the Corps of Engineers.

* * * * * * * * *

I hereby certify that the work authorized by the above-referenced permit, including all the required mitigation, was completed in accordance with the terms and conditions of the permit verification.

_____ _____
Signature of Permittee Date

Linda S. Adams
Acting Secretary for
Environmental Protection

California Regional Water Quality Control Board
Central Valley Region

Katherine Hart, Chair

11020 Sun Center Drive #200, Rancho Cordova, California 95670-6114
Phone (916) 464-3291
http://www.waterboards.ca.gov/centralvalley

Edmund G. Brown Jr.
Governor

7 January 2011

Virginia Strohl
California Department of Transportation
2015 East Shields, Suite 100
Fresno, CA 93726

AMENDED CLEAN WATER ACT §401 TECHNICALLY CONDITIONED WATER QUALITY CERTIFICATION FOR DISCHARGE OF DREDGED AND/OR FILL MATERIALS FOR THE I-5 NORTH STOCKTON CORRIDOR IMPROVEMENTS PROJECT, PACKAGE ONE (WDID#5A39CR00184),* SAN JOAQUIN *COUNTY

Central Valley Regional Water Quality Control Board staff have reviewed your 4 January 2011 request to amend the 2 July 2010 Clean Water Act, Section 401, Technically Conditioned Water Quality Certification (WDID#5A39CR00184) for the subject project. The requested amendment is due to delays in construction. The request for amendment is hereby approved, and the original certification is therefore amended as described below. Please attach this document to the original Water Quality Certification.

REASON FOR AMENDMENT

California Department of Transportation (Caltrans) requests an amendment to the 401 Water Quality Certification to allow an additional season of in-water work to widen structures at the Calaveras River, Five Mile Slough, and Fourteen Mile Slough. Caltrans originally requested one season beginning 1 July 2010 and ending 1 November 2010, timed to avoid impacts to the state and federally listed giant garter snake and the federally listed delta smelt. Nighttime work was also requested.

After more detailed design of the structure was completed and reviewed by Caltrans' bridge engineers, it was determined that the number of the working days required exceeded the number available in a single four-month season and that two five-months season will be required to complete the in-water work: the first beginning 1 July 2011 and ending 1 December 2011 and the second beginning 1 July 2012 and ending 1 December 2012. Nighttime work is still anticipated to be needed for the project. No false work will remain in the waterways after 1 December of any year.

REGIONAL WATER QUALITY CONTROL BOARD CONTACT PERSON:

Skyler Anderson, Environmental Scientist
11020 Sun Center Drive #200
Rancho Cordova, California 95670-6114
sanderson@waterboards.ca.gov
(916) 464-4849

WATER QUALITY CERTIFICATION:

I hereby issue an Order amending the existing Clean Water Act, Section 401, and Technically Conditioned Water Quality Certification for the I-5 North Stockton Corridor Improvements Project, Package one (WDID#5A39CR00184). All other conditions and provisions of the original Water Quality Certifications and any previously approved Amendments remain in full force and effect, except as modified based on the conditions of this Order. Failure to comply with the terms and conditions of the original Water Quality Certification, and subsequently approved Amendments, may result in suspension or revocation of the Water Quality Certification.

Frederick J. Moss

for Pamela C. Creedon
Executive Officer

cc: See enclosure, page 3

DISTRIBUTION LIST

United States Army Corp of Engineers
Sacramento District Office
Regulatory Section, Room 1480
1325 J Street
Sacramento, CA 95814-2922

United States Fish & Wildlife Service
Sacramento Fish & Wildlife Office
2800 Cottage Way
Sacramento, CA 95825

Jeff Drongesen
Department of Fish and Game
1701 Nimbus Road, Suite A
Rancho Cordova, CA 95670

Bill Jennings
CA Sportfishing Protection Alliance
3536 Rainier Avenue
Stockton, CA 95204

(Electronic copy only) Bill Orme
State Water Resources Control Board
401 Certification and Wetlands Unit Chief

(Electronic copy only) Dave Smith
Wetlands Section Chief (W-3)
United States Environmental Protection Agency

State of California – The Natural Resources Agency **EDMUND G. BROWN, JR.** *Governor*
DEPARTMENT OF FISH AND GAME
Bay Delta Region
7329 Silverado Trail
Napa, California 94558
(707) 944-5520
www.dfg.ca.gov

January 31, 2011

Zachary Parker
California Department of Transportation
2015 E. Shields Ave. Suite A-100
Fresno, Ca 93726

Subject: Amendment of Lake or Streambed Alteration Agreement
Notification No. 1600-2010-0160-R3
I-5 North Stockton Corridor Improvements Project

Dear Mr. Parker:

The Department of Fish and Game ("Department") has received your request to amend Lake or Streambed Alteration Agreement 1600-2010-0160-R3 ("Agreement") and the required fee in the amount of $560.25 for a major amendment. Your request to amend the Agreement includes:

Construction will take place over two seasons between July 1st and December 1st of 2011 and 2012.

Five riparian trees will be removed; four near the Calaveras Bridge and 1 next to the culvert at Fourteen Mile Slough.

The Department hereby agrees to amend the agreement with addition of the following conditions:

The Riparian Mitigation Plan shall be designed in coordination with the Department and made available to the Department for approval at least 6 months prior to the start of construction. Mitigation shall be at a minimum 3:1 ratio for linear foot impacts. Mitigation shall be located within known habitat for delta smelt and giant garter snake.

All other conditions in the Agreement remain in effect unless otherwise noted herein.

Please sign and return one copy of this letter to acknowledge the amendment. Copies of the Agreement and this amendment must be readily available at project worksites and must be presented when requested by a Department representative or agency with inspection authority.

If you have any questions regarding this matter, please contact Melissa Escaron at (707) 339-0334 or mescaron@dfg.ca.gov.

Sincerely,

Jim Starr
Environmental Program Manager
Bay Delta Region

cc: Melissa Escaron, Staff Environmental Scientist
Lt. Eric Vielhauer

ACKNOWLEDGEMENT

I hereby agree to the above-referenced amendment.

Print Name: ZACHARY PARKER Date: 2/4/11

Signature: Zachary Parker

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Sacramento Fish and Wildlife Office
2800 Cottage Way, Room W-2605
Sacramento, California 95825-1846

IN REPLY REFER TO:
81420-2009-F-0638-1

MAR 03 2010

Mr. Zachary Parker
Biology Branch Chief
California Department of Transportation, District 6
2015 East Shields Avenue, Suite A-100
Fresno, CA 93726

Subject: Biological Opinion and Appendage to the *Programmatic Biological Opinion on the Effects of Small Highway Projects on the Threatened Giant Garter Snake in Butte, Colusa, Glenn, Sacramento, San Joaquin, Solano, Sutter, Yolo, and Yuba Counties, California* (Service File number 1-1-03-F-0154) for the Interstate-5 North Stockton Corridor Improvements Project, San Joaquin County, California (Caltrans EA 10-0G4700, 10-SJ-5-PM 25.2/37.3)

Dear Mr. Parker:

This is the U.S. Fish and Wildlife Service's (Service) response to the California Department of Transportation's (Caltrans) request for formal consultation on the proposed Interstate-5 (I-5) North Stockton Corridor Improvements Project (project) in San Joaquin County, California. Your request for formal consultation, dated March 13, 2009, was received in this office on March 18, 2009. Caltrans requests that a portion of this project be considered for inclusion with the Service's January 24, 2005, *Programmatic Biological Opinion on the Effects of Small Highway Projects on the Threatened Giant Garter Snake in Butte, Colusa, Glenn, Sacramento, San Joaquin, Solano, Sutter, Yolo, and Yuba Counties, California* (GGS Programmatic). At issue are the effects of this proposed project on the federally-threatened giant garter snake (*Thamnophis gigas*; GGS), the federally-threatened Delta smelt (*Hypomesus transpacificus*; DS), and Delta smelt critical habitat. This biological opinion addresses the Delta smelt and the appendage to the Programmatic addresses the giant garter snake. This response was prepared in accordance with section 7(a)(2) of the Endangered Species Act of 1973, as amended (16 U.S.C. 1531 *et seq.*) (Act).

The findings and recommendations in this formal consultation are based on: (1) your letter, dated March 13, 2009, requesting initiation of section 7 consultation, (2) the *Biological Assessment for the I-5 North Stockton Corridor Improvements Project*, dated January 2009, (3) electronic-mail (e-mail) exchanges and telephone conversations between the Service, Caltrans, and the consultant, LSA Associates, (4) two memorandums from LSA to Caltrans, the first dated

TAKE PRIDE IN AMERICA

June 19, 2009, regarding additional clarification on project impacts to the GGS, and the second, dated October 6, 2009, regarding additional information on potential effects to the DS; and (5) other information available to the Service.

Consultation History

February 9, 2009: The Service received a letter from Caltrans with a request for concurrence with its determination that the project was likely to adversely affect GGS and DS, but did not request to initiate consultation.

February 20, 2009: Susan Jones (Service) e-mailed Zachary Parker (Caltrans) asking for clarification and additional information regarding the Draft Environmental Impact Report (DEIR) and subsequent issues of coordination, and inquiring as to why Caltrans was not initiating formal consultation at that time.

March 19, 2009: The Service received a letter from Caltrans with the request to initiate formal consultation for the GGS and DS. The January 2009 biological assessment (BA) was included in the initiation package.

April 14, 2009: Jen Schofield (Service) e-mailed Charles Walbridge (Caltrans) and Mr. Parker with questions, clarifications, and comments needed in regards to the BA.

May 12, 2009: Mr. Walbridge e-mailed Ms. Schofield requesting a two week late extension to the work window for GGS and a two week early extension to the work window for DS, in order to coordinate in-water work with the salmonid windows per the National Marine Fisheries Service's (NMFS) request to Caltrans.

May 14, 2009: Ms. Schofield received an e-mail from Caltrans and LSA Associates responding to her queries and comments on project design and effects to the species as discussed in the BA.

May 20, 2009: Ms. Schofield e-mailed Mr. Walbridge providing some recommended guidelines for the allowance of construction activity outside of the Service-recommended GGS work window periods. Additional recommendations for DS were forthcoming.

June 2, 2009: After consulting Ryan Olah (Service) for further DS information, an e-mail exchange between Ms. Schofield and Mr. Walbridge discussed the Service's disapproval of the proposed two week earlier extension to the DS window (a new construction date beginning June 15). It was also discussed that Caltrans might possibly request that the NMFS extend the salmonid window later in the season in order to provide the four month window needed for construction.

June 22, 2009: Mr. Walbridge forwarded a memo from the consultant summarizing the impacts to the GGS and providing further clarification.

June 24-26, 2009: E-mail exchanges between Ms. Schofield, Mr. Walbridge, and Rick Harlacher of LSA Associates continued over the issue of extending the project's work period and the extra minimization measures for GGS, since encroachment into the GGS inactive period is likely to increase the potential for further adverse effects to the GGS.

June 29, 2009: Mr. Parker telephoned Ms. Schofield, who confirmed that the Service recommended additional minimization efforts to offset the effects of the extended work window after October 1. Ms. Schofield also updated Mr. Walbridge on the development.

July 14, 2009: Mr. Parker telephoned Ms. Schofield to inform her that the NMFS granted an extension to the salmonid window to allow Caltrans to continue construction until November 1, rather than to begin earlier in June. This would have presented an unacceptable DS window, since the likelihood of DS presence in-water would have been greater in June. The November 1 extension therefore additionally expanded the GGS work window a total of four weeks past October 1, in order to allow for the total four month work period in which all construction activities could be completed in one season.

July 22, 2009: Ms. Schofield requested Mr. Olah's assistance in adding the DS section to the biological opinion.

July 23, 2009: Mr. Parker telephoned Ms. Schofield, who asked for confirmation of the construction window (July 1-November 1). She also relayed to Mr. Parker information she received from Mr. Olah regarding the use of a species Conservation Fund for DS since there are no existing DS banks, and that Caltrans' proposed ratio was acceptable.

August 17, 2009: Mr. Walbridge e-mailed Ms. Schofield and Monica Gutierrez (NMFS) disclosing that four months of night work will be necessary every day in order to complete in-water work on all the structures in one season. Ms. Gutierrez responded the same day that the NMFS had no additional issues with this request.

September 16-17, 2009: Ms. Jones emailed additional information on DS avoidance and minimizations measures received from Mr. Olah. Mr. Parker responded via e-mail that he would discuss the compensation change with San Joaquin County, as it was the project sponsor. He did not foresee any problems.

September 22, 2009: Mr. Parker telephoned Ms. Schofield to discuss several projects, including the topics of DS work window dates, compensation, shading, the in-lieu species conservation fund, and night work for the project considered herein. Ms. Schofield confirmed that following discussion with Mr. Olah, night lighting was unlikely to be problematic to the DS, provided Caltrans keeps construction within the agreed dates. Also, following an earlier discussion with

Jana Affonso (Service), Ms. Schofield also relayed to Mr. Parker that the presence of night work was unlikely to be problematic to the GGS. Even though night work was proposed for each day of the four month period, Mr. Parker thought that this would not involve work simultaneously being carried out on every bridge and waterway throughout this night period.

October 7, 2009: Mr. Walbridge forwarded a memo from the consultant regarding some additional literature research done and clarifications on effects pertaining to the DS.

November 6, 2009: Mr. Olah provided complete DS sections of the BO to Ms. Schofield.

BIOLOGICAL OPINION

Project Description

Caltrans, in cooperation with the City of Stockton Public Works Department, San Joaquin County Public Works Department, and the San Joaquin Council of Governments (SJCOG), proposes to improve a 12.1 mile (mi) segment of I-5 between postmiles (PM) 25.2 and 37.3. The project length extends from Charter Way at the southern end of the corridor to just north of Eight Mile Road at the northern end.

The overall purpose of the project is to meet the need of increased traffic in northwest Stockton. The increased traffic is from new local area development, and from neighboring cities such as Manteca, Tracy, Lathrop, Lodi, and Sacramento which experience high volumes of traffic on the I-5. Congestion and delays have been occurring in the project area. The further issues of noise pollution, declining air quality, and inefficient energy use of stopped/delayed vehicles on the freeway have also become matters requiring attention. The project proposes to undertake several freeway improvements to address these concerns through a 'south to north' construction sequencing plan intended to address the most pressing traffic congestion matters first:

- The freeway will be widened from six to eight lanes by adding two new lanes (one north- and one southbound) in the median, from Country Club Boulevard (PM 28.5) to north of Eight Mile Road (PM 36.2). All bridge work over local roads and waterways will also have lanes completed in the median. These lanes will also operate as High Occupancy Vehicle (HOV) lanes during peak hours, beginning south of Charter Way at PM 25.2.

- New outside auxiliary lanes will be constructed in both north- and southbound directions between March Lane and Benjamin Holt Drive interchanges and between Benjamin Holt Drive and Hammer Lane. The freeway will thus be widened on the outside to provide standard lanes and shoulders for these new additions. Additionally, soundwalls will be constructed as needed along the I-5 and interchange ramps as far north as Hammer Lane.

- In-water construction will occur in two phases, each of which will be completed in one season. This includes casting piles to support temporary construction trestles, casting

piles to support the widened bridge decks, and removal of the temporary trestles and supporting piles. The piles will be 'cast in drilled holes' (CIDH) and will be cast in place, rather than driven, by means of spinning steel casings into the substrate underwater; these casings will be used as forms for the piles. The first phase will include inside widening over the Calaveras River, inside and outside widening at Fourteen Mile Slough, inside and outside widening at Five Mile Slough, inside widening at Mosher Slough, and inside widening at Bear Creek. The second phase will include outside widening at Mosher Slough and outside widening at Bear Creek.

- At the Calaveras River, Fourteen Mile Slough, Mosher Slough, and Bear Creek, bridge widenings will consist of three to 11 spans placed on the outside and/or between the existing left and right reinforced concrete structures. The widenings will consist of precast, pre-stressed concrete double 'T' girders, voided slabs, or 'I' girders set on concrete pile caps and supported by 24 inch concrete piles cast in place. There will be from three to seven piles in each inside widening bent line and two to four piles in each outside widening bent line within the slough banks. A total of approximately186 piles will be required for the four widened bridges at these locations.

- At Five Mile Slough, modification of the bridge will consist of the demolition and reconstruction of the existing parapet and wingwalls at the ends of the existing reinforced concrete box to accommodate the widening of I-5, as well as construction of the barriers and soundwalls along the top of the parapet and wingwalls. This proposed construction may use cofferdams and undertake stream diversion and the dewatering of the construction area within the slough.

- Operations within these aquatic habitats will be conducted using temporary construction trestles, ranging from 30 to 40 feet (ft) wide and extending from existing top of the bank to existing top of the bank. The trestles will be supported on CIDH piles within the creek beds and pile casting equipment will be necessary to spin the steel casings into the substrate. The temporary trestle bents will be spaced at approximately 32 ft on center, and the piles spaced at approximately seven feet on center along each bent line. As each trestle span is placed, it will be used as a platform for construction of each successive trestle bent and span. Construction of each temporary trestle is anticipated to take around two weeks and access to the trestle for construction equipment will be from the roadway median or shoulder.

- The existing inside lanes in the eight-lane section between Charter Way and Country Club Boulevard will be restriped to mark HOV use.

Several improvement elements will occur in later phases in northern section of the project area from Hammer Lane up past Eight Mile Road.

- Two new interchanges will be constructed: one at Otto Drive, with additional construction on associated auxiliary lanes and soundwalls between Otto Drive and Hammer Lane, located one mile south of Otto Drive, and the second at North Gateway Boulevard, located approximately 1.3 mi north of Eight Mile Road at the northern end of the project area.

- Two existing interchanges will be modified at Hammer Lane and at Eight Mile Road, including additional auxiliary lane construction between Eight Mile Road and Otto Drive and soundwalls between the interchanges.

Project Schedule

In-water work for the first phase, which includes the inside widening over the Calaveras River, inside and outside widening at Fourteen Mile Slough, inside and outside widening at Five Mile Slough, inside widening at Mosher Slough, and inside widening at Bear Creek, is anticipated to begin no earlier than July 1, 2012, and be completed by November 1, 2012. In-water work for the second phase, which includes the outside widenings at Mosher Slough and Bear Creek, is anticipated to begin on July 1, 2014, and be completed by November 1, 2014.

Night work is proposed to take place every day during the four month construction window (July 1 – November 1).

Proposed Avoidance and Minimization Measures for the Delta Smelt

According to the biological assessment, updates from the two memos provided by the consultant, and further discussion with Caltrans biologists, Caltrans proposes to implement the following avoidance and minimization measures for the DS.

1. All in-water work will be conducted between July 1 and November 1 to avoid the main spawning season and to time work to be conducted when Delta smelt are less likely to be in the project area. Since the DS population is at low numbers, it is important to minimize potential adverse effects to the species and to avoid direct take. Each of the two phases of water work in the sloughs and channels will be confined to one season so as limit further potential effects and disturbance brought about by multiple seasons of construction activity.

2. Caltrans will implement its Construction Site Best Management Practices (BMPs) including the Storm Water Pollution Prevention Program (SWPPP) and the Water Pollution Control Program (WPCP) to minimize adverse effects such as siltation, sedimentation, and erosion.

3. Emergent and submergent aquatic vegetation will be retained and all other vegetation will be preserved to the extent practical. If removal is necessary, vegetation will be cut at the ground line but the root systems left alone.

4. To the extent practical, new roadways and facilities will be situated perpendicular, not adjacent to, waterways to reduce disturbance to riparian areas.

5. To the extent practical, new bridge and road footings will be located outside of high water zones and riparian habitats.

6. Construction buffers at a minimum of 100 ft will be in place throughout the construction process and such areas will be marked with stakes, fencing, or other appropriate materials visible to personnel and heavy equipment operators in order to protect waterway habitat from unnecessary traffic. These buffers will remain within the boundaries of the project site and will be measured from the tops of the banks to extend outwards away from the waterways; once the distance is established the buffers will then run parallel to the waterways.

7. Temporary water diversion and dewatering will be necessary prior to excavation at Five Mile Slough, requiring installation of a cofferdam. This will be installed using steel sheet piles vibrated into place around the foundation perimeter. Subsequent to placement of the temporary waterway diversion and the sealing of the cofferdam, the construction area will then be dewatered by pumping the water into a suitable off-site location. This dewatering and sealing will minimize potential water quality adverse effects as well as direct contact with the DS during in-water construction activities.

8. To minimize the effects of the permanent loss of 0.734 acres (ac) of DS habitat, Caltrans proposes using a 3:1 compensation ratio to pay a fee of $88,080 ($40,000/ac x 2.202 credits) into the Delta Smelt Conservation Fund prior to ground-breaking. The Conservation Fund is held by the Center for Natural Lands Management (CNLM) and the associated agreement exists between the CNLM, the Service, and the Conservation Fund Participant (Caltrans in this case). Once an open DS conservation bank becomes available, these funds will then be used to purchase credits.

Action Area

The action area is defined in 50 CFR § 402.02, as "all areas to be affected directly or indirectly by the Federal action and not merely the immediate area involved in the action." For the proposed project, the action area incorporates the project footprint, consisting of the existing 12.1 mi segment of the I-5 freeway including six travel lanes, five existing interchanges, and 15 bridges; the cut and fill slopes; and 0.734 ac of permanently affected habitat and 1.47 ac of temporarily affected habitat due to construction activities, including those areas in which temporary trestles will be built and contractor activities will occur, and in which the access and staging areas will be located. Construction will be limited in large part to the existing freeway

and inside median, which consists of portions of multi-purpose open space lands (including a segment of a cultivated park and golf course), and the shoulders of the I-5, but effects will extend beyond these immediate locations at the sites of the newly proposed and modified interchanges, and at bridge locations where activities will be more extensive. The interchange construction sites are comprised of portions of ruderal habitat, row and field crops, natural lands (including valley oak woodland, and a segment of a dead-end slough), as well as urban/industrial and built-up lands. The proposed bridge work sites are comprised of portions of natural lands (waterways, freshwater emergent wetland, and ruderal habitat). A total of 392.98 ac comprises the extent of construction activities.

In addition, segments of the three sloughs, Bear Creek, and the Calaveras River up to one mile downstream of in-water construction activities are also included in the action area because of possible downstream effects of sedimentation, erosion, and siltation; once these waterways meet and flow into neighboring tributaries (at a distance spanning from approximately 0.2 mi to 1.3 mi from different bridge sites), such effects would be considerably diluted. The exception is Mosher Slough, which does not mix with another waterway for approximately a distance of two miles away from bridge site work. However, effects at this distance likely will be similarly diluted.

The action area is located within the Terminous, Lodi South, and Stockton West United States Geological Survey (USGS) 7.5- minute quadrangles. The UTM coordinates marking the approximate center of the linear project area are 10S 644685.53 E and 4207748.92 N, WGS 84.

Status of the Species and Environmental Baseline

Delta Smelt

Status of the Species

DS was federally listed as threatened on March 5, 1993, (58 FR 12854) (Service, 1993a) and critical habitat was designated on December 19, 1994, (59 FR 65256) (Service, 1994a). The DS is included in the Sacramento-San Joaquin Delta Native Fishes Recovery Plan (Service, 1996) and the Five Year Status Review for this species was completed on March 31, 2004 (Service, 2004).

The DS belongs to the family Osmeridae, a more ancestral member of the order Salmoniformes which also includes the family Salmonidae (salmon, trout, whitefish, and graylings) (Moyle and Cech, 1988). The DS is a slender-bodied fish with a small mouth and large eyes. Adults are typically 2.36–2.76 inches long (measured from tip of the snout to origin of the caudal fin) but can be up to 4.72 inches. DS are nearly translucent with a steely-blue sheen to their sides. Some individuals have a chromatophore (cellular organelle containing pigment) between the mandibles. They have been described as unsteady, intermittent, slow speed swimmers that rely on a "stroke and glide" technique (Swanson and Cech, 1995).

The DS is a euryhaline fish (tolerate a wide range of salinities) endemic to the upper Sacramento-San Joaquin estuary. They occur in the Delta primarily downstream of Isleton on the Sacramento River, downstream of Mossdale on the San Joaquin River, and in Suisun Bay. They move into freshwater when spawning (ranging from February to July) and can occur in: (1) the Sacramento River as high as the confluence with the Feather River, (2) the Mokelumne River system, (3) the Cache Slough region, (4) the Delta, and, (5) Montezuma Slough, (6) Suisun Bay, (7) Suisun Marsh, (8) Carquinez Strait, (9) Napa River, (10) Napa Marsh, and 11) San Pablo Bay. It is not known if DS in San Pablo Bay are a permanent population or if they are washed into the Bay during high outflow periods. Since 1982, the center of DS abundance has been the northwestern Delta in the channel of the Sacramento River. In any month, two or more life stages (adult, larvae, and juveniles) of DS have the potential to be present in Suisun Bay (California Department of Water Resources (DWR) and the Bureau of Reclamation (Reclamation), 1994; Moyle, 1976; and Wang, 1991). DS are also captured seasonally in Suisun Marsh.

DS of all sizes are found in the main channels of the Delta and Suisun Marsh and the open waters of Suisun Bay where the waters are well oxygenated and temperatures relatively cool, usually less than 68-71.6 Fahrenheit (°F) in summer. When not spawning, they tend to be concentrated near the zone where incoming salt water and out flowing freshwater mix. This rich productive area is referred to as the mixing zone, the 2 ppt isohaline, or X2. This area has the highest primary productivity and is where zooplankton populations (on which DS feed) are usually most dense (Knutson and Orsi, 1983; Orsi and Mecum, 1986). At all life stages, DS are found in greatest abundance within the top seven feet of the water column and usually not in close association with the shoreline. The spawning season varies from year to year and appears to peak in April and May (Wang, 1991; Wang and Brown, 1993 as cited in DWR and Reclamation, 1994).

DS spawn in shallow, fresh, or slightly brackish water upstream of the mixing zone (Wang, 1991). Most spawning occurs in tidally-influenced backwater sloughs and channel edgewaters (Moyle, 1976, 2002; Wang, 1986, 1991; Moyle *et al.*, 1992). Although DS spawning behavior has not been observed in the wild (Moyle *et al.*, 1992), some researchers believe the adhesive, demersal eggs attach to substrates such as cattails, tules, tree roots, and submerged branches in shallow waters (Moyle, 1976, 2002; Wang, 1991).

Laboratory observations have indicated that DS are broadcast spawners (DWR and Reclamation, 1994) and eggs are demersal (sinks to the bottom) and adhesive, sticking to hard substrates such as: rock, gravel, tree roots or submerged branches, and submerged vegetation (Moyle, 1976, 2002; Wang, 1986). At 57.2-60.8 °F, embryonic development to hatching takes nine to 14 days and feeding begins four to five days later (R. Mager, UCD, unpublished data). Newly hatched DS have a large oil globule that makes them semi-buoyant, allowing them to maintain themselves just off the substrate floor (R. Mager, UCD, unpublished data), where they feed on rotifers (microscopic crustaceans used by fish for food) and other microscopic prey. Once the swim bladder (a gas-filled organ that allows fish to maintain neutral buoyancy) develops, larvae become more buoyant and rise up higher into the water column. At this stage, 0.63-0.71 inches total in length, most are presumably washed downstream until they reach the mixing zone or the

area immediately upstream of it. Growth is rapid and juvenile fish are 1.57-1.97 inches long by early August (Erkkila *et al.*, 1950; Ganssle, 1966; Radtke, 1966). By this time, young-of-year fish dominate trawl catches of DS, and adults become rare. DS reach 2.17-2.76 inches standard length in seven to nine months (Moyle, 1976, 2002). Growth during the next 3 months slows down considerably (only 0.12-0.35 inches total), presumably because most of the energy ingested is being directed towards gonadal development (Erkkila *et al.*, 1950; Radtke, 1966). There is no correlation between size and fecundity, and females between 2.32-2.76 inches standard lengths lay 1,200 to 2,600 eggs (Moyle *et al.*, 1992). The abrupt change from a single-age, adult cohort during spawning in spring to a population dominated by juveniles in summer suggests that most adults die after they spawn (Radtke, 1966 and Moyle, 1976, 2002). However, in El Nino years when temperatures rise above 64.4 °F before all adults have spawned, some fraction of the unspawned population may also hold over as two-year-old fish and spawn in the subsequent year. These two-year-old adults may enhance reproductive success in years following El Nino events.

In a near-annual fish like DS, a strong relationship would be expected between number of spawners present in one year and number of recruits to the population the following year. Instead, the stock-recruit relationship for DS is weak, accounting for about a quarter of the variability in recruitment (Sweetnam and Stevens, 1993). This relationship does indicate, however, that factors affecting numbers of spawning adults (e.g., entrainment, toxics, and predation) can have an effect on DS numbers the following year.

DS feed primarily on (1) planktonic copepods (small crustaceans used by fish for food), (2) cladocerans (small crustaceans used by fish for food), (3) amphipods (small crustaceans used by fish for food) and, to a lesser extent, (4) on insect larvae. Larger fish may also feed on the opossum shrimp (*Neomysis mercedis*). The most important food organism for all sizes seems to be the euryhaline copepod (*Eurytemora affinis*) although in recent years the exotic species, *Pseudodiaptomus forbesi*, has become a major part of the diet (Moyle *et al.*, 1992).

DS are a minor prey item of juvenile and subadult striped bass (*Morone saxatilis*), in the Sacramento-San Joaquin Delta (Stevens, 1966). They also have been reported from the stomach contents of white catfish (*Ameiurus catus*) (Turner, 1966 in Turner and Kelley (eds), 1966) and black crappie (*Pomoxis nigromaculatus*) (Turner, 1966 in Turner and Kelley (eds), 1966) in the Delta.

Reasons for Decline and Threats to Survival - DS were once one of the most common pelagic (living in open water away from the bottom) fish in the upper Sacramento-San Joaquin estuary, as indicated by its abundance in California Department of Fish and Game trawl catches (Erkkila *et al.*, 1950; Radtke, 1966; Stevens and Miller, 1983). Since the 1850s, however, the amount and extent of suitable habitat for the DS has declined dramatically due in large part to hydraulic mining, agricultural development in the Delta, agricultural practices, water diversion, and recreation.

In addition to the degradation and loss of estuarine habitat, DS have been increasingly subject to entrainment, upstream or reverse flows of waters in the Delta and San Joaquin River, and

constriction of low salinity habitat to deep-water river channels of the interior Delta (Moyle *et al.*, 1992). These adverse conditions are primarily a result of the steadily increasing proportion of river flow being diverted from the Delta and occasional droughts (Monroe and Kelly, 1992).

Reduced water quality from agricultural runoff, effluent discharge and boat effluent has the potential to harm the pelagic larvae and reduce the availability of the planktonic food source. When the mixing zone is located in Suisun Bay where there is extensive shallow water habitat within the euphotic zone (depths less than four meters), high densities of phytoplankton and zooplankton may accumulate (Arthur and Ball, 1978, 1979, 1980). The introduction of the Asian clam (*Potamocorbula amurensis*), a highly efficient filter feeder, presently reduces the concentration of phytoplankton in this area.

DS abundance from year to year has fluctuated greatly in the past, but between 1982 and 1992 their population was consistently low. The decline became precipitous in 1982 and 1983 due to extremely high outflows and continued through the drought years 1987-1992 (Moyle *et al.*, 1992). In 1993, numbers increased considerably, apparently in response to a wet winter and spring. During the period 1982-1992, most of the population was confined to the Sacramento River channel between Collinsville and Rio Vista (D. Sweetnam, CDFG unpublished data). This was still an area of high abundance in 1993, but DS were also abundant in Suisun Bay. The abundance indices have shown a consistently low population for the last 10 years and a precipitous decline in the past few years (Stevens *et al.*, 1990, Souza and Bryant, 2002, CDFG, 2001). The actual size of the DS population is not known. However, the pelagic life style of DS, short life span, spawning habits, and relatively low fecundity indicate that a fairly substantial population probably is necessary to keep the species from becoming extinct. The health of the Delta and the declining DS population has become a critical issue with wide ranging implications that have included proposals to up-list the species to endangered status.

More comprehensive information regarding the biology of the DS and its current status (including abundance monitoring) are included in the species' five year review and in the biological opinion for the Operations Criteria and Plan (OCAP) for the continued operation of the Federal Central Valley Project (CVP) and State Water Project (SWP) on the Service's DS recovery webpage (http://www.fws.gov/sacramento/es/delta_smelt.htm).

Environmental Baseline

As a result of long-term monitoring in the Delta, instances of DS have been recorded approximately two to three miles from the bridge site work location over the Calaveras River. According to the 2002-2008 survey results gathered by the California Department of Fish and Game (CDFG), which monitors the post-larval/juvenile distribution and relative abundance in the Sacramento-San Joaquin Delta and San Francisco Estuary throughout its historic spring range (CDFG, Bay Delta Region, 2008), three individuals in 2004 were detected here, at the Rough and Ready Island monitoring station. Three further individuals were detected as recently as 2008 at two other stations (Rindge Tract and Medford Island) located within 10 river miles downstream of other bridge work sites. A total of 32 DS were found at four of these sampling stations between 2002 and 2008, three of these stations being situated between two and 10 miles

downstream of the action area. Therefore, the Service believes that the DS is reasonably certain to occur within the action area given the biology and ecology of the animal, the presence of suitable habitat in and adjacent to the action area, as well as the recent records of this species.

The project is located in what is considered the "central Delta smelt zone" (Service, 2004). Adult DS spawn in central Delta sloughs from February to July in shallow water areas having submerged aquatic plants and other suitable substrates and refugia. These shallow water areas have been identified in the Delta Native Fishes Recovery Plan (Service, 1996) as essential to the long-term survival and recovery of DS and other resident fish. A no net loss strategy of the DS population and habitat was proposed in this recovery plan.

There are several unrelated Federal projects that have completed formal consultation with the Service for the DS and whose action areas lie within and/or adjacent to the action area of the project considered herein. These have resulted in both direct and indirect effects to the DS. These include the Stockton Downtown Marina Redevelopment Project (Service File No. 81420-2008-F-0216), the Navy Bridge Replacement Project (Service File No. 81420-2008-F-0078), and the Aksland Avenue/Trinity Parkway Bridge Bank Protection Project (Service File No. 81420-2008-F-1337).

Delta Smelt Critical Habitat

Status of Delta Smelt Critical Habitat

In determining which areas to designate as critical habitat, the Service considers those physical and biological features that are essential to a species' conservation and that may require special management considerations or protection (50 CFR 424.12(b).

The Service is required to list the known primary constituent elements (PCEs) together with the critical habitat description. Such physical and biological features include, but are not limited to, the following:

1. Space for individual and population growth, and for normal behavior;

2. Food, water, air, light, minerals, or other nutritional or physiological requirements;

3. Cover or shelter;

4. Sites for breeding, reproduction, and the rearing of offspring; and

5. Generally, habitats that are protected from disturbance or are representative of the historic geographical and ecological distributions of a species.

In designating critical habitat for the DS, the Service identified the following primary constituent elements essential to the conservation of the species: physical habitat, water, river flow, and

salinity concentrations required to maintain DS habitat for spawning, larval and juvenile transport, rearing, and adult migration. These elements are organized by habitat conditions required for each life stage of the DS and are further described as follows.

The spawning habitat element is described as shallow, fresh or slightly brackish backwater sloughs and edgewaters with suitable water quality and substrates for egg attachment to ensure egg hatching and larval viability. Specific areas that have been identified as important DS spawning habitat include Barker, Lindsey, Cache, Prospect, Georgiana, Beaver, Hog, and Sycamore sloughs and the Sacramento River in the Delta, and tributaries of northern Suisun Bay.

The larval and juvenile transport element includes the ability to transport larvae from where they were hatched to shallow, productive rearing or nursery habitat. Adequate river flow is necessary to transport larvae from upstream spawning areas to rearing habitat in Suisun Bay and to ensure that rearing habitat is maintained in Suisun Bay. To ensure this, X2 (the 2 ppt isohaline mixing zone) must be located westward of the confluence of the Sacramento-San Joaquin Rivers, located near Collinsville (Confluence), during the period when larvae or juveniles are being transported, according to historical salinity conditions. X2 is important because the "entrapment zone" or zone where particles, nutrients, and plankton are "trapped," leading to an area of high productivity, is associated with its location. Habitat conditions suitable for transport of larvae and juveniles may be needed by the species as early as February 1 and as late as August 31, because the spawning season varies from year to year and may start as early as December and extend until July.

The rearing habitat element includes areas that support a food-rich environment that allow larval and juvenile DS to mature into adulthood. An area extending eastward from Carquinez Strait, including Suisun, Grizzly, and Honker Bays, Montezuma Slough and its tributary sloughs, up the Sacramento River to its confluence with Three Mile Slough, and south along the San Joaquin River including Big Break, defines the specific geographic area critical to the maintenance of suitable rearing habitat. The Three Mile Slough represents the approximate location of the most upstream extent of historical tidal incursion. Rearing habitat is vulnerable to impacts of export pumping and salinity intrusion from the beginning of February to the end of August.

The adult migration element reflects the importance of providing unrestricted access to suitable spawning habitat. Adequate flow and suitable water quality is needed to attract migrating adults in the Sacramento and San Joaquin river channels and their associated tributaries, including Cache and Montezuma sloughs and their tributaries. These areas are vulnerable to physical disturbance and flow disruption during migratory periods.

The geographical boundaries of the designated critical habitat for the DS includes all water and submerged lands below ordinary high water and the entire water column bounded by and contained in Suisun Bay (including Grizzly and Honker Bays); the length of Goodyear, Suisun, Cutoff, First Mallard (Spring Branch), and Montezuma sloughs; and the existing contiguous waters contained within the Delta.

Refer to the federal register announcement for additional information regarding these primary constituent elements and a map of the critical habitat (http://ecos.fws.gov/docs/federal_register/fr2751.pdf).

DS critical habitat has been affected by past activities that have destroyed spawning and refugial areas and have changed hydrology patterns in Delta waterways. Critical habitat also has been affected by diversions that have shifted the position of X2 upstream of the confluence of the Sacramento and San Joaquin rivers. This shift has caused a decreased abundance of smelt. Existing baseline conditions and implementation of the Service's 1994 and 1995 biological opinions concerning the operation of the CVP and the SWP, provide a substantial part of the necessary positive riverine flows and estuarine outflows to transport smelt larvae downstream to suitable rearing habitat in Suisun Bay outside the influence of marinas, agricultural diversions, and Federal and State pumping plants.

The demands on surface water resources in the Central Valley have increased and there are several proposed diversion projects that would likely result in lower Delta outflows and increased entrainment.

Environmental Baseline of Delta Smelt Critical Habitat

The proposed Interstate-5 North Stockton Corridor Improvements Project is within the geographical boundary of designated critical habitat for the DS. The action area, which includes multiple waterways, contains the PCEs defined for DS critical habitat.

Analytical Framework for the Jeopardy/No Jeopardy Determination

The following analysis relies on four components to support the jeopardy determination for the DS: (1) the *Status of the Species*, which evaluates the species' rangewide condition, the factors responsible for that condition, and its survival and recovery needs; (2) the *Environmental Baseline*, which evaluates the condition of the species in the action area, the factors responsible for that condition, and the role of the action area in the species' survival and recovery; (3) the *Effects of the Action*, which determines the direct and indirect effects of the proposed Federal action and the effects of any interrelated or interdependent activities on the species; and (4) *Cumulative Effects*, which evaluates the effects of future, non-Federal activities in the action area on the species.

In accordance with the implementing regulations for section 7 and Service policy, the jeopardy determination is made in the following manner: the effects of the proposed Federal action are evaluated in the context of the aggregate effects of all factors that have contributed to the species' current status and, for non-Federal activities in the action area, those actions likely to affect the species in the future, to determine if implementation of the proposed action is likely to cause an appreciable reduction in the likelihood of both the survival and recovery of the species in the wild.

The following analysis places an emphasis on using the range-wide survival and recovery needs of the species and the role of the action area in providing for those needs as the context for evaluating the significance of the effects of the proposed Federal action, taken together with cumulative effects, for purposes of making the jeopardy determination.

Analytical Framework for the Adverse Modification/No Adverse Modification Determination

This biological opinion does not rely on the regulatory definition of "destruction or adverse modification" of critical habitat at 50 CFR 402.02. Instead, we have relied upon the statutory provisions of the ESA to complete the following analysis with respect to DS critical habitat.

The following analysis relies on four components to support the adverse modification determination: (1) the *Status of Critical Habitat*, which evaluates the range-wide condition of designated critical habitat for the DS in terms of primary constituent elements (PCEs), the factors responsible for that condition, and the intended recovery function of the critical habitat overall, as well as the intended recovery function of discrete critical habitat units; (2) the *Environmental Baseline*, which evaluates the condition of the critical habitat in the action area, the factors responsible for that condition, and the recovery role of the critical habitat in the action area; (3) the *Effects of the Action*, which determines the direct and indirect impacts of the proposed Federal action and the effects of any interrelated or interdependent activities on the PCEs and how that will influence the recovery role of affected critical habitat unit; and (4) *Cumulative Effects*, which evaluates the effects of future, non-Federal activities in the action area on the PCEs and how that will influence the recovery role of affected critical habitat unit.
In accordance with Service policy and guidance, the adverse modification determination is made in the following manner: the effects of the proposed Federal action on critical habitat are evaluated in the context of the aggregate effects of all factors that have contributed to the current status of the critical habitat range-wide and, for non-Federal activities in the action area, those actions likely to affect the critical habitat in the future, to determine if the critical habitat would remain functional (or retain the current ability for the PCEs to be functionally established in areas of currently unsuitable but capable habitat) to serve the intended recovery role for the species with implementation of the proposed Federal action.

The following analysis places an emphasis on using the intended range-wide recovery function of DS critical habitat and the role of the action area relative to that intended function as the context for evaluating the significance of effects of the proposed Federal action, taken together with cumulative effects, for purposes of making the adverse modification determination.

Effects of the Action

Delta smelt

The in-water construction work could detrimentally affect the DS by: decreasing water quality as a result of mobilizing bottom material leading to increased turbidity (i.e. produced from spinning the steel casings necessary for casting the piles to support temporary construction

trestles, the widened bridge decks, and removal of the temporary trestles and supporting piles); creating predator habitat through additional cover and shading, resulting from the addition of lanes over presently open, exposed habitat; restricting channels during cofferdam and pile installation work, thus also restricting access by DS to areas of open water habitat; and changing water velocities during waterway diversion and dewatering activities. Although Caltrans proposes to keep intact the aquatic vegetation in order to maintain its proper function for the DS, this vegetation upon which the DS depends for egg attachment and refugia could become silted over as well as lose functionality for the species as a consequence of shade cover created by the lane additions. Caltrans' proposed implementation of its BMPs, including the SWPPP and the WPCP to minimize adverse effects such as siltation, sedimentation, and erosion, will aid in diminishing the adverse effects to water quality and aquatic vegetation posed by some in-water activities.

Individual DS could also be affected through potential contact with equipment or by capture or entrapment in cofferdams during dewatering when present in the work area. By following the work window restrictions of performing all in-water work from July 1 through November 30, however, these interactions will be minimized and the number of affected DS is likely to be low.

The project will permanently affect DS habitat. According to the biological assessment and further updates from Caltrans and consultant biologists, the project will result in the casting of new piles in the Calaveras River, at Fourteen Mile Slough, Mosher Slough and Bear Creek, effectively removing 0.018 ac of open water habitat from use. Because Five Mile Slough is not directly connected to downstream channels, the segment of this particular waterway within the action area is inaccessible to the species. The project will result in the shading of aquatic habitat through the construction of additional bridge lanes in the median, covering 0.716 ac of open water. This shading could further reduce the function of aquatic vegetation as no longer contributing to the DS's food chain and could further alter its sheltering function as a means instead of cover for predatory fish. It is possible that the shading effect and the resulting increase in predatory cover could also reduce the DS's detection of predators, thereby leading to increased DS predation. To help minimize the effects of the permanent loss of a total of 0.734 ac (0.018 ac due to open water loss + 0.716 ac of loss due to shading) of DS habitat, Caltrans proposes paying $88,080 into the Delta Smelt Conservation Fund, prior to ground-breaking; these funds will be used to purchase conservation bank credits once a bank becomes available.

Temporary effects to an additional 1.47 ac are also anticipated to occur during the placement of temporary piles at four of the bridge locations due to general disturbance during these in-water activities, resulting in noise, physical disruptions, and water quality issues. Since each phase of in-water work will take place during one construction season (July 1-November 1), effects are expected to be minimal.

Critical Habitat for the Delta Smelt

The proposed project is located within critical habitat for the DS, which includes all water and all submerged lands below ordinary high water within the action area.

The proposed project will affect associated feeding, breeding, and sheltering habitats for the DS within the critical habitat unit in terms of what biological elements they can provide for the species. As discussed in the section above, a decrease in vegetative function, as pertains to the DS (submergent vegetation comprises an integral part of the species' dietary needs and is also used as spawning substrates), is one such outcome. However, direct effects to critical habitat PCEs will likely be limited to the 0.734 ac of habitat that will be lost or modified from the new piles and permanent coverage by additional bridge lanes over previously open aquatic habitat.

Therefore, the effects to the PCEs in the project area are not likely to reduce the overall value of DS habitat. Furthermore, the proposed project is not expected to diminish the long-term value of the critical habitat for the DS, or prevent critical habitat from sustaining its role in the conservation and recovery of the species.

Cumulative Effects

Cumulative effects include the effects of future State, Tribal, local or private actions that are reasonably certain to occur in the action area considered in this biological opinion. Future Federal actions that are unrelated to the proposed action are not considered in this section because they require separate consultation pursuant to section 7 of the Act.

Delta Smelt and its Critical Habitat

Cumulative effects to the DS and its designated critical habitat include non-Federal actions in which point and non-point source chemical contaminant discharges result. These contaminants include selenium and numerous pesticides associated with discharges related to private and local agricultural and urban activities; discharges from agricultural and urban lands in which such actions are reasonably certain to occur could enter the waterways within the action area from upstream, facilitated by the high connectivity between waterways in the Delta. Considered to be potential sources of mortality for DS, these contaminants could adversely affect DS reproductive success and survival rates. Spawning habitat could also be affected if submerged aquatic plants used as substrates for adhesive egg attachment are lost due to toxic substances.

Additional cumulative effects may result from any continuing or future non-Federal diversions of water that entrain adult or larval fish or that may decrease outflows incrementally, thus shifting the position of the DS's preferred habitat upstream. These diversions also include municipal and industrial uses. State or local levee maintenance may also lead to the loss of critical habitat by disturbing spawning or rearing habitat and releasing contaminants into the water, and if such activities are located nearby, contaminants could easily travel into the project's waterways.

Conclusion

After reviewing the current status of the DS and its critical habitat, the environmental baseline, the effects of the project, and the cumulative effects, it is the Service's biological opinion that the proposed Interstate-5 North Stockton Corridor Improvements Project is not likely to jeopardize the continued existence of the DS, or result in the destruction or adverse modification of its

critical habitat. We base this determination on the fact that the number of DS likely to be taken is low, minimized in part by the restriction of in-water work to times when DS are not likely to be present; implementation of water quality BMPs, and the compensation of lost and shaded habitat at a 3:1 ratio. The extent of take is also likely to be minimal based on the rangewide DS population throughout the Delta. Additionally, acreage amounts of habitat loss and disturbance are small for the given project and effects to the species are not likely to extend more than one mile downstream of bridge work sites.

Appending to the Giant Garter Snake Programmatic Biological Opinion

The Service has determined that it is appropriate to append the GGS portion of this proposed project to the Programmatic. This letter is an agreement by the Service to append the project to the Programmatic and represents the Service's biological opinion on the effects of the proposed action. The FHWA proposed to append up to 18 projects per year to the Programmatic, each of which may permanently affect up to three acres (=54 ac/year) and temporarily affect up to 20 ac (=360 ac/year) throughout the action area over the Programmatic's five year term. Thus far, according to the Service file, 25 projects have been appended to the Programmatic over the five year term with five currently pending or in progress, inclusive of the project considered herein.

According to the biological assessment, the GGS memo provided by the consultant, and further discussion with Caltrans biologists, a total of 0.08 ac of GGS habitat will be permanently affected by the project while 1.47 ac will be temporarily affected due to the proposed construction activities.

Conservation measures for projects appended to the Programmatic involve following the *FHWA Giant Garter Snake Programmatic Biological Opinion Guidelines for Restoration and/or Replacement of Giant Garter Snake Habitat* (Guidelines), the *Standard Avoidance and Minimization Measures During Construction Activities in Giant Garter Snake Habitat* (Avoidance Measures), and the *Terms and Conditions* of the Programmatic, except as approved by the Service. It is the Service's intent that following these Guidelines and Avoidance Measures will reduce habitat degradation while increasing the protected habitat areas across the species' range. A copy of the Guidelines and Avoidance Measures are found as appendices to the Programmatic and are available from the Service upon request.

The Service is tracking losses of habitat permitted under the Programmatic within the range of the GGS, and minimization for those losses, in each county under the jurisdiction of the Sacramento Fish and Wildlife Office. The Service reevaluates the effectiveness of this Programmatic annually to ensure that continued implementation will not result in unacceptable impacts to the GGS or to the quality and quantity of habitats upon which it depends.

Conservation Measures

Conservation measures for projects appended to the Programmatic shall be in accordance with the Guidelines and Avoidance Measures unless otherwise provided by the Service. The Programmatic identifies three levels of seasonal project duration, the maximum extent of affected acreage allowed, and the appropriate conservation measure response (pp. 3-9). These measures include the following:

1. avoidance of take and disturbance of habitat (Levels 1, 2, and 3)

2. minimization of disturbance and habitat loss (Levels 1, 2, and 3)

3. restoration of temporary habitat disturbance and associated impacts to GGS habitat (Levels 1 and 2)

4. replacement of permanent and temporary habitat loss (Levels 2 and 3)

5. management and monitoring of restored and replacement habitat (Levels 1, 2, and 3)

6. a management plan for the long-term protection of the restored and replaced habitat area(s) to protect the area(s) in perpetuity as habitat for the GGS (Levels 2 and 3)

Effects of the Proposed Action

The northern segment of I-5 under construction is located approximately one mile east of the southern part of the White Slough Wildlife Area (WSWA), which is home to one of the last known extant populations of GGS in the Delta Basin, the Caldoni Marsh/White Slough population. According to the 2000 San Joaquin County Multi-Species Habitat Conservation and Open Space Plan (SJMCP), known occupied habitat for the GGS in the Delta vicinity includes the WSWA, Shin Kee Tract, area west of I-5 on Terminous Tract, and Rio Blanco Tract; all of these, however, lie outside of the given action area. The waterways in which construction activities are to occur in the Calaveras River, Fourteen Mile Slough, Five Mile Slough, Mosher Slough, and Bear Creek, feature hydrological connectivity through which GGS can disperse, as well as suitable aquatic habitat providing water during the GGS active period and associated upland bank habitat with vegetation for basking and cover. According to the California Natural Diversity Database (CNDDB), there have been five historical documented occurrences of GGS within the action area. Recent surveys were conducted by species expert and consulting biologist, Eric Hansen, throughout the WSWA to update information on snake presence there. In July 2009, 13 adult GGS individuals (eight males and five females) were captured, with four individuals captured more than once. Capture sites were located approximately three miles northwest of the northern segment of I-5 included in the action area and approximately five miles northwest of Bear Creek, the northernmost waterway in which bridge construction activities will

occur. Fourteen developing embryos were also detected in one of the snakes, indicating that at least some reproduction has been occurring (Hansen, 2009, unpublished data).

Since GGS can move approximately five miles over several days (Wylie *et al.*, 1997), and because of the proximity of the historical snake observation records and current surveying efforts, the presence of suitable habitat in and/or around the five waterways in which in-water construction activities will occur during bridge widenings, and the biology and ecology of the species, the Service believes that the GGS is reasonably certain to occur within the action area and, therefore, the proposed project is likely to adversely affect the GGS through permanent habitat loss and temporary disturbance.

Construction activities associated with the proposed project are likely to adversely affect the GGS through the permanent loss of a total of 0.08 ac of habitat (Level 3 effects) and disturbance (Level 1 effects) to 1.47 ac of aquatic, bank and upland habitat at the Calaveras River, three sloughs and Bear Creek locations. Permanent effects include the casting of piles in steel casings, in order to support the widened bridge lane decks; and the shading of previously open bank habitat along the waterways due to the extension with additional lanes of the bridge decks and the box culvert at Five Mile Slough. The casting of permanent piles down into the water and rooted into the substrate at four of the bridge locations (minus Five Mile Slough) will result in the loss of approximately 0.018 ac of aquatic habitat. The shading of available sun-exposed bankside habitat segments (0.062 ac) within the open median of the existing highway will permanently alter the capacity of this habitat to support effective GGS thermoregulation and will change basking behavior. Individuals that could presently stop there to bask would no longer find them suitable areas; once the median is covered over by the additional bridge lanes, the bank habitat below will be continuously out of the sun. As a result, individuals would end up having to locate other portions of suitable bank habitat along the waterways on which to bask.

Temporary effects are expected to disturb 1.47 ac of habitat for one season (generally defined by the calendar year period between May 1 and October 1). Note, with the proposed work window extension, the season will be defined as July 1 to November 1 for this project. The 1.47 ac pertain to the waterway segments and banks, and to areas extending out from the waterways that will be disrupted by noise. Temporary effects due to construction activities include increased equipment and vehicle access and movement; increased worker foot traffic; temporary additional shading of habitat due to trestle structure presence during bridge work; habitat disturbance due to the temporary placement of supporting piles during trestle work and their eventual removal; and reduced water quality in the waterways due to erosion and sedimentation. This last effect is expected to be minimized through appropriate implementation of water quality BMPs.

As a result of these permanent and temporary effects, harassment or harm of GGS individuals is likely to occur to those GGS present in the construction area when activities begin or when they enter the action area once activities are underway. However, because construction will continue for a month into the GGS inactive period, additional effects to the species are likely to occur. Increased risk is posed to the GGS during this over-wintering period, because at this time of year, the species cannot thermo-regulate as effectively, and has a low resulting body temperature; this

impairs rapid movement, and hence the ability to relocate quickly to avoid danger. Additionally, individuals move to hibernacula during this season, occupying underground burrows or crevices, where they are more susceptible to ground disturbance and overland construction work. Consequently, continued personnel foot traffic, in addition to equipment and machinery presence, would lead to injury and mortality of GGS individuals as a result of vehicle hits or being unearthed from their burrows. To minimize the impact of these additional dangers to the GGS, Caltrans has agreed to compensate for effects likely to occur during the one quarter portion of the work period window falling outside of the GGS active season by purchasing an extra 1.163 credits at the closest Service-approved conservation bank (see discussion on next page).

The agreed upon conservation responsibilities of Caltrans, in cooperation with the City of Stockton, San Joaquin County, and the SJCOG are as follows:

1. Caltrans shall implement the *Conservation Measures* and the *Reasonable and Prudent Measures* and *Terms and Conditions* as found in the Programmatic.

2. Caltrans shall implement the proposed conservation measures for Level 1 and Level 3 effects.

    a. **Level 1 effects**: No replacement minimization is proposed for temporary effects to 1.47 ac of habitat, as it is expected that each of the two phases of work potentially affecting GGS will only take one season to complete. Thus, Level 1 restoration measures to re-vegetate the areas with native grasses and forbs shall be undertaken to create higher habitat quality than that of pre-project conditions. The waterways shall also be restored to pre-construction conditions after project completion.

    b. **Level 3 effects**: The proposed project will result in the permanent loss of a total of 0.08 ac of giant garter snake aquatic and bank habitat. Prior to groundbreaking, Caltrans shall minimize for permanent habitat loss and adverse effects to the GGS by using a 3:1 compensation ratio to purchase a total of 0.24 credits at the closest Service-approved conservation bank.

The Service generally requires that the conservation bank's service area appropriately covers the project area but since no GGS banks currently exist within San Joaquin County or the wider valley, in this instance, the Service is making an exception to allow Caltrans to use the closest approved bank for its minimization efforts (Service, pers. comm., 2009).

As described in the Guidelines (p.1), if there are plans for construction activity to proceed outside of the GGS active season, the Service will be contacted as soon as possible to determine what additional measures may be necessary to minimize take of GGS. On May 12, 2009, Caltrans requested a two-week work period extension beyond October 1, the approximate end

date marking the species' active period, in order to coordinate a four-month work schedule with other listed species' windows. On July 14, 2009, Caltrans informed the Service that following approval from the NMFS to extend its construction window later in the year, the GGS window would be extended a further two weeks, until November 1, resulting in a total of four extra weeks encroaching into the GGS inactive period. Following discussion with Caltrans biologists, the Service approved this request, given that Caltrans agreed to adhere to the following measures:

1. To reflect the one month extension into the GGS active period (one quarter of the work window), the Service shall request extra minimization measures applied at a 0.75:1 ratio to the total of permanently and temporarily affected habitat (0.75 x 1.55 ac = 1.163 ac). Additional effects to the 1.55 ac total at this compensation ratio shall be minimized through the purchase of 1.163 credits at the closest Service-approved conservation bank and shall be finalized before construction activities continue into the extended work window.

    a. The Service shall require assurances that work can be completed within the requested extension period. A letter, specifically outlining the activities occurring during this period, along with any encountered problems and/or instances of take, shall be sent to the Service within 30 days of project completion.

    b. A Service-approved biologist shall be on-site to monitor construction activities between October 1 and November 1, the last day of the extension request.

INCIDENTAL TAKE STATEMENT

Section 9 of the Endangered Species Act and Federal regulations pursuant to section 4(d) of the Act, prohibit take of endangered and threatened species, respectively, without special exemption. Take is defined as harass, harm, pursue, hunt, shoot, wound, kill, trap, capture or collect, or attempt to engage in any such conduct. The Service defines harassment as an intentional or negligent act or omission that creates the likelihood of injury to listed species by annoying it to such an extent as to significantly disrupt normal behavior patterns which include, but are not limited to, breeding, feeding, or sheltering. The Service defines harm to include significant habitat modification or degradation that results in death or injury to listed species by significantly impairing behavioral patterns such as breeding, feeding, or sheltering. Incidental take is defined as take that is incidental to, and not the purpose of, the carrying out of an otherwise lawful activity. Under the terms of section 7(b)(4) and section 7(o)(2), take that is incidental to and not intended as part of the agency action is not considered to be prohibited, provided such taking is in compliance with this Incidental Take Statement.

The measures described below are nondiscretionary, and must be implemented by Caltrans so that they become binding conditions of any grant or permit issued to the applicant, as appropriate, for the exemption under section 7(o)(2) to apply. Caltrans has a continuing duty to

regulate the activity that is covered by this incidental take statement. If Caltrans (1) fails to require the applicant or any of its contractors to adhere to the terms and conditions of the incidental take statement through enforceable terms, and/or (2) fails to retain oversight to ensure compliance with these terms and conditions, the protective coverage of section 7(o)(2) may lapse.

Amount or Extent of Take

Giant Garter Snake

The Service anticipates that incidental take of the GGS will be difficult to detect for the following reasons: the GGS is cryptically colored, secretive, and aversive to human activities, thereby making detection highly difficult, as the GGS can retreat to burrows, soil crevices, or vegetative cover. Most close-range observations represent chance encounters that are difficult to predict. It is difficult to quantify an exact number of GGS that will be taken as a result of the proposed action, so in instances in which take calculations are problematic, the Service may estimate take in numbers of individuals per acre of permanently lost or degraded habitat as a result of the project action, as these impacts reflect a significant biological effect to the species. Therefore, the Service anticipates take incidental to this project as all GGS inhabiting, using, or moving through the 0.08 ac of suitable aquatic and bank habitat that will be permanently lost and shaded, as well as the 1.47 ac of suitable habitat that will be temporarily disturbed. Upon implementation of the Programmatic's *Reasonable and Prudent Measures* and *Terms and Conditions*, and the *Conservation Measures*, incidental take within this acreage in the forms of harm or harassment to the GGS stemming from habitat loss, bridge widening activities, casting piles, and increased equipment, vehicular, and human presence, will become exempt from the prohibitions described under section 9 of the Act. Injury to or mortality of two GGS individuals over the course of the Programmatic's term, stemming from being run over or unearthed during overland activities during the species' inactive period (in this project's case), also will become exempt from section 9 prohibitions.

Delta Smelt

The Service anticipates that incidental take of the DS will be difficult to detect and quantify because of its small body size and its secretive nature. Furthermore, turbid conditions in the Delta and associated waterways create challenges with identification, while the species' annual abundance fluctuates based on seasonal conditions, making steady detection problematic. Therefore, it is difficult to provide precise numbers of DS that will be injured, harassed, harmed, or killed due to the proposed action. Accordingly, the Service anticipates take incidental to this project as all DS within the 0.734 ac of DS habitat that will be affected by the project due to permanent open water loss and shading, as well as the 1.47 ac of suitable habitat that will be temporarily disturbed. Upon implementation of the following *Reasonable and Prudent Measures* and *Terms and Conditions*, and the *Conservation Measures* considered herein, incidental take associated with the project in the form of harm, harassment, injury, or mortality stemming from habitat loss, bridge widening, casting of piles, and shading effects which could

provide cover to predatory fish and disrupt elements of the DS's food chain, will become exempt from the prohibitions described under section 9 of the Act.

Effect of Take

In the accompanying biological opinion, the Service has determined that this level of anticipated take is not likely to jeopardize the continued existence of the GGS or the DS.

Reasonable and Prudent Measures

The following reasonable and prudent measures are necessary and appropriate to minimize the impact of the I-5 North Stockton Corridor Improvements Project on the DS due to the effects of construction activities.

1. All of the avoidance and minimization measures proposed in the biological assessment and the *Project Description* must be fully implemented and adhered to.

2. To maintain an active accounting of project progress and take levels, reporting updates must be submitted to the Service at appropriate intervals during construction.

Terms and Conditions

In order to be exempt from the prohibitions of section 9 of the Act, Caltrans, as well as any contractor acting on its behalf, must comply with the following terms and conditions, which implement the Reasonable and Prudent Measures described above. These Terms and Conditions are nondiscretionary.

The following Term and Condition implements Reasonable and Prudent Measure two:

1. Caltrans shall inform the Service in writing of the intended construction start date 30 days prior to this date. Caltrans shall also submit a progress update halfway through the four month work windows of each of the two construction phases. The updates shall detail any changes to the project footprint and to the extent of DS habitat directly or indirectly affected by the project, to the level of take, as well as to known effects of construction activities on the DS.

Reporting Requirements

1. A post-construction report detailing compliance with the project design criteria described under the *Description of the Proposed Action* section of this biological opinion shall be provided to the Service within 30 calendar days of completion of the project. The report shall include: (1) dates of project groundbreaking and completion; (2) pertinent information concerning the success of the project in meeting compensation and other conservation measures; (3) an explanation of failure to meet such measures, if any; (4)

known project effects on the DS, if any; (5) occurrences of incidental take of the DS, and; (6) any other pertinent information.

2. New sightings of DS or any other sensitive animal species should be reported to the California Natural Diversity Database of the California Department of Fish and Game (CNDDB). A copy of the reporting form and a topographic map clearly marked with the location in which the animals were observed also should be provided to the Service.

Disposition of Individuals Taken

The Sacramento Fish and Wildlife Office is to be notified by Caltrans via email or telephone within one working day of the observation of and/or finding of any dead or injured DS and/or any other listed species occurring as a result of project activities. In the case of a dead animal, the individual animal should be preserved, as appropriate, and held in a secure location until instructions are received from the Service regarding the disposition of the specimen or until the Service takes custody of the specimen. Caltrans must report to the Service immediately any information about take or suspected take of federally-listed species not authorized in this opinion. Caltrans must notify the Service within 24 hours of receiving such information. Notification must include the date, time, and location of the incident or of the finding of a dead or injured animal. The Service contacts are Mr. Daniel Russell, Division Chief, Endangered Species Program, Sacramento, at (916) 414-6600 and Mr. Daniel Crum, the Resident Agent-in-Charge of the Service's Law Enforcement Division at (916) 414-6660. The California Department of Fish and Game contact is Mr. Paul Hoffman, Wildlife Biologist, at (530) 934-9309.

Any contractor or employee who, during routine operations and maintenance activities inadvertently kills or injures a listed wildlife species must immediately report the incident to his representative at his contracting/employment firm or to Caltrans. This representative must contact the Service immediately in the case of a federally-listed species and contact the CDFG in the case of a dead or injured State-listed species. The California Department of Fish and Game contact for immediate assistance is State Dispatch at (916) 445-0045.

CONSERVATION RECOMMENDATIONS

Conservation recommendations are suggestions of the Service regarding discretionary measures to minimize or avoid adverse effects of a proposed action on listed species or critical habitat, or regarding the development of new information. These measures may serve to minimize or avoid further adverse effects of a proposed action on listed, proposed, or candidate species, or on designated critical habitat. They may also serve as suggestions on how action agencies can assist species conservation in furtherance of their responsibilities under section 7(a)(1) of the Act, or recommend studies improving an understanding of a species' biology or ecology. Wherever possible, conservation recommendations should be tied to tasks identified in recovery plans. The Service is providing you with the following conservation recommendations:

1. It is recommended that Caltrans assist the Service in implementing recovery actions identified in the recovery plan for the Sacramento-San Joaquin Delta native fishes.

2. It is recommended that Caltrans examine the use of natural fiber check dams enhanced with polyacrylamide as a BMP to control sediment discharge.

In order for the Service to be kept informed of actions minimizing or avoiding adverse effects or benefiting listed and/or proposed species or their habitats, the Service requests notification of the implementation of these recommendations.

RE-INITIATION—CLOSING STATEMENT

This concludes the Service's review of the proposed I-5 North Stockton Corridor Improvements Project as submitted in your March 13, 2009 letter. As provided in 50 CFR § 402.16, re-initiation of formal consultation is required where discretionary Federal agency involvement or control over the action has been maintained (or is authorized by law) and if (1) the amount or extent of incidental take is exceeded, (2) new information reveals effects of the agency action that may affect listed species or critical habitat in a manner or an extant not considered in this biological opinion, (3) the agency action is subsequently modified in a manner that causes an effect to the listed species or critical habitat that was not considered in this biological opinion, or (4) a new species is listed or critical habitat designated that may be affected by the action. In instances where the amount or extent of incidental take is exceeded, any operations causing such take must cease pending reinitiation; otherwise, the project will be out of compliance with this biological opinion.

Please contact Jen Schofield, Ryan Olah, or Susan P. Jones at the letterhead address or at (916) 414-6600 if you have any questions regarding this letter on the I-5 North Stockton Corridor Improvements Project. The Service wishes to thank you for your continued efforts and dedication to the conservation of America's wildlife resources.

Sincerely,

For Susan K. Moore
Field Supervisor

Enclosures
Instructions for providing Funds to the Delta Smelt Conservation Fund
Two-Party Agreement between the United States Fish and Wildlife Service and the Center for Natural Lands Management
Three-Party Agreement between the United States Fish and Wildlife Service, Delta Smelt Conservation Fund Participant, and the Center for Natural Lands Management
Delta Smelt Conservation Fund Account Payment Receipt

cc:
Mr. Walter C. Waidelich, Jr., Division Administrator, Federal Highways Administration, Sacramento, California
Mr. Dan Gifford, California Department of Fish and Game, Rancho Cordova, California

Literature Cited

Arthur, J.F. and M.D. Ball 1978. Entrapment of suspended materials in the San Francisco Bay-Delta Estuary. U.S. Dept. Interior, Bureau of Reclamation, Sacramento, California.

_____ 1979. Factors influencing the entrapment of suspended material in the San Francisco Bay-Delta Estuary. Pages 143-174 in T.J. Conomos (ed.). Pacific Division, Amer. Assoc. Advance. Sci., San Francisco, California

_____ 1980. The significance of the entrapment zone location to the phytoplankton standing crop in the San Francisco Bay-Delta Estuary. U.S. Dept. Interior, Water and Power Resources Service.

California Department of Fish and Game. 2001. 2001 Summer Townet Survey [database on the internet]. http://www.delta.dfg.ca.gov/data/skt/skt2002/index.html. Accessed on September 18, 2002.

California Department of Fish and Game, Bay Delta Region. 2008. 20mm Delta smelt survey. http://www.delta.dfg.ca.gov/data/20mm/

California Department of Water Resources and U.S. Bureau of Reclamation. 1994. Effects of the Central Valley Project and State Water Project on Delta smelt and Sacramento splittail. Sacramento, California. 230 pp.

California Natural Diversity Data Base (CNDDB). 2009. RareFind 3. California Department of Fish and Game. Sacramento, California.

Erkkila, L.F., J.F. Moffett, O.B. Cope, B.R. Smith, and R.S. Nelson. 1950. Sacramento-San Joaquin Delta fishery resources: effects of Tracy pumping plant and delta cross channel. U.S. Fish and Wildlife Services Special Report. Fisheries 56. 109 pp.

Ganssle, D. 1966. Fishes and decapods of San Pablo and Suisun bays. Pp.64-94 in D.W. Kelley, ed. Ecological studies of the Sacramento-San Joaquin Estuary, Part 1. Calif. Dept. Fish and Game, Fish Bulletin No. 133.

Hansen, E. 2009. Unpublished data. U. S. Fish and Wildlife Service (FWS) Agreement No. 802709G514 – Report of Progress to Date. November 15, 2009.

Knutson, A.C., Jr. and J.J. Orsi. 1983. Factors regulating abundance and distribution of the shrimp *Neomysis mercedis* in the Sacramento-San Joaquin Estuary. Transactions of the American Fisheries Society 112:476-485.

Monroe, M.W. and J. Kelly 1992. State of the Estuary: A report on conditions and problems in the San Francisco Bay/Sacramento-San Joaquin Delta Estuary. San Francisco Estuary Project, Oakland, California.

Moyle, P.B. 1976. Inland Fishes of California. University of California Press, Berkeley, California. 405 pp.

Moyle, Peter B. 2002. Inland Fishes of California. University of California Press. Berkeley, California. 576 pp.

Moyle, P. B. and J. Cech, Jr. 1988. Fishes: An introduction to Ichthyology. Prentice Hall, Englewood Cliffs, New Jersey. 559 pages.

Moyle, P. B., B. Herbold, D. E. Stevens, and L. W. Miller. 1992. Life history and status of Delta smelt in the Sacramento-San Joaquin Estuary, California. Trans. Am. Fish. Soc.121:67-77.

Orsi, J.J. and W.L. Mecum. 1986. Zooplankton distribution and abundance in the Sacramento-San Joaquin Delta in relation to certain environmental factors. Estuaries 9(4B):326-339.

Radtke, L. D. 1966. Distribution of smelt, juvenile sturgeon, and starry flounder in the Sacramento-San Joaquin Delta. Pp. 115-119 in J. L. Turner and D. W. Kelley (eds.). Ecological studies of the Sacramento-San Joaquin Estuary, Part 2. California Department of Fish and Game Fish Bulletin No. 136.

Souza, K. and M. Bryant. 2002. Townet Survey and Fall Midwater Trawl. Interagency Ecological Program for the San Francisco Estuary. Newsletter 15(2):21-24.

Stevens, D. E. and S. W. Miller. 1983. Effects of river flow on abundance of young chinook salmon, American shad, longfin smelt, and Delta smelt in the Sacramento-San Joaquin river system. North American Journal of Fisheries Management 3:425-437.

Stevens, D. E., S. W. Miller, and B. C. Bolster. 1990. Report to the Fish and Game Commission: A status review of the Delta smelt (*Hypomesus transpacificus*) in California. California Department of Fish and Game Candidate Species Status Rept. 90-2. 149 pages.

Swanson, C. And J. J. Cech, Jr. 1995. Environmental tolerances and requirements of the Delta smelt, *Hypomesus transpacificus*. Final Report. Davis, California. 77 pp.

Turner, J.L. and D.W. Kelly. 1966. Ecological studies of the Sacramento-San Joaquin Delta. Calif. Dept. of Fish and Game Bulletin No. 136.

U.S. Fish and Wildlife Service (Service). 1991. Proposed Rule to List the Giant Garter Snake, *Thamnophis gigas* as an Endangered Species. Federal Register 56:67046.

_____ 1994. Endangered and threatened wildlife and plants: Critical habitat determination for the Delta smelt. December 19, 1994. Federal Register 59(242): 65256-65279.

_____ 1996. Sacramento-San Joaquin Delta Native Fishes Recovery Plan. Portland, Oregon.

_____ 1997. Guidelines for the Restoration and/or Replacement of Giant Garter Snake Habitat. In Appendix A of the Programmatic Formal Consultation for U.S. Army Corps of Engineers 404 Permitted Projects with Relatively Small Effects on the Giant Garter Snake within Butte, Colusa, Glenn, Fresno, Merced, Sacramento, San Joaquin, Solano, Stanislaus, Sutter and Yolo Counties, California. File # 1-1-F-97-149. Issued on November 13, 1997. Available at: http://www.fws.gov/sacramento/es/documents/ggs%20appendix%20a.PDF. Sacramento Fish and Wildlife Service Field Office. Sacramento, California.

_____ 2004. Formal Programmatic Consultation on the Issuance of Section 10 and 404 Permits for Projects with Relatively Small Effects on the Delta Smelt (*Hypomesus transpacificus*) and its Critical Habitat within the Jurisdiction of the Sacramento Fish and Wildlife Office of the U.S. Fish and Wildlife Service, California. File # 1-1-04-F-0345. Sacramento, California. 30 pages.

Wang, J.C.S. 1986. Fishes of the Sacramento-San Joaquin estuary and adjacent waters, California: A guide to the early life histories. Interagency Ecological Study Program for the Sacramento-San Joaquin Estuary. Sacramento, California. Technical Report 9.

Wang, J.C.S. 1991. Early life stages and early life history of the Delta smelt, *Hypomesus transpacificus*, in the Sacramento-San Joaquin Estuary, with comparison of early life stages of the longfin smelt, *Spirinchus thaleichthys*. Interagency Ecological Studies Program for the Sacramento-San Joaquin Estuary. Sacramento, California. Technical Report 28.

Wylie, G. D., M. L. Casazza, and J. K. Daugherty. 1997. 1996 progress report for the giant garter snake study. Unpublished (preliminary) report. U.S. Geological Survey, Biological Resources Division, Dixon Field Station, Dixon, California. May 1, 1997. 6 pp. + Figures.

PERSONAL COMMUNICATIONS

Sanchez, Kenneth and J. Knight. 2009. U.S. Fish and Wildlife Service, Sacramento, California. Because there are no existing giant garter snake banks in the San Joaquin Valley and based on a previous understanding between Caltrans and Kenneth Sanchez (Service) that Caltrans would be able to use a Sacramento Valley conservation bank for this project, authorization to use the 'closest Service-approved conservation bank' was agreed upon by the Service and Caltrans for a total of four projects within San Joaquin County.

Instructions for providing Funds to the Delta Smelt In-Lieu Conservation Fund

When fully executed, the enclosed Deposit Agreement fulfills, in part, the conservation measures described for the I-5 North Stockton Corridor Improvements project, in accordance with the biological opinion (file no. 81420-2009-F-0638-1). This biological opinion, issued by the U.S. Fish and Wildlife Service (Service) on March 3, 2010, describes conservation measures needed to minimize the adverse effects of this project on the federally threatened Delta smelt.

The Deposit Agreement (i.e., the enclosed document entitled "Agreement Between the United States Fish and Wildlife Service, Delta Smelt Conservation Fund Participant, and The Center for Natural Lands Management") is between the Service, the California Department of Transportation (Participant), and The Center for Natural Lands Management. If you are satisfied with the Deposit Agreement, sign this agreement. Also enclosed is a partially completed Payment Receipt. Please ensure that the Participant Information section of the Payment Receipt is accurate. Then forward the Deposit Agreement, the Payment Receipt, and a check made payable to the Delta Smelt Conservation Fund Account for the amount specified in the Deposit Agreement, to:

The Center for Natural Lands Management
215 West Ash Street
Fallbrook, California 92028

You may retain the enclosed copy of the document entitled "Agreement Between the United States Fish and Wildlife Service and The Center for Natural Lands Management" for your records. In addition, you and the Service will receive copies of the signed Deposit Agreement and completed Payment Receipt after processing by The Center for Natural Lands Management. When the Service receives the payment receipt and a copy of the signed Deposit Agreement, completion of this portion of the conservation measures for the I-5 North Stockton Corridor Improvements project will be considered fulfilled.

AGREEMENT BETWEEN THE
UNITED STATES FISH AND WILDLIFE SERVICE
AND
THE CENTER FOR NATURAL LANDS MANAGEMENT

This Agreement is made this 18th day of OCTOBER 2004, by and between the United States Fish and Wildlife Service ("Service") and the Center for Natural Lands Management ("CNLM"), hereinafter referred to collectively as the "Parties."

I. RECITALS

This Agreement is based on the following facts, intentions and expectations:

A. The Service and the CNLM wish to cooperate in facilitating the development of a regional program to conserve federally listed delta smelt in California. A major goal of this program is to provide an effective, efficient means by which consulting Federal or State agencies and/or private individuals (hereafter collectively referred to as the "Proponents"), can, in accordance with the Endangered Species Act of 1973, as amended, 16 U.S.C. 1533 et seq. (ESA), minimize and mitigate certain impacts of their projects on delta smelt (*Hypomesus transpacificus*), and carry out their obligations in a manner that allows the Service to maximize protection for these species and their habitat. Pursuant to this Agreement, third parties who voluntarily elect to do so, will, at the direction of the Service, deposit fees into one or more accounts as defined in Paragraph II.A.2(a). Upon receipt of written instructions from the Service and solely at the direction of the Service, the CNLM will cause the disbursement of the funds held in the account to Service-approved delta smelt conservation banks, or for the purchase of land or conservation easements to provide permanent protection and perpetual management of habitat for delta smelt. The Service has developed this process in consultation with representatives of the development and environmental communities in an effort to facilitate planned natural resource conservation while ensuring the implementation of conservation measures to minimize and mitigate the adverse effects of development projects on delta smelt.

B. The Service, a constituent agency within the U.S. Department of the Interior, is mandated under the ESA and other Federal conservation laws, to protect and conserve wildlife, fish and plant species. The Service has jurisdiction over the conservation, protection, enhancement and management of fish, wildlife, native plants and habitat necessary for biologically sustainable populations of these species.

C. The CNLM is a non-profit corporation of the State of California that was created in 1990 to preserve native plants and animals in their natural environments and protect the diversity of species with consideration for their complicated interrelationships.

D. The Service is authorized to enter into this Agreement by the ESA, 15 U.S.C. 1531 et seq., the Fish and Wildlife Coordination Act, 16 U.S.C. 661-666c, and the Fish and Wildlife Act of 1956, 16 U.S.C. 742(f) et seq.

E. The purpose of this Agreement is to facilitate the preservation, restoration, enhancement, and creation of delta smelt habitat by providing a means for third parties to expedite their compliance with the ESA through payment of a Delta Smelt Conservation Fee to be used to conserve delta smelt habitat.

II. AGREEMENT

In consideration of the recitals set forth above, the covenants herein and other consideration, the receipt and adequacy of which are hereby acknowledged, the Parties agree as follows:

A. Obligations of the Parties

1. The Service

(a) The Service shall establish Delta Smelt Conservation Fees to minimize the effects of projects resulting in take of delta smelt, shallow water habitat, or otherwise adversely affect the delta smelt and its habitat that require review under section 7 or section 10 of the ESA. The Service shall notify the Federal agency and Proponent, if any is involved, of the availability and amount of the voluntary Delta Smelt Conservation Fee, the payment of which will satisfy, in whole or in part, the Proponent's obligations under the ESA to address impacts on the delta smelt or its habitat,

(b) The Service shall inform each Proponent in writing that payment of a Delta Smelt Conservation Fee into the Delta Smelt Conservation Fund Account (as defined herein below) is voluntary and that other options are available to satisfy the Proponent's obligation under the ESA to address the impacts of its project on delta smelt and their habitat. The Service shall provide each Proponent with a copy of this Agreement prior to the Proponent's payment of a Delta Smelt Conservation Fee and advise the Proponent that the CNLM's Fee for administering the Delta Smelt Conservation Fund Account shall be the amount set forth below in Paragraph II.A.2(b) of this Agreement. An additional agreement, a form of which is attached hereto as Exhibit A, shall be executed by the Service, the CNLM and the Proponent for each fee deposited to the Delta Smelt Conservation Fund Account.

(c) The Service will direct all Delta Smelt Conservation Fees deposited in the Delta Smelt Conservation Fund Account in accordance with this Agreement, and interest and earnings thereon, and all disbursements from the Account, with the exception of the Management Fee identified under Paragraph II.A.2(b) below, to finance the acquisition and management of delta smelt habitat to offset project impacts to such habitat as provided in Paragraph II.A.1(a) above.

(d) The Service shall be solely responsible for identifying and approving appropriate delta smelt conservation lands to be acquired and managed with Delta Smelt Conservation Fees and shall direct the CNLM to disburse funds from the Delta Smelt Conservation Fund Account for such acquisition and management including restoration and creation when appropriate. The Service shall keep a record of all conservation bank transactions and shall provide the CNLM with reasonable access and copies of such records.

2. The CNLM

(a) Delta Smelt Conservation Fees will be deposited into one or more interest-bearing accounts ("Delta Smelt Conservation Fund Account") at a reputable financial institution reasonably acceptable to the Service. Any interest or earnings accrued shall remain with the account.

(b) Upon deposit by the CNLM of the Delta Smelt Conservation Fee in the Delta Smelt Conservation Fund Account the CNLM shall deduct from the Delta Smelt Conservation Fund Account, a Management Fee, set at the rate of two point five percent (2.5%) of each deposit in the Delta Smelt Conservation Fund Account to defray the costs associated with administration of the Account.

(c) The CNLM shall provide to the Service on or before December 31, an annual accounting showing the deposits, interest and earnings received, and disbursements of all sums made pursuant to this Agreement during the period beginning October 1 and ending on September 30.

(d) Duly authorized employees of the CNLM shall disburse all funds in the Delta Smelt Conservation Fund Account solely at the written direction of the Service.

III. SPECIAL TERMS AND CONDITIONS

A. Delta Smelt Conservation Fees deposited with the CNLM in accordance with this Agreement shall not be used for the "creation" of delta smelt habitat, or acquisition of land for "creation" of delta smelt habitat, unless approved in writing by the Service.

B. This Agreement does not impose upon the CNLM any obligation to acquire or manage any delta smelt habitat.

C. This Agreement does not impose upon the CNLM any obligations to maintain an accounting of the biological values associated with Delta Smelt Conservation Fees deposited or disbursed pursuant to this Agreement or to match Delta Smelt Conservation Fees deposited with specific delta smelt habitat acquisitions.

D. Nothing in this Agreement shall prohibit the disbursement of Delta Smelt Conservation Fees to enable the purchase of a land parcel otherwise appropriate for use as a

conservation bank or other conservation purpose because a portion of the parcel is not suitable for delta smelt habitat conservation. Because delta habitat may occur within a larger land parcel that is only available as a single unit, disbursements from the Delta Smelt Conservation Fund Account may be made to acquire an entire parcel.

E. Notwithstanding anything contained herein to the contrary, the CNLM shall have no obligation pursuant to this Agreement other than to deposit and disburse the Delta Smelt Conservation Fees in accordance with this Agreement, subject to the terms and conditions hereof.

IV. EFFECTIVE DATE

This Agreement shall take effect immediately upon execution by the Service and the CNLM.

V. TERM OF AGREEMENT

Except as otherwise provided herein, this Agreement shall be in effect for a period of four (4) years from the effective date hereof.

VI. AMENDMENTS

Amendments to this Agreement may be proposed by either Party and shall become effective upon the written agreement of both Parties.

VII. TERMINATION

This Agreement may be terminated by either Party without cause at any time upon thirty (30) days written notice to the other Party. Upon termination, the CNLM shall provide the Service with an accounting for the Delta Smelt Conservation Fund Account in accordance with the procedure set forth in Paragraph II.A.2(c) of this Agreement. All funds remaining in the account shall be transferred to an entity designated by the Service to receive such funds. Within 30 days following final disbursal of funds, the CNLM shall provide the Service a final accounting showing the deposits (including interest accrued thereon) and disbursements of all sums received pursuant to this Agreement, from the date of the last annual accounting through the date of final disbursement.

VIII. MISCELLANEOUS PROVISIONS

A. Entire Agreement

This Agreement and its related Exhibits contain the entire agreement of the Parties with respect to the matters covered by this Agreement, and no other agreement, statement, or promise

made by either Party, or to any employee, officer, or agent of either Party, which is not contained in this Agreement shall be binding or valid.

B. Interpretation and Headings

The language in all parts of this Agreement shall in all cases be simply construed according to its fair meaning and not strictly for or against either Party. Headings of the paragraphs of this Agreement are for the purpose of convenience only and the words contained in such headings shall in no way be held to explain, modify, amplify, or aid in the interpretation, construction, or meaning of the provisions of this Agreement.

C. Notices

All notices, demands, or requests from one Party to the other Party may be personally delivered, sent by facsimile, sent by recognized overnight delivery service, or sent by mail, certified or registered, postage prepaid, to the addresses stated in this paragraph and shall be effective at the time of personal delivery, facsimile, transmission, or mailing.

The Service: U.S. Fish and Wildlife Service
2800 Cottage Way, W-2605
Sacramento, CA 95825-1846
Attention: Chief, Endangered Species Division
Telephone: 916/ 414-6600
Facsimile: 916/ 414-6712

The CNLM: Center for Natural Lands Management
425 East Alvarado Street, Suite H
Fallbrook, CA 92028-2960
Attention: Sherry Teresa, Executive Director
Telephone: 760/731-7790
Facsimile: 760/731-7791
E-mail: cnlmpres@aol.com

Either Party may change the address to which such notices, demands, requests or other communications may be sent by giving the other Party written notice of such change. The Parties agree to accept facsimile transmitted signed documents and agree to rely on such documents as if they bore original signatures. Each Party agrees to provide to the other Party, within seventy-two (72) hours after transmission, such documents bearing the original signatures.

D. Successors and Assigns

This Agreement, and the rights and obligations thereunder shall not be transferred or otherwise assigned by the CNLM without the prior written approval of the proposed transferee/assignee by the Service.

E. Execution

This Agreement may be executed in several counterparts and all counterparts so executed shall constitute one agreement which shall be binding on all of the parties, notwithstanding that all of the parties are not signatory to the original or the same counterpart. If any provision of this Agreement is held invalid, the other provisions shall not be affected thereby. This Agreement represents the entire agreement of the parties and may not be amended, except in writing signed by each party hereto. Each party to this Agreement warrants to the other that it is duly organized, validly existing and, if a corporation, qualified to do business in the State of California, and that it and the respective signatories have full right and authority to enter into and consummate this Agreement and all related documents.

IN WITNESS WHEREOF, the Parties have executed this Agreement as of the date set forth above:

UNITED STATES FISH AND WILDLIFE SERVICE

By: Chf Wagano

Name: Christopher D. Wagano

Title: Deputy Asst. F.S. - SSP.

Date: 10/12/06

THE CENTER FOR NATURAL LANDS MANAGEMENT

By: Michael C. Stroud

Name: Michael C. Stroud

Title: Director of operations

Date: 10/18/06

Enclosure (Form of Three Party Agreement and Payment Receipt)

AGREEMENT BETWEEN THE UNITED STATES FISH AND WILDLIFE SERVICE, DELTA SMELT CONSERVATION FUND PARTICIPANT AND THE CENTER FOR NATURAL LANDS MANAGEMENT

This Agreement is entered into this 3rd day of March, 2010 ; by and between the United States Fish and Wildlife Service (the "Service"), The Center for Natural Lands Management (the "CNLM"), and the California Department of Transportation (the "Participant").

For good and valuable consideration, the receipt and adequacy of which are hereby acknowledged, the parties hereby agree as follows:

1. The biological opinion for the proposed Interstate-5 North Stockton Corridor Improvements Project, dated March 3, 2010, between the Service and the California Department of Transportation, issued under section 7 of the Endangered Species Act of 1973, as amended, (ESA), evaluates the impacts of the Participant's activities on the Delta smelt (*Hypomesus transpacificus*) and its habitat and specifies alternative measures available to the Participant to minimize the take of Delta smelt incidental to the Participant's activities. In accordance with the biological opinion, the Participant elects and hereby agrees to deposit a Delta smelt fee established by the Service in the amount of eighty-eight thousand, eighty dollars ($88,080.00) (the "Delta Smelt Conservation Fee") into one or more accounts at a financial institution reasonably acceptable to the CNLM and the Service in such investments as are approved by the Service (the "Delta Smelt Conservation Fund Account"), as a means of minimizing the incidental take of Delta smelt resulting from the Participant's activities and otherwise reducing the impacts of the Participant's activities on Delta smelt and their habitat. The Delta Smelt Conservation Fee shall be labeled in the CNLM records as "For use in San Joaquin County only."

2. The Service agrees that the total amount of the Delta Smelt Conservation Fee deposited by the Participant in the Delta Smelt Conservation Fund Account, with the exception of the management fee identified in paragraph 6 of this Agreement, shall be used, at the direction of the Service, to purchase 2.202 conservation credits in one or more of the Service approved Delta smelt conservation banks or for the reimbursement of the purchase of land or conservation easements, adequately managed, endowed and protected to provide permanent protection and perpetual management of Delta smelt habitat.

3. The Service agrees that any portion of the Delta Smelt Conservation Fee deposited by the Participant into the Delta Smelt Conservation Fund Account in excess of the amount used to purchase 2.202 conservation credits or for the purchase of land or conservation easements shall be refunded at the direction of the Service to the Participant within 30 days of receipt by the CNLM of written notice from the Service setting forth the amount to be refunded to the Participant.

4. The Service and the Participant jointly acknowledge and agree that the Delta Smelt Conservation Fee shall be managed and dispersed in accordance with the "AGREEMENT

BETWEEN THE UNITED STATES FISH AND WILDLIFE SERVICE AND THE CENTER FOR NATURAL LANDS MANAGEMENT" dated October 18, 2006 (Service-CNLM Agreement), the receipt of a copy of which the Participant hereby acknowledges. If for any reason the Service-CNLM Agreement is terminated, the Service shall ensure that the Delta Smelt Conservation Fee deposited by the Participant into the Delta Smelt Conservation Fund Account will be used in accordance with this Agreement to satisfy the Participant's obligations under the biological opinion referenced above in paragraph 1, and the CNLM shall have no further liabilities or obligations hereunder.

5. The Participant acknowledges and agrees that it has voluntarily elected to deposit the Delta Smelt Conservation Fee into the Delta Smelt Conservation Fund Account in satisfaction of its obligations under the section 7 biological opinion and in lieu of carrying out other alternative conservation measures identified by the Service as available to the Participant to minimize the impacts of the Participant's activities on delta smelt and their habitat.

6. The Participant further acknowledges that the CNLM's fee for administering the Delta Smelt Conservation Fund Account (Administration Fee) is 2.5% of the Delta Smelt Conservation Fee specified above in paragraph 1 and agrees to the deduction of said Administration Fee from the total Delta Smelt Conservation Fee deposited.

7. **Participant's check made payable to the Delta Smelt Conservation Fund Account shall be forwarded together with a fully executed copy of this Agreement and the Payment Receipt form to The Center for Natural Lands Management at 215 West Ash Street, Fallbrook, California 92028.**

8. Participant acknowledges and agrees that the CNLM's sole obligation pursuant to the terms and conditions of the AGREEMENT BETWEEN THE UNITED STATES FISH AND WILDLIFE SERVICE AND THE CENTER FOR NATURAL LANDS MANAGEMENT and this Agreement (the "Agreements") is to accept the Delta Smelt Conservation Fee, deposit the Delta Smelt Conservation Fee into the Delta Smelt Conservation Fund Account and to disburse the Delta Smelt Conservation Fee solely at the direction of the Service, subject to the terms and conditions of the Agreements. The CNLM is accepting the Delta Smelt Conservation Fee solely as an accommodation to the Service and the Participant. Participant acknowledges and agrees that acceptance, deposit and disbursement of the Delta Smelt Conservation Fee by the CNLM does not create any liability or duty to Participant and Participant hereby releases the CNLM from any and all liability or claims due to the existence of these Agreements or Participant's development.

9. Participant acknowledges and agrees that the CNLM has made no representations or warranties to the Participant whatsoever and Participant assumes all risks related to its proceeding with development activities, in reliance on the section 7 authorization issued by the Service. Participant does hereby agree to indemnify the CNLM, defend and hold the CNLM harmless from and against any and all claims, damages, losses, liabilities, costs and expenses, including without limitation, attorneys' fees arising out of or in any way connected with or related to the Agreements or Participant's development.

10. Participant acknowledges and agrees that the Service has made no representations or warranties to the Participant whatsoever and Participant assumes all risks related to its proceeding with development activities, in reliance on the section 7 authorization issued by the Service. Participant does hereby agree to indemnify the Service, defend and hold the Service harmless from and against any and all claims, damages, losses, liabilities, costs and expenses, including without limitation, attorneys' fees arising out of or in any way connected with or related to the Agreements, the Delta Smelt Conservation Fund Account, Participant's development and/or any action related to section 7 of the ESA. In addition, Participant agrees to indemnify the Service, defend and hold the Service harmless from and against any and all claims, damages, losses, liabilities, costs and expenses, including without limitation, attorneys' fees in the event that any claim is brought against the Service for any act or omission arising out of or any acts it takes pursuant to the Agreements or Participant's development.

11. This Agreement may be executed in several counterparts and all counterparts so executed shall constitute one agreement which shall be binding on all of the parties, notwithstanding that all of the parties are not signatory to the original or the same counterpart. If any provision of this Agreement is held invalid, the other provisions shall not be affected thereby. This Agreement represents the entire agreement of the parties and may not be amended except by a writing signed by each party hereto. Each party to this Agreement warrants to the other that it is duly organized, validly existing and, if a corporation, qualified to do business in the State of California, and that it and the respective signatories have full right and authority to enter into and consummate this Agreement and all related documents.

In witness whereof, this Agreement is executed as of the date and year first above written, at the Fish and Wildlife Service Sacramento Field Office, Sacramento, California.

UNITED STATES FISH AND WILDLIFE SERVICE

By: Kenneth Sanchez

Name: KENNETH SANCHEZ

Title: ASST FIELD SUPERVISOR

THE CENTER FOR NATURAL LANDS MANAGEMENT

By:

Name:

Title:

THE CALIFORNIA DEPARTMENT OF TRANSPORTATION (PARTICIPANT)

By:

Name:

Title:

U. S. FISH & WILDLIFE SERVICE
DELTA SMELT CONSERVATION FUND ACCOUNT
PAYMENT RECEIPT

PROJECT PARTICIPANT INFORMATION

Name: California Department of Transportation

Address: 2015 East Shields Avenue, Suite A-100, Fresno, CA 93726

Telephone: (559) 243-8196

Contact: Zachary Parker

PROJECT INFORMATION

Project Name: Interstate-5 North Stockton Corridor Improvements Project, San Joaquin Co., CA

Project Location: A segment of I-5 extending from Charter Way at the southern end of the corridor to just north of Eight Mile Road at the northern end and within the Terminous, Lodi South, and Stockton West United States Geological Survey (USGS) 7.5- minute quadrangles.

County: San Joaquin

Project Description: Caltrans, in cooperation with the City of Stockton Public Works Department, San Joaquin County Public Works Department, and the San Joaquin Council of Governments, proposes to improve a 12.1 mile segment of I-5; widen the freeway from six to eight lanes, construct new auxiliary lanes, new interchanges, and modify existing interchanges.

Service File #: 81420-2009-F-0638 Corps/Other File #:

Total Acres Impacted: 0.734 acres

FEE INFORMATION

Credits Purchased: 2.202 credits (at 3:1)

Total Payment Amount: $88,080 ($40,000/credit)

PAYMENT INFORMATION

Payee: Delta Smelt Conservation Fund Account- (For use in San Joaquin County only)

Payer: _____

Amount: _____

Method of Payment: Check No. _____ Money Order No. _____ Wire Transaction ____

Received by: Name:_____ Title: _____

_____ Date: _____
(Signature)

United States Department of the Interior

FISH AND WILDLIFE SERVICE
San Francisco Bay-Delta Fish and Wildlife Office
650 Capitol Mall, Fifth Floor
Sacramento, California 95814

IN REPLY REFER TO:
81420-2009-F-0638-R001-1

JAN 3 2011

Mr. Zachary Parker
Biology Branch Chief
California Department of Transportation, District 6
2015 East Shields Avenue, Suite A-100
Fresno, California 93726

Subject: Reinitiation of the Biological Opinion and Appendage to the *Programmatic Biological Opinion on the Effects of Small Highway Projects on the Threatened Giant Garter Snake in Butte, Colusa, Glenn, Sacramento, San Joaquin, Solano, Sutter, Yolo, and Yuba Counties, California* for the Interstate-5 North Stockton Corridor Improvements Project, San Joaquin County, California (California Department of Transportation EA 10-0G4700, 10-SJ-5-PM 25.2/37.3)

This is the U.S. Fish and Wildlife Service's (Service) response to the California Department of Transportation's (Caltrans) request to amend the biological opinion (BO) for the proposed Interstate-5 North Stockton Corridor Improvements Project (project) in San Joaquin County, California (Service file number 81420-2009-F-0638), issued on March 3, 2010. Your letter, dated October 26, 2010, was received in this office via electronic-mail (e-mail) on October 27, 2010, with the hard copy received on October 29, 2010. Under consideration is Caltrans' request to add an additional season of in-water construction work at several bridge structure locations and to extend both work seasons from a four-month to a five-month period. At issue are the continued effects these changes will have on the federally-threatened giant garter snake (*Thamnophis gigas*; GGS) and potential effects to the federally-threatened Delta smelt (*Hypomesus transpacificus*; DS). As stated in the BO's closing statement, reinitiation of formal consultation is required in the event of certain occurrences; in the present circumstances, this is when the agency action is subsequently modified in a manner that causes an effect to the listed species that was not considered in the BO. The Service and Caltrans have also discussed the recent availability of a new Service-approved DS conservation bank in the Legal Delta; as the DS Conservation Fund has not yet been paid into, as outlined in the BO, it has been agreed that compensation measures for the DS can be served better through utilization of a Service-approved conservation bank. This response was prepared in accordance with section 7(a)(2) of the Endangered Species Act of 1973, as amended (16 U.S.C. 1531 *et seq.*) (Act).

In reviewing the request, the Service has relied upon: (1) the Service's March 3, 2010, BO; (2) Caltrans' October 26, 2010, amendment request letter to the Service; (3) e-mail correspondence

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between Caltrans and the Service dated November and December 2010; and (4) other information available to the Service.

Consultation History

October 27&29, 2010: The Service received an electronic copy of a letter from Caltrans requesting an amendment to the BO regarding an extension of the number of work days in the construction season window, as well as an allowance for a second season of in-water work the subsequent calendar year. The original hard copy letter was received on October 29.

November 8, 2010: The Service e-mailed Caltrans with questions, comments, and recommendations pertaining to its request letter.

November 9, 2010. Caltrans responded via e-mail to the Service's questions and made special note to address the additional compensation credits and scheduling of restoration work as commented on by the Service. Caltrans confirmed that restoration of bank habitat will be part of the winterization of the project, with erosion control Best Management Practices (BMPs) incorporated at the end of each season. Once structure construction is complete, final bank stabilization treatment will take place.

November 17-18, 2010. The Service e-mailed Caltrans for confirmation regarding whether the conservation measures proposed, i.e. increasing GGS compensation and restricting restoration work within the two season construction window, were agreeable to Caltrans. Caltrans replied that the lead engineer felt confident that erosion control and bank re-vegetation could be finished by the end of the second season on December 1, 2012. Caltrans also acknowledged the expected increase in compensation.

December 9, 2010. The Service e-mailed Caltrans to inquire whether it had already fulfilled its DS compensation obligations by using the DS Conservation Fund; if not, a new DS conservation bank had recently become available which could be utilized instead. Caltrans replied that the Conservation Fund had not yet been used; it would therefore inform the San Joaquin Council of Governments, one of its cooperative partners, of the change.

December 15 & 16, 2010. The Service e-mailed Caltrans to disclose the fact that due to the changes in the effects levels resulting from the increased work window length, additional compensation would be anticipated for temporary effects to the GGS. Caltrans acknowledged this clarification on December 16.

According to Caltrans, the request to increase the number of seasons and work days per-season to two five-month seasons derived from the completion of more detailed designs of the bridge structures at the Calaveras River, Five Mile Slough, and Fourteen Mile Slough. These updated designs determined that the original number of work days/seasons, as assessed in the BO, was insufficient for concluding the necessary in-water work at these locations. In addition, the

schedule for the overall project was moved forward so that the first season of Phase 1 work is now due to begin on July 1, 2011, rather than on July 1, 2012. The addition of one extra month in each of the two seasons will likely involve activities pertaining to falsework installation and removal at the Calaveras River and Fourteen Mile Slough structures. No falsework will remain in the waterways following December 1 of each season.

The Service approves these changes to the project description. Given that two fifths of each work period will now be extended into the GGS inactive season, it is likely that there will be significant continuing effects to the GGS and potential suitable habitat during these periods. However, since a work season scheduled later in the year is integral to avoiding the main DS spawning season (February – June), the DS is much less likely to be in the project area during these later months; it is therefore improbable that there will be significant supplementary effects to the DS in addition to what was previously analyzed and addressed in the BO.

The Service has also recommended that with the availability of a new Service-approved DS conservation bank in the Legal Delta, compensation efforts for the DS would be better served here than by using the DS Conservation Fund. Caltrans has acknowledged and agreed to this change.

The following changes are to be made to the March 3, 2010 BO. All modifications and additions are in **bold**:

Page 6, under *Project Schedule*, is currently written as:

"In-water work for the first phase, which includes the inside widening over the Calaveras River, inside and outside widening at Fourteen Mile Slough, inside and outside widening at Five Mile Slough, inside widening at Mosher Slough, and inside widening at Bear Creek, is anticipated to begin no earlier than July 1, 2012, and be completed by November 1, 2012. In-water work for the second phase, which includes the outside widening at Mosher Slough and Bear Creek, is anticipated to begin on July 1, 2014, and be completed by November 1, 2014.

Night work is proposed to take place every day during the four month construction window (July 1 – November 1)."

Modify the schedule to reflect an increase in the number of seasons and work days per season. This is amended to read:

"In-water work for the first phase, which includes the inside widening over the Calaveras River, inside and outside widening at Fourteen Mile Slough, inside and outside widening at Five Mile Slough, inside widening at Mosher Slough, and inside widening at Bear Creek, is anticipated to **take place over two consecutive seasons of five months each, with the first beginning no earlier than July 1, 2011, and continuing through December 1, 2011, and the second initiating on July 1, 2012 and continuing through December 1, 2012.** In-water work for the

second phase, which includes the outside widening at Mosher Slough and Bear Creek, is anticipated to begin on July 1, 2014, and be completed by November 1, 2014.

Night work is proposed to take place every day during **both five**-month construction windows (July 1 – **December** 1)."

Page 6, #1 under Proposed Avoidance and Minimization Measures for the Delta Smelt, is currently written as:

1. "All in-water work will be conducted between July 1 and November 1 to avoid the main spawning season and to time work to be conducted when Delta smelt are less likely to be in the project area. Since the DS population is at low numbers, it is important to minimize potential adverse effects to the species and to avoid direct take. Each of the two phases of water work in the sloughs and channels will be confined to one season so as limit further potential effects and disturbance brought about by multiple seasons of construction activity."

Modify the dates and number of seasons necessary per phase. This is amended to read:

1. "All in-water work will be conducted between July 1 and **December 1** to avoid the main spawning season and to time work to be conducted when Delta smelt are less likely to be in the project area. Since the DS population is at low numbers, it is important to minimize potential adverse effects to the species and to avoid direct take. **The first phase of water work in the sloughs and channels will consist of two seasons, but the second phase** will be confined to one season so as to limit further potential effects and disturbance brought about by multiple seasons of construction activity."

Page 7, #8 under Proposed Avoidance and Minimization Measures for the Delta Smelt, is currently written as:

8. "To minimize the effects of the permanent loss of 0.734 acres (ac) of DS habitat, Caltrans proposes using a 3:1 compensation ratio to pay a fee of $88,080 ($40,000/ac x 2.202 credits) into the Delta Smelt Conservation Fund prior to ground-breaking. The Conservation Fund is held by the Center for Natural Lands Management (CNLM) and the associated agreement exists between the CNLM, the Service, and the Conservation Fund Participant (Caltrans in this case). Once an open DS conservation bank becomes available, these funds will then be used to purchase credits."

Delete the Conservation Fund language and include new reference to utilization of the available DS conservation bank. This is amended to read:

8. "To minimize the effects of the permanent loss of 0.734 acres (ac) of DS habitat, Caltrans proposes using a 3:1 compensation ratio to **purchase 2.202 credits at an available Service-approved DS conservation bank, prior to ground-breaking, whose service area covers the project area."**

Page 16, second and fourth paragraphs under Effects of the Action, *Delta Smelt*, is currently written as:

"Individual DS could also be affected through potential contact with equipment or by capture or entrapment in cofferdams during dewatering when present in the work area. By following the work window restrictions of performing all in-water work from July 1 through November 30, however, these interactions will be minimized and the number of affected DS is likely to be low."

"Temporary effects to an additional 1.47 ac are also anticipated to occur during the placement of temporary piles at four of the bridge locations due to general disturbance during these in-water activities, resulting in noise, physical disruptions, and water quality issues. Since each phase of in-water work will take place during one construction season (July 1-November 1), effects are expected to be minimal."

Modify the work window dates. This is amended to read:

"Individual DS could also be affected through potential contact with equipment or by capture or entrapment in cofferdams during dewatering when present in the work area. By following the work window restrictions of performing all in-water work from July 1 through **December 1**, however, these interactions will be minimized and the number of affected DS is likely to be low."

"Temporary effects to an additional 1.47 ac are also anticipated to occur during the placement of temporary piles at four of the bridge locations due to general disturbance during these in-water activities, resulting in noise, physical disruptions, and water quality issues. **Although the first phase of in-water work will take place over two seasons (July 1-December 1), which is still considered to be a period in which impacts remain temporary, and the second phase will take place during a single construction season, overall effects are expected to be minimal.**"

Page 16, third paragraph, last sentence under Effects of the Action, *Delta Smelt*, is currently written as:

"To help minimize the effects of the permanent loss of a total of 0.734 ac (0.018 ac due to open water loss + 0.716 ac of loss due to shading) of DS habitat, Caltrans proposes paying $88,080 into the Delta Smelt Conservation Fund, prior to ground-breaking; these funds will be used to purchase conservation bank credits once a bank becomes available."

Delete the Conservation Fund language and reference utilization of the recently available DS conservation bank. Add further discussion of the significance of habitat compensation to recovery efforts. This is amended to read:

"To help minimize the effects of the permanent loss of a total of 0.734 ac (0.018 ac due to open water loss + 0.716 ac of loss due to shading) of DS habitat, Caltrans **proposes purchasing 2.202 credits at an available Service-approved DS conservation bank, prior to ground-breaking, whose service area covers the project area. This compensation measure will help lead to preservation and enhancement of suitable DS habitat and will contribute to protecting and managing the habitat for the conservation of the species in perpetuity. Resulting protected waterways and water bodies will provide habitat commensurate with, or better than, aquatic habitat lost as a result of the project, ensuring that the DS can continue to meet all its life cycle functions. This habitat will also help maintain the geographic distribution of the species in the Delta and will contribute to the recovery of the species by increasing the amount of habitat that is secure from development threats."**

Page 20, third paragraph under Effects of the Proposed Action, is currently written as:

"Temporary effects are expected to disturb 1.47 ac of habitat for one season (generally defined by the calendar year period between May 1 and October 1). Note, with the proposed work window extension, the season will be defined as July 1 to November 1 for this project."

Modify the season window. This is amended to read:

"Temporary effects are expected to disturb 1.47 ac of habitat for **two** seasons **during the first phase (a season is** generally defined by the calendar year period between May 1 and October 1). Note, with the proposed work window extension, **each** season will be defined as July 1 to **December 1** for this project."

Add a new paragraph addressing GGS habitat compensation and the link to recovery under Effects of the Proposed Action, beginning page 19. This is amended to read:

"Caltrans proposes to minimize both permanent and temporary effects to the GGS and its habitat by purchasing credits at the closest Service-approved conservation bank. Caltrans' compensation measures will help lead to preservation and enhancement of suitable GGS habitat and will contribute to protecting and managing the habitat for the conservation of the species in perpetuity. The protected land will provide habitat commensurate with, or better than, habitat lost as a result of the project, ensuring that the GGS can continue to breed, forage, shelter, and meet all its life cycle functions. This land will also help maintain the geographic distribution of the species and will contribute to the recovery of the species by increasing the amount of habitat that is secure from development threats and other factors that threaten the species."

Page 20-21, fourth paragraph, beginning with the second sentence, under Effects of the Proposed Action, is currently written as:

"However, because construction will continue for a month into the GGS inactive period, additional effects to the species are likely to occur. Increased risk is posed to the GGS during this over-wintering period, because at this time of year, the species cannot thermo-regulate as effectively, and has a low resulting body temperature; this impairs rapid movement, and hence the ability to relocate quickly to avoid danger. Additionally, individuals move to hibernacula during this season, occupying underground burrows or crevices, where they are more susceptible to ground disturbance and overland construction work. Consequently, continued personnel foot traffic, in addition to equipment and machinery presence, would lead to injury and mortality of GGS individuals as a result of vehicle hits or being unearthed from their burrows. To minimize the impact of these additional dangers to the GGS, Caltrans has agreed to compensate for effects likely to occur during the one quarter portion of the work period window falling outside of the GGS active season by purchasing an extra 1.163 credits at the closest Service-approved conservation bank (see discussion on next page)."

Make changes to the work time expected to occur outside of the GGS active period and to the number of compensation credits. This is amended to read:

"However, because **first phase** construction will continue for **two** months into the GGS inactive period **over two seasons**, additional effects to the species are likely to occur. Increased risk is posed to the GGS during this over-wintering period, because at this time of year, the species cannot thermo-regulate as effectively, and has a low resulting body temperature; this impairs rapid movement, and hence the ability to relocate quickly to avoid danger. Additionally, individuals move to hibernacula during this season, occupying underground burrows or crevices, where they are more susceptible to ground disturbance and overland construction work. Consequently, continued personnel foot traffic, in addition to equipment and machinery presence, would lead to injury and mortality of GGS individuals as a result of vehicle hits or being unearthed from their burrows. To minimize the impact of these additional dangers to the GGS, Caltrans has agreed to compensate for effects likely to occur during the **two fifths** portion of **each** work period window falling outside of the GGS active season by purchasing an extra **3.72** credits at the closest Service-approved conservation bank (see discussion on next page); **this total is separate from those credits to be purchased for Level 2 temporary effects and Level 3 permanent effects to the GGS.**"

Page 21, conservation responsibility #2.a under Effects of the Proposed Action, is currently written as:

9. "Caltrans shall implement the proposed conservation measures for Level 1 and Level 3 effects.

a. **Level 1 effects**: No replacement minimization is proposed for temporary effects to 1.47 ac of habitat, as it is expected that each of the two phases of work potentially affecting GGS will only take one season to complete. Thus, Level 1 restoration measures to re-vegetate the areas with native grasses and forbs shall be undertaken to create higher habitat quality than that of pre-project conditions. The waterways shall also be restored to pre-construction conditions after project completion."

The increase in the number of seasons elevates the Level 1 temporary effects to Level 2 temporary effects. Habitat replacement at a compensation ratio of 1:1, in addition to the original conservation measure of restoration, is now appropriate. This is amended to read:

2. "Caltrans shall implement the proposed conservation measures for Level **2** and Level 3 effects.

   a. **Level 2 effects**: **Replacement minimization at 1:1** is proposed for temporary effects to 1.47 ac of habitat **(=1.47 ac)**, as it is expected that **the first phase of work potentially affecting the GGS will take two seasons to complete; however, no replacement minimization for temporary effects is proposed for the second phase since it will only take one season to complete**. Restoration measures to re-vegetate the areas with native grasses and forbs shall be undertaken to create higher habitat quality than that of pre-project conditions. The waterways shall also be restored to pre-construction conditions after project completion."

Pages 21-22, beginning with the last paragraph under <u>Effects of the Proposed Action</u>, are currently written as:

"As described in the Guidelines (p.1), if there are plans for construction activity to proceed outside of the GGS active season, the Service will be contacted as soon as possible to determine what additional measures may be necessary to minimize take of GGS. On May 12, 2009, Caltrans requested a two-week work period extension beyond October 1, the approximate end date marking the species' active period, in order to coordinate a four-month work schedule with other listed species' windows. On July 14, 2009, Caltrans informed the Service that following approval from the NMFS to extend its construction window later in the year, the GGS window would be extended a further two weeks, until November 1, resulting in a total of four extra weeks encroaching into the GGS inactive period. Following discussion with Caltrans biologists, the Service approved this request, given that Caltrans agreed to adhere to the following measures:

1. To reflect the one month extension into the GGS active period (one quarter of the work window), the Service shall request extra minimization measures applied at a 0.75:1 ratio to the total of permanently and temporarily affected habitat (0.75 x 1.55 ac = 1.163 ac). Additional effects to the 1.55 ac total at this compensation ratio shall be minimized

through the purchase of 1.163 credits at the closest Service-approved conservation bank and shall be finalized before construction activities continue into the extended work window.

a. The Service shall require assurances that work can be completed within the requested extension period. A letter, specifically outlining the activities occurring during this period, along with any encountered problems and/or instances of take, shall be sent to the Service within 30 days of project completion.

b. A Service-approved biologist shall be on-site to monitor construction activities between October 1 and November 1, the last day of the extension request."

Add new information pertaining to the additional request for a work window extension, modify the calculations to reflect a new credit total, and modify the window in which a Service-approved biologist shall be on-site to monitor activities. This is amended to read:

"As described in the Guidelines (p.1), if there are plans for construction activity to proceed outside of the GGS active season, the Service will be contacted as soon as possible to determine what additional measures may be necessary to minimize take of GGS. On May 12, 2009, Caltrans requested a two-week work period extension beyond October 1, the approximate end date marking the species' active period, in order to coordinate a four-month work schedule with other listed species' windows. On July 14, 2009, Caltrans informed the Service that following approval from the NMFS to extend its construction window later in the year, the GGS window would be extended a further two weeks, until November 1, resulting in a total of four extra weeks encroaching into the GGS inactive period. **On October 26, 2010, Caltrans requested an additional season in which to conduct water-based work activities as well as an extra month of work days extending into the GGS inactive period for each of the two seasons. This will result in two five-month construction seasons for the first phase.** Following discussion with Caltrans biologists, the Service has approved this request, given that Caltrans agrees to adhere to the following measures:

1. To reflect the **two** month extensions into the GGS active period **for two seasons (two-fifths** of each work window), the Service shall request extra minimization measures applied at a **1.2:1** ratio to the total of permanently and temporarily affected habitat: **(2/5 x 1.55 ac at 3:1) x 2 seasons = 3.72 ac**. Additional effects to the 1.55 ac total at this compensation ratio shall be minimized through the purchase of **3.72** credits at the closest Service-approved conservation bank and shall be finalized before construction activities continue into the extended work window.

   a. The Service shall require assurances that work can be completed within the requested extension period. A letter, specifically outlining the activities occurring during this period, along with any encountered problems and/or instances of take, shall be sent to the Service within 30 days of project completion.

b. A Service-approved biologist shall be on-site to monitor construction activities between October 1 and **December 1**, the last day of the extension request."

Page 24, Term and Condition number one, under Terms and Conditions, is currently written as:

1. "Caltrans shall inform the Service in writing of the intended construction start date 30 days prior to this date. Caltrans shall also submit a progress update halfway through the four month work windows of each of the two construction phases. The updates shall detail any changes to the project footprint and to the extent of DS habitat directly or indirectly affected by the project, to the level of take, as well as to known effects of construction activities on the DS."

Modify the work window length. This is amended to read:

1. "Caltrans shall inform the Service in writing of the intended construction start date 30 days prior to this date. Caltrans shall also submit a progress update halfway through **each of the two five-month work windows of the first construction phase; it shall also submit a progress update halfway through the single five-month work window of the second construction phase**. The updates shall detail any changes to the project footprint and to the extent of DS habitat directly or indirectly affected by the project, to the level of take, as well as to known effects of construction activities on the DS."

Page 26, under Enclosures, is currently written as:

"*Instructions for providing Funds to the Delta Smelt Conservation Fund*
Two-Party Agreement between the United States Fish and Wildlife Service and the Center for Natural Lands Management
Three-Party Agreement between the United States Fish and Wildlife Service, Delta Smelt Conservation Fund Participant, and the Center for Natural Lands Management
Delta Smelt Conservation Fund Account Payment Receipt"

Delete all the enclosure references; these are no longer relevant since utilization of the Conservation Fund is being replaced with utilization of an available Service-approved DS conservation bank.

The remainder of the March 3, 2010, BO is unchanged. This concludes the reinitiation of formal consultation for the Interstate-5 North Stockton Corridor Improvements Project. As provided in 50 CFR § 402.16, reinitiation of formal consultation is required where discretionary Federal agency involvement or control over the action has been maintained (or is authorized by law) and if: (1) the amount or extent of incidental take is exceeded; (2) new information reveals effects of the agency action that may affect listed species or critical habitat in a manner or to an extent not considered in this opinion; (3) the agency action is subsequently modified in a manner that causes an effect to the listed species or critical habitat that was not considered in this opinion; or

(4) a new species is listed or critical habitat designated that may be affected by the action. In instances where the amount or extent of incidental take is exceeded, any operations causing such take must cease pending re-initiation.

Please contact Jen Schofield or Michael Welsh at the letterhead address or at (916) 414-6630 if you have any questions regarding this letter.

Sincerely,

Jennifer Norris
Assistant Field Supervisor

cc:
Mr. Dan Gifford, California Department of Fish and Game, Rancho Cordova, California
Ms. Kelly Souza, California Department of Fish and Game, Stockton, California

UNITED STATES DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
NATIONAL MARINE FISHERIES SERVICE
Southwest Region
501 West Ocean Boulevard, Suite 4200
Long Beach, California 90802-4213

APR 15 2009

In response refer to:
2009/00440

Zachary K. Parker
Chief
Central Region Biology Branch
California Department of Transportation
2015 East Shields Avenue, Suite A-100
Fresno, California 93726-5428

Dear Mr. Parker:

This letter is in response to your February 5, 2009, request for initiation of section 7 consultation with NOAA's National Marine Fisheries Service (NMFS), pursuant to the Endangered Species Act (ESA) and the Magnuson-Stevens Fishery Conservation and Management Act (MSA), concerning the Interstate 5 North Stockton Corridor Improvement project located in San Joaquin County, California. You have determined that the proposed project may affect, but is not likely to adversely affect Central Valley (CV) Steelhead (*Oncorhynchus mykiss*) or their designated critical habitat. In addition, the proposed project area has been designated as Essential Fish Habitat (EFH) of Pacific salmon pursuant to the Magnuson-Stevens Fishery Conservation and Management Act (MSA). This letter also serves as consultation under the authority of, and in accordance with, the provisions of the Fish and Wildlife Coordination act of 1934 (FWCA), as amended. NMFS recognizes that the California Department of Transportation (Caltrans) is acting in conjunction with the Federal Highway Administration (FHWA) for this project and has assumed FHWA's responsibilities under Federal environmental laws as allowed by the Memorandum of Understanding between FHWA and Caltrans, which became effective on July 1, 2007.

Caltrans proposes to add mainline lanes to Interstate 5 between Country Club Boulevard and Eight Mile Road, construct auxiliary lanes between interchanges, modify two existing interchanges and construct two new interchanges in the North Stockton area of San Joaquin County. These proposed changes will reduce existing traffic congestion, improve traffic operations and accommodate as much as possible of the anticipated travel demand in the corridor through the year 2035. The existing structures must be widened to accommodate the improvements at each of the overcrossings or bridge widenings as a result of additional lanes (inside widening) and/or auxiliary lane improvements. Improvements may include the addition of new columns (within waterways or on dry land) to support the widened structures. The proposed bridge widenings at the Calaveras River, Fourteen Mile Slough, Mosher Slough, and Bear Creek will consist of three to eleven spans placed on the outside and/or between the existing left and right reinforced concrete structures. The widenings will consist of precast-pre-

stressed concrete double 'T' girders, voided slabs or 'I' girders set on concrete pile caps, supported by 24" precast concrete driven piles. There will be a total of 186 piles required for the four widened bridges. Bridge construction operations within aquatic habitats will be conducted with the use of temporary construction trestles. The temporary trestles will range from 30 to 40 feet wide. The temporary trestles will be supported on steel piles within the creek beds. Pile driving equipment, including the use of a truck crane and vibratory hammer, will be required for the construction of the temporary trestles. The construction of each trestle will take approximately 2 weeks. Access to the trestle for construction equipment will be from the roadway median and/or shoulder.

All proposed in-channel work will be conducted between June 15-October 15 and during daylight hours, thus no night time work will be conducted. Temporary stream diversion and dewatering will be required prior to excavation. This will involve installation of cofferdams using steel sheet piles vibrated into place with a vibratory hammer around the perimeter of the foundations. After the temporary stream diversion has been placed and the cofferdam perimeter has been sealed, the sealed construction area will then be dewatered by pumping the water into a suitable off-site location. During the demolition of any bridge work, a tarp or other approved method will be used below the bridge and left in place until demolition is completed. During construction, Environmentally Sensitive Area fencing will be installed within the stream zone, and upstream and downstream of the work area to protect against unanticipated habitat disturbance. Best Management Practices (BMPs), Storm Water Pollution Prevention Plan (SWPPP) and a Water Pollution Control Plan (WPCP) will be implemented during construction activities.

ESA Section 7 Consultation

Based on our review of the material provided with your request and the best scientific and commercial information currently available, NMFS concurs that the I-5 North Stockton Corridor Improvement project is not likely to adversely affect the CV steelhead or it's designated critical habitat. NMFS has reached this determination based on the following reasons:

1. All in-channel work will be conducted during June 15-October 15, when salmonids are least likely to be present in the action area and will least likely be exposed to the effects of the proposed construction activities. In addition, all-in channel work will be conducted during daylight hours to allow for an extended period of inactivity (*i.e.* night time) for salmonids, if present, to migrate undisturbed through the action area.

2. Protective fencing will be placed to keep construction activities and vehicles from impacting riparian vegetation adjacent to the project site. Thus, any vegetation not planned for removal/disturbance will be fenced to provide appropriate protection from construction impacts. Any areas within the biologically sensitive area that may be temporarily disturbed during construction shall be reseeded with erosion control seeding consisting of a sterile, non-proliferating grass species such as cereal barley or Regreen, to minimize the effects to CV steelhead and its critical habitat resulting from erosion, siltation, and other water quality impacts.

3. The project will conform to the water pollution control standards through the implementation of a SWPPP, WPCP, and BMPs. In addition, cofferdams will be placed along the perimeter of the bridge foundations. This will minimize siltation and turbidity issues that can potentially cause harm to anadromous listed fish.

4. During any bridge demolition work, a tarp or other approved method will be used below the bridge to prevent debris and foreign materials from falling into the channels that could potentially have adverse effects on anadromous listed fish. The tarp will be left in place until the demolition is completed.

This concludes ESA consultation for the I-5 Stockton Corridor Improvement project. This concurrence does not provide incidental take authorization pursuant to section 7(b)(4) and section 7(o)(2) of the ESA. Re-initiation of the consultation is required where discretionary Federal agency involvement or control over the action has been retained (or is authorized by law), and if: (1) new information reveals effects of the action that may affect listed species or critical habitat in a manner or to an extent not considered; (2) the action is subsequently modified in a manner that causes adverse effects to listed species or critical habitat; or (3) a new species is listed or critical habitat designated that may be affected by this action.

Essential Fish Habitat (EFH) Consultation

With regards to EFH consultation, the proposed project area has been identified as EFH for Chinook salmon in Amendment 14 of the Pacific Salmon Fishery Management Plan pursuant to the Magnuson Stevens Act (MSA). Federal action agencies are mandated by the MSA (section 305(b)(2)) to consult with NMFS on all actions that may adversely affect EFH and NMFS must provide EFH conservation recommendations to those agencies (section 305(b)(4)(A)). Because the proposed action has incorporated specific measures (described above) to minimize impacts to the habitat of salmonids, NMFS has determined that the proposed project will not adversely affect EFH, and additional EFH Conservation Recommendations are not being provided at this time; however, if there is substantial revision to the action, the lead Federal agency will need to re-initiate EFH consultation.

Fish and Wildlife Coordination Act (FWCA)

The purpose of the FWCA is to ensure that wildlife conservation receives equal consideration, and is coordinated with other aspects of water resources development (16 U.S.C. 661). The FWCA establishes a consultation requirement for Federal departments and agencies that undertake any action that proposes to modify any stream or other body of water for any purpose, including navigation and drainage (16 U.S.C 662(a)). Consistent with this consultation requirement, NMFS provides recommendations and comments to Federal action agencies for the purpose of conserving fish and wildlife resources. The FWCA allows the opportunity to offer recommendations for the conservation of species and habitats beyond those currently managed under the ESA and MSA. Because the proposed project is designed to avoid environmental impacts to aquatic habitat within the action area, NMFS has no additional FWCA comments to provide.

Please contact Monica Gutierrez at (916) 930-3657, or via e-mail at Monica.Gutierrez@noaa.gov if you have any questions or require additional information concerning this project.

Sincerely,

for Rodney R. McInnis
Regional Administrator

cc: Copy to File ARN # 151422SWR2009SA00086
NMFS-PRD, Long Beach, CA
Bryant Chesney, Long Beach, CA

UNITED STATES DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
NATIONAL MARINE FISHERIES SERVICE
Sacramento Area Office
650 Capitol Mall, Suite 5-200
Sacramento, California 95814-4706

DEC 29 2010

Mr. Zachary Parker,
Chief
Central Region Biology Branch
Department of Transportation,
District 6
2015 East Shields Avenue, Suite A-100
Fresno, California 93726-5428

Dear Mr. Parker:

This letter is in response to your October 26, 2010, request for an in-water work extension to widen structures at the Calaveras River, Five Mile Slough, and Fourteen Mile Slough in San Joaquin County, California. The proposed project will add mainline lanes to Interstate 5 (between Country Club Boulevard and Eight Mile Road), construct auxiliary lanes between interchanges, modify two existing interchanges, and construct two new interchanges in the North Stockton area of San Joaquin County.

Enclosed with your letter was an updated in-water work schedule detailing the activities of the project to continue through December 1, 2012. The current agreed upon in-water work window is limited from July 1, 2011, through November 1, 2011. After more detailed design of the structures was completed and reviewed by California Department of Transportation's bridge engineers, it was decided the number of working days required exceeded the number available in a single four-month season and that two five-month seasons would be required to complete the in-water work with the first beginning July 1, 2011, and ending December 1, 2011, and a second beginning July 1, 2012, and ending December 1, 2012.

NOAA's National Marine Fisheries Service (NMFS) has determined that in order to avoid the potential for adversely impacting Central Valley steelhead (*Oncorhynchus mykiss*) utilizing the Calaveras River watershed for spawning and rearing, any in-channel work should be limited to the period when the Calaveras River is in the dry. NMFS maintains that limiting in-channel activities to the fall and late-fall periods when the Calaveras River is in the dry will avoid and minimize potential impacts to listed anadromous fish in the action area. In addition, nighttime work is still anticipated for the project and all falsework will be removed before December 1 for both seasons.

After reviewing the information submitted with your letter, NMFS has determined that the proposed extension of in-water work activities to extend through December 1, 2012, will not result in adverse effects to listed species or designated critical habitat beyond those that were previously considered in the original informal consultation (April 15, 2009); therefore re-initiation of consultation is not warranted at this time.

Please contact Dylan Van Dyne at (916) 930-3725, or via e-mail at Dylan.VanDyne@noaa.gov if you have any questions or require additional information concerning this project.

Sincerely,

For Maria Rea
Central Valley Office Supervisor

cc: Copy to File ARN # 151422SWR2009SA00086
NMFS-PRD, Long Beach, CA

FOUNDATION REPORT
COUNTRY CLUB BLVD. UC (WIDEN)
(BRIDGE NO. 29-0189)
STOCKTON, CALIFORNIA
10-SJ-05, PM 28.53 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

PARIKH CONSULTANTS, INC.
2360 Qume Drive, Suite A, San Jose, CA 95131
(408) 452-9000

June 11, 2010

Job No. 206127.BR

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FOUNDATION REPORT
COUNTRY CLUB BLVD. UC (WIDEN)
(BRIDGE NO. 29-0189)
STOCKTON, CALIFORNIA
10-SJ-05 PM 28.53 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Country Club Blvd. Undercrossing (Widen) (Bridge No. 29-0189)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Country Club Blvd. Undercrossing (UC) (Widen) (Bridge No. 29-0189).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter

unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are single-span structures. Based on the as-built general plan and foundation plan of the Country Club Blvd. UC, the structures were constructed in 1970. The existing left and right bridges are approximately 126 feet long. The left bridge (Bridge No. 29-0189L) is 52 feet 11 inches wide and the right bridge (Bridge No. 29-0189R) is 64 feet 2 inches wide. The left and right bridges have not been widened since their construction in 1970.

The abutments of the existing left and right bridges are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class I 45 tons) based on the as-built data. The specified tip elevation is -65 (NAVD 29) feet at the left bridge. The specified tip elevation is -60 feet (NAVD 29) at Abutment 1 and -55 feet (NAVD 29) at Abutment 2 of the right bridge.

Proposed Construction

The widening of the Country Club Blvd. UC is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. There will be no outside widening for the Country Club Blvd. UC.

According to the designer, the proposed Country Club Blvd. UC (Widen) will match the existing span with supports at Abutments 1 and 2. The widen structure will be a cast-in-place prestressed concrete box girder with approximate length of 126 feet measured along the "I-5" Line. The width of the inside widening will be 36 feet 11 inches. The abutments will incorporate diaphragm type abutments.

Based on our discussions with the designer, the proposed bridge abutments are to be supported on Caltrans Standard Precast Prestressed Class 140 Alternate "X" (Mod) (14-inch square) piles.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The Country Club Blvd. UC is located at the intersection of Interstate 5 (I-5) and Country Club Blvd. It is approximately 0.2 miles southeast of Michigan Avenue UC. The Project Location Map, Plate 1, shows the location of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the Country Club Blvd. UC (February 1964) have been used as a reference (Ref. 1). Based on the as-built LOTB, Borings B-2 and B-4 were drilled and Cone Penetration Test (CPT) at B-1 and B-3 were performed by Caltrans in 1964. The soil borings were drilled to the approximate depths between 76 feet and 86 feet and to the bottom elevation between approximately -75 feet and -86 feet. The CPTs were pushed to the approximate depths between 61.5 feet and 82 feet and to the bottom elevation between approximately -61.0 feet and -81.5 feet.

Field Exploration in 2009

Additional field exploration was conducted in March and April 2009. Boring A-09-CC-001 was drilled on the north side and CPT-09-CC-001 was performed on the south side of Country Club Blvd. UC. The soil boring was drilled to the depth of 111.5 feet and the CPT was pushed to the approximate depth of 99.5 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A. Laboratory tests were performed on selected soil samples collected during field exploration to

evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit weight, Plasticity Index and grain size classification of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built Borings B-2 and B-4, the subsurface soils generally consist of soft peaty clay, underlain by very stiff to hard silt, underlain by intermittent layers of soft to stiff silty clay/clayey silt and loose to medium dense sands, underlain by stiff to very stiff clays (with pockets of loose sand or soft clays), underlain by medium dense to dense sands.

Based on Boring A-09-CC-001 and CPT-09-CC-001 (in the median), the subsurface soils generally consist of dense silty sand (embankment fill), underlain by very stiff lean clay, underlain by dense poorly graded/clayey sand, underlain by very stiff to hard lean clays.

Groundwater was recorded at approximate depths between 12 feet (Elevation -12 feet) and 13 feet (Elevation -13 feet) in the CPTs in February 1964. No groundwater was recorded in the soil borings in February 1964. Groundwater was measured at Elevation -4.5 feet (the depth of 28 feet below existing ground surface) during drilling for A-09-CC-001 in March 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project site (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of medium dense to dense silty sand, underlain by very stiff to hard lean clays with intermittent layers of medium dense to very dense silty sand. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for

fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring A-09-CC-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-CC-001 included in this report, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure, different foundation systems have been evaluated with respect to their design capacity and construction considerations. Caltrans standard Class 140 Alt X (Mod) Piles are recommended to be the foundation system for the abutments of the proposed bridge structure. These piles may be designed for service load of 70 tons at the abutments to the indicated pile tip elevations. Pertinent foundation design information provided by Biggs Cardosa Associates (BCA), including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2: FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 140 Alt X (Mod)	16.80	12.80	4.00	30.67	1.00	9
Abut 2	WSD	Class 140 Alt X (Mod)	16.80	12.80	4.00	34.50	1.00	9

TABLE 3: FOUNDATION DESIGN LOADS

Support No.	**Service-I Limit State (kips)**			**Strength Limit State (Controlling Group, kips)**				**Extreme Limit State (Controlling Group, kips)**			
	Total Load		**Permanent Loads**	**Compression**		**Tension**		**Compression**		**Tension**	
	Per Support	**Per Pile**	**Per Support**	**Per Support**	**Max. Per Pile.**	**Per Support**	**Max. Per Pile.**	**Per Support**	**Max. Per Pile.**	**Per Support**	**Max. Per Pile.**
Abut 1	1,215	135	885	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1,210	135	880	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads", based upon WSD is presented in Table 4 below.

The evaluation of Load Demands on the piles, based upon WSD is presented in Table 3 above. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Table 4 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	Class 140 Alt X (Mod)	12.80 ±	1,215	885	135	270	-50.0 (a)	-50.0
Abut 2	Class 140 Alt X (Mod)	12.80 ±	1,210	880	135	270	-45.5 (a)	-50.0

Notes: (i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively. The design tip elevation due to the lateral loads are estimated by BCA and not included in the table above.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

TABLE 5: PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 140 Alt X (Mod)	270	0	-50.0 (a)	-50.0	270
Abut 2	Class 140 Alt X (Mod)	270	0	-45.5 (a)	-50.0	270

Notes
(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression (b) Lateral Load.
(ii) The design tip elevation due to the lateral loads are estimated by BCA and not included in the pile data table above.
(iii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iv) Based on Caltrans review comment, it was recommended to keep the same pile length (specified tip elevation) for "ease of pile fabrication".

The estimation of the driven pile capacity in clay is based on the adhesion factor per FHWA manual (Tomlinson 1980). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 5, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles should not be spaced closer than 3 times the pile diameter measured center-to-center. For

piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter, there is no group effect for vertical pile capacity.

It is anticipated that the pile capacity will develop after driving as a result of soil "freeze" and dissipation of excess pore water pressures. The final pile capacity can be verified and the gain of pile capacity after initial driving may be evaluated based on "re-driving" after 24-hour (min.) set up. Caltrans Standard Specifications (Section 49-1.08) may be used as the pile driving acceptance criteria. As an option, Pile Driving Analyzer (PDA) may be used to evaluate the pile capacity of the driven piles when unanticipated conditions arise. Typical applications of the PDA include capacity evaluation (for both during driving and re-driving) and integrity testing for piles that have experienced hard driving.

It is prudent to make the contractor aware of these conditions so that appropriate steps can be taken to comply with the standards and maintain the integrity of the piles.

Predrilling to the bottom of the existing water line is required to minimize the vibration impact on the existing Cal Water Service water lines. The bottom of predrill elevation is -2.0 feet according to BCA. Based on the available boring/CPT information, dense sands were encountered in the embankment fill. It is recommended to follow Caltrans standard specifications (Section 49-1.06) (Ref. 7) for oversize predrilling through the existing embankment to the bottom of predrill elevation described above.

Due to the hard driving condition that may be encountered during pile driving, pile driving should be allowed to terminate short of the specified tip elevation provided the following conditions are satisfied:

- Other requirements including tension and lateral demands are met;
- Pile attaining its capacity and refusal within 5 feet above the specified tip elevation.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. It is our understanding that the structural designer will perform lateral load analyses for the piles using LPILE Program (v. 5). "Generalized Vertical Soil Profile

and Strength Parameters" based on A-09-CC-001 and CPT-09-CC-001 are included in Appendix C. To account for group effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D, respectively.

Generally, abutment piles are designed for service loads only, and batter piles are used for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for

the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for new structures. Settlement is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Boring A-09-CC-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
A-09-CC-001	31.0	1070	6.9	32.5	45.0

Based on the corrosion test results, the native subsurface soils in Boring A-09-CC-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.8 Construction Observation

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

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

REGISTERED PROFESSIONAL ENGINEER
GARY HIRAN PARIKH
No. 666
EXP. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Country Club Blvd. UC (Widen) Foundation Report (6-2010).rpt

REFERENCES

1. Country Club Blvd. UC As-built Log of Test Borings (February 1964).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

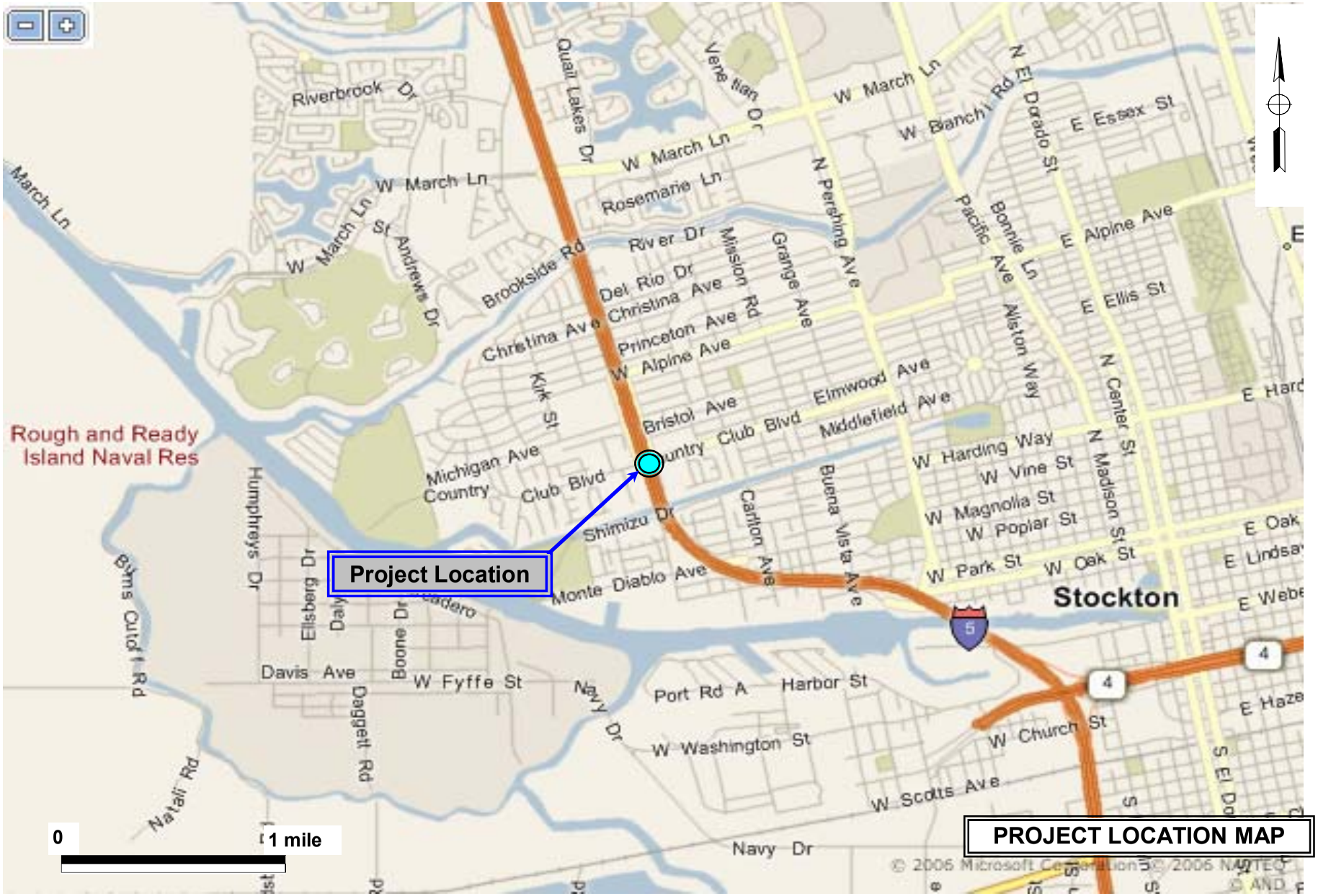
3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

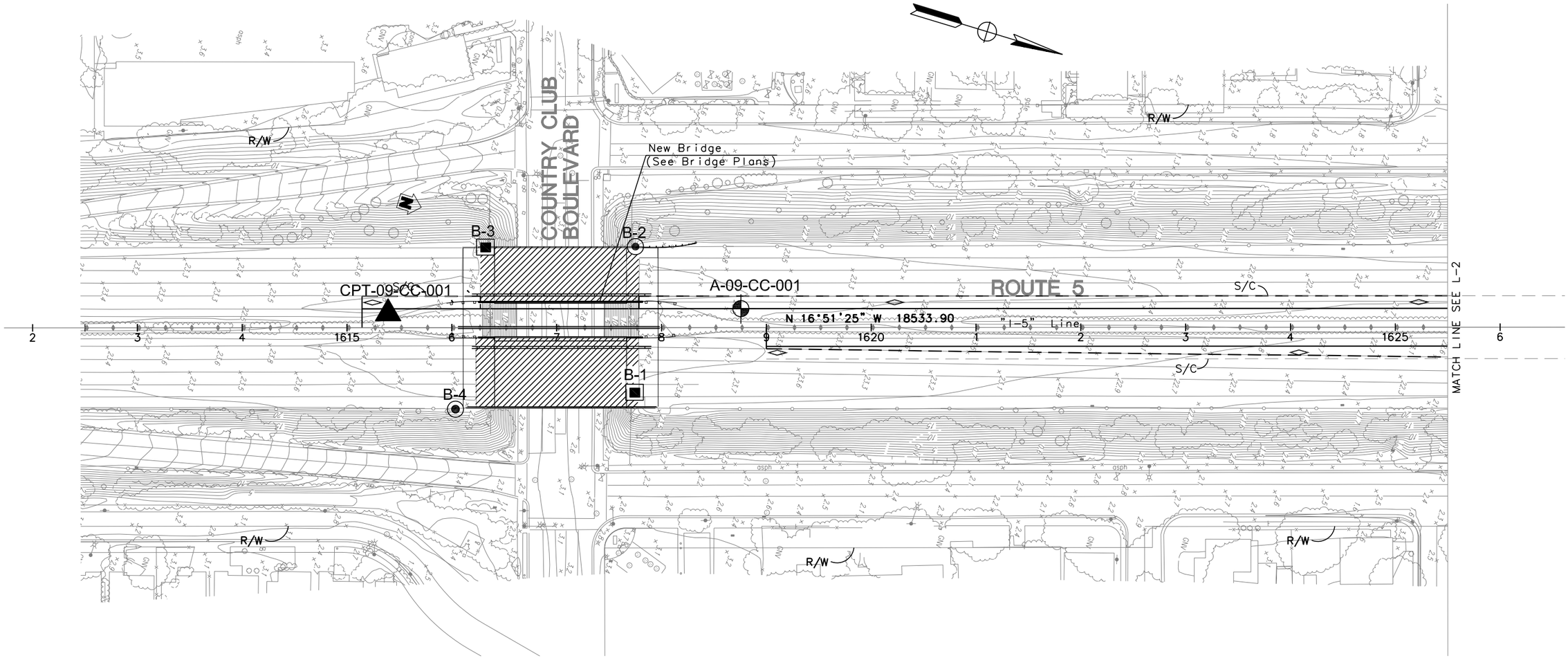
7. State of California Department of Transportation Standard Specifications May 2006



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

COUNTRY CLUB BOULEVARD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0189)	PLATE NO.: 1



LEGEND

A-09-CC-001
Approximate Boring Location

B-3
Approximate CPT location performed in 1964

B-4
Approximate Boring location drilled in 1964

CPT-09-CC-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

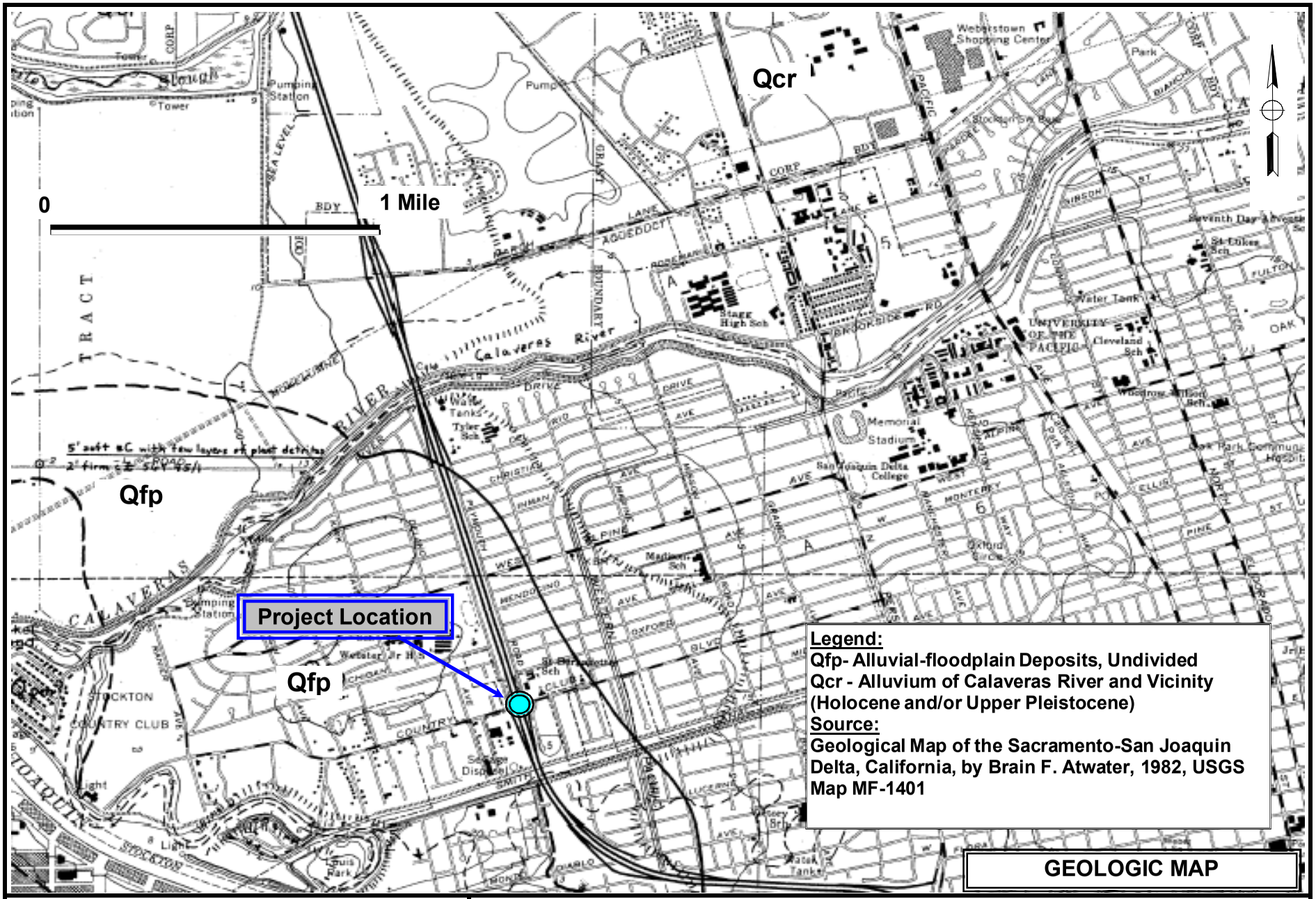
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

COUNTRY CLUB BLVD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

PLATE NO: 2



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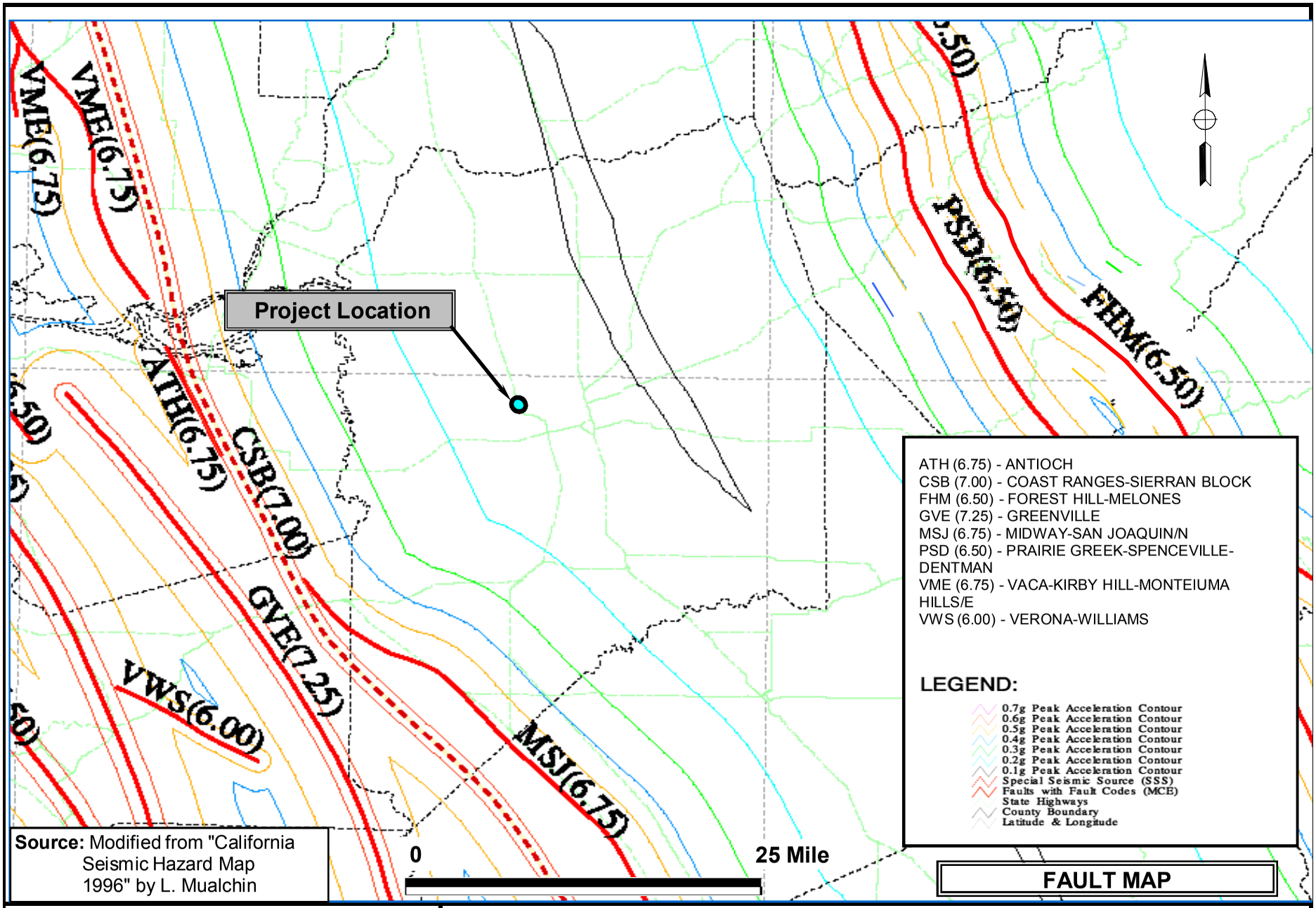
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

COUNTRY CLUB BOULEVARD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0189)	PLATE NO.: 3



Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

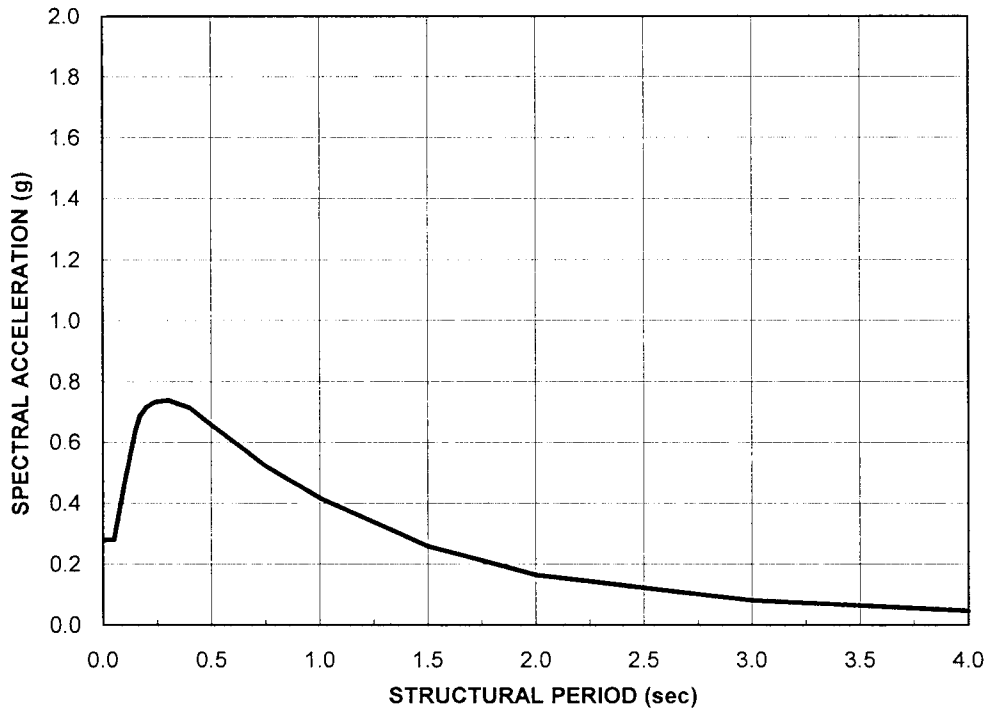
FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

COUNTRY CLUB BOULEVARD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0189)	PLATE NO.: 4

ARS DESIGN CURVE
COUNTRY CLUB BLVD UC (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

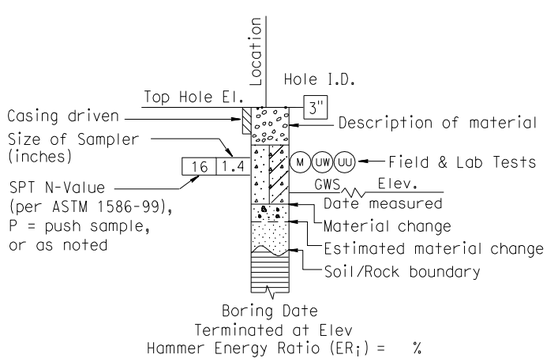
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

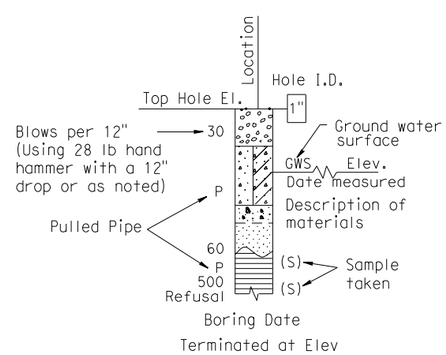
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

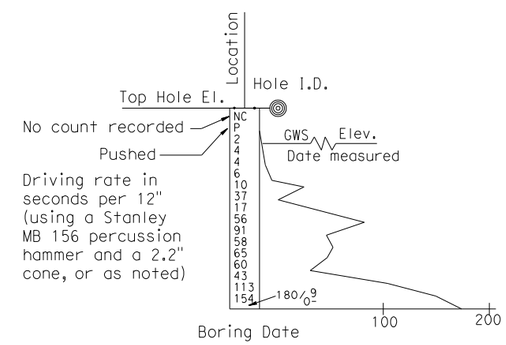
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



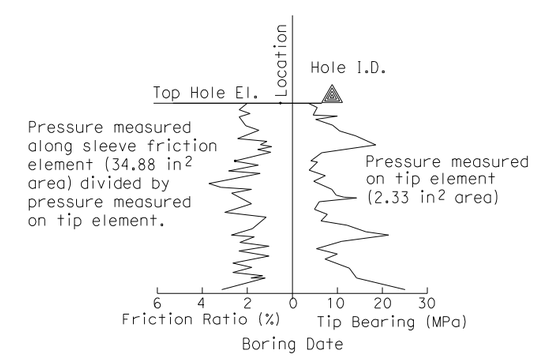
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY: O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER	BRIDGE NO. 29-0189 POST MILES 28.53
SIGN OFF DATE	CHECKED BY: D. WANG				

COUNTRY CLUB BLVD. UC (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 25 OF 28

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REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

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

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)
- (VS) Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0189
POST MILES 28.53

COUNTRY CLUB BLVD. UC (WIDEN)
SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 26 OF 28

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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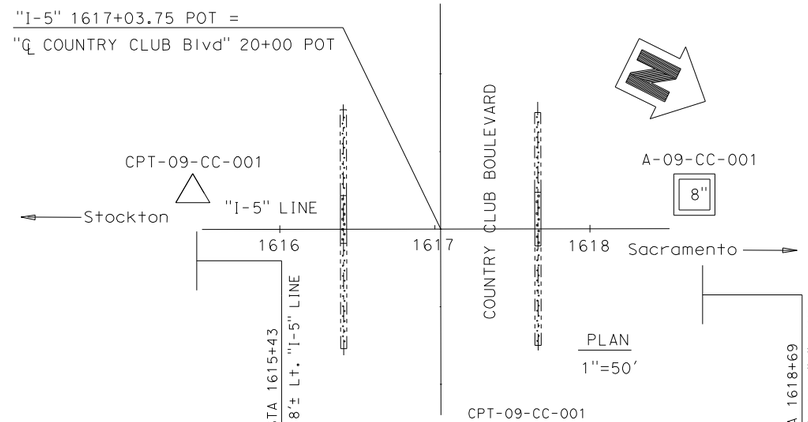
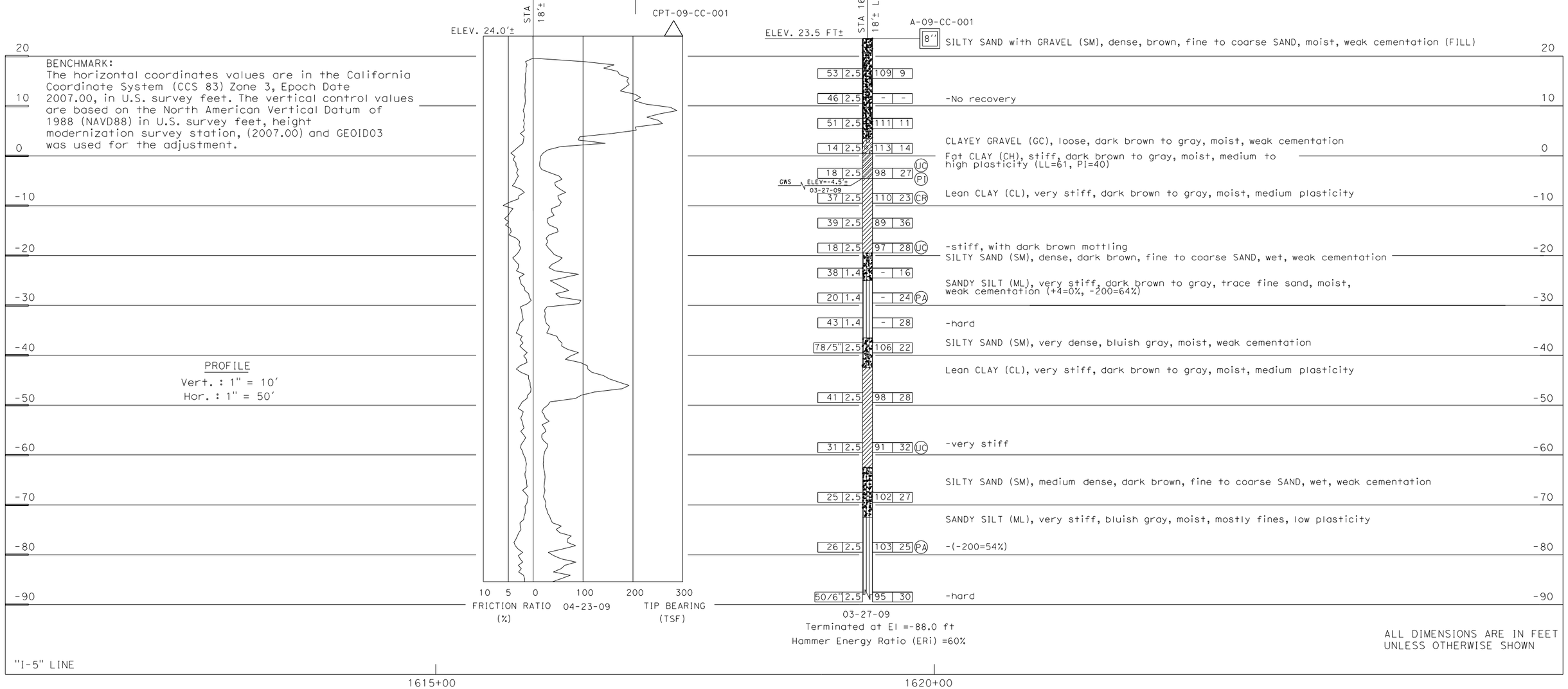
SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131





DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER

BRIDGE NO. 29-0189
POST MILES 28.53

COUNTRY CLUB BLVD. UC (WIDEN)
LOG OF TEST BORINGS 1 OF 2

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES → 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 27 OF 28

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DIST.	COUNTY	ROUTE	SECTION	SHEET NO.	TOTAL SHEETS
10	SJ	5	255/315	294	332

September 5, 1967

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES
As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5	—	—	—

REGISTERED CIVIL ENGINEER — DATE

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

COUNTRY CLUB BLVD. UC (WIDEN)

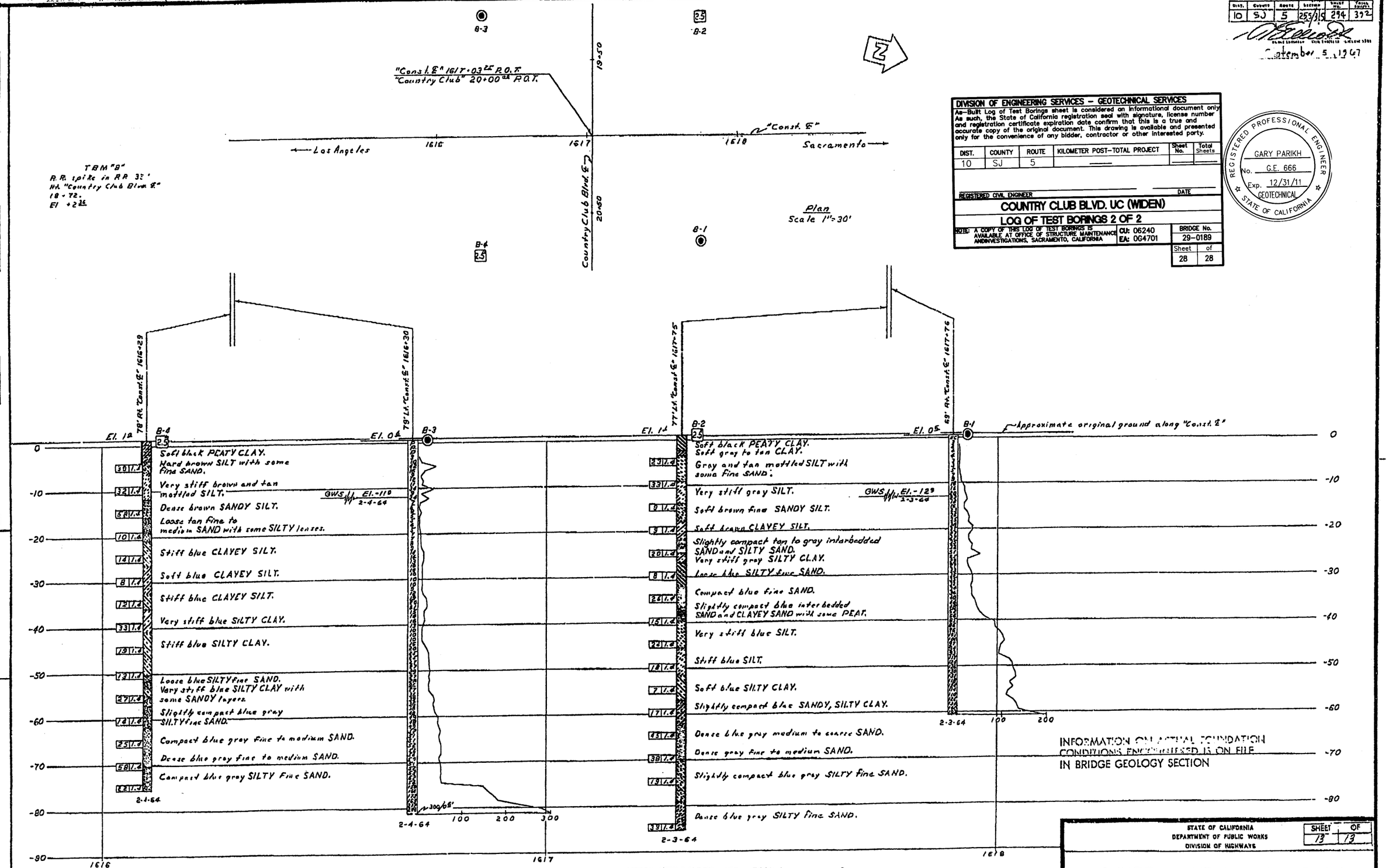
LOG OF TEST BORINGS 2 OF 2

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 064701

BRIDGE No. 29-0189

Sheet 28 of 28



INFORMATION ON ACTUAL FOUNDATION CONDITIONS ENCOUNTERED IS ON FILE IN BRIDGE GEOLOGY SECTION

LEGEND OF BORING OPERATIONS

Penetrometer; 2½" Core Penetrometer; Sonic Boring; Rotary Boring; Auger Boring; Jet Boring; Core Boring; Test Pit

LEGEND OF EARTH MATERIALS

Gravel; Sand; Silt; Clay; Sandy Clay or Clayey Sand; Sandy Silt or Silty Sand; Silty Clay or Clayey Silt; Peat or Organic Matter; Fill Material; Igneous Rock; Sedimentary Rock; Metamorphic Rock

CLASSIFICATION OF MATERIAL: BASED ON STANDARD GRADE SIZE LIMITS

Diagram shows the basis for estimated classification of earth materials by the unified system. It shows, as present in approximate proportions, the "clay-silt-sand" and "gravel-cobbles" fractions. The terms "gravelly," "sandy," "silty," "clayey," "trace," "little," "some," "and" when used to describe sand, and gravel, refer to grain size limits.

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

AS BUILT PLANS
Contract No. 10-080924
Date Completed
Document No. 000001-99

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

SHEET 13 OF 13

COUNTRY CLUB BOULEVARD U. C.

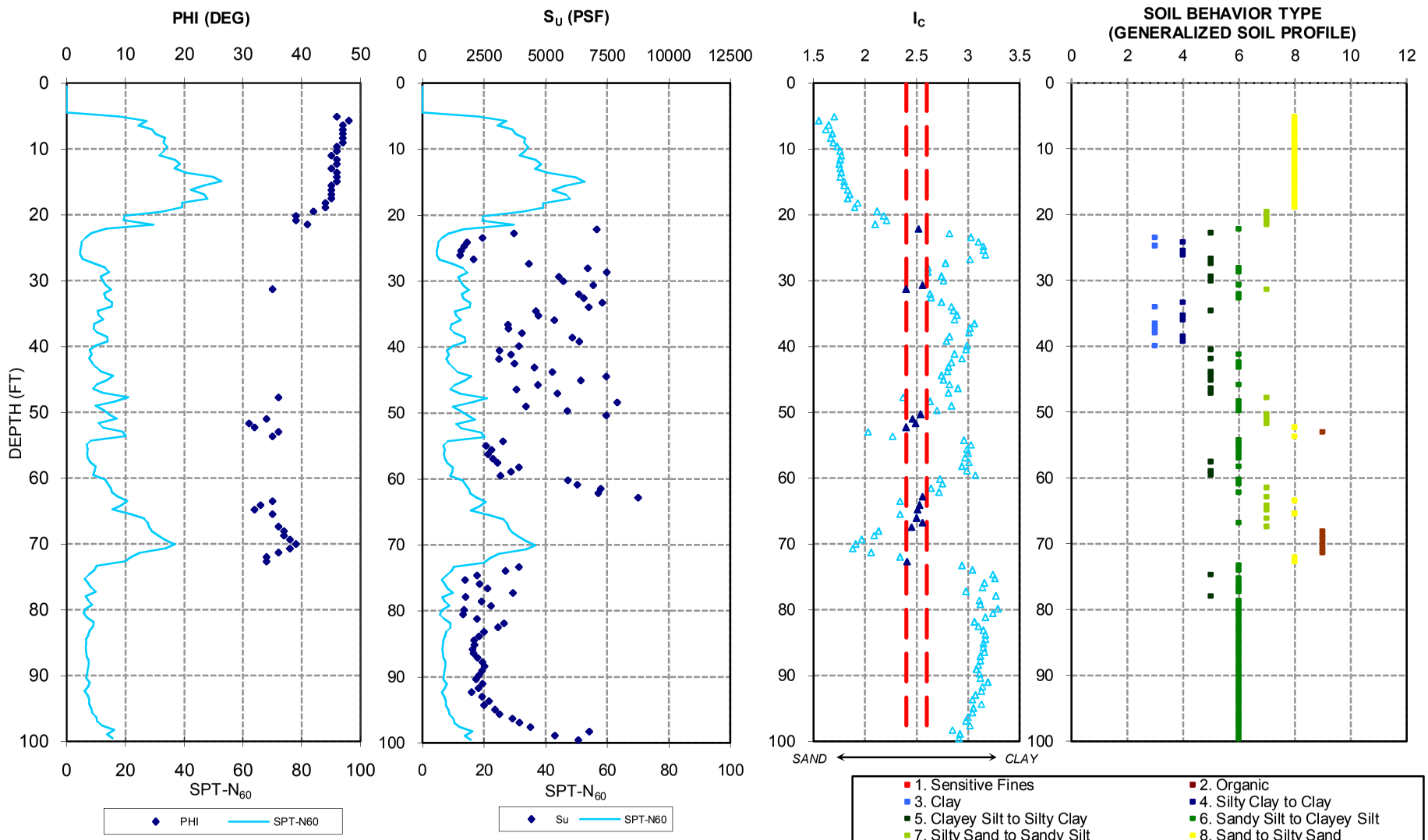
LOG OF TEST BORINGS

SCALE As Noted | BRIDGE 29-189 R | FILE | DRAWING 29189-B

Discard prints bearing earlier numbers → PREL. DRAWING No. PR-

CPT DATA CORRELATION

CPT-09-CC-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

COUNTRY CLUB BOULEVARD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0189)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

	I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-COUNTRY		
PROJECT NAME	***CLUB BLVD UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***206127.BR (BR NO. 29 0189)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-CC-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)	Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
5.085	8 Sand to Silty Sand	101.565	1.564	0.308		0.308	1.7	1.45889	0.99	0.129	1.700	1.46	148.17	1.038	153.781	0.42	1.00	1	
5.741	8 Sand to Silty Sand	162.324	2.051	0.347		0.347	1.55	1.42211	0.99	0.129	1.550	1.42	230.84	1.000	230.842		1.00	1	
6.398	8 Sand to Silty Sand	141.504	2.036	0.387		0.387	1.65	1.38626	0.99	0.128	1.650	1.39	196.16	1.003	196.816		1.00	1	
7.054	8 Sand to Silty Sand	169.399	2.367	0.427		0.427	1.62	1.35218	0.99	0.128	1.620	1.35	229.06	1.000	229.058		1.00	1	
7.71	8 Sand to Silty Sand	173.13	2.698	0.466		0.466	1.68	1.32053	0.98	0.128	1.680	1.32	228.62	1.024	234.170		1.00	1	
8.366	8 Sand to Silty Sand	191.16	2.957	0.506		0.506	1.67	1.28957	0.98	0.128	1.670	1.29	246.51	1.017	250.797		1.00	1	
9.022	8 Sand to Silty Sand	187.98	2.865	0.546		0.546	1.69	1.26002	0.98	0.128	1.690	1.26	236.86	1.031	244.221		1.00	1	
9.678	8 Sand to Silty Sand	192.567	3.182	0.585		0.585	1.73	1.23249	0.98	0.127	1.730	1.23	237.34	1.058	251.117		1.00	1	
10.335	8 Sand to Silty Sand	183.829	3.059	0.625		0.625	1.76	1.20548	0.98	0.127	1.760	1.21	221.60	1.078	238.984		1.00	1	
10.991	8 Sand to Silty Sand	174.581	2.783	0.665		0.665	1.77	1.17962	0.98	0.127	1.770	1.18	205.94	1.085	223.517		1.00	1	
11.647	8 Sand to Silty Sand	204.422	3.415	0.704		0.704	1.76	1.15546	0.98	0.127	1.760	1.16	236.20	1.078	254.729		1.00	1	
12.303	8 Sand to Silty Sand	215.297	3.437	0.744		0.744	1.75	1.13169	0.97	0.127	1.750	1.13	243.65	1.072	261.094		1.00	1	
12.959	8 Sand to Silty Sand	202.937	3.124	0.784		0.784	1.76	1.10887	0.97	0.126	1.760	1.11	225.03	1.078	242.682		1.00	1	
13.615	8 Sand to Silty Sand	225.973	3.716	0.823		0.823	1.77	1.08749	0.97	0.126	1.770	1.09	245.74	1.085	266.718		1.00	1	
14.272	8 Sand to Silty Sand	275.511	4.93	0.863		0.863	1.76	1.06641	0.97	0.126	1.760	1.07	293.81	1.078	316.853		1.00	1	
14.928	8 Sand to Silty Sand	288.155	5.744	0.903		0.903	1.8	1.04612	0.97	0.126	1.800	1.05	301.45	1.107	333.598		1.00	1	
15.584	8 Sand to Silty Sand	253.43	4.487	0.942		0.942	1.8	1.02708	0.97	0.126	1.800	1.03	260.29	1.107	288.055		1.00	1	
16.24	8 Sand to Silty Sand	228.208	3.95	0.982		0.982	1.83	1.00825	0.97	0.126	1.830	1.01	230.09	1.129	259.814		1.00	1	
16.896	8 Sand to Silty Sand	250.819	4.833	1.022		1.022	1.85	0.9901	0.96	0.125	1.850	0.99	248.34	1.145	284.364		0.99	1	
17.552	8 Sand to Silty Sand	259.545	4.614	1.062		1.062	1.83	0.97259	0.96	0.125	1.830	0.97	252.43	1.129	285.040		0.98	1	
18.209	8 Sand to Silty Sand	204.479	3.966	1.101		1.101	1.93	0.95611	0.96	0.125	1.930	0.96	195.50	1.218	238.214		0.97	1	
18.865	8 Sand to Silty Sand	206.747	3.607	1.141		1.141	1.9	0.93977	0.96	0.125	1.900	0.94	194.29	1.189	230.986		0.95	1	
19.521	7 Silty Sand to Sandy Silt	157.198	4.035	1.18		1.18	2.12	0.92437	0.96	0.125	2.120	0.92	145.31	1.492	216.824		0.95	1	
20.177	7 Silty Sand to Sandy Silt	93.298	1.729	1.218		1.218	2.18	0.90984	0.96	0.124	2.180	0.91	84.89	1.619	137.445	0.32	0.95	1	
20.833	7 Silty Sand to Sandy Silt	90.287	1.742	1.257		1.257	2.21	0.8954	0.95	0.124	2.210	0.90	80.84	1.692	136.756	0.32	0.95	1	
21.49	7 Silty Sand to Sandy Silt	144.815	2.998	1.295		1.295	2.1	0.88176	0.95	0.124	2.100	0.88	127.69	1.455	185.762		0.93	1	
22.146	6 Sandy Silt to Clayey Silt	54.343	1.586	1.333		1.333	2.52	0.86854	0.95	0.124	2.520	0.87	47.20	2.872	135.537	0.31	0.94	1	
22.802	5 Clayey Silt to Silty Clay	29.138	1.071	1.371		1.371	2.82	0.8557	0.95	0.123							0.94	1	
23.458	3 Clay	19.703	0.883	1.407		1.407	3.03	0.84388	0.95	0.123							0.93	1	
24.114	4 Silty Clay to Clay	14.884	0.55	1.445		1.445	3.1	0.83176	0.94	0.123							0.93	1	
24.77	3 Clay	14.039	0.57	1.481		1.481	3.15	0.82059	0.94	0.123							0.92	1	
25.427	4 Silty Clay to Clay	13.069	0.455	1.519	0.013	1.506	3.15	0.81301	0.94	0.123							0.92	1	
26.083	4 Silty Clay to Clay	12.837	0.442	1.556	0.034	1.523	3.17	0.80793	0.94	0.125							0.92	1	
26.739	5 Clayey Silt to Silty Clay	17.078	0.522	1.594	0.054	1.54	3.02	0.80292	0.94	0.126							0.92	1	
27.395	5 Clayey Silt to Silty Clay	33.723	1.134	1.632	0.075	1.557	2.78	0.79797	0.93	0.127							0.92	1	
28.051	6 Sandy Silt to Clayey Silt	50.876	1.557	1.669	0.095	1.574	2.61	0.79308	0.93	0.128							0.91	1	
28.707	6 Sandy Silt to Clayey Silt	56.353	1.956	1.707	0.115	1.591	2.61	0.78825	0.93	0.129							0.91	1	
29.364	5 Clayey Silt to Silty Clay	42.309	1.589	1.744	0.136	1.609	2.74	0.7832	0.92	0.130							0.91	1	
30.02	5 Clayey Silt to Silty Clay	43.845	1.866	1.782	0.156	1.626	2.76	0.77849	0.92	0.131							0.91	1	
30.676	6 Sandy Silt to Clayey Silt	52.949	1.342	1.82	0.177	1.643	2.56	0.77383	0.92	0.132	2.560	0.77	40.97	3.091	126.631	0.27	0.91	1	2.43
31.332	7 Silty Sand to Sandy Silt	64.788	1.143	1.858	0.197	1.661	2.4	0.76896	0.91	0.133	2.400	0.77	49.82	2.312	115.201	0.22	0.90	1	1.99
31.988	6 Sandy Silt to Clayey Silt	49.374	1.412	1.896	0.218	1.678	2.63	0.76442	0.91	0.134							0.90	1	
32.644	6 Sandy Silt to Clayey Silt	50.802	1.525	1.933	0.238	1.695	2.64	0.75993	0.91	0.134							0.90	1	
33.301	4 Silty Clay to Clay	56.206	2.689	1.971	0.259	1.712	2.74	0.75549	0.90	0.135							0.90	1	
33.957	3 Clay	52.565	3.179	2.008	0.279	1.728	2.84	0.75137	0.90	0.136							0.90	1	
34.613	5 Clayey Silt to Silty Clay	35.805	1.478	2.045	0.3	1.745	2.86	0.74703	0.89	0.136							0.89	1	
35.269	4 Silty Clay to Clay	36.616	1.718	2.083	0.32	1.763	2.89	0.74249	0.89	0.137							0.89	1	
35.925	4 Silty Clay to Clay	41.951	2.116	2.12	0.341	1.78	2.87	0.73826	0.88	0.137							0.89	1	
36.581	3 Clay	27.89	1.608	2.157	0.361	1.796	3.06	0.73431	0.88	0.137							0.89	1	
37.238	3 Clay	28.015	1.381	2.193	0.382	1.812	3.02	0.73041	0.87	0.138							0.89	1	
37.894	3 Clay	32.358	1.806	2.23	0.402	1.828	3.01	0.72655	0.87	0.138							0.89	1	
38.55	4 Silty Clay to Clay	47.61	2.207	2.268	0.423	1.845	2.82	0.7225	0.86	0.138							0.88	1	
39.206	4 Silty Clay to Clay	49.186	2.2	2.305	0.443	1.862	2.79	0.71848	0.86	0.138							0.88	1	
39.862	3 Clay	31.428	1.548	2.342	0.464	1.878	2.99	0.71475	0.85	0.138							0.88	1	
40.518	5 Clayey Silt to Silty Clay	25.095	0.871	2.379	0.484	1.895	2.98	0.71082	0.85	0.138							0.88	1	
41.175	6 Sandy Silt to Clayey Silt	28.599	0.804	2.417	0.504	1.913	2.87	0.70671	0.84	0.138							0.88	1	
41.831	5 Clayey Silt to Silty Clay	24.783	0.732	2.455	0.525	1.93	2.94	0.70288	0.83	0.138							0.88	1	
42.487	6 Sandy Silt to Clayey Silt	29.551	0.772	2.492	0.545	1.947	2.84	0.69908	0.83	0.138							0.88	1	
43.143	6 Sandy Silt to Clayey Silt	35.153	1.033	2.53	0.566	1.964	2.81	0.69532	0.82	0.138							0.87	1	
43.799	5 Clayey Silt to Silty Clay	40.409	1.405	2.567	0.586	1.981	2.8	0.69161	0.82	0.137							0.87	1	
44.455	5 Clayey Silt to Silty Clay	57.277	2.417	2.605	0.607	1.998	2.74	0.68793	0.81	0.137							0.87	1	
45.112	5 Clayey Silt to Silty Clay	49	1.813	2.643	0.627	2.015	2.76	0.68429	0.80	0.137							0.87	1	
45.768	6 Sandy Silt to Clayey Silt	36.225	1.114	2.68	0.648	2.032	2.82	0.68069	0.80	0.136							0.87	1	
46.424	5 Clayey Silt to Silty Clay	30.045	0.928	2.718	0.668	2.05	2.9	0.67692	0.79	0.136							0.87	1	
47.08	5 Clayey Silt to Silty Clay	43.006	1.484	2.755	0.689	2.067	2.81	0.6734	0.78	0.136							0.86	1	
47.736	7 Silty Sand to Sandy Silt	91.104	1.629	2.794	0.709	2.085	2.37	0.66971	0.78	0.135	2.370	0.67	61.01	2.194	133.866	0.30	0.86	1	2.67
48.392	6 Sandy Silt to Clayey Silt	61.989	1.753	2.832	0.73	2.102	2.63	0.66626	0.77	0.135							0.86	1	
49.049	6 Sandy Silt to Clayey Silt	33.929	0.886	2.869	0.75	2.119	2.84	0.66285	0.76	0.134							0.86	1	
49.705	6 Sandy Silt to Clayey Silt	46.306	1.134	2.907	0.771	2.136	2.7	0.65947	0.76	0.134							0.86	1	
50.361	7 Silty Sand to Sandy Silt	58.808	1.049	2.945	0.791	2.154	2.54	0.65593	0.75	0.133	2.540	0.66	38.57	2.979	114.911	0.22	0.86	1	1.98
51.017	7 Silty Sand to Sandy Silt	71.418	1.202	2.984	0.812	2.172	2.46	0.65243	0.74	0.133	2.460	0.65	46.60	2.574	119.949	0.24	0.86	1	2.16
51.673	7 Silty Sand to Sandy Silt	44.974	0.4	3.023	0.832	2.191	2.49	0.64878	0.74	0.132	2.490	0.65	29.18	2.718	79.317	0.13	0.85	1	1.14
52.329	8 Sand to Silty Sand	53.257	0.436	3.062	0.852	2.21	2.4	0.64516	0.73	0.131	2.400	0.65	34.36	2.312	79.452	0.13	0.85	1	1.15
52.986	9 Sand	95.793	0.387	3.103	0.873	2.23	2.03	0.6414	0.72	0.131	2.030	0.64	61.44	1.341	82.406	0.13	0.85	1	1.20
53.642	8 Sand to Silty Sand	91.682	1.015	3.143	0.893	2.249	2.27	0.63787	0.72	0.130	2.270	0.64	58.48	1.856	108.545	0.20	0.85	1	1.82
54.298	6 Sandy Silt to Clayey Silt	26.376	0.717	3.18	0.914	2.266	2.96	0.63474	0.71	0.130							0.85	1	
54.954	6 Sandy Silt to Clayey Silt	20.797	0.526	3.218	0.934	2.284	3.03	0.63146	0.70	0.129							0.85	1	
55.61	6 Sandy Silt to Clayey Silt	22.492	0.547	3.256	0.955	2.301	2.99	0.62839	0.70	0.128							0.85	1	
56.266	6 Sandy Silt to Clayey Silt	21.421	0.493	3.293	0.975	2.318	3	0.62536	0.69	0.128							0.85	1	
56.923	6 Sandy Silt to Clayey Silt	22.906	0.496	3.331	0.996	2.335	2.97	0.62235	0.69	0.127							0.84	1	
57.579	5 Clayey Silt to Silty Clay	24.846	0.699	3.368	1.016	2.352	3.01	0.61937	0.68	0.127							0.84	1	
58.235	6 Sandy Silt to Clayey Silt	32.25	0.931	3.406	1.037	2.369	2.94	0.61642	0.67	0.126							0.84	1	
58.891	5 Clayey Silt to Silty Clay	30.164	0.934	3.443	1.057	2.386	2.99	0.6135	0.67	0.125							0.84	1	
59.547	5 Clayey Silt to Silty Clay	26.852	0.92	3.481	1.078	2.403	3.07	0.6106	0.66	0.125							0.84	1	
60.203	6 Sandy Silt to Clayey Silt	47.7	1.06	3.519	1.098	2.421	2.73	0.60757	0.66	0.124							0.84	1	
60.86	6 Sandy Silt to Clayey Silt	50.519	1.313	3.556	1.119	2.438	2.75	0.60473	0.65	0.124							0.84	1	
61.516	7 Silty Sand to Sandy Silt	57.748	1.182	3.595	1.139	2.456	2.64	0.60175	0.65	0.123							0.84	1	
62.172	6 Sandy Silt to Clayey Silt	57.102	1.535	3.633	1.16	2.473	2.72	0.59897	0.64	0.123							0.83	1	
62.828	7 Silty Sand to Sandy Silt	69.416	1.364	3.671	1.18	2.491	2.56	0.59604	0.64	0.122	2.560	0.60	41.38	3.091	127.872	0.27	0.83	1	2.68
63.484	8 Sand to Silty Sand	91.036	1.122	3.711	1.201	2.51	2.34	0.59299	0.63	0.122	2.340	0.59	53.98	2.084	112.489	0.21	0.83	1	2.08
64.14	7 Silty Sand to Sandy Silt	75.069	1.422	3.749	1.221	2.528	2.53	0.59013	0.63	0.121	2.530	0.59	44.30	2.925	129.568	0.28	0.83	1	2.78
64.797	7 Silty Sand to Sandy Silt	64.325	0.878	3.788	1.241	2.547	2.51	0.58714	0.62	0.121	2.510	0.59	37.77	2.820	106.486	0.19	0.83	1	1.90
65.453	8 Sand to Silty Sand	94.869	1.193	3.828	1.262	2.566	2.34	0.58417	0.62	0.120	2.340	0.58	55.42	2.084	115.482	0.22	0.83	1	2.22
66.109	7 Silty Sand to Sandy Silt	108.714	2.911	3.866	1.282	2.584	2.5	0.5814	0.61	0.120	2.500	0.58	63.21	2.768	174.981		0.83	1	
66.765	6 Sandy Silt to Clayey Silt	109.44	3.697	3.904	1.303	2.601	2.56	0.5788	0.61	0.119	2.560	0.58	63.34	3.091	195.766		0.83	1	
67.421	7 Silty Sand to Sandy Silt	117.662	2.896	3.943	1.323	2.619	2.45	0.57607	0.61	0.119	2.450	0.58	67.78	2.528	171.366		0.82	1	
68.077	9 Sand	140.88	1.303	3.983	1.344	2.64	2.13	0.57292	0.60	0.118	2.130	0.57	80.71	1.512	122.017	0.25	0.80	1	2.51
68.734	9 Sand	152.282	1.379	4.024	1.364	2.66	2.09	0.56995	0.60	0.118	2.090	0.57	86.79	1.437	124.715	0.26	0.79	1	2.64

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-COUNTRY CLUB BLVD UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***206127.BR (BR NO. 29 0189)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-CC-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
69.39	9	Sand	172.376	1.109	4.065	1.385	2.68	1.97	0.56701	0.60	0.117	1.970	0.57	97.74	1.263	123.417	0.25	0.78	1	2.59
70.046	9	Sand	192.3	1.142	4.106	1.405	2.7	1.91	0.5641	0.59	0.117	1.910	0.56	108.48	1.198	129.999	0.28	0.76	1	2.90
70.702	9	Sand	178.511	0.796	4.146	1.426	2.721	1.88	0.56108	0.59	0.117	1.880	0.56	100.16	1.171	117.244	0.23	0.77	1	2.35
71.358	9	Sand	123.587	0.621	4.187	1.446	2.741	2.06	0.55823	0.58	0.116	2.060	0.56	68.99	1.387	95.671	0.16	0.81	1	1.66
72.014	8	Sand to Silty Sand	96.666	1.092	4.227	1.467	2.76	2.34	0.55556	0.58	0.116	2.340	0.56	53.70	2.084	111.905	0.21	0.82	1	2.17
72.671	8	Sand to Silty Sand	84.912	1.069	4.266	1.487	2.779	2.41	0.5529	0.58	0.115	2.410	0.55	46.95	2.354	110.499	0.21	0.82	1	2.12
73.327	6	Sandy Silt to Clayey Silt	33.146	0.739	4.304	1.508	2.796	2.94	0.55055	0.58	0.115							0.81	1	
73.983	6	Sandy Silt to Clayey Silt	28.803	0.763	4.342	1.528	2.814	3.04	0.54808	0.57	0.115							0.81	1	
74.639	5	Clayey Silt to Silty Clay	20.31	0.633	4.379	1.549	2.831	3.24	0.54577	0.57	0.114							0.81	1	
75.295	6	Sandy Silt to Clayey Silt	16.182	0.346	4.417	1.569	2.848	3.26	0.54348	0.57	0.114							0.81	1	
75.951	6	Sandy Silt to Clayey Silt	20.701	0.484	4.454	1.589	2.865	3.16	0.54121	0.56	0.114							0.81	1	
76.608	6	Sandy Silt to Clayey Silt	23.383	0.604	4.492	1.61	2.882	3.14	0.53895	0.56	0.114							0.81	1	
77.264	6	Sandy Silt to Clayey Silt	31.332	0.72	4.53	1.63	2.899	2.98	0.53672	0.56	0.113							0.81	1	
77.92	5	Clayey Silt to Silty Clay	16.533	0.363	4.567	1.651	2.916	3.27	0.5345	0.56	0.113							0.81	1	
78.576	6	Sandy Silt to Clayey Silt	21.307	0.417	4.605	1.671	2.933	3.11	0.5323	0.55	0.113							0.81	1	
79.232	6	Sandy Silt to Clayey Silt	25.1	0.632	4.642	1.692	2.95	3.12	0.53012	0.55	0.113							0.81	1	
79.888	6	Sandy Silt to Clayey Silt	16.08	0.343	4.68	1.712	2.968	3.29	0.52783	0.55	0.112							0.80	1	
80.545	6	Sandy Silt to Clayey Silt	15.212	0.257	4.718	1.733	2.985	3.24	0.52569	0.55	0.112							0.80	1	
81.201	6	Sandy Silt to Clayey Silt	19.425	0.407	4.755	1.753	3.002	3.17	0.52356	0.54	0.112							0.80	1	
81.857	6	Sandy Silt to Clayey Silt	27.55	0.705	4.793	1.774	3.019	3.06	0.52145	0.54	0.112							0.80	1	
82.513	6	Sandy Silt to Clayey Silt	26.28	0.71	4.83	1.794	3.036	3.1	0.51936	0.54	0.112							0.80	1	
83.169	6	Sandy Silt to Clayey Silt	22.112	0.508	4.868	1.815	3.053	3.15	0.51728	0.54	0.111							0.80	1	
83.825	6	Sandy Silt to Clayey Silt	20.463	0.445	4.905	1.835	3.07	3.17	0.51522	0.54	0.111							0.80	1	
84.482	6	Sandy Silt to Clayey Silt	18.864	0.341	4.943	1.856	3.087	3.17	0.51318	0.53	0.111							0.80	1	
85.138	6	Sandy Silt to Clayey Silt	18.83	0.315	4.981	1.876	3.105	3.15	0.51103	0.53	0.111							0.80	1	
85.794	6	Sandy Silt to Clayey Silt	18.07	0.274	5.018	1.897	3.122	3.15	0.50902	0.53	0.111							0.80	1	
86.45	6	Sandy Silt to Clayey Silt	18.552	0.305	5.056	1.917	3.139	3.16	0.50703	0.53	0.111							0.80	1	
87.106	6	Sandy Silt to Clayey Silt	19.686	0.312	5.093	1.938	3.156	3.12	0.50505	0.53	0.110							0.79	1	
87.762	6	Sandy Silt to Clayey Silt	21.358	0.393	5.131	1.958	3.173	3.12	0.50309	0.52	0.110							0.79	1	
88.419	6	Sandy Silt to Clayey Silt	21.835	0.384	5.169	1.978	3.19	3.1	0.50114	0.52	0.110							0.79	1	
89.075	6	Sandy Silt to Clayey Silt	21.081	0.312	5.206	1.999	3.207	3.08	0.49921	0.52	0.110							0.79	1	
89.731	6	Sandy Silt to Clayey Silt	20.157	0.297	5.244	2.019	3.224	3.11	0.49729	0.52	0.110							0.79	1	
90.387	6	Sandy Silt to Clayey Silt	19.317	0.264	5.281	2.04	3.242	3.12	0.49527	0.52	0.110							0.79	1	
91.043	6	Sandy Silt to Clayey Silt	21.829	0.503	5.319	2.06	3.259	3.19	0.49338	0.52	0.110							0.79	1	
91.699	6	Sandy Silt to Clayey Silt	20.775	0.336	5.357	2.081	3.276	3.14	0.49151	0.52	0.110							0.79	1	
92.356	6	Sandy Silt to Clayey Silt	17.962	0.205	5.394	2.101	3.293	3.13	0.48965	0.51	0.109							0.79	1	
93.012	6	Sandy Silt to Clayey Silt	21.2	0.272	5.432	2.122	3.31	3.07	0.4878	0.51	0.109							0.79	1	
93.668	6	Sandy Silt to Clayey Silt	23.269	0.324	5.469	2.142	3.327	3.04	0.48597	0.51	0.109							0.79	1	
94.324	6	Sandy Silt to Clayey Silt	22.01	0.396	5.507	2.163	3.344	3.13	0.48415	0.51	0.109							0.79	1	
94.98	6	Sandy Silt to Clayey Silt	25.293	0.425	5.545	2.183	3.361	3.05	0.48235	0.51	0.109							0.78	1	
95.636	6	Sandy Silt to Clayey Silt	26.734	0.462	5.582	2.204	3.379	3.04	0.48045	0.51	0.109							0.78	1	
96.293	6	Sandy Silt to Clayey Silt	31.253	0.602	5.62	2.224	3.396	3	0.47868	0.51	0.109							0.78	1	
96.949	6	Sandy Silt to Clayey Silt	33.01	0.67	5.657	2.245	3.413	2.98	0.47691	0.50	0.109							0.78	1	
97.605	6	Sandy Silt to Clayey Silt	36.758	1.02	5.695	2.265	3.43	3.02	0.47516	0.50	0.109							0.78	1	
98.261	6	Sandy Silt to Clayey Silt	55.486	1.472	5.733	2.286	3.447	2.85	0.47342	0.50	0.108							0.78	1	
98.917	6	Sandy Silt to Clayey Silt	45.036	1.128	5.77	2.306	3.464	2.92	0.4717	0.50	0.108							0.78	1	
99.573	6	Sandy Silt to Clayey Silt	52.129	1.531	5.808	2.326	3.481	2.91	0.46999	0.50	0.108							0.78	1	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected sample of the fine-grained materials. The result was used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The result of the test is presented on Plate B-2, "Plasticity Chart".

Grain Size Classification

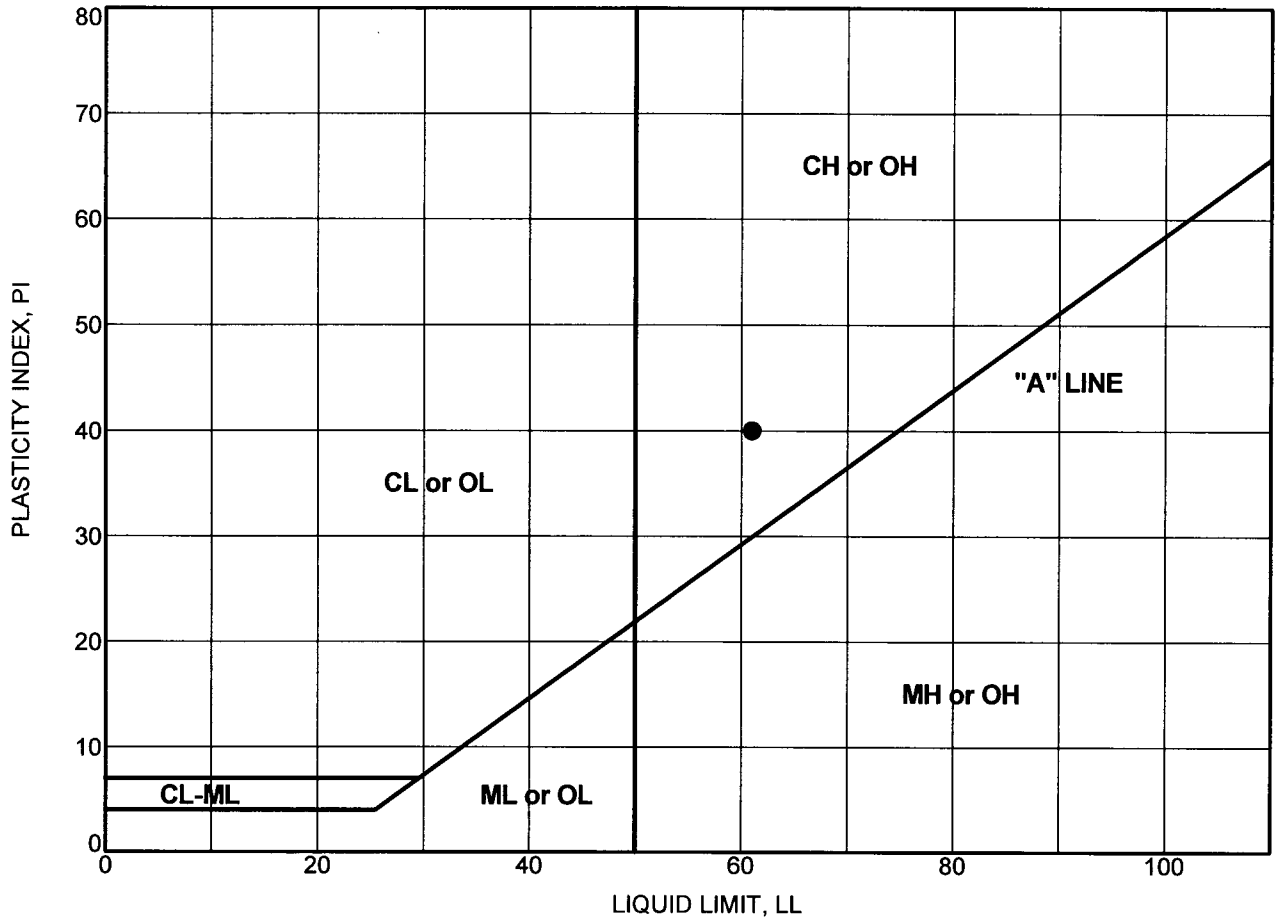
Grain size classification test (ASTM Test Method D 420) was performed on selected sample of granular soil to aid in the classification. The result is presented on Plate B-3, "Grain Size Distribution Curves".

Unconfined Compression Test

Strength tests were performed on selected undisturbed samples using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-5A through B-5C.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-4.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A09-CC-001	MC-5	26.0	●	27	61	21	40	FAT CLAY (CH)

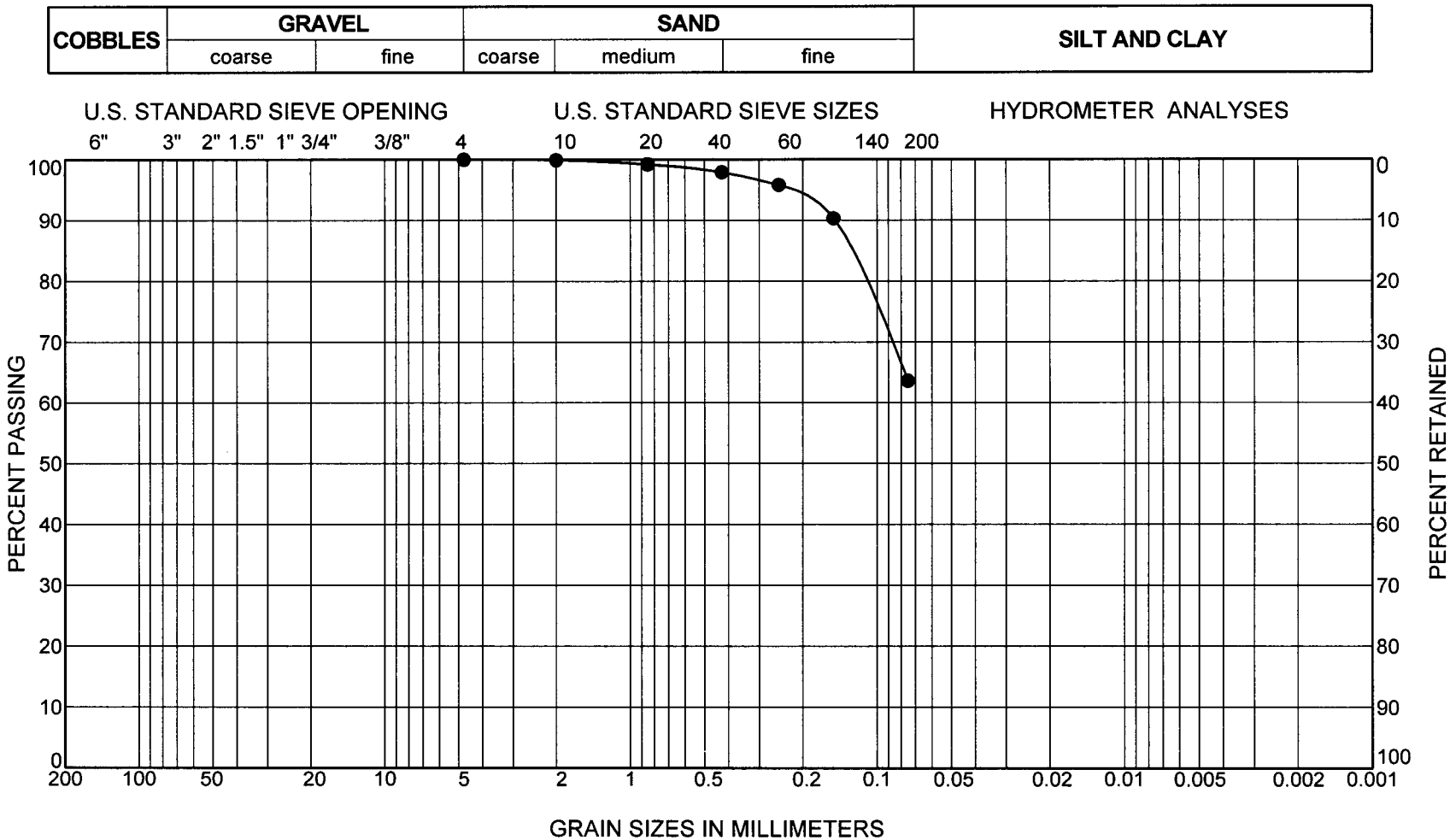
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Country Club Blvd. UC (Widen)
Stockton, California

JOB NO: 206127.BR (CCB)

PLATE NO: B-2

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A09-CC-001	SPT-10	51.0	●			SANDY SILT (ML)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

JOB NO: 206127.BR (CCB)

Country Club Blvd. UC (Widen)
Stockton, California

PLATE NO: B-3

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/29/2009
Date Submitted 04/24/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR Site ID : A-09-CC-001#6@31.0'
Thank you for your business.

* For future reference to this analysis please use SUN # 55554-111851.

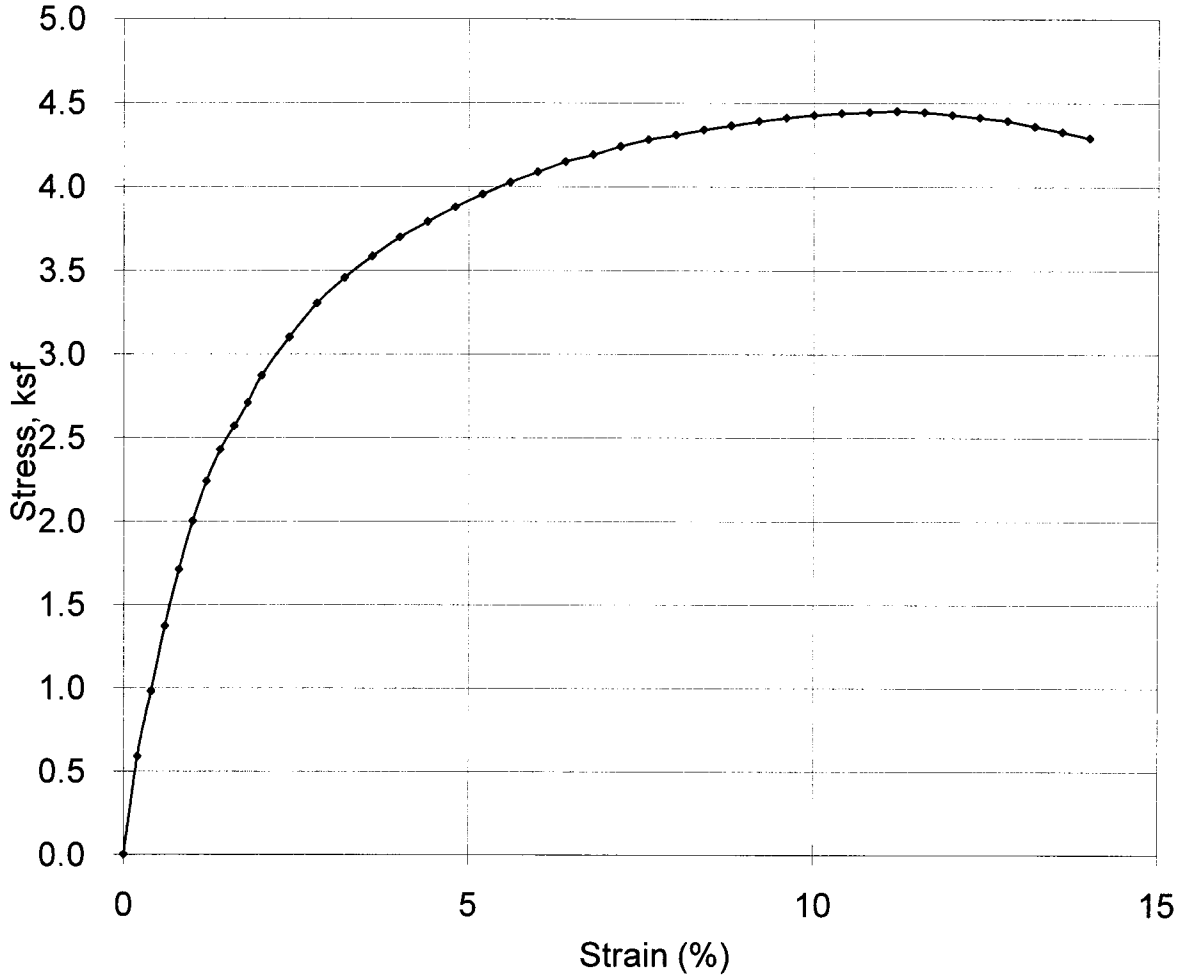
EVALUATION FOR SOIL CORROSION

Soil pH	6.88	
Minimum Resistivity	1.07 ohm-cm (x1000)	
Chloride	32.5 ppm	00.00325 %
Sulfate	45.0 ppm	00.00450 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 1 2009
By

UNCONFINED COMPRESSION TEST



Boring No.: A-09-CC-001

Sample No. : MC-5

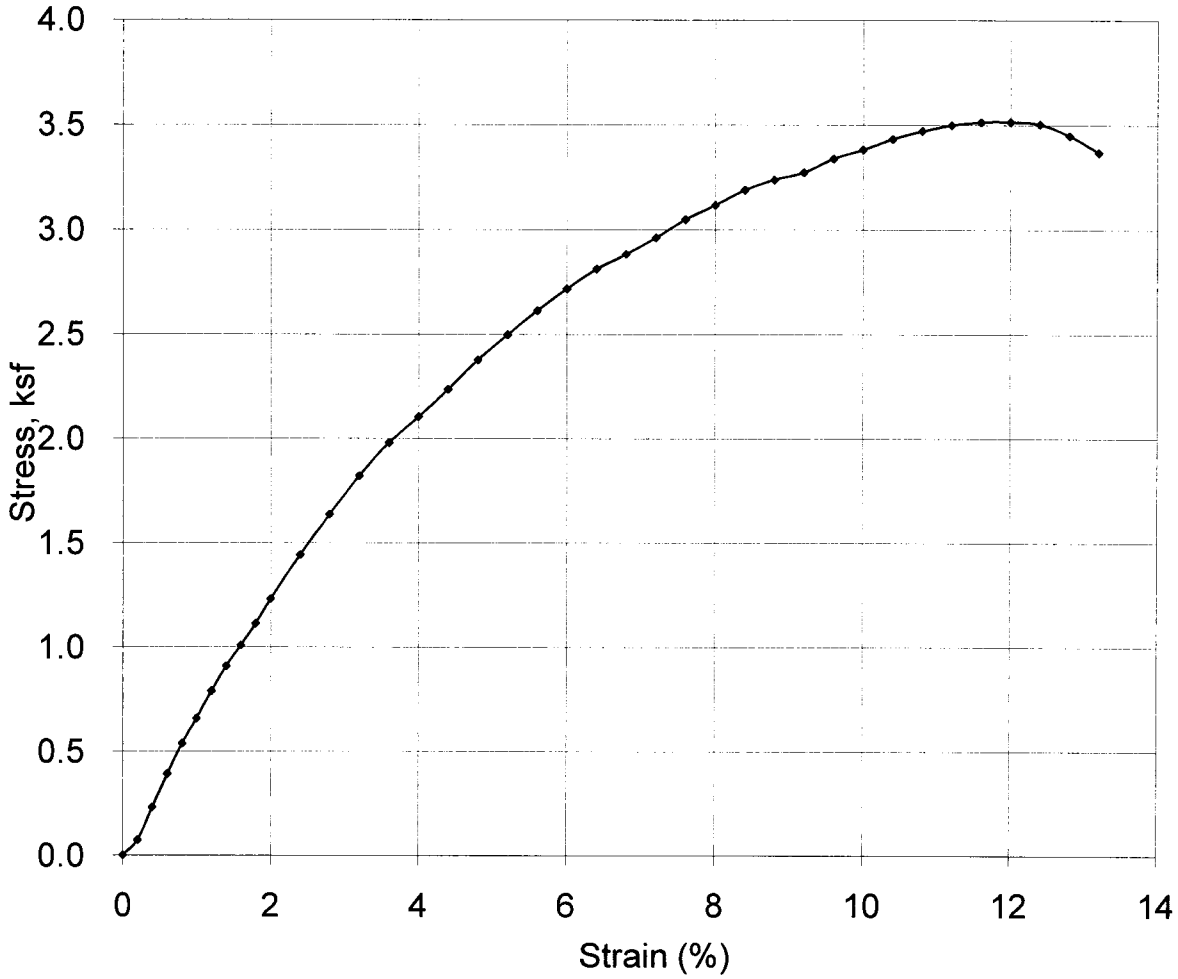
Depth (feet): 26

Material Description:
Fat Clay

Unconfined Compressive Stren 4.4 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**COUNTRY CLUB BLVD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5A

UNCONFINED COMPRESSION TEST



Boring No.: A-09-CC-001

Sample No. : MC-8

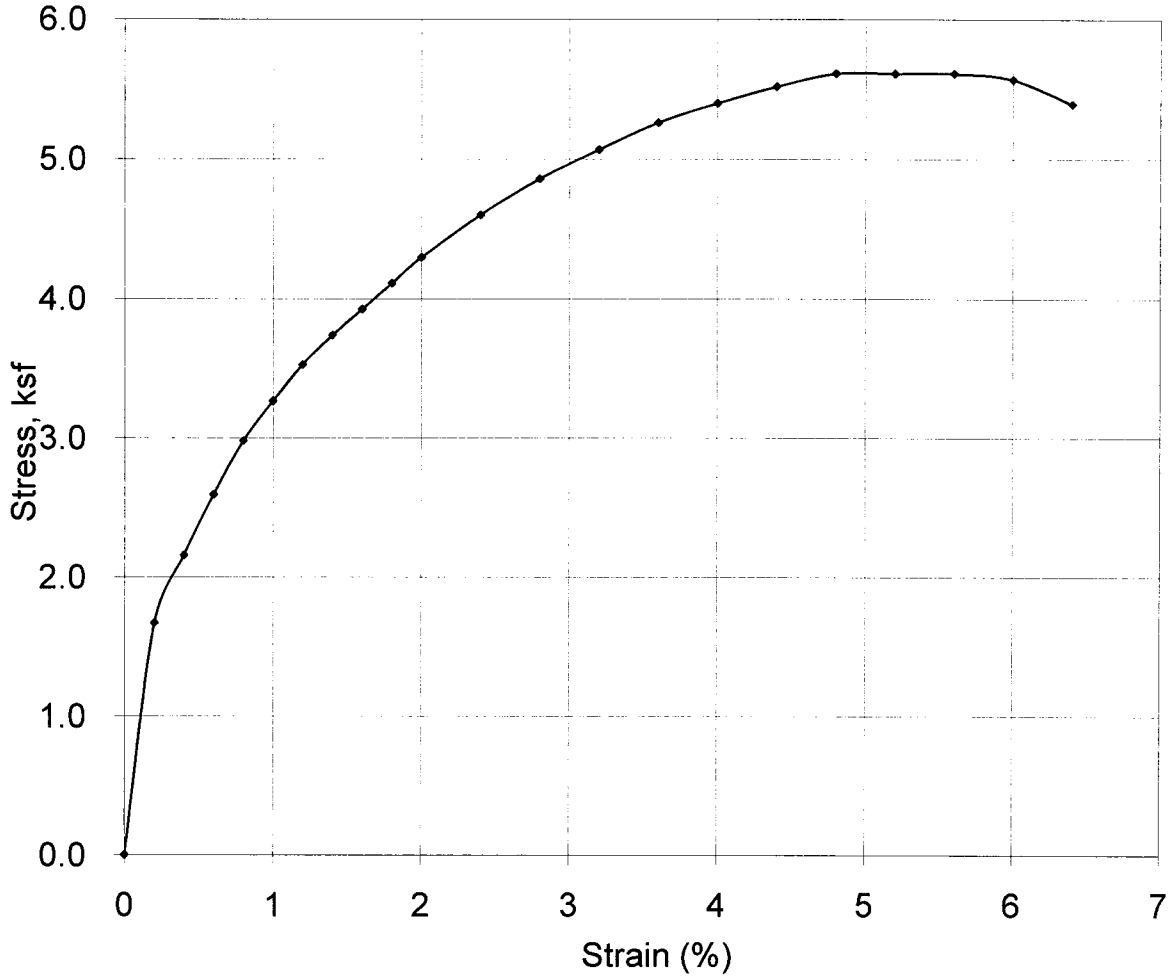
Depth (feet): 41

Material Description:
Lean Clay, Very Stiff

Unconfined Compressive Strength: 3.5 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**COUNTRY CLUB BLVD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5B

UNCONFINED COMPRESSION TEST



Boring No.: A-09-CC-001

Sample No. : MC-14

Depth (feet): 81

Material Description:
Lean Clay

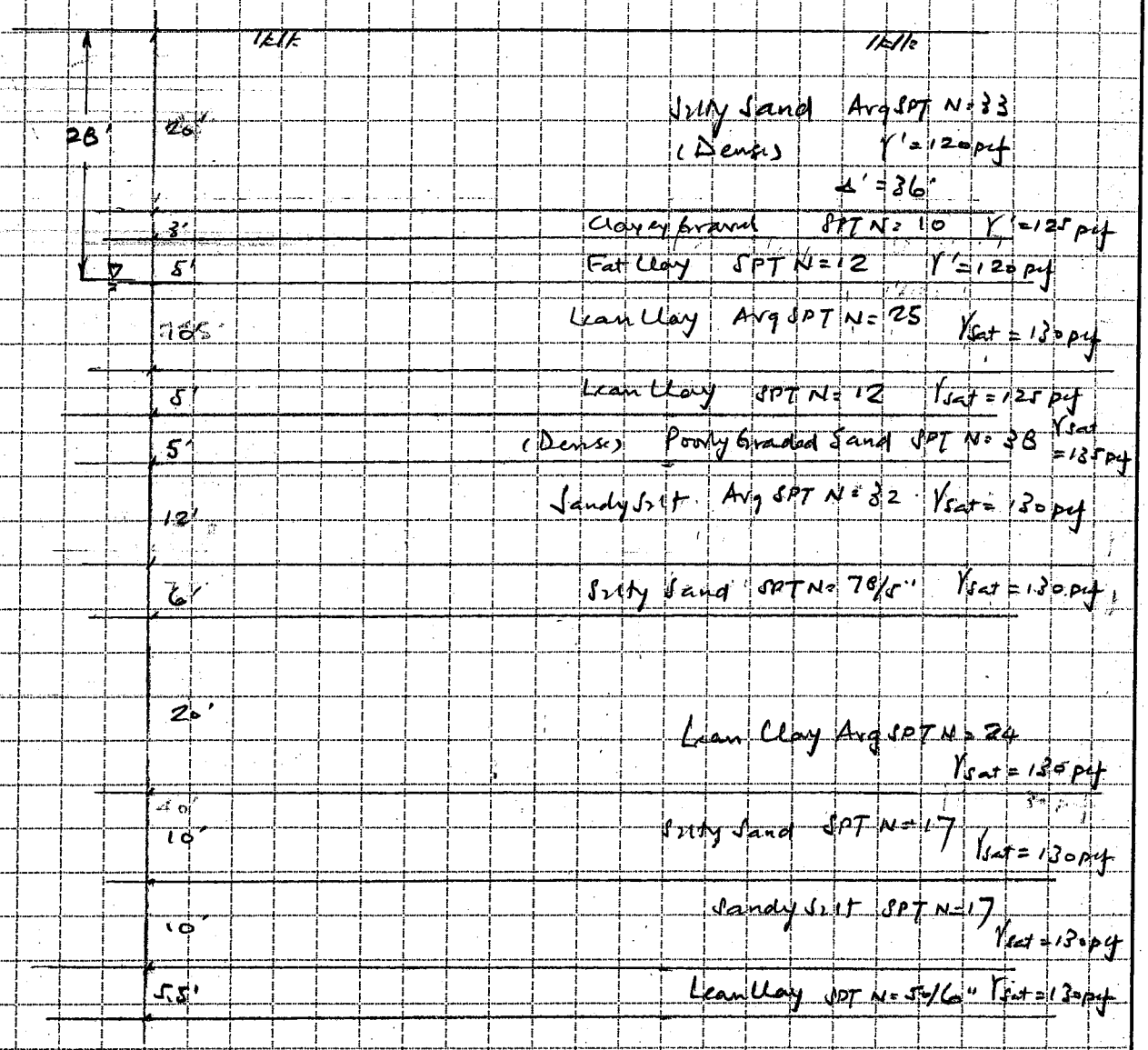
Unconfined Compressive Strength: 5.6 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**COUNTRY CLUB BLVD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5C

APPENDIX C

Evaluation of Liquefaction Potential

Evaluation of Liquefaction Potential - County Club Blvd UC (Boring A-09-001)



LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-001 [Country Club Blvd. UC (Widen)]***

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

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 28

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR$_{7.0}$/CSR)*MSF*Kσ*Kα*

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,cs}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	20.0	1	33	SPT	2400.0	2400.0	0.91	0.12	38	0.91	1	0.95	1.2	1.15	45.57	15%	50.26		0.95	1	
2	23.0	1	10	SPT	2775.0	2775.0	0.89	0.12	12	0.85	1	0.80	1.2	1.15	10.81	15%	13.83	0.15	0.91	1	1.39
3	28.0	2	12	SPT	3375.0	3375.0	0.86		14	0.77	1	0.80	1.2	1.15	11.77	55%			0.85	1	
4	38.0	2	25	SPT	4674.0	4050.0	0.77		29	0.70	1	0.80	1.2	1.15	22.38	55%			0.81	1	
5	43.0	2	12	SPT	5299.0	4362.5	0.72		14	0.68	1	0.80	1.2	1.15	10.35	55%			0.79	1	
6	48.0	1	38	SPT	5974.0	4725.0	0.67	0.11	44	0.65	1	0.80	1.2	1.15	31.49	15%	35.51		0.77	1	
7	60.0	2	32	SPT	7534.0	5535.0	0.58		37	0.60	1	0.80	1.2	1.15	24.50	55%			0.74	1	
8	66.0	1	50	SPT	8314.0	5940.0	0.55	0.10	58	0.58	1	0.80	1.2	1.15	36.96	15%	41.23		0.72	1	
9	86.0	2	24	SPT	10914.0	7290.0	0.49		28	0.52	1	0.80	1.2	1.15	16.01	55%			0.68	1	
10	96.0	1	17	SPT	12214.0	7965.0	0.47	0.09	20	0.50	1	0.80	1.2	1.15	10.85	15%	13.87	0.15	0.66	1	1.24
11	106.0	2	17	SPT	13514.0	8640.0	0.46		20	0.48	1	0.80	1.2	1.15	10.42	55%			0.64	1	
12	111.5	2	50	SPT	14229.0	9011.3	0.46		58	0.47	1	0.80	1.2	1.15	30.01	55%			0.64	1	

Vertical Pile Capacity Calculations

FOUNDATION DESIGN INFORMATION FROM BRIDGE DESIGNER TO GEOTECHNICAL ENGINEER

COUNTRY CLUB BLVD UC (WIDEN)

Table 1. Foundation Design Data Sheet

Location	Design Method	Pile Type	Finish Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	No. of Piles Per Support
					B	L		
Abut 1	WSD	Class 140	16.80	12.80	4.00	30.67	1.00	9
Abut 2	WSD	Class 140	16.80	12.80	4.00	34.50	1.00	9

Note: Finish Grade is at the front face of abutment diaphragm; Pile Cut-off Elevations are the Bottom of Footing Elevations

Table 2. Foundation Design Loads

Location	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent	Compression		Tension		Compression		Tension	
	Per Support	Max per Pile	Per Support	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile
Abut 1	1,215	**135**	885	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1,210	**135**	880	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

PARIKH

Practicing in the Geosciences

JOB I-5 North Stockton Corridor - Country Club Blvd UC Widening 20602-BR

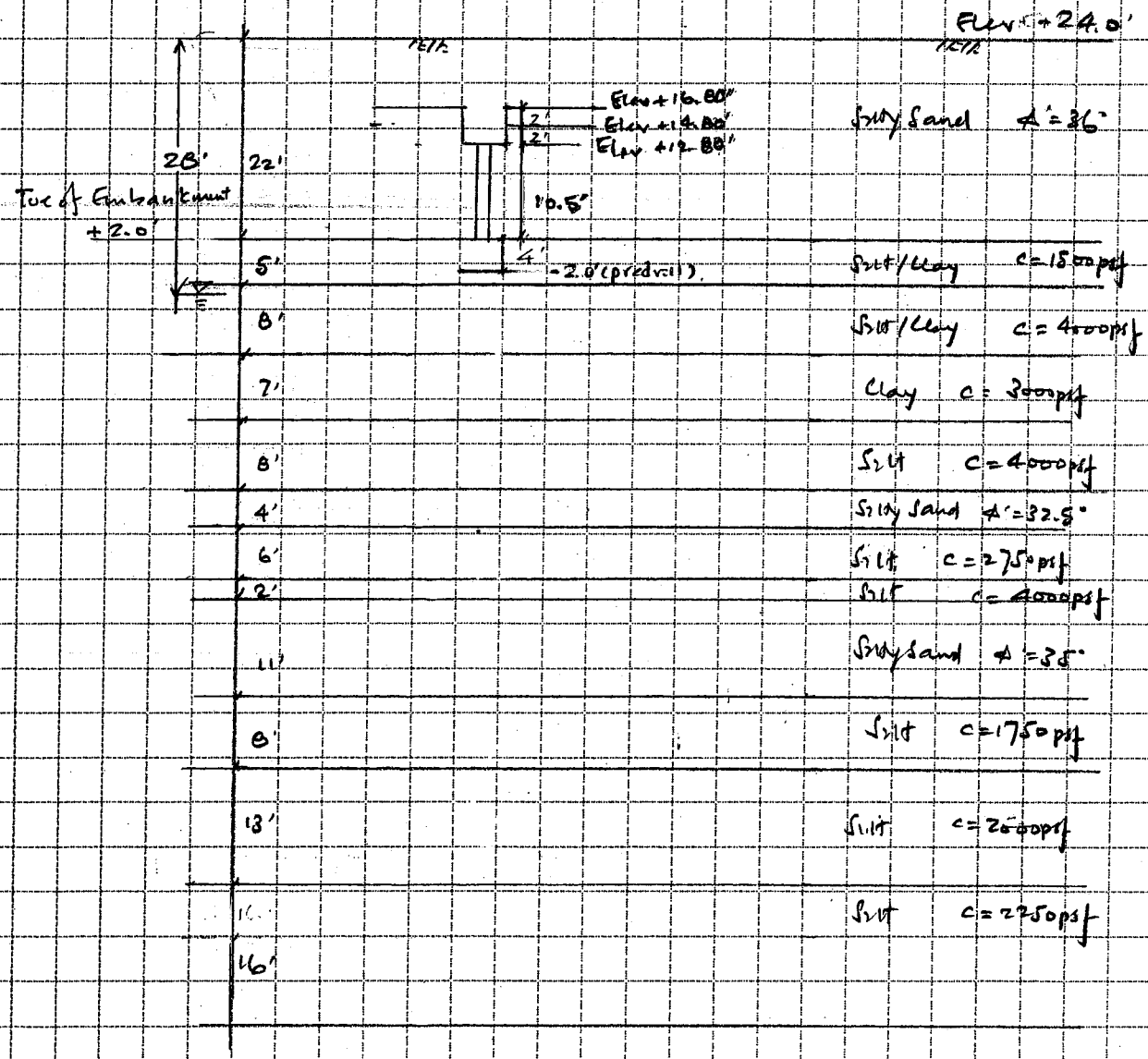
SHEET NO 1 OF 5

CALCULATED BY A Lam DATE 6/2009

CHECKED BY DATE

SCALE

Pile Capacity Analyses - Country Club Blvd UC (CPT-09-001) Abutment 1 - 14" □ precast concrete pile 64.5 tons service load



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Country Club Blvd. UC (Widen)*** ***Abut 1 - 65 tons service load (130 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 12/02/09

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS INCREAMENTS

1- GRAVELS, SANDS (FT)= 5

AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : CPT-09-CC-001 (PARIKH 2009)

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 140 Alt. "X" - 14" square precast prestressed

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	BT2 FTG	1	215	430	215	SAND		0		0.00	0.00	0.00	0.00	0.00
2.0	7.0		1	125	625	743	SAND	DENSE	0	36	0.00	11.50	11.50	3.85	3.85
7.0	12.5		1	125	687.5	1399	SAND	MED DEN	0	35	0.00	22.79	34.30	7.64	11.49
12.5	17.5		2	115	575	2030	CLAY	STIFF	1500	0	0.80	13.50	47.80	13.50	24.99
17.5	18.5		2	120	120	2378	CLAY	V.STIFF	4000	0	0.25	2.25	50.05	2.25	27.24
18.5	21.5		2	68	204	2540	CLAY	V.STIFF	4000	0	0.25	6.75	56.80	6.75	33.99
21.5	25.5		2	68	272	2574	CLAY	V.STIFF	4000	0	0.25	9.00	65.80	9.00	42.99
25.5	29.0		2	68	238	3033	CLAY	V.STIFF	3000	0	0.35	8.27	74.07	8.27	51.26
29.0	32.5		2	68	238	3271	CLAY	V.STIFF	3000	0	0.35	8.27	82.33	8.27	59.53
32.5	36.5		2	68	272	3526	CLAY	V.STIFF	4000	0	0.25	9.00	91.33	9.00	68.53
36.5	40.5		2	68	272	3526	CLAY	V.STIFF	4000	0	0.25	9.00	100.33	9.00	77.53
40.5	44.5		1	68	272	4070	SAND	MED DEN	0	33	0.00	15.46	115.80	5.18	82.71
44.5	50.5		2	68	408	4410	CLAY	V.STIFF	2750	0	0.55	20.42	136.22	0.00	82.71
50.5	52.5		2	68	136	4682	CLAY	V.STIFF	4000	0	0.30	5.40	141.62	6.69	89.40
52.5	56.5		1	68	272	4886	SAND	MED DEN	0	35	0.00	16.31	157.92	0.00	89.40
56.5	60.5		1	68	272	5158	SAND	MED DEN	0	34	0.00	16.12	174.05	0.00	89.40
60.5	63.5		1	68	204	4308	SAND	MED DEN	0	34	0.00	11.88	185.93	4.05	93.45
63.5	64.5		2	68	68	4648	CLAY	STIFF	1750	0	0.85	3.35	189.28 ←	3.35	96.80
64.5	71.5		2	68	476	5736	CLAY	STIFF	1750	0	0.85	23.43	212.71	23.43	120.23
71.5	78.0		2	68	442	5787	CLAY	STIFF	1750	0	0.85	24.86	237.57	24.86	145.09
78.0	84.5		2	68	442	6263	CLAY	STIFF	2000	0	0.75	21.94	259.51	24.86	169.95
84.5	100.5		2	68	1088	7028	CLAY	STIFF	2000	0	0.75	60.75	320.26	54.00	223.95

Abutment 1 --> Pile cut-off elevation is +12.80 feet. Assumed finished grade elevation is +14.80 feet

Pre-drill through the exsiting embankment to bottom of pipe. Elevation of bottom of pipe is Elev. -2.0 feet (assumed to the depth of 17.5 feet)

Pile Length @ Abutment 1 = 64.5' - 2.0' = 62.5 '

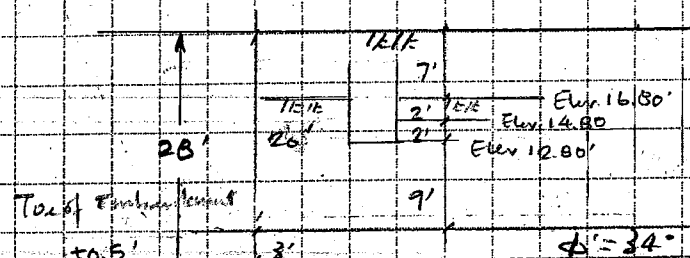
Pile tip at 62.5' (comp) below pile cut-off (pile tip @ Elev. -50.0', pile cut-off @ Elev. +12.8'), Ult. Pile Capacity =189.3 tons (friction)-47.8 tons (Predrilled)=141.5 tons > 270k/135T (Service Limit State), OK

I-5 Country Club Blvd. Abut 1 Class 140 Alt X (12-02-09)

Sheet 2 of 5

Pile Capacity Analyses - County Club Blvd UC (Boring A-09-001) Abutment 2

14" □ precast concrete pile - 64.5 tons service load



Silty Sand Avg SPT N = 33
(Dense) $\gamma' = 120$ pcf
$\phi' = 41°$

Elev 3.5'

$\phi' = 34°$ Clayey Gravel SPT N = 10 $\gamma' = 125$ pcf

$c = 1250$ psf Fat Clay SPT N = 12 $\gamma' = 120$ pcf

$c = 3000$ psf Lean Clay Avg SPT N = 25 $\gamma_{sat} = 130$ pcf

5' $c = 1250$ psf Lean Clay SPT N = 12 $\gamma_{sat} = 125$ pcf

5' $\phi' = 38°$ (Dense) Poorly Graded Sand SPT N = 38 $\gamma_{sat} = 135$ pcf

12' $c = 3500$ psf Sandy Silt Avg SPT N = 32 $\gamma_{sat} = 130$ pcf

6' $\phi' = 39°$ Silty Sand SPT N = 78/5" $\gamma_{sat} = 130$ pcf

20' $c = 2750$ psf Lean Clay Avg SPT N = 24 $\gamma_{sat} = 130$ pcf

10' $\phi' = 34°$ Silty Sand SPT N = 17 $\gamma_{sat} = 130$ pcf

10' $c = 2000$ psf Sandy Silt SPT N = 17 $\gamma_{sat} = 130$ pcf

5.5' Lean Clay SPT N = 50/6" $\gamma_{sat} = 130$ pcf
$c = 4000$ psf

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Country Club Blvd. UC (Widen)*** ***Abut 2 - 65 tons service load (130 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 12/02/09

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS

1- GRAVELS, SANDS AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : A-09-CC-001 (PARIKH 2009)

CLAY, PSILT

INCREAMENTS (FT)= 5

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) : 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 140 Alt. "X" - 14" square precast prestressed c

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	BT2 FTG	1	120	240	120	SAND		0		0.00	0.00	0.00	0.00	0.00
2.0	11.0		1	120	1080	780	SAND	V. DENSE	0	41	0.00	25.15	25.15	8.43	8.43
11.0	14.0		1	125	375	1508	SAND	MED DEN	0	34	0.00	13.04	38.20	4.37	12.80
14.0	16.5		2	120	300	1845	CLAY	STIFF	1250	0	0.85	5.98	44.17	5.98	18.77
16.5	19.0		2	120	300	2145	CLAY	STIFF	1250	0	0.85	5.98	50.15	5.98	24.75
19.0	24.0		2	68	340	2465	CLAY	V.STIFF	3000	0	0.25	8.44	58.59	8.44	33.19
24.0	29.0		2	68	340	2805	CLAY	V.STIFF	3000	0	0.25	8.44	67.02	8.44	41.62
29.0	34.0		2	63	315	3133	CLAY	STIFF	1250	0	1.00	14.06	81.09	14.06	55.69
34.0	39.0		1	73	365	3473	SAND	DENSE	0	38	0.00	30.73	111.82	10.29	65.98
39.0	45.0		2	68	408	3859	CLAY	V.STIFF	3500	0	0.30	14.18	125.99	14.18	80.16
45.0	51.0		2	68	408	4267	CLAY	V.STIFF	3500	0	0.30	14.18	140.17	14.18	94.33
51.0	57.0		1	68	408	4675	SAND	V. DENSE	0	39	0.00	37.56	177.72	12.58	106.91
57.0	60.0		2	68	204	4981	CLAY	V.STIFF	2750	0	0.55	10.21	187.93 ←	10.21	117.12
60.0	67.0		2	68	476	5321	CLAY	V.STIFF	2750	0	0.55	23.82	211.76	23.82	140.94
67.0	72.0		2	68	340	5729	CLAY	V.STIFF	2750	0	0.55	17.02	228.77	17.02	157.96
72.0	77.0		2	68	340	6069	CLAY	V.STIFF	2750	0	0.55	17.02	245.79	17.02	174.98
77.0	82.0		1	68	340	6409	SAND	MED DEN	0	34	0.00	21.20	266.98	7.10	182.08
82.0	87.0		1	68	340	6749	SAND	MED DEN	0	34	0.00	21.11	288.09	7.07	189.15
87.0	92.0		2	68	340	7089	CLAY	STIFF	2000	0	0.85	19.13	307.22	19.13	208.27
92.0	97.0		2	68	340	7429	CLAY	STIFF	2000	0	0.85	19.13	326.34	19.13	227.40
97.0	102.0		2	68	340	7769	CLAY	V.STIFF	4000	0	0.30	13.50	339.84	13.50	240.90

Abutment 2 --> Pile cut-off elevation is +12.80 feet. Assumed finished grade elevation is +14.80 feet

Pre-drill through the exsiting embankment to bottom of pipe. Elevation of bottom of pipe is Elev. -2.0 feet (assumed to the depth of 16.5 feet)

Pile Length @ Abutment 2 = 60.0' - 2.0' = 58 '

Pile tip at 58' below pile cut-off (pile tip @ Elev. -45.5', pile cut-off @ Elev. +12.8'), Ult. Pile Capacity =187.9 tons (friction)-44.2 tons (Predrilled)=143.7 tons > 270k/135T (Service Limit State), OK

I-5 Country Club Blvd. Abut 2 Class 140 Alt X (12-02-09)

Sheet 4 of 5

COUNTRY CLUB BLVD. UC (WIDEN)
(Sheet 5 of 5)

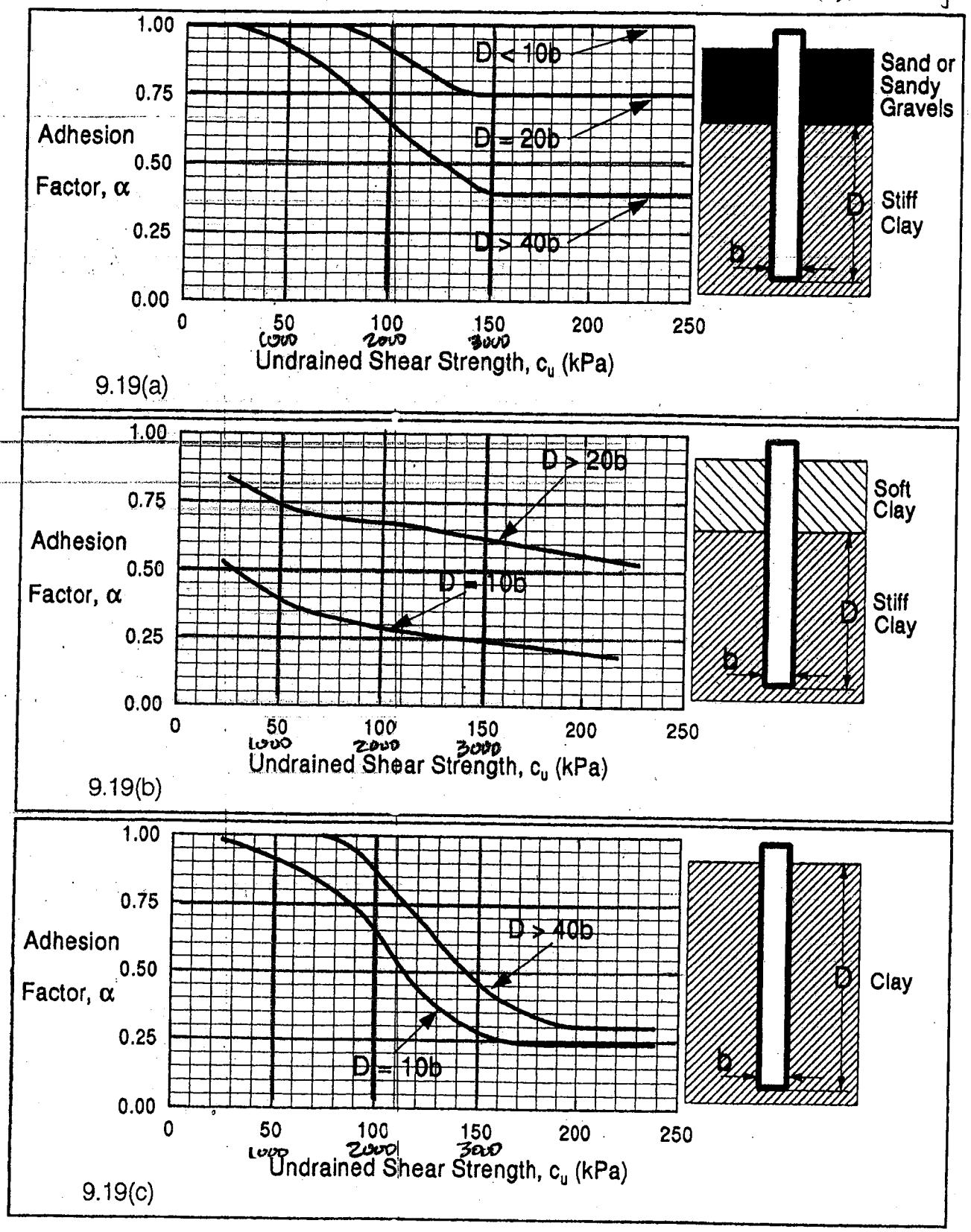


Figure 9.19 Adhesion Factors for Driven Piles in Clay (after Tomlinson, 1980)

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Country Club Blvd. UC (Widen)

CPT-09-CC-001 (Abutment 1) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
24.0* to 2.0	0 to 22	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	225	-	0.069	-
2.0 to -3.0	22 to 27	Clay (firm to stiff)	ϕ = 0°; c=1500 psf	3, Clay	500	0.007	0.067	-
-3.0 to -4.0	27 to 28	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.069	-
-4.0 to -11.0	28 to 35	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 28 feet deep below existing ground surface for design.
-11.0 to -18.0	35 to 42	Clay (very stiff)	ϕ = 0°; c=3000 psf	3, Clay	1000	0.005	0.039	-
-18.0 to -26.0	42 to 50	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-26.0 to -30.0	50 to 54	Sand (medium dense)	ϕ = 32.5°; c= 0 psf	4, Sand	60	-	0.039	-
-30.0 to -36.0	54 to 60	Clay (very stiff)	ϕ = 0°; c=2750 psf	3, Clay	1000	0.005	0.039	-
-36.0 to -38.0	60 to 62	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-38.0 to -49.0	62 to 73	Sand (dense)	ϕ = 35°; c= 0 psf	4, Sand	125	-	0.042	-
-49.0 to -57.0	73 to 81	Clay (very stiff)	ϕ = 0°; c=1750 psf	3, Clay	500	0.007	0.039	-
-57.0 to -70.0	81 to 94	Clay (very stiff)	ϕ = 0°; c=2000 psf	3, Clay	500	0.007	0.039	-
-70.0 to -86.0	94 to 110	Clay (very stiff)	ϕ = 0°; c=2250 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +16.80 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Country Club Blvd. UC (Widen)

Boring A-09-CC-001 (Abutment 2)(14-inch square Alt. "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
23.5* to 3.5	0 to 20	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.069	-
3.5 to 0.5	20 to 23	Sand (loose)	$\phi = 31°$; c= 0 psf	4, Sand	25	-	0.069	-
0.5 to -4.5	23 to 28	Clay (firm to stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.069	-
-4.5 to -14.5	28 to 38	Clay (very stiff)	$\phi = 0°$; c=3000 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 28 feet deep below existing ground surface for design.
-14.5 to -19.5	38 to 43	Clay (firm to stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.036	-
-19.5 to -24.5	43 to 48	Sand (dense)	$\phi = 35°$; c= 0 psf	4, Sand	125	-	0.042	-
-24.5 to -36.5	48 to 60	Clay (very stiff)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-36.5 to -42.5	60 to 66	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-
-42.5 to -62.5	66 to 86	Clay (very stiff)	$\phi = 0°$; c=2750 psf	3, Clay	1000	0.005	0.039	-
-62.5to -72.5	86 to 96	Sand (medium dense)	$\phi = 30.5°$; c= 0 psf	4, Sand	60	-	0.039	-
-72.5 to -82.5	96 to 106	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.039	-
-82.5 to -88	106 to 111.5	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +16.80 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__) __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection __ 65% PS&E Unchecked Details _X PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 7/8/2010 **Structure Name*:** Country Club Blvd UC_ **Br No*:** 29-0189 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	_____	_____	_____	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 4th review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated June 11, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___) __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** Country Club Blvd UC_ **Br No*:** 29-0189 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	6/04/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	9	Please provide the Nominal Driving Resistance in Table 5.	Comment is incorporated. "Nominal Driving Resistance" is provided in Table 5.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection _X 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Country Club Blvd UC **Br No*:** 29-0189 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 EA: 0G4701 Project Name: I-5 North Stockton Widening OSFP Liaison: John Fujimoto Phone: 916-227-8757 e-mail: john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	Reviewer Name: Brandon Badeker Functional Unit: Geotechnical Services Cost Center: 59-323 Phone Number: 916-227-1046 e-mail: brandon_badeker@dot.ca.gov Date of Review: 12/16/2009 Structure Name*: Country Club Blvd UC Br No*: 29-0189 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
3	FR	8	Tables 2 and 3, Although the pile tip appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	
4	FR	9	Table 4: Pile data table shows two different design tip elevations for the two abutments. It is recommended to keep the same pile length for ease of pile fabrication.	Comment is incorporated. Same pile length is used in the design of specified pile tip elevation.	
5	FR	LOTB	As is stated in the Soil Legend, the sample number or	Comment is incorporated.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

			unconfined compressive strength is generally not located to the left of the graphic on the LOTB.		
6	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #3 above.	
7	FR	NA	C2 – Approved subject to OSFP Verification	NA	

FOUNDATION REPORT
MICHIGAN AVE. UC (WIDEN)
(BRIDGE NO. 29-0190)
STOCKTON, CALIFORNIA
10-SJ-05, PM 28.75 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
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June 11, 2010

Job No. 206127.BR

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APPENDIX B

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Evaluation of Liquefaction Potential
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APPENDIX D

Office of Special Funded Projects Comment & Response Form (with PARIKH Consultants, Inc. Responses)

FOUNDATION REPORT
MICHIGAN AVENUE UC (WIDEN)
(BRIDGE NO. 29-0190)
STOCKTON, CALIFORNIA
10-SJ-05 PM 28.75 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Michigan Ave. Undercrossing (Widen) (Bridge No. 29-0190)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Michigan Ave. Undercrossing (UC) (Widen) (Bridge No. 29-0190).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter

unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are single-span structures. Based on the as-built general plan and foundation plan of the Michigan Avenue UC, the structures were constructed in 1970. The existing left and right bridges are approximately 100 feet long. The left bridge (Bridge No. 29-0190L) and the right bridge (Bridge No. 29-0190R) is 53 feet wide. The left and right bridges have not been widened since their construction in 1970.

The abutments of the existing left and right bridge are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class I 45 tons) based on the as-built data. The specified tip elevation is -60 feet (NAVD 29).

Proposed Construction

The widening of the Michigan Avenue UC is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. There will be no outside widening for the Michigan Avenue UC.

According to Rajappan & Meyer Consulting Engineers Inc., the proposed Michigan Avenue UC (Widen) will match the existing span with supports at Abutments 1 and 2. The widen structure will be a cast-in-place prestressed concrete box girder with approximate length of 100 feet measured along the "I-5" Line. The width of the inside widening will be 54 feet. The abutments will incorporate diaphragm type abutments.

Based on our discussions with the designer, the proposed bridge abutments are to be supported on Caltrans Standard Precast Prestressed Class 140 Alternate "X" (Mod) (14-inch square) piles.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The Michigan Ave. UC is located at the intersection of Interstate 5 (I-5) and Michigan Ave. It is approximately 0.3 miles southeast of the Michigan Ave. UC. The Project Location Map, Plate 1, shows the location of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the Michigan Avenue UC (February 1964) (Ref. 1) have been used as a reference. Based on the as-built LOTB, Borings B-2 and B-4 were drilled and Cone Penetration Test (CPT) at B-1 and B-3 were performed by Caltrans in 1964. The soil borings were drilled to the approximate depths between 77 feet and 82 feet and to the bottom elevation between approximately -78 feet and -82 feet. The CPTs were pushed to the approximate depths between 52.5 feet and 103 feet and to the bottom elevation between approximately -53.0 feet and -103 feet.

Field Exploration in 2009

Additional field exploration was conducted in March and April 2009. Boring A-09-MA-001 was drilled on the south side and CPT-09-MA-001 was performed on the north side of Michigan Avenue UC. The soil boring was drilled and the CPT was pushed to the depth of approximately 110 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit weight, and Plasticity Index of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built Borings B-2 and B-4, the subsurface soils generally consist of soft peaty clay, underlain by stiff to hard silt/silty clay, underlain by interbedded layers of soft to very stiff clays/medium dense sands, underlain by medium dense to dense sands.

Based on Boring A-09-MA-001 and CPT-09-MA-001, the subsurface soils generally consist of dense silty sand (embankment fill), underlain by stiff to very stiff lean clay with intermittent layers of medium dense to dense silty sands.

Groundwater was recorded at approximate depths between 13 feet (Elevation -13 feet) and 14 feet (Elevation -14 feet) in the CPTs in February 1964. No groundwater was recorded in the soil borings in February 1964. Groundwater was measured at Elevation -11 feet (the depth of 32 feet below existing ground surface) during drilling for A-09-MA-001 in March 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of dense silty sand, underlain by stiff to very stiff lean clays with intermittent layers of medium dense to dense silty sand. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground

shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring A-09-MA-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-MA-001 included in this report, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the

project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure, different foundation systems have been evaluated with respect to their design capacity and construction considerations. Caltrans standard Class 140 Alternative "X" (Mod) Piles are recommended to be the foundation system for the abutments of the proposed bridge structure. These piles may be designed for service load of 70 tons at the abutments to the indicated pile tip elevations. Pertinent foundation design information provided by Biggs Cardosa Associates (BCA), including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2: FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 140 Alt X (Mod)	13.95	10.95	2.5	48.0	1.00	10
Abut 2	WSD	Class 140 Alt X (Mod)	13.81	10.81	2.5	48.0	1.00	10

TABLE 3: FOUNDATION DESIGN LOADS

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	1,280	127	1,030	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1,280	127	1,030	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads", based upon WSD is presented in Table 4 below.

The evaluation of Load Demands on the piles, based upon WSD is presented in Table 3

above. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Table 4 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	Class 140 Alt X (Mod)	10.95	1,280	1,030	130	260	-45.5 (a)	-50.0
Abut 2	Class 140 Alt X (Mod)	10.81	1,280	1,030	130	260	-50.0 (a)	-50.0

Notes: (i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively. The design tip elevation due to the lateral loads are estimated by BCA and not included in the table above.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

TABLE 5: PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 140 Alt X (Mod)	260	0	-45.5 (a)	-50.0	260
Abut 2	Class 140 Alt X (Mod)	260	0	-50.0 (a)	-50.0	260

Notes
(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression (b) Lateral Load.
(ii) The design tip elevation due to the lateral loads are estimated by BCA and not included in the pile data table above.
(iii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iv) Based on Caltrans review comment, it was recommended to keep the same pile length (specified tip elevation) for "ease of pile fabrication".

The estimation of the driven pile capacity in clay is based on the adhesion factor per FHWA manual (Tomlinson 1980). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 5, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles should not be spaced closer than 3 times the pile diameter measured center-to-center. For piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter, there is no group effect for vertical pile capacity.

It is anticipated that the pile capacity will develop after driving as a result of soil "freeze" and dissipation of excess pore water pressures. The final pile capacity can be verified and the gain of pile capacity after initial driving may be evaluated based on "re-driving" after 24-hour (min.) set up. Caltrans Standard Specifications (Section 49-1.08) may be used as the pile driving acceptance criteria. As an option, Pile Driving Analyzer (PDA) may be used to evaluate the pile capacity of the driven piles when unanticipated conditions arise. Typical applications of the PDA include capacity evaluation (for both during driving and re-driving) and integrity testing for piles that have experienced hard driving.

It is prudent to make the contractor aware of these conditions so that appropriate steps can be taken to comply with the standards and maintain the integrity of the piles.

Predrilling to the bottom of the existing water line is required to minimize the vibration impact on the existing Cal Water Service water lines. The bottom of predrill elevation is -1.0 feet according to Biggs Cardosa Associates. Based on the available boring/CPT information, dense sands were encountered in the embankment fill. It is recommended to follow Caltrans standard specifications (Section 49-1.06) (Ref. 7) for oversize predrilling through the existing embankment to the bottom of predrill elevation described above.

Due to the hard driving condition that may be encountered during pile driving, pile driving should be allowed to terminate short of the specified tip elevation provided the following conditions are satisfied:

- Other requirements including tension and lateral demands are met;
- Pile attaining its capacity and refusal within 5 feet above the specified tip elevation.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. It is our understanding that the structural designer will perform lateral load analyses for the piles using LPILE Program (v. 5). "Generalized Vertical Soil Profile and Strength Parameters" based on A-09-MA-001 and CPT-09-MA-001 are included in Appendix C. To account for group effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D, respectively.

Generally, abutment piles are designed for service loads only, and batter piles are used for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design

if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for new structures. Settlement is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Boring A-09-MA-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
A-09-MA-001	29.5	780	6.8	106.9	49.6

Based on the corrosion test results, the native subsurface soils in Boring A-09-MA-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.8 Construction Observation

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

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Michigan Ave. UC (Widen) Foundation Report (6-2010).rpt

REGISTERED PROFESSIONAL ENGINEER
GARY M. PARIKH
No. 666
EXP. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

REFERENCES

1. Michigan Avenue UC As-built Log of Test Borings (February 1964).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

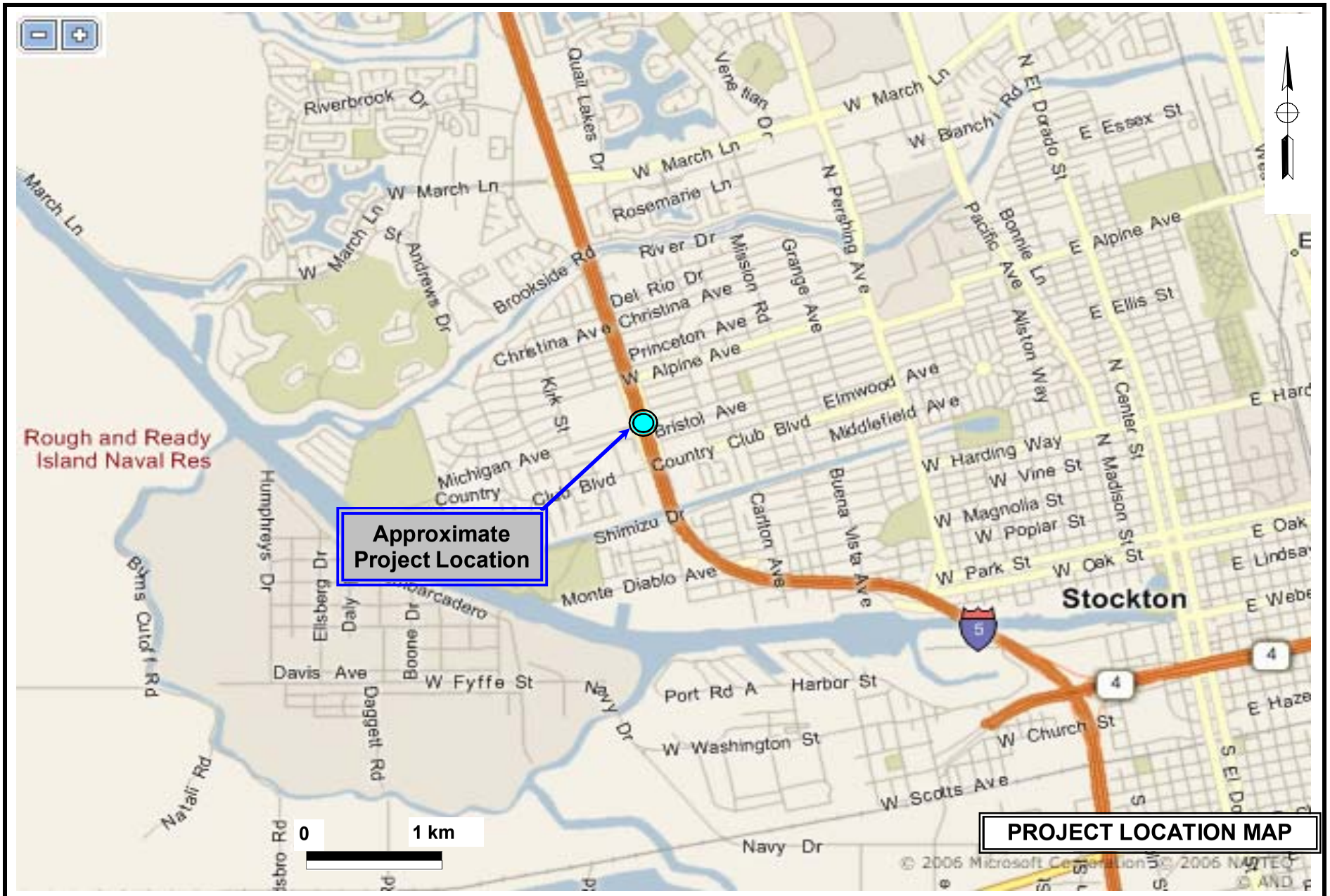
3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

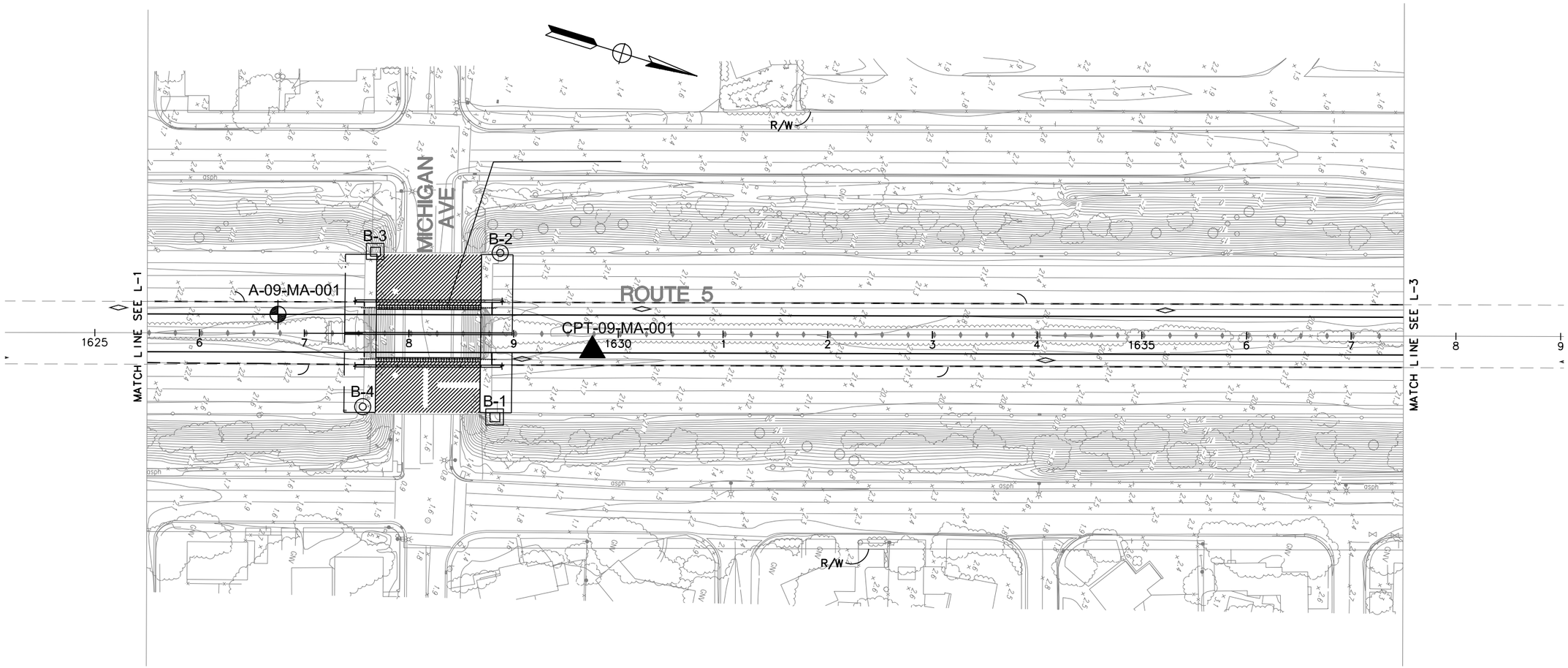
7. State of California Department of Transportation Standard Specifications May 2006.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MICHIGAN AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0190)	PLATE NO.: 1



LEGEND

A-09-MA-001
Approximate Boring Location

B-3
Approximate CPT location performed in 1964

B-4
Approximate Boring location drilled in 1964

CPT-09-MA-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

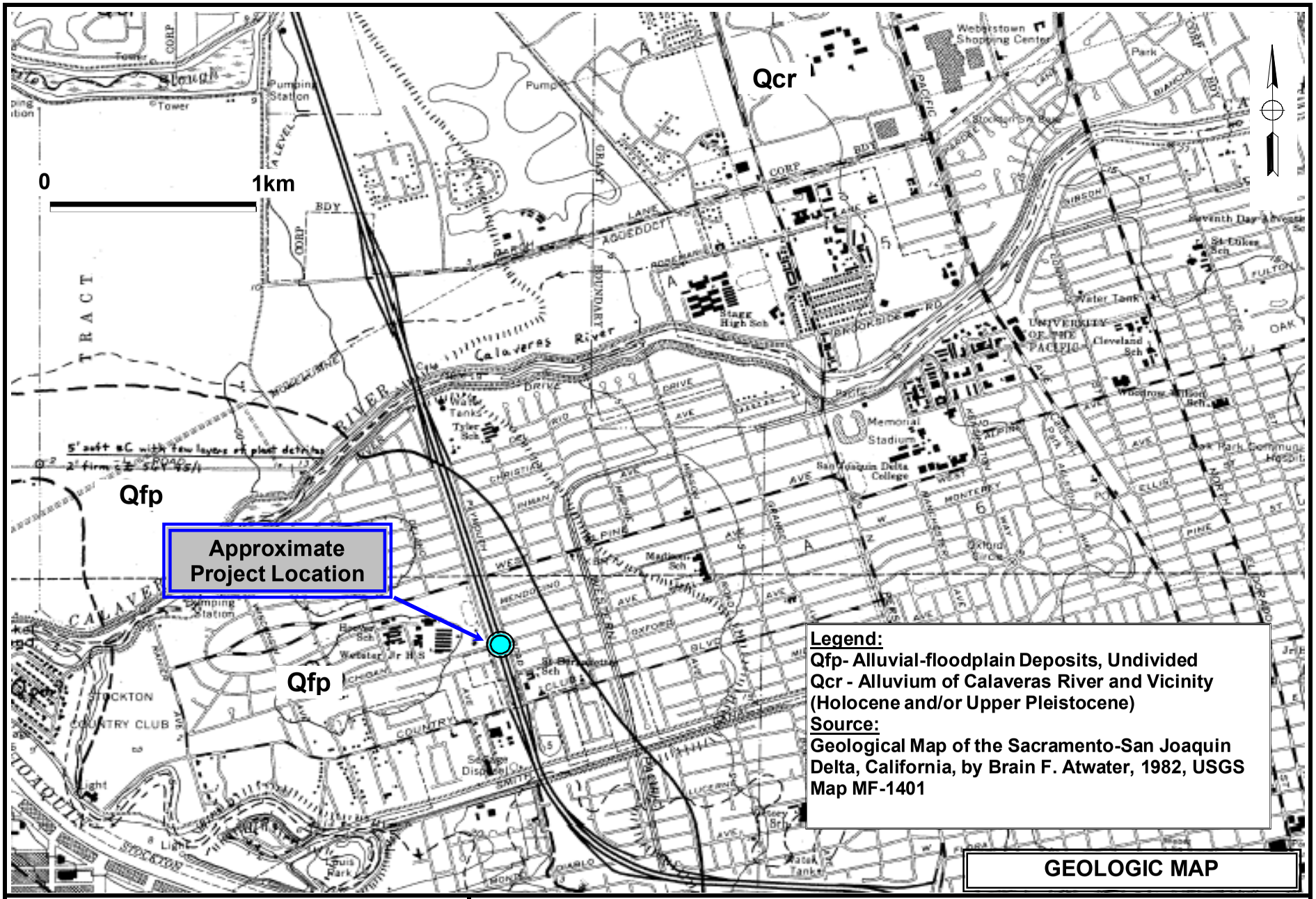
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

MICHIGAN AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

PLATE NO: 2

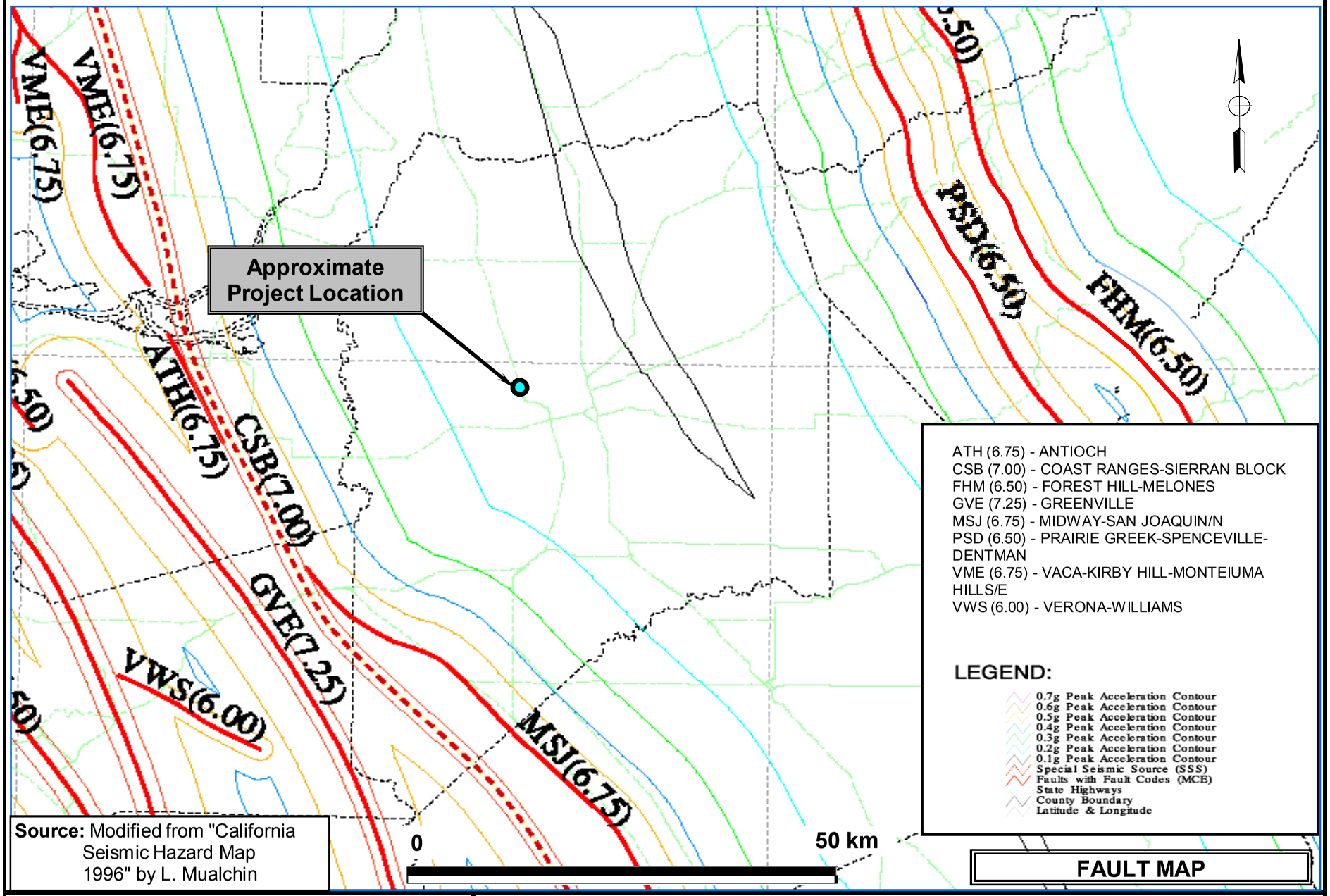


GEOLOGIC MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MICHIGAN AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0190)	PLATE NO.: 3



Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

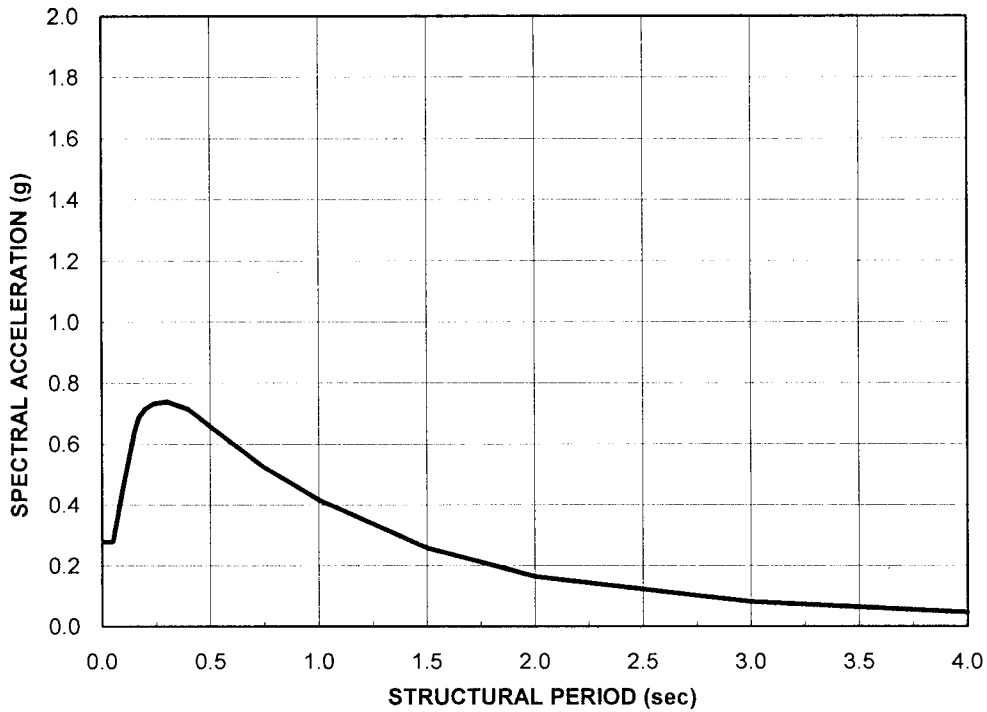
FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MICHIGAN AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0190) | **PLATE NO.: 4**

ARS DESIGN CURVE
MICHIGAN AVENUE UC (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

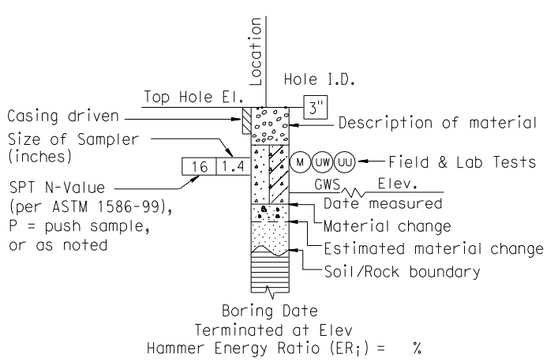
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

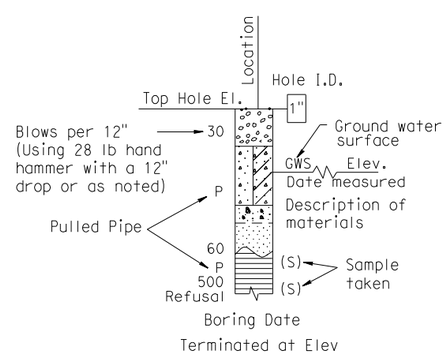
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

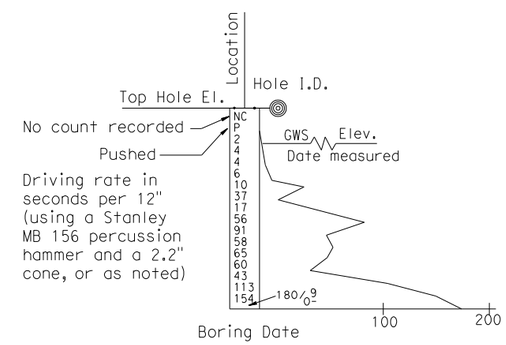
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



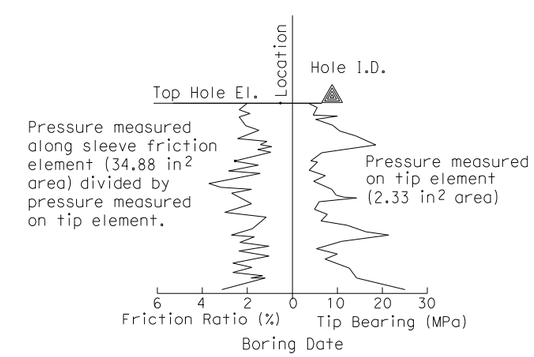
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM	BRIDGE NO. 29-0190
SIGN OFF DATE	CHECKED BY D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER	POST MILES 28.75

MICHIGAN AVENUE UC (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 21 OF 24

...\29-0190-z-ltb01.dgn 6/8/2010 9:50:09 AM

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

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

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)
- (VS) Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0190
POST MILES 28.75

MICHIGAN AVENUE UC (WIDEN)
SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 22 OF 24

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

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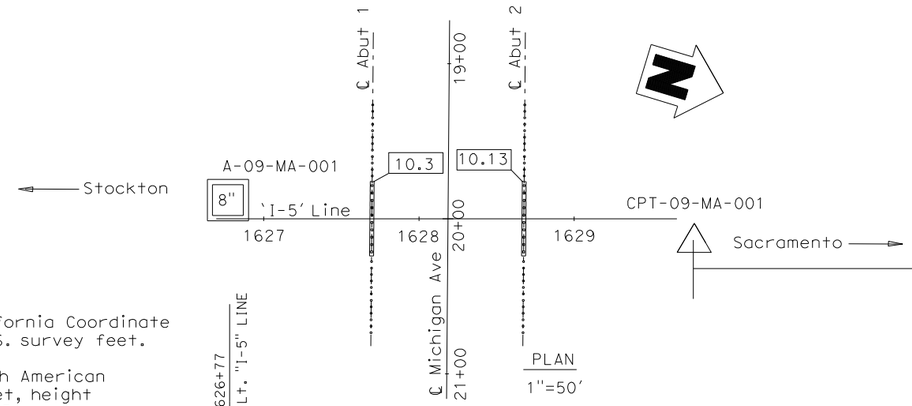
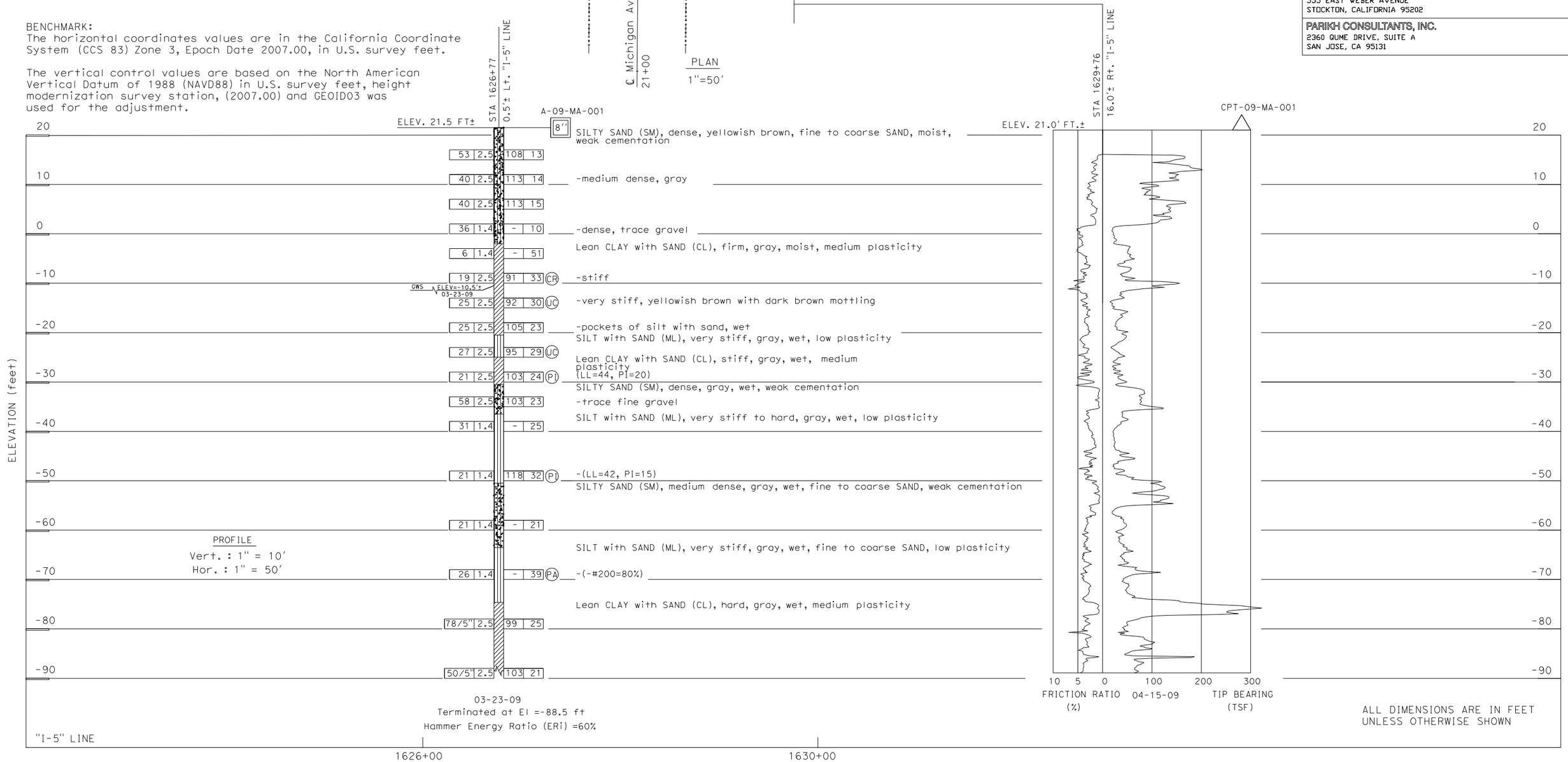
SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131



BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.

The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.



ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER

BRIDGE NO. 29-0190
POST MILES 28.75

MICHIGAN AVENUE UC (WIDEN)
LOG OF TEST BORINGS 1 OF 2

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 23 OF 24

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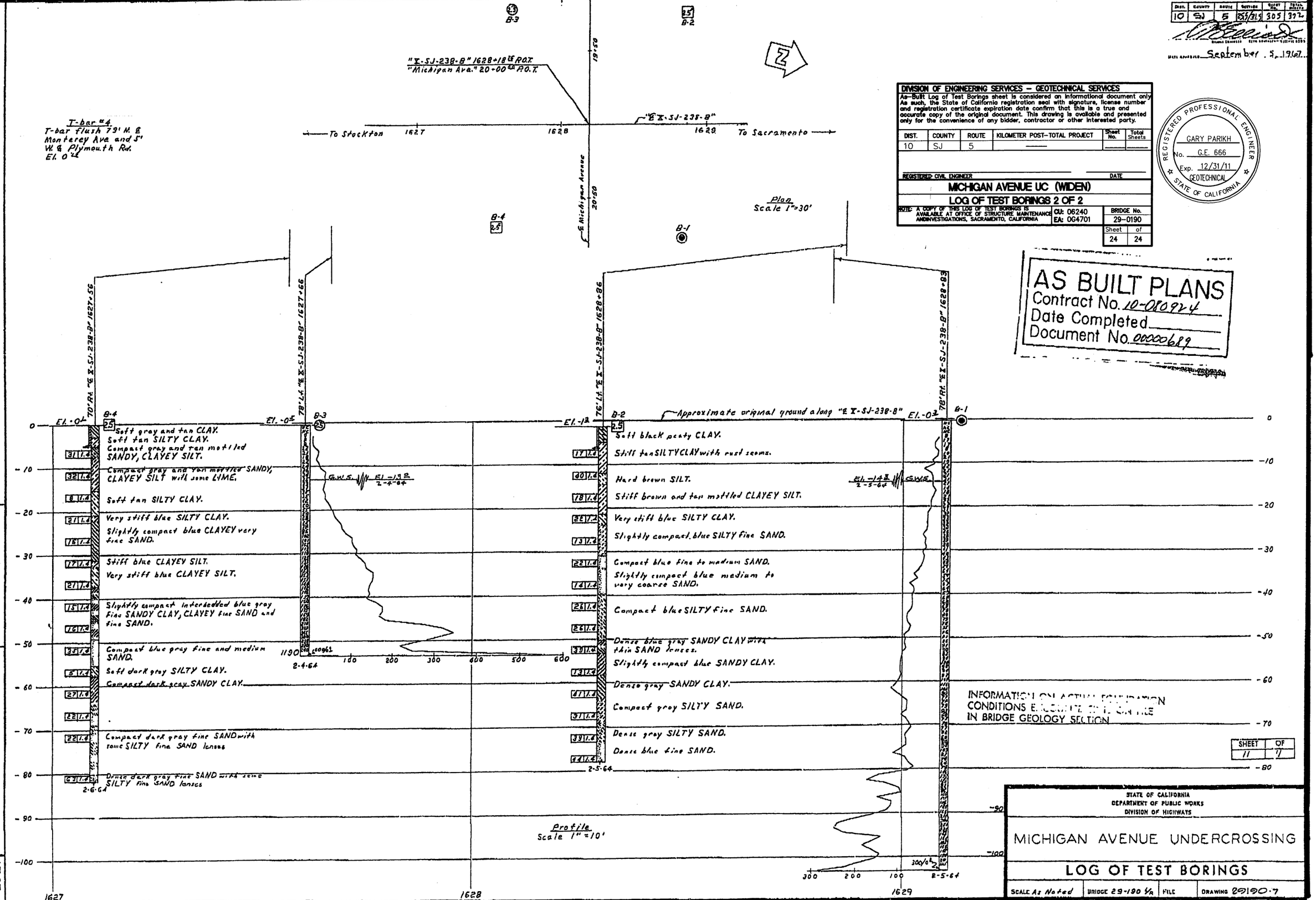
DIST.	COUNTY	ROUTE	SECTION	SHEET NO.	TOTAL SHEETS
10	SJ	5	25/31.5	305	372

DATE APPROVED September 5, 1967

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES					
As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.					
DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5	—	—	—
REGISTERED CIVIL ENGINEER			DATE		
MICHIGAN AVENUE UC (WIDEN)					
LOG OF TEST BORINGS 2 OF 2					
NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA			CU: 06240 EA: 0G4701	BRIDGE No. 29-0190	
				Sheet 24	of 24

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

AS BUILT PLANS
Contract No. 10-080924
Date Completed
Document No. 00000689



SHEET	OF
11	11

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

MICHIGAN AVENUE UNDERCROSSING

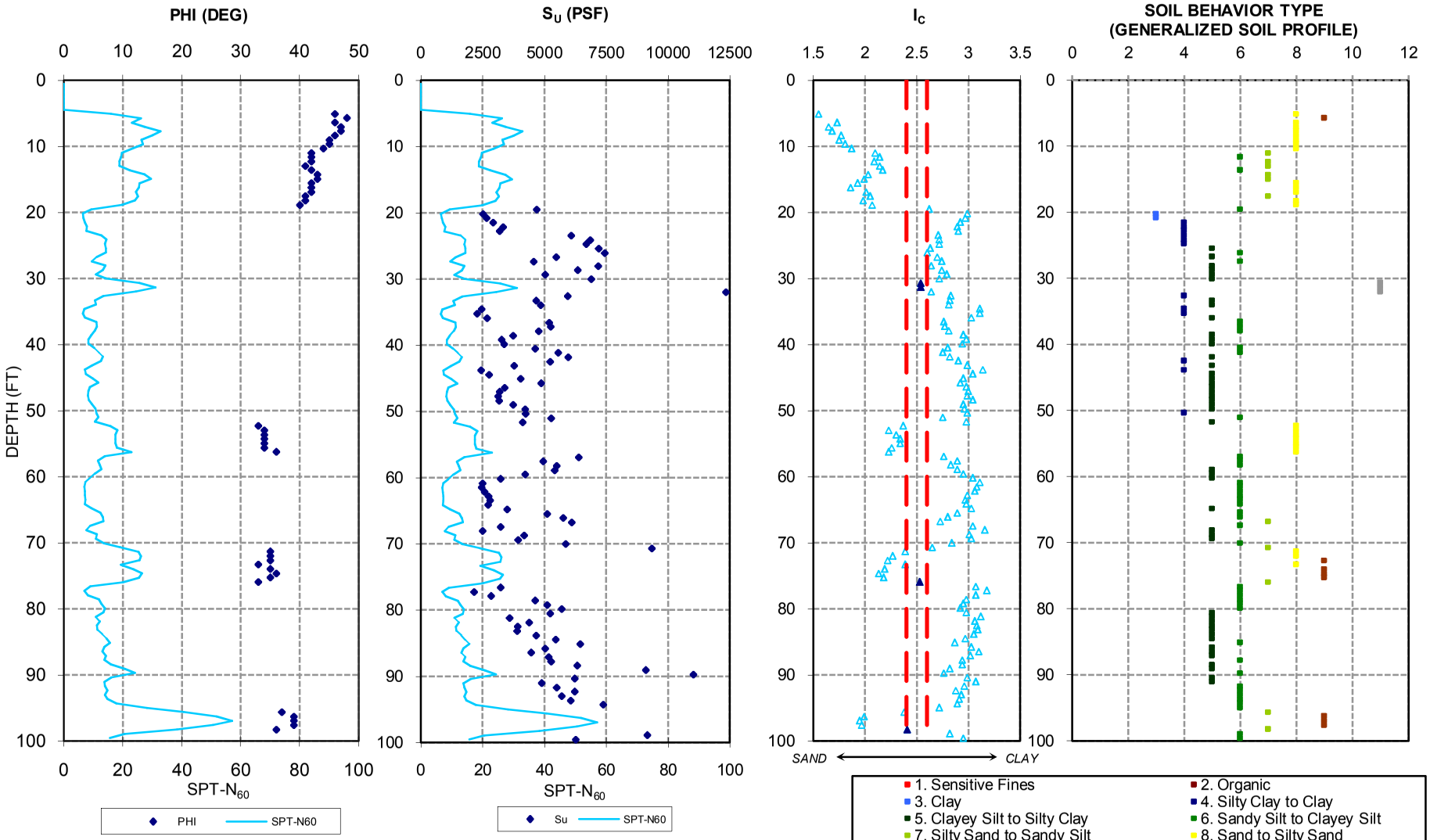
LOG OF TEST BORINGS

SCALE As Noted | BRIDGE 29-190 S/R | FILE | DRAWING 29190-7

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

CPT DATA CORRELATION

CPT-09-MA-002



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

MICHIGAN AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0190)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-MICHIGAN AVENUE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0190)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-MA-002***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
5.085	8	Sand to Silty Sand	94.778	0.905	0.308		0.308	1.55	1.45889	0.99	0.129	1.550	1.46	138.27	1.000	138.270	0.33	1.00	1	
5.741	9	Sand	159.08	1.721	0.348		0.348	1.49	1.42119	0.99	0.129	1.490	1.42	226.08	1.000	226.083		1.00	1	
6.398	8	Sand to Silty Sand	129.534	2.191	0.388		0.388	1.73	1.38539	0.99	0.128	1.730	1.39	179.46	1.058	189.874		1.00	1	
7.054	8	Sand to Silty Sand	157.827	2.267	0.428		0.428	1.65	1.35135	0.99	0.128	1.650	1.35	213.28	1.003	213.991		1.00	1	
7.71	8	Sand to Silty Sand	187.583	3.063	0.467		0.467	1.68	1.31974	0.98	0.128	1.680	1.32	247.56	1.024	253.566		1.00	1	
8.366	8	Sand to Silty Sand	166.474	3.059	0.507		0.507	1.77	1.28881	0.98	0.128	1.770	1.29	214.55	1.085	232.865		1.00	1	
9.022	8	Sand to Silty Sand	146.556	2.256	0.547		0.547	1.75	1.2593	0.98	0.128	1.750	1.26	184.56	1.072	197.772		1.00	1	
9.678	8	Sand to Silty Sand	146.215	2.541	0.586		0.586	1.81	1.2318	0.98	0.127	1.810	1.23	180.11	1.114	200.642		1.00	1	
10.335	8	Sand to Silty Sand	125.356	2.148	0.626		0.626	1.87	1.20482	0.98	0.127	1.870	1.20	151.03	1.162	175.474		1.00	1	
10.991	7	Silty Sand to Sandy Silt	96.11	2.544	0.665		0.665	2.1	1.17962	0.98	0.127	2.100	1.18	113.37	1.455	164.931		1.00	1	
11.647	6	Sandy Silt to Clayey Silt	93.434	2.56	0.702		0.702	2.14	1.15668	0.98	0.127	2.140	1.16	108.07	1.532	165.563		1.00	1	
12.303	7	Silty Sand to Sandy Silt	92.243	2.033	0.741		0.741	2.09	1.13344	0.97	0.127	2.090	1.13	104.55	1.437	150.232	0.40	1.00	1	
12.959	7	Silty Sand to Sandy Silt	90.484	2.217	0.78		0.78	2.14	1.11111	0.97	0.126	2.140	1.11	100.54	1.532	154.020	0.42	1.00	1	
13.615	6	Sandy Silt to Clayey Silt	106.838	3.161	0.817		0.817	2.17	1.09073	0.97	0.126	2.170	1.09	116.53	1.596	186.028		1.00	1	
14.272	7	Silty Sand to Sandy Silt	137.461	3.177	0.856		0.856	2.03	1.07004	0.97	0.126	2.030	1.07	147.09	1.341	197.277		1.00	1	
14.928	7	Silty Sand to Sandy Silt	150.791	3.187	0.894		0.894	1.99	1.05062	0.97	0.126	1.990	1.05	158.42	1.287	203.917		1.00	1	
15.584	8	Sand to Silty Sand	134.014	2.065	0.934		0.934	1.93	1.03093	0.97	0.126	1.930	1.03	138.16	1.218	168.341		1.00	1	
16.24	8	Sand to Silty Sand	136.174	1.65	0.974		0.974	1.86	1.01196	0.97	0.126	1.860	1.01	137.80	1.153	158.934	0.45	1.00	1	
16.896	8	Sand to Silty Sand	123.263	2.114	1.013		1.013	2.01	0.99413	0.96	0.125	2.010	0.99	122.54	1.313	160.928		1.00	1	
17.552	7	Silty Sand to Sandy Silt	125.435	2.38	1.052		1.052	2.05	0.97691	0.96	0.125	2.050	0.98	122.54	1.371	168.007		0.99	1	
18.209	8	Sand to Silty Sand	123.701	1.764	1.092		1.092	1.98	0.95986	0.96	0.125	1.980	0.96	118.74	1.275	151.357	0.40	0.98	1	
18.865	8	Sand to Silty Sand	97.76	1.393	1.131		1.131	2.07	0.9438	0.96	0.125	2.070	0.94	92.27	1.403	129.442	0.28	0.97	1	
19.521	6	Sandy Silt to Clayey Silt	36.179	1.066	1.169		1.169	2.62	0.92866	0.96	0.125							0.97	1	
20.177	3	Clay	20.06	0.993	1.206		1.206	2.99	0.91438	0.96	0.124							0.96	1	
20.833	3	Clay	21.239	1.006	1.242		1.242	2.97	0.9009	0.95	0.124							0.96	1	
21.49	4	Silty Clay to Clay	23.099	1.005	1.28		1.28	2.92	0.8871	0.95	0.124							0.95	1	
22.146	4	Silty Clay to Clay	26.189	1.161	1.317		1.317	2.89	0.87406	0.95	0.124							0.95	1	
22.802	4	Silty Clay to Clay	25.163	1.035	1.355		1.355	2.9	0.86106	0.95	0.123							0.94	1	
23.458	4	Silty Clay to Clay	46.986	2.082	1.393		1.393	2.71	0.84844	0.95	0.123							0.94	1	
24.114	4	Silty Clay to Clay	52.792	2.686	1.43		1.43	2.72	0.8365	0.94	0.123							0.93	1	
24.77	4	Silty Clay to Clay	51.5	2.42	1.468		1.468	2.72	0.82459	0.94	0.123							0.93	1	
25.427	5	Clayey Silt to Silty Clay	55.302	2.047	1.505		1.505	2.63	0.81331	0.94	0.122							0.92	1	
26.083	6	Sandy Silt to Clayey Silt	55.939	1.91	1.543		1.543	2.6	0.80204	0.94	0.122							0.92	1	
26.739	5	Clayey Silt to Silty Clay	41.424	1.405	1.581		1.581	2.7	0.79108	0.94	0.122							0.91	1	
27.395	6	Sandy Silt to Clayey Silt	34.558	1.035	1.618		1.618	2.74	0.7807	0.93	0.121							0.91	1	
28.051	5	Clayey Silt to Silty Clay	53.898	1.938	1.656		1.656	2.64	0.77031	0.93	0.121							0.90	1	
28.707	5	Clayey Silt to Silty Clay	48.444	2.007	1.693		1.693	2.74	0.76046	0.93	0.120							0.90	1	
29.364	5	Clayey Silt to Silty Clay	38.119	1.455	1.731		1.731	2.79	0.7506	0.92	0.120							0.90	1	
30.02	5	Clayey Silt to Silty Clay	52.469	2.128	1.768		1.768	2.72	0.74124	0.92	0.120							0.89	1	
30.676	11	Sitff Fine-Grained	103.288	4.659	1.811	0.021	1.79	2.54	0.73579	0.92	0.121	2.540	0.74	76.00	2.979	226.395		0.89	1	
31.332	11	Sitff Fine-Grained	125.203	6.56	1.854	0.041	1.813	2.54	0.73017	0.91	0.121	2.540	0.73	91.42	2.979	272.336		0.89	1	
31.988	11	Sitff Fine-Grained	93.996	5.11	1.897	0.062	1.835	2.64	0.72488	0.91	0.122							0.89	1	
32.644	4	Silty Clay to Clay	46.346	2.147	1.935	0.082	1.852	2.83	0.72084	0.91	0.123							0.88	1	
33.301	5	Clayey Silt to Silty Clay	36.588	1.243	1.972	0.103	1.869	2.82	0.71685	0.90	0.124							0.88	1	
33.957	5	Clayey Silt to Silty Clay	37.966	1.275	2.01	0.123	1.886	2.81	0.7129	0.90	0.124							0.88	1	
34.613	4	Silty Clay to Clay	20.344	0.819	2.047	0.144	1.904	3.11	0.70876	0.89	0.125							0.88	1	
35.269	4	Silty Clay to Clay	19.056	0.686	2.085	0.164	1.921	3.11	0.7049	0.89	0.125							0.88	1	
35.925	5	Clayey Silt to Silty Clay	21.778	0.713	2.122	0.185	1.938	3.03	0.70108	0.88	0.126							0.88	1	
36.581	6	Sandy Silt to Clayey Silt	39.775	1.192	2.16	0.205	1.955	2.76	0.69731	0.88	0.126							0.87	1	
37.238	6	Sandy Silt to Clayey Silt	40.347	1.263	2.198	0.226	1.972	2.77	0.69357	0.87	0.127							0.87	1	
37.894	6	Sandy Silt to Clayey Silt	37.115	1.158	2.235	0.246	1.989	2.81	0.68987	0.87	0.127							0.87	1	
38.55	5	Clayey Silt to Silty Clay	30.039	1.096	2.273	0.267	2.006	2.95	0.68621	0.86	0.127							0.87	1	
39.206	5	Clayey Silt to Silty Clay	26.359	0.895	2.31	0.287	2.023	2.98	0.68259	0.86	0.127							0.87	1	
39.862	5	Clayey Silt to Silty Clay	27.113	0.81	2.348	0.307	2.041	2.94	0.6788	0.85	0.127							0.87	1	
40.518	6	Sandy Silt to Clayey Silt	36.14	1.007	2.386	0.328	2.058	2.8	0.67526	0.85	0.128							0.87	1	
41.175	6	Sandy Silt to Clayey Silt	43.018	1.254	2.423	0.348	2.075	2.75	0.67176	0.84	0.128							0.86	1	
41.831	5	Clayey Silt to Silty Clay	46.403	1.833	2.461	0.369	2.092	2.82	0.66829	0.83	0.128							0.86	1	
42.487	4	Silty Clay to Clay	41.396	1.844	2.498	0.389	2.109	2.9	0.66485	0.83	0.128							0.86	1	
43.143	5	Clayey Silt to Silty Clay	30.329	1.221	2.536	0.41	2.126	2.99	0.66146	0.82	0.127							0.86	1	
43.799	4	Silty Clay to Clay	20.627	0.77	2.574	0.43	2.143	3.14	0.65809	0.82	0.127							0.86	1	
44.455	5	Clayey Silt to Silty Clay	22.713	0.695	2.611	0.451	2.16	3.04	0.65476	0.81	0.127							0.86	1	
45.112	5	Clayey Silt to Silty Clay	32.069	1.168	2.649	0.471	2.178	2.95	0.65127	0.80	0.127							0.86	1	
45.768	5	Clayey Silt to Silty Clay	38.646	1.583	2.686	0.492	2.195	2.92	0.64801	0.80	0.127							0.85	1	
46.424	5	Clayey Silt to Silty Clay	27.743	0.86	2.724	0.512	2.212	2.98	0.64478	0.79	0.126							0.85	1	
47.08	5	Clayey Silt to Silty Clay	26.065	0.804	2.762	0.533	2.229	3	0.64159	0.78	0.126							0.85	1	
47.736	5	Clayey Silt to Silty Clay	25.804	0.732	2.799	0.553	2.246	2.99	0.63842	0.78	0.126							0.85	1	
48.392	5	Clayey Silt to Silty Clay	26.127	0.891	2.837	0.574	2.263	3.04	0.63529	0.77	0.125							0.85	1	
49.049	5	Clayey Silt to Silty Clay	30.663	0.917	2.874	0.594	2.28	2.95	0.63218	0.76	0.125							0.85	1	
49.705	5	Clayey Silt to Silty Clay	34.326	1.273	2.912	0.615	2.297	2.96	0.62911	0.76	0.125							0.85	1	
50.361	4	Silty Clay to Clay	34.649	1.453	2.95	0.635	2.315	2.99	0.62589	0.75	0.124							0.85	1	
51.017	6	Sandy Silt to Clayey Silt	42.547	0.944	2.987	0.655	2.332	2.75	0.62288	0.74	0.124							0.84	1	
51.673	5	Clayey Silt to Silty Clay	33.713	1.248	3.025	0.676	2.349	2.98	0.61989	0.74	0.123							0.84	1	
52.329	8	Sand to Silty Sand	68.935	0.684	3.064	0.696	2.368	2.37	0.61659	0.73	0.123	2.370	0.62	42.50	2.194	93.257	0.16	0.84	1	1.51
52.986	8	Sand to Silty Sand	84.533	0.615	3.104	0.717	2.387	2.23	0.61333	0.72	0.122	2.230	0.61	51.85	1.743	90.392	0.15	0.84	1	1.45
53.642	8	Sand to Silty Sand	78.795	0.693	3.144	0.737	2.406	2.3	0.61009	0.72	0.122	2.300	0.61	48.07	1.949	93.680	0.16	0.84	1	1.53
54.298	8	Sand to Silty Sand	77.366	0.762	3.183	0.758	2.426	2.34	0.60673	0.71	0.121	2.340	0.61	46.94	2.084	97.812	0.17	0.84	1	1.65
54.954	8	Sand to Silty Sand	77.315	0.762	3.223	0.778	2.445	2.34	0.60357	0.70	0.121	2.340	0.60	46.66	2.084	97.238	0.17	0.84	1	1.64
55.61	8	Sand to Silty Sand	82.469	0.634	3.263	0.799	2.464	2.26	0.60044	0.70	0.120	2.260	0.60	49.52	1.827	90.458	0.15	0.83	1	1.48
56.266	8	Sand to Silty Sand	106.288	1.069	3.303	0.819	2.483	2.23	0.59734	0.69	0.120	2.230	0.60	63.49	1.743	110.692	0.21	0.83	1	2.06
56.923	6	Sandy Silt to Clayey Silt	50.252	1.417	3.34	0.84	2.5	2.76	0.59459	0.69	0.119							0.83	1	
57.579	6	Sandy Silt to Clayey Silt	38.845	1.227	3.378	0.86	2.517	2.89	0.59188	0.68	0.119							0.83	1	
58.235	6	Sandy Silt to Clayey Silt	42.655	1.25	3.415	0.881	2.535	2.83	0.58902	0.67	0.118							0.83	1	
58.891	5	Clayey Silt to Silty Clay	42.706	1.49	3.453	0.901	2.552	2.89	0.58635	0.67	0.118							0.83	1	
59.547	5	Clayey Silt to Silty Clay	33.929	1.064	3.49	0.922	2.569	2.95	0.58371	0.66	0.117							0.83	1	
60.203	5	Clayey Silt to Silty Clay	26.785	0.776	3.528	0.942	2.586	3.04	0.58109	0.66	0.117							0.83	1	
60.86	6	Sandy Silt to Clayey Silt	21.296	0.539	3.566	0.963	2.603	3.11	0.57849	0.65	0.116							0.83	1	
61.516	6	Sandy Silt to Clayey Silt	20.655	0.453	3.603	0.983	2.62	3.08	0.57592	0.65	0.116							0.82	1	
62.172	6	Sandy Silt to Clayey Silt	21.51	0.474	3.641	1.004	2.637	3.06	0.57336	0.64	0.115							0.82	1	
62.828	6	Sandy Silt to Clayey Silt	22.714	0.4	3.678	1.024	2.654	2.99	0.57084	0.64	0.115							0.82	1	
63.484	6	Sandy Silt to Clayey Silt	23.207	0.381	3.716	1.044	2.672	2.97	0.56818	0.63	0.114							0.82	1	
64.14	6	Sandy Silt to Clayey Silt	22.419	0.358	3.754	1.065	2.689	2.98	0.5657	0.63	0.114							0.82	1	
64.797	5	Clayey Silt to Silty Clay	28.701	0.866	3.791	1.085	2.706	3.03	0.56324	0.62	0.114							0.82	1	
65.453	6	Sandy Silt to Clayey Silt	41.668	1.191	3.829	1.106	2.723	2.89	0.5608	0.62	0.113							0.82	1	
66.109	6	Sandy Silt to Clayey Silt	46.715	1.143	3.866	1.126	2.74	2.8	0.55838	0.61	0.113							0.82	1	
66.765	7	Silty Sand to Sandy Silt	49.436	0.954	3.905	1.147	2.758	2.73	0.55584	0.61	0.112							0.82	1	
67.421	6	Sandy Silt to Clayey Silt	27.51	0.714	3.943	1.167	2.775	3.04	0.55346	0.61	0.112							0.82	1	
68.077	5	Clayey Silt to Silty Clay	21.597	0.605	3.98	1.188	2.792	3.16	0.5511	0.60	0.112							0.81	1	
68.734	5	Clayey Silt to Silty Clay	34.62	1.166	4.018	1.208	2.81	3.01	0.54863	0.60	0.111							0.81	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-MICHIGAN AVENUE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0190)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-MA-002***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
69.39	5	Clayey Silt to Silty Clay	33.152	1.086	4.055	1.229	2.827	3.03	0.54631	0.60	0.111							0.81	1	
70.046	6	Sandy Silt to Clayey Silt	46.845	1.312	4.093	1.249	2.844	2.84	0.54402	0.59	0.111							0.81	1	
70.702	7	Silty Sand to Sandy Silt	73.283	1.836	4.132	1.27	2.862	2.65	0.54161	0.59	0.110							0.81	1	
71.358	8	Sand to Silty Sand	109.423	1.75	4.171	1.29	2.881	2.39	0.53908	0.58	0.110	2.390	0.54	58.99	2.272	134.022	0.30	0.81	1	3.29
72.014	8	Sand to Silty Sand	118.852	1.348	4.211	1.311	2.9	2.27	0.53659	0.58	0.110	2.270	0.54	63.77	1.856	118.370	0.23	0.81	1	2.55
72.671	9	Sand	119.629	1.076	4.252	1.331	2.921	2.22	0.53385	0.58	0.109	2.220	0.53	63.86	1.717	109.666	0.20	0.81	1	2.21
73.327	8	Sand to Silty Sand	83.399	0.823	4.291	1.352	2.94	2.39	0.5314	0.58	0.109	2.390	0.53	44.32	2.272	100.692	0.17	0.81	1	1.91
73.983	9	Sand	111.164	0.771	4.332	1.372	2.96	2.19	0.52885	0.57	0.109	2.190	0.53	58.79	1.643	96.568	0.16	0.80	1	1.79
74.639	9	Sand	127.907	0.843	4.373	1.393	2.98	2.13	0.52632	0.57	0.109	2.130	0.53	67.32	1.512	101.770	0.18	0.80	1	1.96
75.295	9	Sand	120.19	0.899	4.414	1.413	3.001	2.18	0.52368	0.57	0.108	2.180	0.52	62.94	1.619	101.913	0.18	0.80	1	1.97
75.951	7	Silty Sand to Sandy Silt	81.499	1.389	4.452	1.433	3.019	2.53	0.52145	0.56	0.108	2.530	0.52	42.50	2.925	124.296	0.26	0.80	1	2.85
76.608	6	Sandy Silt to Clayey Silt	27.04	0.69	4.49	1.454	3.036	3.07	0.51936	0.56	0.108							0.80	1	
77.264	6	Sandy Silt to Clayey Silt	19.357	0.404	4.527	1.474	3.053	3.18	0.51728	0.56	0.108							0.80	1	
77.92	6	Sandy Silt to Clayey Silt	24.244	0.486	4.565	1.495	3.07	3.07	0.51522	0.56	0.107							0.80	1	
78.576	6	Sandy Silt to Clayey Silt	37.784	1.158	4.603	1.515	3.087	2.98	0.51318	0.55	0.107							0.80	1	
79.232	6	Sandy Silt to Clayey Silt	40.88	1.292	4.64	1.536	3.104	2.95	0.51115	0.55	0.107							0.80	1	
79.888	6	Sandy Silt to Clayey Silt	45.546	1.475	4.678	1.556	3.122	2.92	0.50902	0.55	0.107							0.80	1	
80.545	5	Clayey Silt to Silty Clay	42.887	1.482	4.715	1.577	3.139	2.98	0.50703	0.55	0.107							0.80	1	
81.201	5	Clayey Silt to Silty Clay	31.275	1.079	4.753	1.597	3.156	3.12	0.50505	0.54	0.107							0.79	1	
81.857	5	Clayey Silt to Silty Clay	36.934	1.324	4.791	1.618	3.173	3.06	0.50309	0.54	0.106							0.79	1	
82.513	5	Clayey Silt to Silty Clay	33.708	1.091	4.828	1.638	3.19	3.08	0.50114	0.54	0.106							0.79	1	
83.169	5	Clayey Silt to Silty Clay	33.583	1.134	4.866	1.659	3.207	3.09	0.49921	0.54	0.106							0.79	1	
83.825	5	Clayey Silt to Silty Clay	39.684	1.439	4.903	1.679	3.224	3.05	0.49729	0.54	0.106							0.79	1	
84.482	5	Clayey Silt to Silty Clay	45.847	1.552	4.941	1.7	3.241	2.97	0.49538	0.53	0.106							0.79	1	
85.138	6	Sandy Silt to Clayey Silt	53.195	1.519	4.979	1.72	3.259	2.87	0.49338	0.53	0.106							0.79	1	
85.794	5	Clayey Silt to Silty Clay	42.587	1.543	5.016	1.741	3.276	3.03	0.49151	0.53	0.106							0.79	1	
86.45	5	Clayey Silt to Silty Clay	38.419	1.531	5.054	1.761	3.293	3.1	0.48965	0.53	0.105							0.79	1	
87.106	5	Clayey Silt to Silty Clay	43.964	1.628	5.091	1.781	3.31	3.02	0.4878	0.53	0.105							0.79	1	
87.762	6	Sandy Silt to Clayey Silt	44.741	1.237	5.129	1.802	3.327	2.94	0.48597	0.52	0.105							0.79	1	
88.419	5	Clayey Silt to Silty Clay	51.982	1.814	5.167	1.822	3.344	2.94	0.48415	0.52	0.105							0.79	1	
89.075	5	Clayey Silt to Silty Clay	71.764	2.702	5.204	1.843	3.361	2.82	0.48235	0.52	0.105							0.78	1	
89.731	6	Sandy Silt to Clayey Silt	86.806	3.276	5.242	1.863	3.378	2.76	0.48056	0.52	0.105							0.78	1	
90.387	5	Clayey Silt to Silty Clay	50.876	2.069	5.279	1.884	3.395	2.99	0.47878	0.52	0.105							0.78	1	
91.043	5	Clayey Silt to Silty Clay	41.135	1.619	5.317	1.904	3.413	3.07	0.47691	0.52	0.105							0.78	1	
91.699	6	Sandy Silt to Clayey Silt	44.815	1.4	5.355	1.925	3.43	2.96	0.47516	0.52	0.105							0.78	1	
92.356	6	Sandy Silt to Clayey Silt	49.442	1.327	5.392	1.945	3.447	2.88	0.47342	0.51	0.104							0.78	1	
93.012	6	Sandy Silt to Clayey Silt	45.886	1.315	5.43	1.966	3.464	2.93	0.4717	0.51	0.104							0.78	1	
93.668	6	Sandy Silt to Clayey Silt	48.455	1.407	5.467	1.986	3.481	2.91	0.46999	0.51	0.104							0.78	1	
94.324	6	Sandy Silt to Clayey Silt	59.506	2.052	5.505	2.007	3.498	2.89	0.46828	0.51	0.104							0.78	1	
94.98	6	Sandy Silt to Clayey Silt	104.094	3.952	5.542	2.027	3.515	2.72	0.4666	0.51	0.104							0.78	1	
95.636	7	Silty Sand to Sandy Silt	176.09	3.862	5.581	2.048	3.534	2.38	0.46472	0.51	0.104	2.380	0.46	81.83	2.233	182.699		0.76	1	
96.293	9	Sand	264.846	2.301	5.622	2.068	3.554	1.99	0.46277	0.51	0.104	1.990	0.46	122.56	1.287	157.757	0.45	0.69	1	5.11
96.949	9	Sand	294.846	2.611	5.663	2.089	3.574	1.95	0.46083	0.50	0.104	1.950	0.46	135.87	1.240	168.465		0.67	1	
97.605	9	Sand	260.214	2.008	5.703	2.109	3.594	1.97	0.45891	0.50	0.104	1.970	0.46	119.41	1.263	150.787	0.40	0.69	1	4.58
98.261	7	Silty Sand to Sandy Silt	164.949	3.562	5.742	2.13	3.612	2.41	0.45719	0.50	0.104	2.410	0.46	75.41	2.354	177.496		0.76	1	
98.917	6	Sandy Silt to Clayey Silt	71.838	2.459	5.78	2.15	3.63	2.82	0.45549	0.50	0.104							0.77	1	
99.573	6	Sandy Silt to Clayey Silt	50.871	1.653	5.817	2.17	3.647	2.95	0.45389	0.50	0.104							0.77	1	
100.23	6	Sandy Silt to Clayey Silt	47.332	1.507	5.855	2.191	3.664	2.98	0.4523	0.50	0.103							0.77	2	
100.886	5	Clayey Silt to Silty Clay	42.173	1.646	5.892	2.211	3.681	3.09	0.45073	0.50	0.103							0.77	3	
101.542	4	Silty Clay to Clay	43.448	1.91	5.93	2.232	3.698	3.12	0.44916	0.50	0.103							0.77	4	
102.198	6	Sandy Silt to Clayey Silt	46.255	1.42	5.968	2.252	3.715	2.99	0.44761	0.49	0.103							0.77	5	
102.854	6	Sandy Silt to Clayey Silt	79.458	2.403	6.005	2.273	3.732	2.76	0.44607	0.49	0.103							0.77	6	
103.511	6	Sandy Silt to Clayey Silt	93.638	3.087	6.043	2.293	3.749	2.73	0.44453	0.49	0.103							0.77	7	
104.167	6	Sandy Silt to Clayey Silt	87.356	2.801	6.08	2.314	3.767	2.74	0.44292	0.49	0.103							0.77	8	
104.823	6	Sandy Silt to Clayey Silt	86.347	3.264	6.118	2.334	3.784	2.8	0.44141	0.49	0.103							0.77	9	
105.479	5	Clayey Silt to Silty Clay	67.08	2.934	6.155	2.355	3.801	2.95	0.43991	0.49	0.103							0.77	10	
106.135	5	Clayey Silt to Silty Clay	49.527	1.794	6.193	2.375	3.818	3.01	0.43842	0.49	0.103							0.76	11	
106.791	7	Silty Sand to Sandy Silt	117.18	2.19	6.232	2.396	3.836	2.51	0.43685	0.49	0.103	2.510	0.44	51.19	2.820	144.333	0.36	0.76	12	4.17
107.448	6	Sandy Silt to Clayey Silt	68.243	2.371	6.269	2.416	3.853	2.87	0.43538	0.49	0.103							0.76	13	
108.104	5	Clayey Silt to Silty Clay	75.109	2.95	6.307	2.437	3.87	2.87	0.43393	0.49	0.103							0.76	14	
108.76	5	Clayey Silt to Silty Clay	64.626	2.588	6.344	2.457	3.887	2.93	0.43247	0.48	0.103							0.76	15	
109.416	5	Clayey Silt to Silty Clay	66.633	2.727	6.382	2.478	3.905	2.93	0.43095	0.48	0.103							0.76	16	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

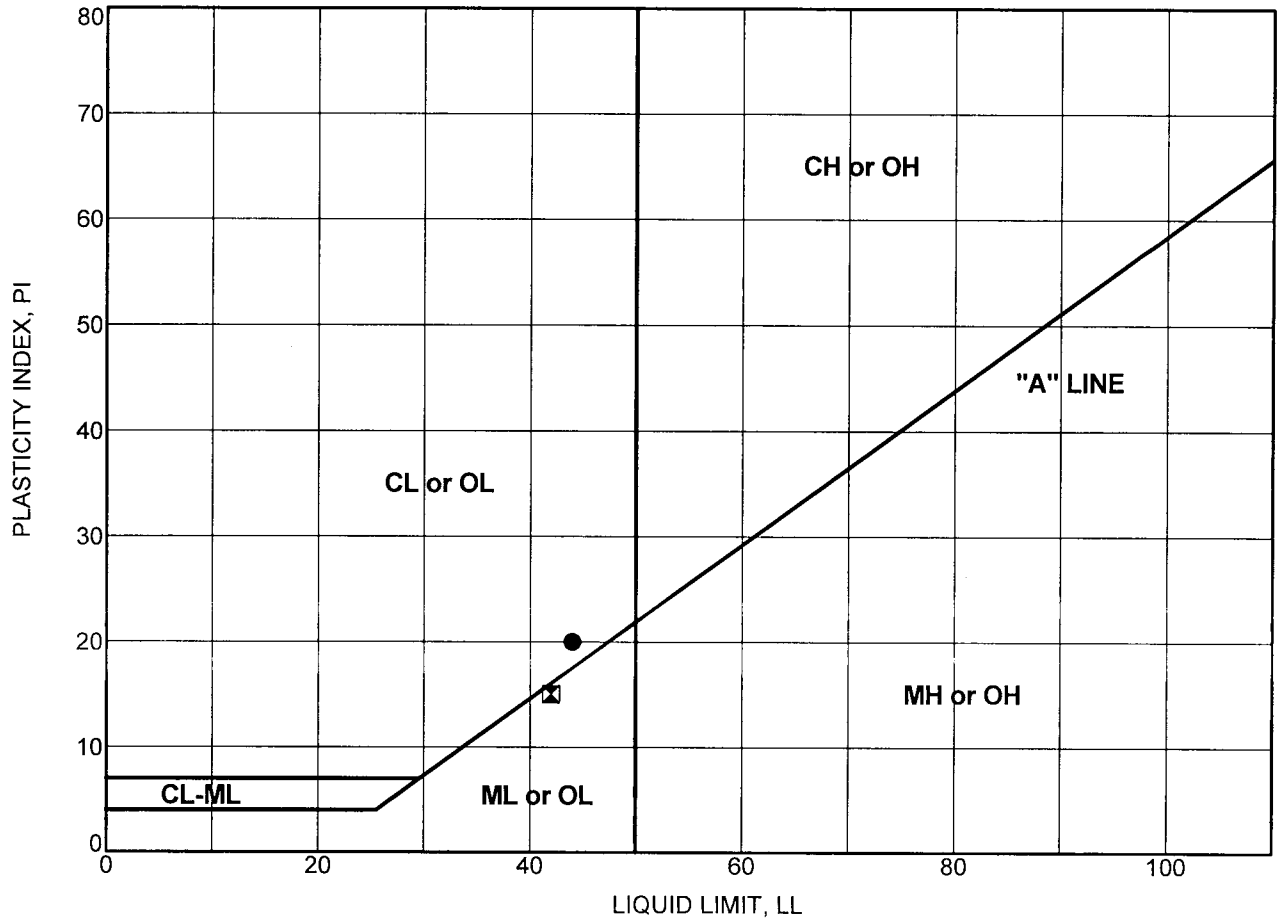
The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The results of these tests are presented on Plate B-2, "Plasticity Chart".

Unconfined Compression Test

Strength tests were performed on selected undisturbed samples using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-4A and B-4B.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-3.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A09-MA-001	MC-10	49.5	●	29	44	24	20	LEAN CLAY (CL)
A09-MA-001	SPT-13	69.5	⊠	32	42	27	15	SILT (ML)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Michigan Avenue UC (Widen)
Stockton, California

JOB NO: 206127.BR (MAB)

PLATE NO: B-2

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/29/2009
Date Submitted 04/24/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR Site ID : A-09-MA-001#6@29.5'
Thank you for your business.

* For future reference to this analysis please use SUN # 55554-111858.

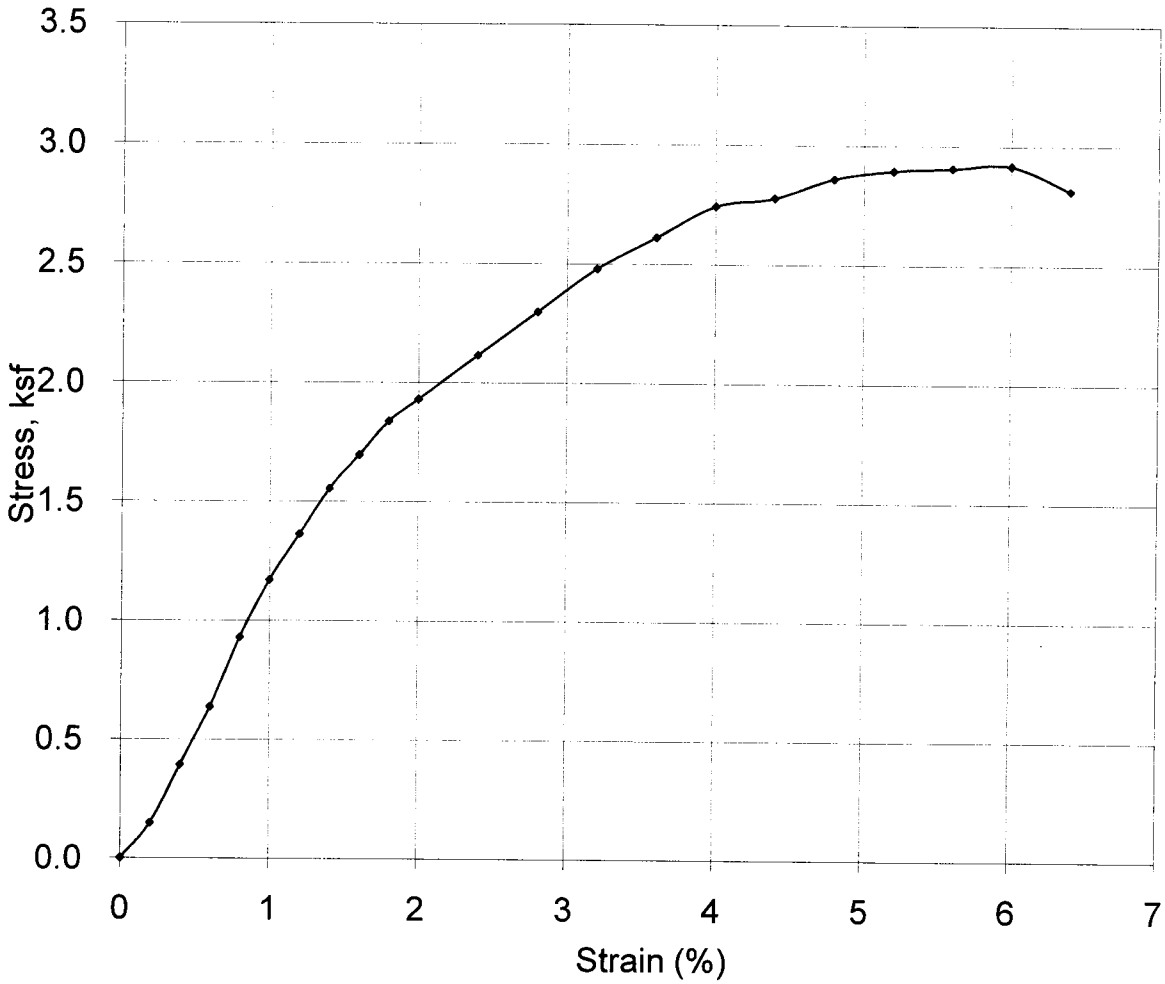
EVALUATION FOR SOIL CORROSION

Soil pH	6.83		
Minimum Resistivity	0.78 ohm-cm (x1000)		
Chloride	106.9 ppm	00.01069	%
Sulfate	49.6 ppm	00.00496	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 1 2009
By

UNCONFINED COMPRESSION TEST



Boring No.: A-09-MA-001

Sample No. : MC-7

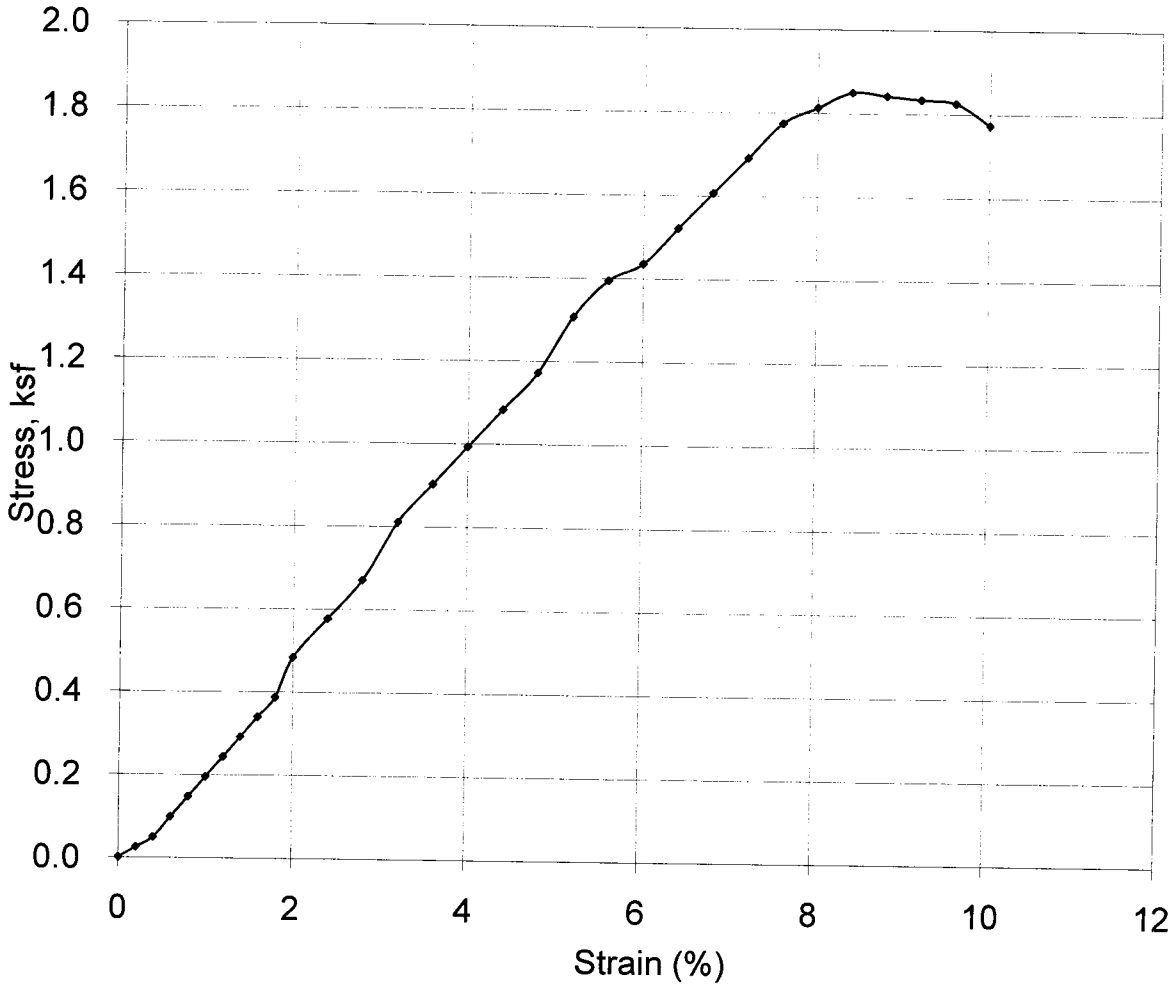
Depth (feet): 34.5

Material Description:
Lean Clay

Unconfined Compressive Strength: 2.9 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**MICHIGAN AVENUE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-4A

UNCONFINED COMPRESSION TEST



Boring No.: A-09-MA-001

Sample No. : MC-9

Depth (feet): 44.5

Material Description:
Silty Clay

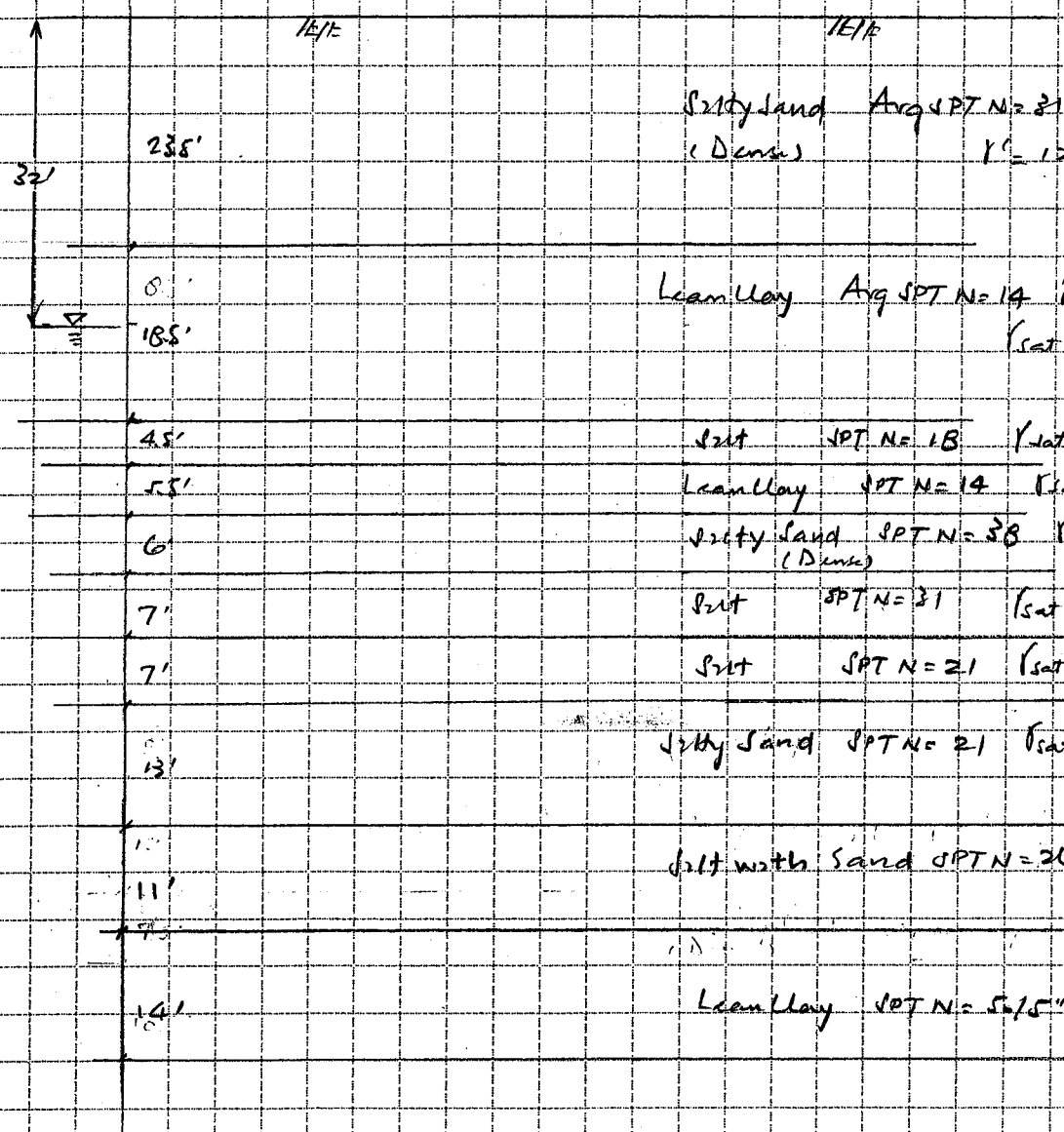
Unconfined Compressive Strength: 1.9 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	MICHIGAN AVENUE. UC (WIDEN) STOCKTON, CALIFORNIA	
	JOB NO.: 206127.BR	PLATE NO.: B-4B

APPENDIX C

Evaluation of Liquefaction Potential

Evaluation of Liquefaction Potential - Mechergan Ave UC (Boring A-09-MA-001)



LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-MA-001 (Michigan Avenue UC)(Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 32

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR$_{7.0}$/CSR)*MSF*Kσ*Kα*

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	23.5	1	31	SPT	2820.0	2820.0	0.89	0.12	36	0.84	1	0.95	1.2	1.15	39.49	15%	43.89		0.90	1	
2	42.0	2	14	SPT	5097.5	4472.5	0.73		16	0.67	1	0.80	1.2	1.15	11.93	55%			0.79	1	
3	46.5	2	18	SPT	5682.5	4776.3	0.69		21	0.65	1	0.80	1.2	1.15	14.84	55%			0.77	1	
4	52.0	2	14	SPT	6397.5	5147.5	0.64		16	0.62	1	0.80	1.2	1.15	11.12	55%			0.75	1	
5	58.0	1	38	SPT	7207.5	5582.5	0.59	0.10	44	0.60	1	0.80	1.2	1.15	28.97	15%	32.87		0.73	1	
6	65.0	2	31	SPT	8117.5	6055.0	0.55		36	0.57	1	0.80	1.2	1.15	22.70	55%			0.72	1	
7	72.0	2	21	SPT	9027.5	6527.5	0.52		24	0.55	1	0.80	1.2	1.15	14.81	55%			0.70	1	
8	85.0	1	21	SPT	10717.5	7405.0	0.49	0.09	24	0.52	1	0.80	1.2	1.15	13.90	15%	17.07	0.18	0.68	1	1.58
9	96.0	2	26	SPT	12147.5	8147.5	0.47		30	0.50	1	0.80	1.2	1.15	16.41	55%			0.66	1	
10	110.0	2	50	SPT	13967.5	9092.5	0.46		58	0.47	1	0.80	1.2	1.15	29.87	55%			0.63	1	

Vertical Pile Capacity Calculations

PROJECT: Michigan Ave UC
BRIDGE NO: 29-0190 R&L

FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Diaphragm Abutment Size (ft)		Permissible Settlement (in.).	No. of Piles per Support Per Bridge
					B	L		
Abutment 1	WSD	14" SQ. Alt. "X"	13.95'	10.95'	2.5'	48.0'	1"	10
Abutment 2	WSD	14" SQ. Alt. "X"	13.81'	10.81'	2.5'	48.0'	1"	10

Note: Pile Cut-off Elevations at Abuts 1 & 2 are the bottom of diaphragm abutment elevations.

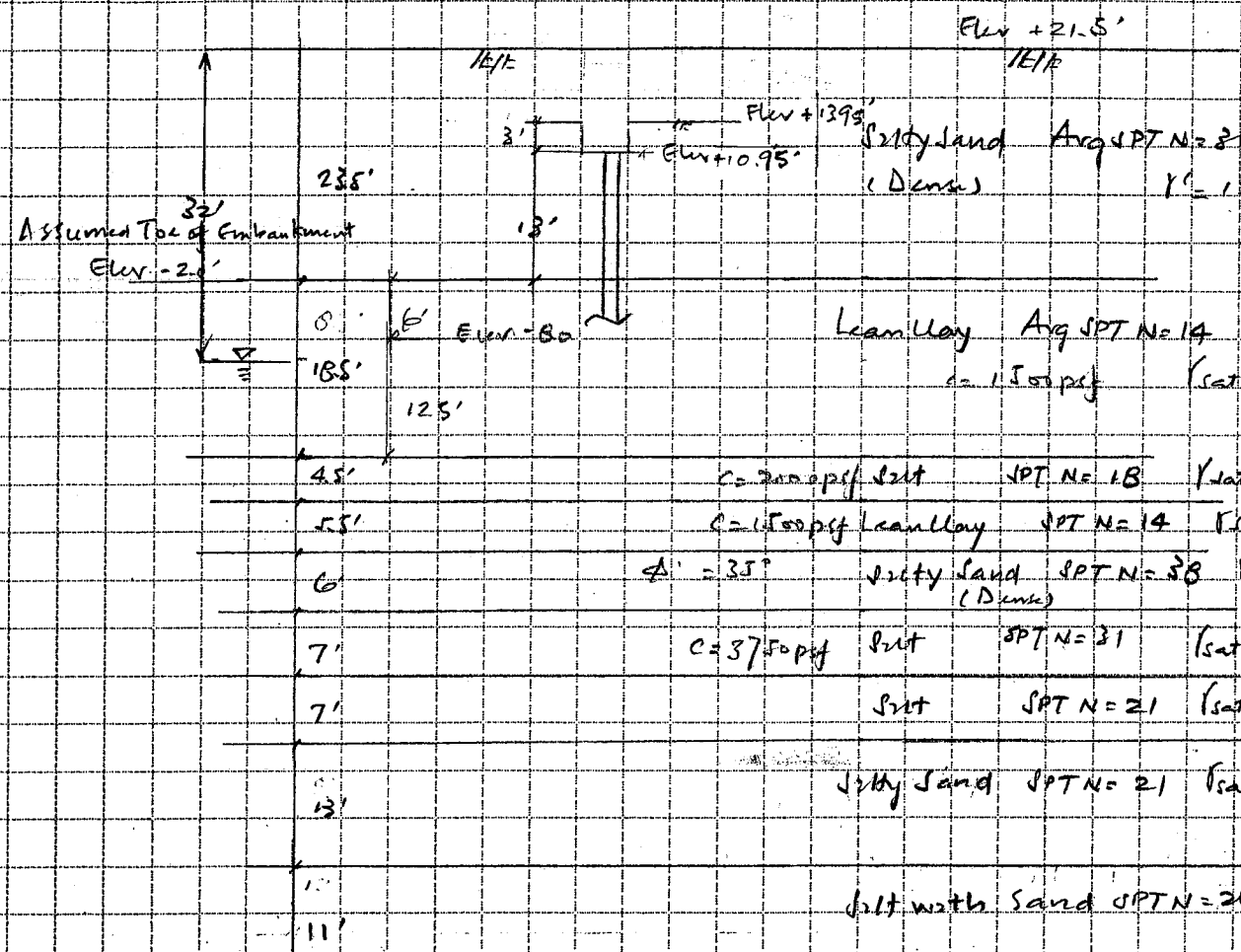
FOUNDATION LOADS

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abutment 1	1280	127	1030	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abutment 2	1280	127	1030	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes: Total Load for Service-I Limit State @ Abutments 1 & 2 = DL + LL (HL93)

Extreme Limit State = DL + Seismic

Pile Capacity Analyses - Mariposa Ave UC (Boring A-09-001) About 1.14" □ precast concrete pile - 63.5 tons service load.



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Michigan Ave. UC (Widen)*** ***Abut 1 - 65 tons service load (130 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 12/03/09

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS INCREAMENTS

1- GRAVELS, SANDS (FT)= 5

AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : A-09-MA-001 (PARIKH 2009)

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) : 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 140 Alt. "X" - 14" square precast prestressed (

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	120	360	180	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	16.0		1	120	1560	1140	SAND	V. DENSE	0	40	0.00	51.57	51.57	17.28	17.28
16.0	18.5		2	115	287.5	2064	CLAY	STIFF	1500	0	0.75	6.33	57.90	6.33	23.60
18.5	24.5		2	115	690	2553	CLAY	STIFF	1500	0	0.75	15.19	73.09	15.19	38.79
24.5	29.5		2	68	340	3068	CLAY	STIFF	1500	0	0.75	12.66	85.74	12.66	51.45
29.5	34.5		2	68	340	3408	CLAY	STIFF	1500	0	0.75	12.66	98.40	14.63	66.07
34.5	39.0		2	63	283.5	3719	CLAY	STIFF	2000	0	0.65	13.16	111.56	13.16	79.24
39.0	44.5		2	73	401.5	4062	CLAY	STIFF	1500	0	0.80	14.85	126.41	14.85	94.09
44.5	50.5		1	68	408	4467	SAND	DENSE	0	38	0.00	49.81	176.22	16.68	110.77
50.5	55.5		2	68	340	4841	CLAY	V.STIFF	3750	0	0.25	10.55	186.76	10.55	121.32
55.5	57.5		2	68	136	5079	CLAY	V.STIFF	3750	0	0.25	4.22	190.98	4.22	125.54
57.5	59.0		2	68	102	5198	CLAY	V.STIFF	2500	0	0.50	4.22	195.20 ←	4.22	129.75
59.0	64.5		2	68	374	5436	CLAY	V.STIFF	2500	0	0.50	15.47	210.67	0.00	129.75
64.5	69.5		1	68	340	5793	SAND	MED DEN	0	35	0.00	30.04	240.71	0.00	129.75
69.5	74.5		1	68	340	6133	SAND	MED DEN	0	34	0.00	29.89	270.60	0.00	129.75
74.5	77.5		1	68	204	6405	SAND	MED DEN	0	34	0.00	17.87	288.47	5.99	135.74
77.5	83.0		2	68	374	6694	CLAY	V.STIFF	3000	0	0.45	16.71	305.18	16.71	152.45
83.0	88.5		2	68	374	7068	CLAY	V.STIFF	3000	0	0.45	16.71	321.88	16.71	169.15
88.5	95.5		2	68	476	7493	CLAY	V.STIFF	4000	0	0.30	18.90	340.78	18.90	188.05
95.5	102.5		2	68	476	7969	CLAY	V.STIFF	4000	0	0.30	18.90	359.68	18.90	206.95

Abutment 1 --> Pile cut-off elevation is +10.95 feet. Finished grade elevation is +13.95 feet

Pre-drill through the exsiting embankment to bottom of pipe. Elevation of bottom of pipe is Elev. -4.5 feet (assumed to the depth of 18.5 feet)

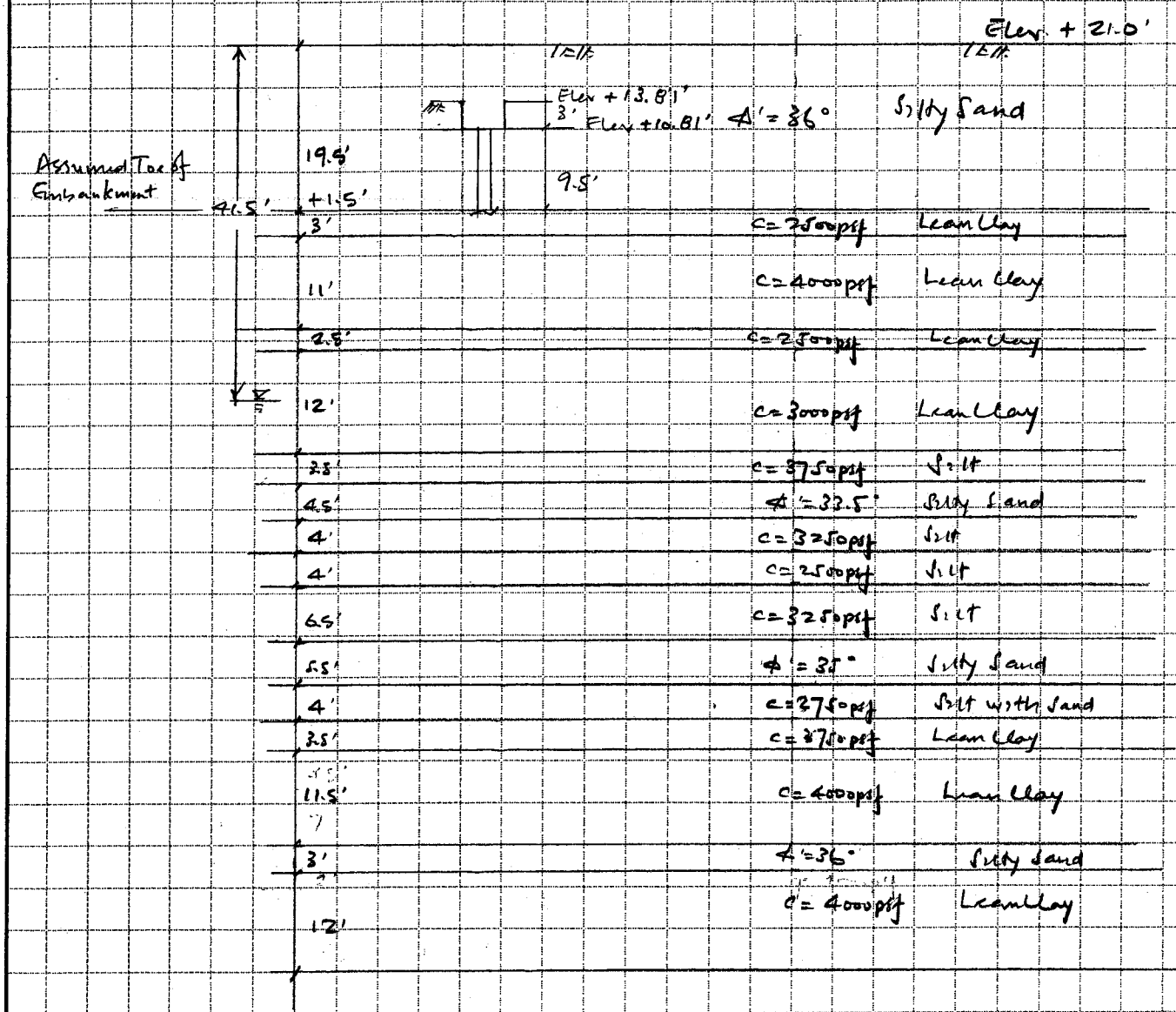
Pile Length @ Abutment 1 = 59.0' - 3.0' = 56.0'

Pile tip at 56.0' below pile cut-off (pile tip @ Elev. -45.5', pile cut-off @ Elev. +10.95'), available Ult. Pile Capacity =195.2 tons (friction)-57.9 tons (Predrilled)=137.3 tons > Nominal Resistance (260k/130T), OK

I-5 Michigan Ave. Abut 1 Class 140 Alt X (12-03-09)

Sheet 2 of 5

Simplified Soil Profile - Michigan Avenue UC (Widen) (CPT-09-001) About 2 14" □ precast concrete pile - 63.5 tons service Load.



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Michigan Ave. UC (Widen)*** ***Abut 2 - 65 tons service load (130 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 12/03/09

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.17 — 1- GRAVELS, SANDS AND NONPLASTIC SILTS — (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 2 (1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 1 (1=CONCRETE ,2=STEEL) — SOIL TYPES — GRAVEL, SAND, NSILT Ref Boring : CPT-09-MA-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) : 4.50 SQ. FT.

DRIVEN/DRILLED = 1 (1=DRIVEN ,2=DRILLED) — CLAY, PSILT — AREA OF THE PILE TIP 1.35 SQ. FT.

Class 140 Alt. "X" - 14" square precast prestressed c

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	120	360	180	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	12.5		1	120	1140	930	SAND	DENSE	0	36	0.00	26.59	26.59	8.91	8.91
12.5	15.5		2	115	345	1673	CLAY	V.STIFF	2500	0	0.40	6.75	33.34	6.75	15.66
15.5	18.5		2	120	360	2025	CLAY	V.STIFF	4000	0	0.25	6.75	40.09	6.75	22.41
18.5	26.5		2	120	960	2685	CLAY	V.STIFF	4000	0	0.25	18.00	58.09	18.00	40.41
26.5	29.0		2	115	287.5	3309	CLAY	V.STIFF	2500	0	0.45	6.33	64.41	6.75	47.16
29.0	34.5		2	120	660	3783	CLAY	V.STIFF	3000	0	0.40	14.85	79.26	14.85	62.01
34.5	41.0		2	68	442	4334	CLAY	V.STIFF	3000	0	0.40	17.55	96.81	17.55	79.56
41.0	44.5		2	68	238	4674	CLAY	V.STIFF	3750	0	0.30	8.86	105.67	8.86	88.42
44.5	49.0		1	68	306	4946	SAND	MED DEN	0	34	0.00	21.44	127.12	0.00	88.42
49.0	53.0		2	68	272	5235	CLAY	V.STIFF	3250	0	0.40	11.70	138.82	11.70	100.12
53.0	57.0		2	68	272	5507	CLAY	V.STIFF	2500	0	0.65	14.63	153.44	14.63	114.74
57.0	60.0		2	68	204	5745	CLAY	V.STIFF	3250	0	0.40	8.78	162.22	9.28	124.02
60.0	63.5		2	68	238	5966	CLAY	V.STIFF	3250	0	0.40	10.24	172.45 ←	9.45	133.47
63.5	69.0		1	68	374	6272	SAND	MED DEN	0	35	0.00	26.76	199.21	0.00	133.47
69.0	73.0		2	68	272	6595	CLAY	V.STIFF	2750	0	0.50	12.38	211.58	12.38	145.85
73.0	76.5		2	68	238	6850	CLAY	V.STIFF	4000	0	0.30	8.86	220.44	9.45	155.30
76.5	88.0		2	68	782	7360	CLAY	V.STIFF	4000	0	0.30	31.05	251.49	31.05	186.35
88.0	91.0		1	68	204	7853	SAND	DENSE	0	36	0.00	18.50	269.99	0.00	186.35
91.0	103.0		2	68	816	8363	CLAY	V.STIFF	4000	0	0.30	32.40	302.39	32.40	218.75

Abutment 2 --> Pile cut-off elevation is +10.81 feet. Finished grade elevation is +13.81 feet

Pre-drill through the exsiting embankment to bottom of pipe. Elevation of bottom of pipe is Elev. -4.5 feet (assumed to the depth of 18.5 feet)

Pile Length @ Abutment 2 = 63.5' - 3.0' = 60.5'

Pile tip at 60.5' below pile cut-off (pile tip @ Elev. -50.0', pile cut-off @ Elev. +10.81'), Ult. Pile Capacity =172.5 tons (Friction)-40.1 tons (Predrilled)=132.4 tons > Nominal Resistance (260k/130T), OK

I-5 Michigan Ave. Abut 2 Class 140 Alt X (12-03-09)

Sheet 4 of 5

MICHIGAN AVE. UC (WIDEN)
(Sheet 5 of 5)

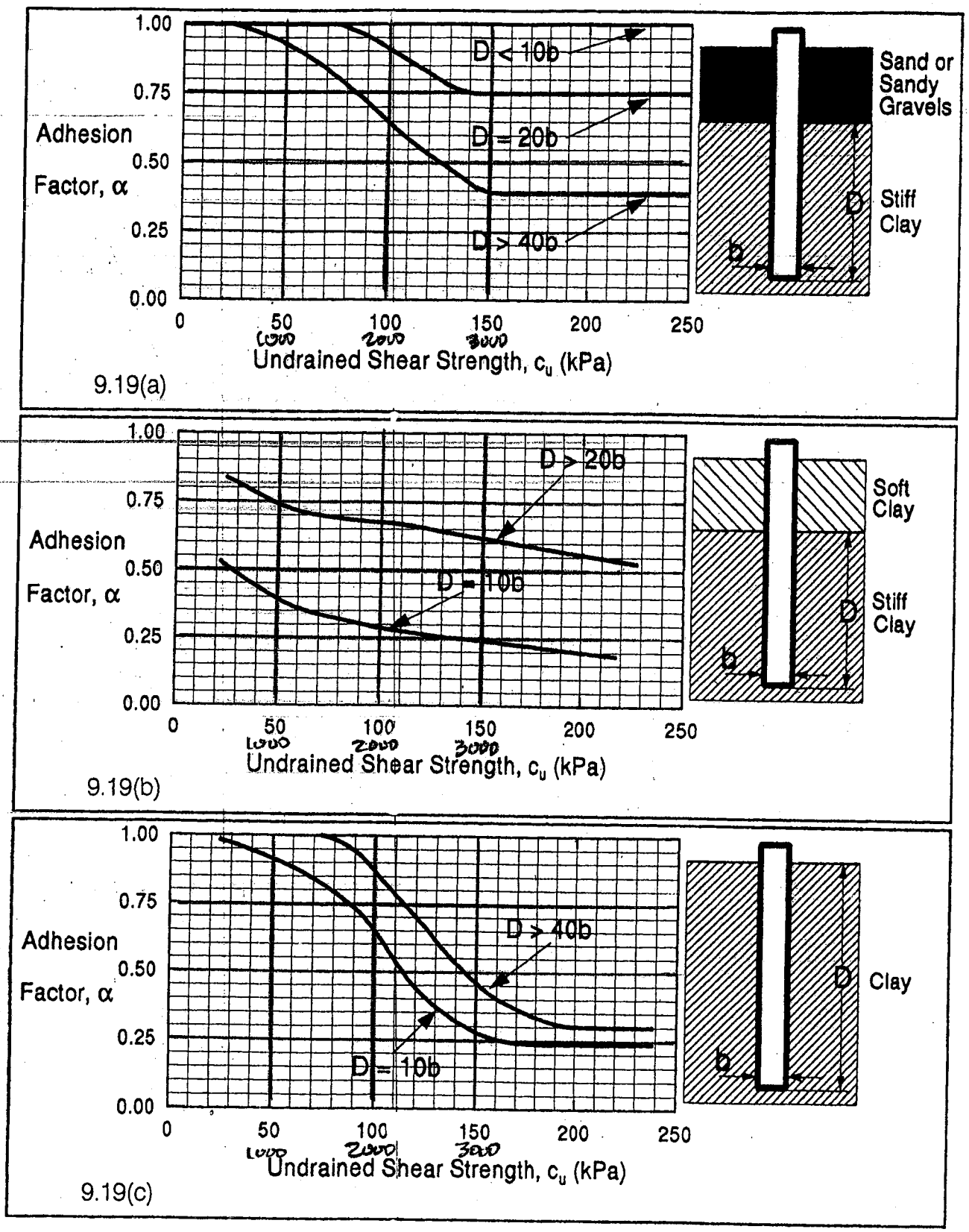


Figure 9.19 Adhesion Factors for Driven Piles in Clay (after Tomlinson, 1980)

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Michigan Avenue UC (Widen)

Boring A-09-MA-001 (Abutment 1) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
21.5* to -2.0	0 to 23.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.072	-
-2.0 to -10.5	23.5 to 32	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.069	-
-10.5 to -20.5	32 to 42	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.036	Assumed GWL at 32 feet deep below existing ground surface for design.
-20.5 to -25.0	42 to 46.5	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.036	-
-25.0 to -30.5	46.5 to 52	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.039	-
-30.5 to -36.5	52 to 58	Sand (Dense)	$\phi = 35°$; c= 0 psf	4, Sand	125	-	0.042	-
-36.5 to -43.5	58 to 65	Clay (very stiff)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-43.5 to -50.5	65 to 72	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-50.5 to -63.5	72 to 85	Sand (medium dense)	$\phi = 32°$; c=0 psf	4, Sand	60	-	0.039	-
-63.5 to -74.5	85 to 96	Clay (very stiff)	$\phi = 0°$; c=3000 psf	3, Clay	1000	0.005	0.039	-
-74.5 to -88.5	96 to 110	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +13.95 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Michigan Avenue UC (Widen)

CPT-09-MA-001 (Abutment 2) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
21.0* to 1.5	0 to 19.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.069	-
1.5 to -1.5	19.5 to 22.5	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.067	-
-1.5 to -12.5	22.5 to 33.5	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.069	-
-12.5 to -15.0	33.5 to 36	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.067	-
-15.0 to -20.5	36 to 41.5	Clay (very stiff)	$\phi = 0°$; c=3000 psf	3, Clay	1000	0.005	0.069	-
-20.5 to -27.0	41.5 to 48	Clay (very stiff)	$\phi = 0°$; c=3000 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 41.5 feet deep below existing ground surface for design.
-27.0 to -30.5	48 to 51.5	Clay (very stiff)	$\phi = 0°$; c=3750 psf	3, Clay	1000	0.005	0.039	-
-30.5 to -35.0	51.5 to 56	Sand (medium dense)	$\phi = 33.5°$; c= 0 psf	4, Sand	60	-	0.039	-
-35.0 to -39.0	56 to 60	Clay (very stiff)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-39.0 to -43.0	60 to 64	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-43.0 to -49.5	64 to 70.5	Clay (very stiff)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-49.5 to -55.0	70.5 to 76	Sand (dense)	$\phi = 35°$; c=0 psf	4, Sand	125	-	0.042	-
-55.0 to -59.0	76 to 80	Clay (very stiff)	$\phi = 0°$; c=2750 psf	3, Clay	1000	0.005	0.039	-
-59.0 to -62.5	80 to 83.5	Clay (very stiff)	$\phi = 0°$; c=3750 psf	3, Clay	1000	0.005	0.039	-
-62.5 to -74.0	83.5 to 95	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-74.0 to -77.0	95 to 98	Sand (dense)	$\phi = 36°$; c=0 psf	4, Sand	125	-	0.042	-
-77.0 to -89.0	98 to 110	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +13.81 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection __ 65% PS&E Unchecked Details _X PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 7/8/2010 **Structure Name*:** Michigan Avenue UC **Br No*:** 29-0190 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	_____	_____	_____	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 4th review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated June 11, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** Michigan Avenue UC **Br No*:** 29-0190 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	6/04/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	9	Please provide the Nominal Driving Resistance in Table 5.	Comment is incorporated. "Nominal Driving Resistance" is provided in Table 5.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection _X 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Michigan Avenue UC **Br No*:** 29-0190 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 EA: 0G4701 Project Name: I-5 North Stockton Widening OSFP Liaison: John Fujimoto Phone: 916-227-8757 e-mail: john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	Reviewer Name: Brandon Badeker Functional Unit: Geotechnical Services Cost Center: 59-323 Phone Number: 916-227-1046 e-mail: brandon_badeker@dot.ca.gov Date of Review: 12/16/2009 Structure Name*: Michigan Avenue UC Br No*: 29-0190 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
3	FR	8	Tables 2 and 3, Although the pile tip appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	
4	FR	9	Table 4: Pile data table shows two different design tip elevations for the two abutments. It is recommended to keep the same pile length for ease of pile fabrication.	Comment is incorporated. Same pile length is used in the design of specified pile tip elevation.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

5	FR	12, S9.6	The first sentence of the second paragraph states that the corrosion tests were for Alpine Avenue, this should be corrected to Michigan Avenue.	Comment noted. The subsurface soils are referred to the boring instead of its vicinity to the bridge structure. An excerpt of the report is attached.	
6	FR	LOTB	As is stated in the Soil Legend, the sample number or unconfined compressive strength is generally not located to the left of the graphic on the LOTB.	Comment is incorporated.	
7	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #3 above.	
8	FR	NA	C2 – Approved subject to OSFP Verification	NA	

FOUNDATION REPORT

ALPINE AVENUE UNDERCROSSING (WIDEN)
(BRIDGE NO. 29-0191)
STOCKTON, CALIFORNIA
10-SJ-05, PM 28.97 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
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June 11, 2010

Job No. 206127.BR

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Laboratory Test ..... Plate B-1
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Office of Special Funded Projects Comment & Response Form (with PARIKH Consultants, Inc. Responses)

FOUNDATION REPORT
ALPINE AVENUE UC (WIDEN)
(BRIDGE NO. 29-0191)
STOCKTON, CALIFORNIA
10-SJ-05 PM 28.97 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Alpine Avenue Undercrossing (Widen) (Bridge No. 29-0191)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Alpine Avenue Undercrossing (UC) (Widen) (Bridge No. 29-0191).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter

unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are single-span structures. Based on the as-built general plan and foundation plan of the Alpine Avenue UC, the structures were constructed in 1970. The existing left bridge (Bridge No. 29-0191L) and right bridge (Bridge No. 29-0191R) are approximately 117 feet long and 53 feet wide. The left and right bridges have not been widened since their construction in 1970.

The abutments of the existing left and right bridge are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class I 45 tons) based on the as-built data. The specified tip elevation is -50 feet (NAVD 29).

Proposed Construction

The widening of the Alpine Ave. UC (Widen) is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. There will be no outside widening for the Alpine Ave. UC.

According to the designer, the proposed Alpine Avenue UC (Widen) will match the existing span with supports at Abutments 1 and 2. The widen structure will be a cast-in-place prestressed concrete box girder with approximate length of 117 feet measured along the "I-5" Line. The width of the inside widening will be 48 feet. The abutments will incorporate diaphragm type abutments.

Based on our discussions with the designer, the proposed bridge abutments are to be supported on

Caltrans Standard Precast Prestressed Class 140 Alternate "X" (Mod) (14-inch square) piles.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The Alpine Ave. UC is located at the intersection of Interstate 5 (I-5) and Alpine Ave. It is approximately 0.2 miles southeast of Telegraph Avenue UC. The Project Location Map, Plate 1, shows the location of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the Alpine Avenue UC (February 1964) have been used as a reference (Ref. 1). Based on the as-built LOTB, Boring B-4 was drilled and Cone Penetration Test (CPT) at B-1, B-2 and B-3 were performed by Caltrans in 1964. The soil borings were drilled to the approximate depth 92 feet and to the bottom elevation between approximately -93 feet. The CPTs were pushed to the approximate depths between 65.5 feet and 68 feet and to the bottom elevation between approximately -66.5 feet and -69 feet.

Field Exploration in 2009

Additional field exploration was conducted in April 2009. Boring R-09-AA-001 was drilled on the south side and CPT-09-AA-001 was performed on the north side of Alpine Avenue UC. The soil boring was drilled and the CPT was pushed to the depth of approximately 110 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are

presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit weight, and grain size classification of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built Boring B-4, the subsurface soils generally consist of soft peaty clay and stiff silty clay, underlain by loose to medium dense silty sand, underlain by interbedded layers of medium dense to dense sands with pockets/lenses of stiff to very stiff clays, underlain by hard silty clay.

Based on Boring R-09-AA-001 and CPT-09-AA-001, the subsurface soils generally consist of medium dense silty/poorly graded sand (embankment fill), underlain by soft to very stiff lean clays with intermittent layers of medium dense silty/poorly graded sands.

Groundwater was recorded at approximate depths between 10 feet and 11 feet (Elevation -11 feet) in the CPTs in February 1964. No groundwater was recorded in the soil borings in February 1964. Groundwater was measured at Elevation -6 feet (the depth of 28 feet below existing ground surface) during drilling for R-09-AA-001 April 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of medium dense sands, underlain by soft to very stiff lean clays with intermittent layers of medium dense sands. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground

shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring R-09-AA-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-AA-001 attached, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure, different foundation systems have been evaluated with respect to their design capacity and construction considerations. Caltrans Class 140 Alt. "X" (Mod) Piles are recommended to be the foundation system for the abutments of the proposed bridge structure. These piles may be designed for service load of 65 tons at the abutments to the indicated pile tip elevations. Pertinent foundation design information provided by Rajappan & Meyer Consulting Engineers, Inc. (R&M), including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2: FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 140 Alt X (Mod)	12.60±	9.40±	2.50	51.00	1.00	14
Abut 2	WSD	Class 140 Alt X (Mod)	12.80±	9.60±	2.50	51.00	1.00	14

TABLE 3: FOUNDATION DESIGN LOADS

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	1,932	138	1,368	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1,932	138	1,368	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

* Entire support remains in overall compression

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads", based upon WSD is presented in Table 3 above.

The evaluation of Load Demands on the piles, based upon WSD is presented in Table 4 below. The estimated specified tip elevations for the anticipated design loading of the

piles are shown in Table 4 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	Class 140 Alt X (Mod)	9.40±	1,932	1,368	140	280	-44.0 (a), -9.0 (b)	-50.7
Abut 2	Class 140 Alt X (Mod)	9.60±	1,932	1,368	140	280	-50.5 (a), -6.0 (b)	-50.5

Notes: (i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

TABLE 5: PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 140 Alt X (Mod)	280	0	-44.0 (a), -9.0 (c)	-50.7	280
Abut 2	Class 140 Alt X (Mod)	280	0	-50.5 (a), -6.0 (c)	-50.5	280

Notes
(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression, (b) Tension, (c) Lateral Load.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iii) Based on Caltrans review comment, it was recommended to keep the same pile length (specified tip elevation) for "ease of pile fabrication".

The estimation of the driven pile capacity in clay is based on the adhesion factor per FHWA manual (Tomlinson 1980). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 5, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles

should not be spaced closer than 3 times the pile diameter measured center-to-center. For piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter, there is no group effect for vertical pile capacity.

It is anticipated that the pile capacity will develop after driving as a result of soil "freeze" and dissipation of excess pore water pressures. The final pile capacity can be verified and the gain of pile capacity after initial driving may be evaluated based on "re-driving" after 24-hour (min.) set up. Caltrans Standard Specifications (Section 49-1.08) may be used as the pile driving acceptance criteria. As an option, Pile Driving Analyzer (PDA) may be used to evaluate the pile capacity of the driven piles when unanticipated conditions arise. Typical applications of the PDA include capacity evaluation (for both during driving and re-driving) and integrity testing for piles that have experienced hard driving.

It is prudent to make the contractor aware of these conditions so that appropriate steps can be taken to comply with the standards and maintain the integrity of the piles.

Based on the available boring/CPT information, dense sands were encountered in the embankment fill. It is recommended to follow Caltrans standard specifications (Section 49-1.06) (Ref. 7) for oversize predrill through the existing embankment.

Due to the hard driving condition that may be encountered during pile driving, pile driving should be allowed to terminate short of the specified tip elevation provided the following conditions are satisfied:

- Other requirements including tension and lateral demands are met;
- Pile attaining its capacity and refusal within 5 feet above the specified tip elevation,

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. Lateral load analyses for the piles were performed using LPILE Program (v. 5). "Generalized Vertical Soil Profile and Strength Parameters" based on R-09-AA-001 and CPT-09-AA-001 are included in Appendix C. To account for group effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D,

respectively.

Generally, abutment piles are designed for service loads only, and batter piles are used for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live

load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for new structures. Settlement is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Boring R-09-AA-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
R-09-AA-001	31.0	1470	6.8	60.8	47.6

Based on the corrosion test results, the native subsurface soils in Boring R-09-AA-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.8 Construction Observation

To a degree, the performance of any structure is dependent upon construction procedures

and quality. Hence, geotechnical observation of foundation excavation, and pile installations should be carried out by regulating agencies. If the subsurface conditions different from those forming the basis of our recommendations are encountered, this office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,

PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Alpine Ave. UC (Widen) Foundation Report (6-2010).rpt

REFERENCES

1. Alpine Ave. UC As-built Log of Test Borings (February 1964).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

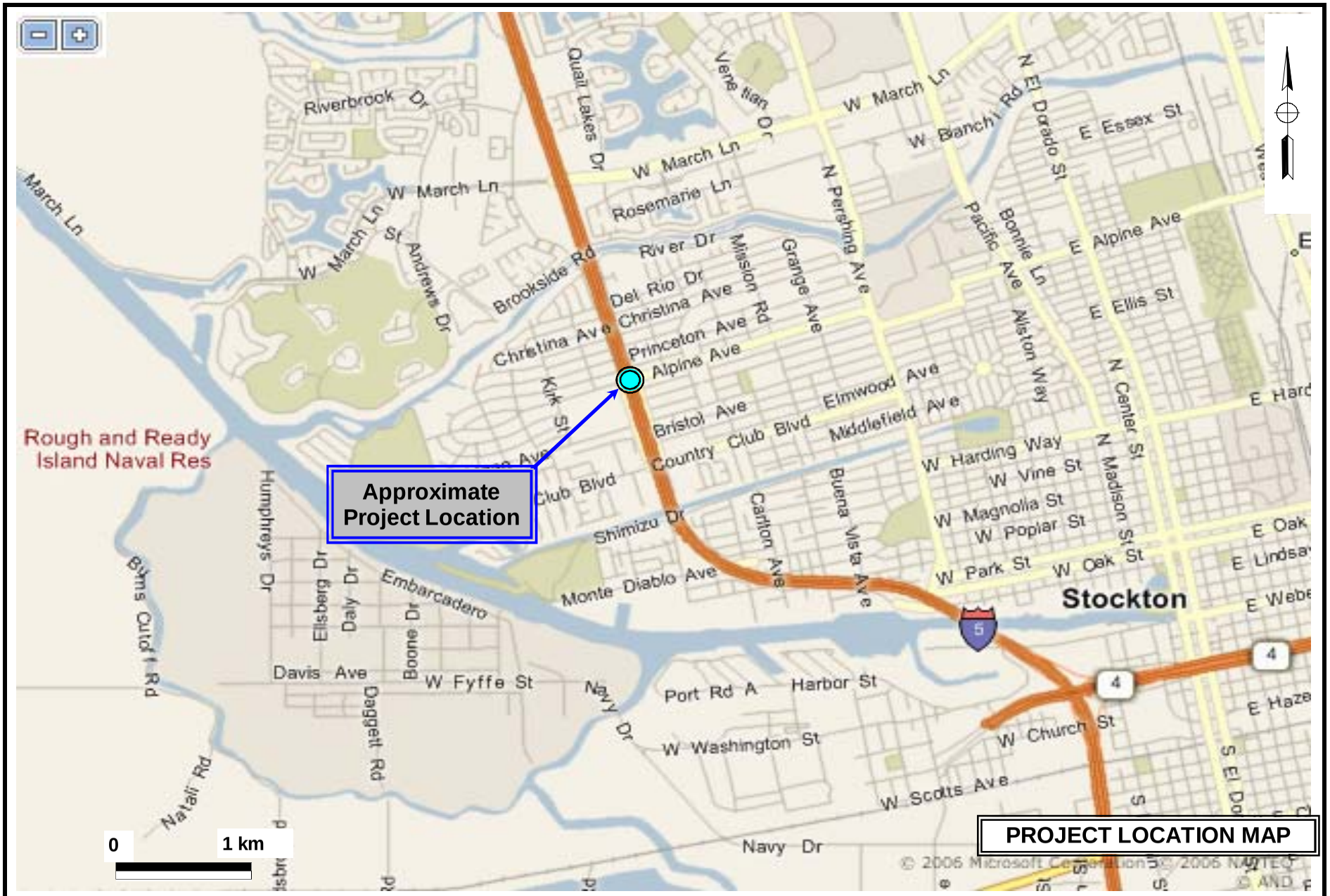
3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

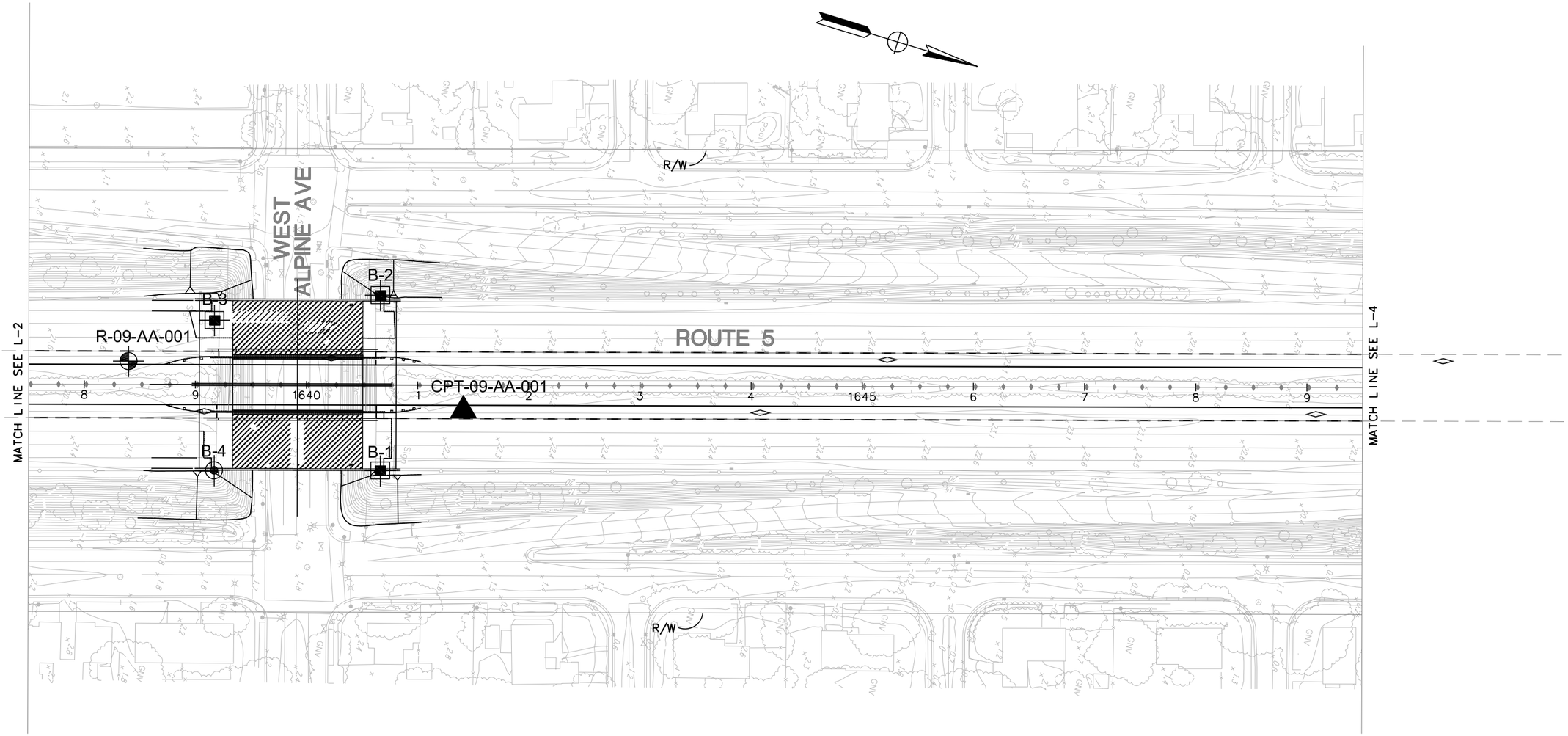
7. State of California Department of Transportation Standard Specifications May 2006.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

ALPINE AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0191)	**PLATE NO.: 1**



LEGEND

A-09-AA-001
Approximate Boring Location

B-3
Approximate CPT location performed in 1964

B-4
Approximate Boring location drilled in 1964

CPT-09-AA-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

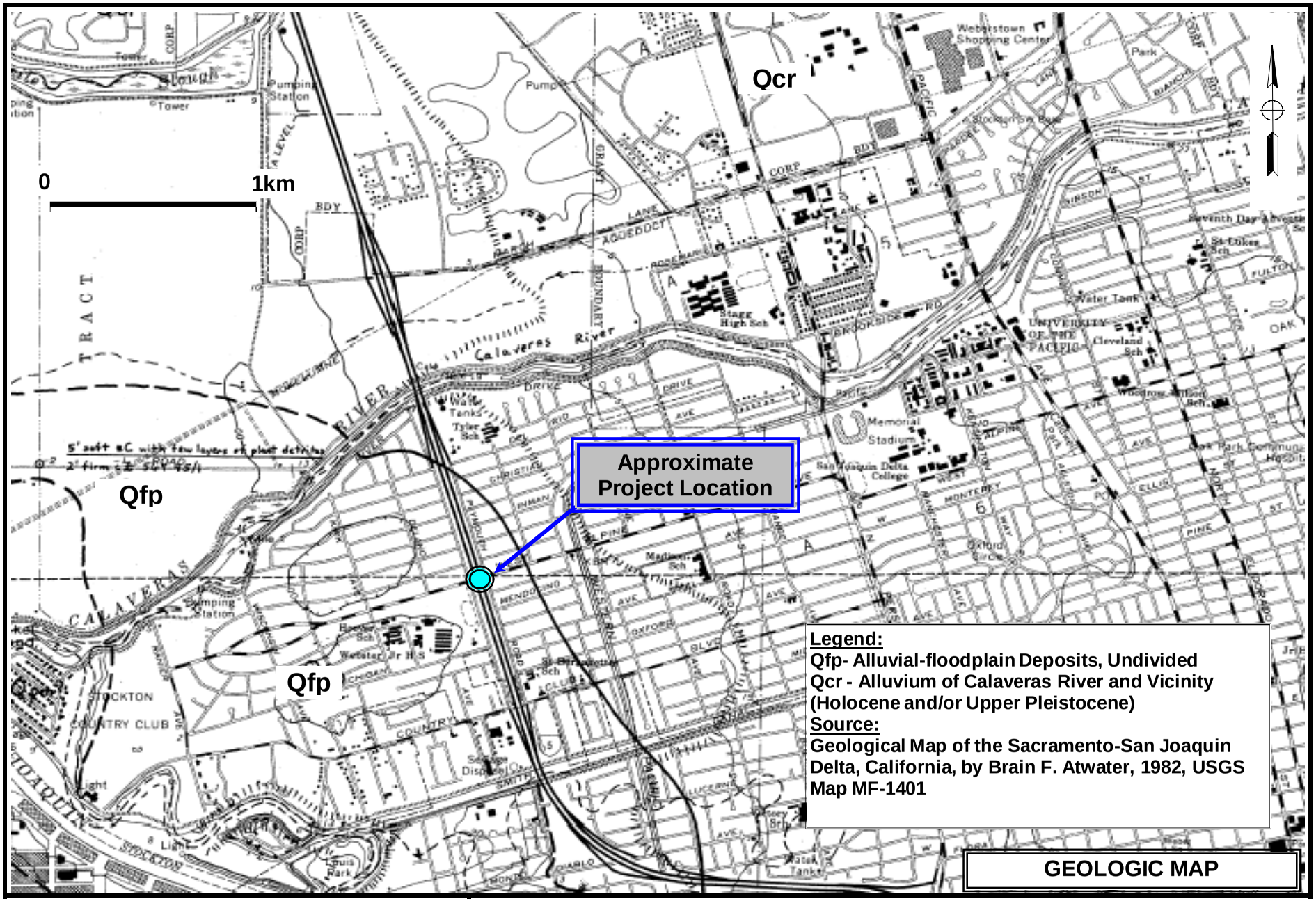
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

ALPINE AVENUE UC (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

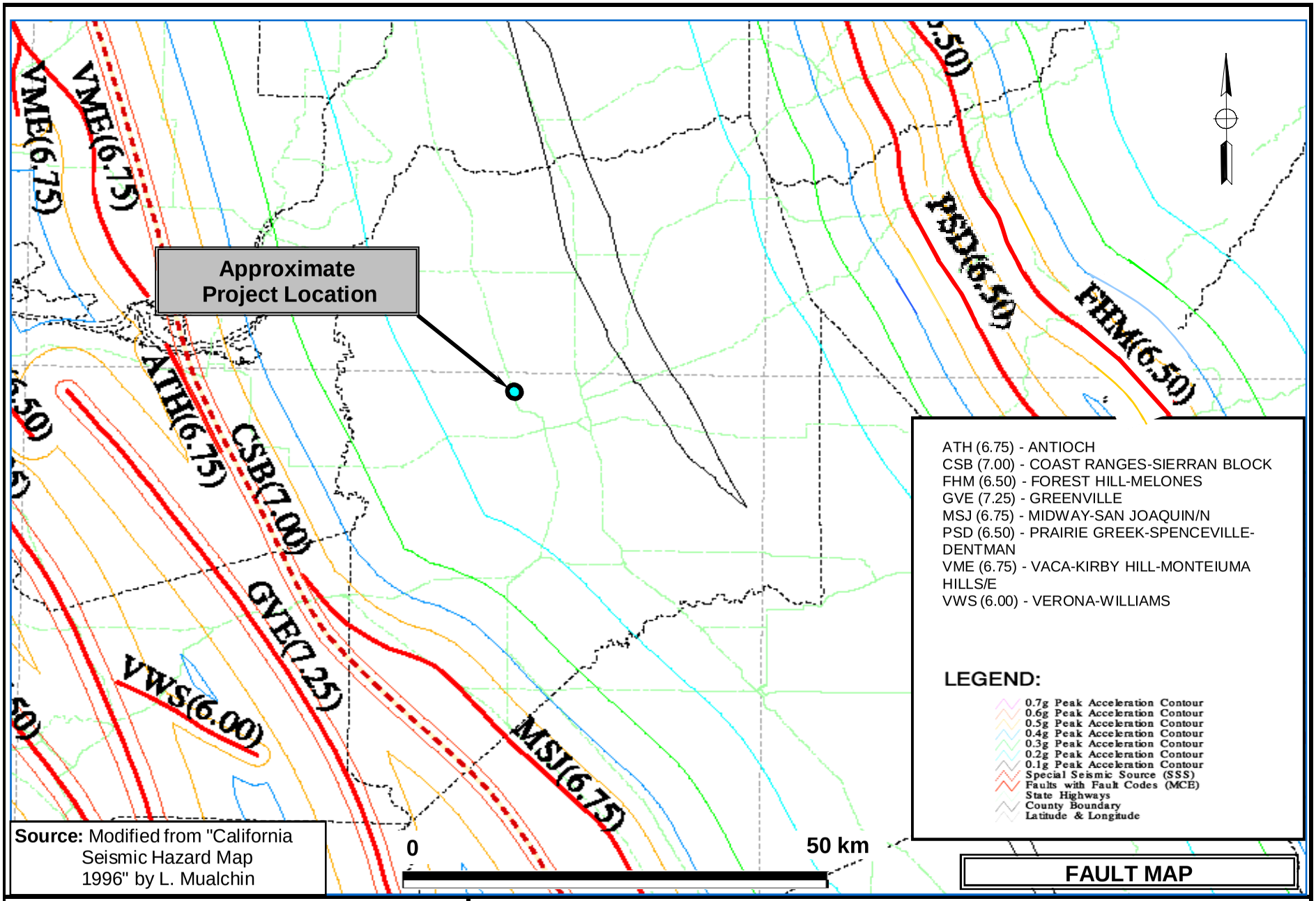
PLATE NO: 2



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

ALPINE AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0191)	PLATE NO.: 3



Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

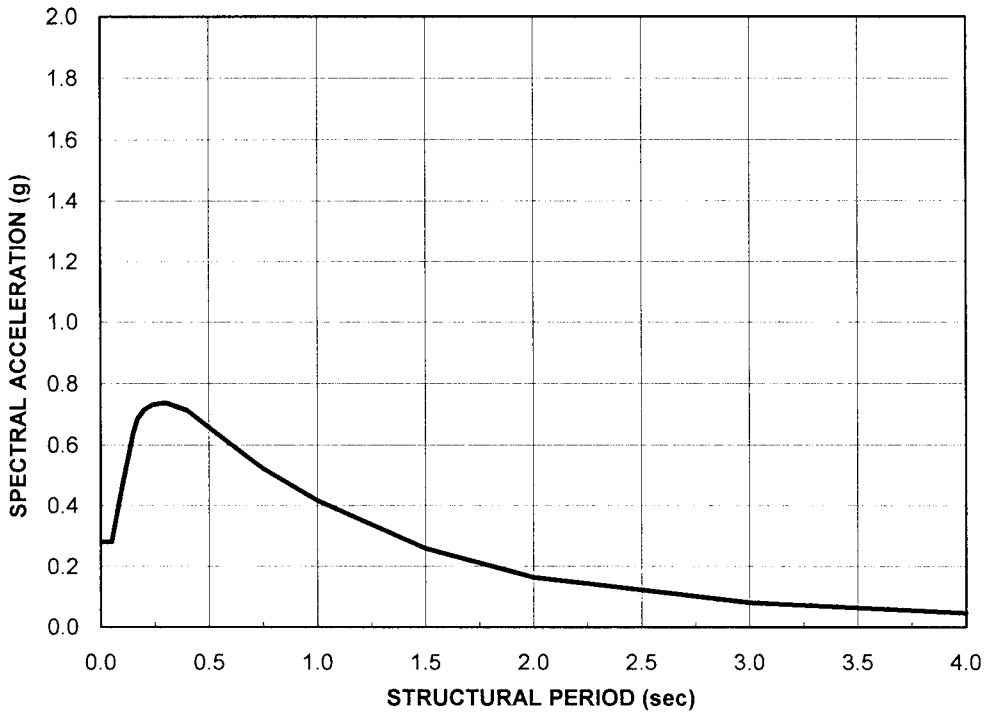
FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

ALPINE AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0191) | **PLATE NO.: 4**

ARS DESIGN CURVE
WEST ALPINE AVENUE UC (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

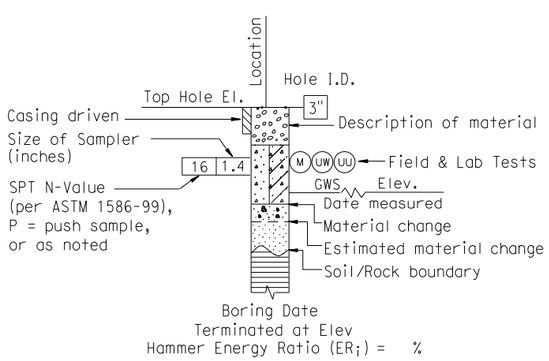
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

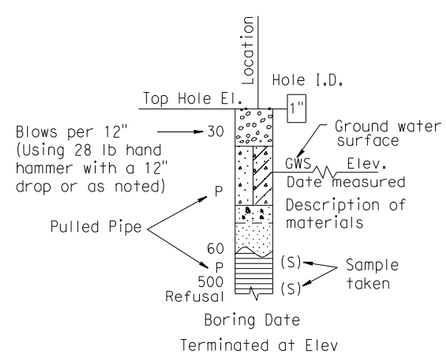
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

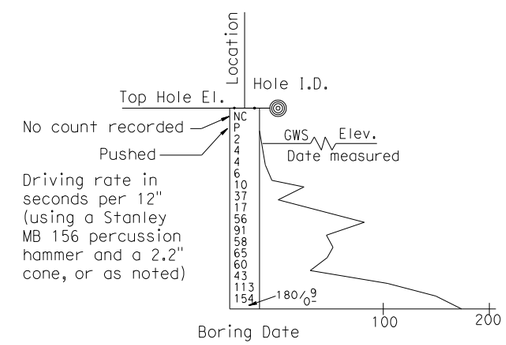
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



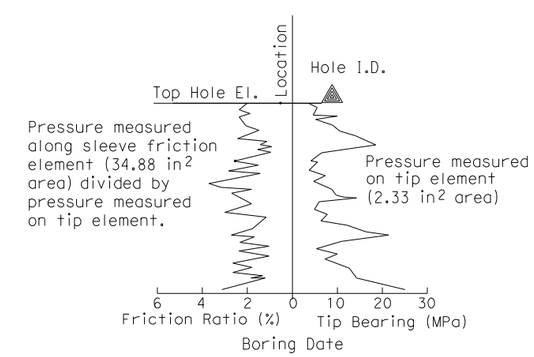
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER
SIGN OFF DATE	CHECKED BY	D. WANG	DATE: MARCH 2009 - APRIL 2009		

BRIDGE NO. 29-0191
POST MILES 28.97

ALPINE AVENUE UC (WIDEN)
SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 26 OF 29

...\29-0191-z-ltb01.dgn 6/8/2010 9:16:24 AM

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

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

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)
- (VS) Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0191
POST MILES 28.97

ALPINE AVENUE UC (WIDEN)
SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 27 OF 29

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

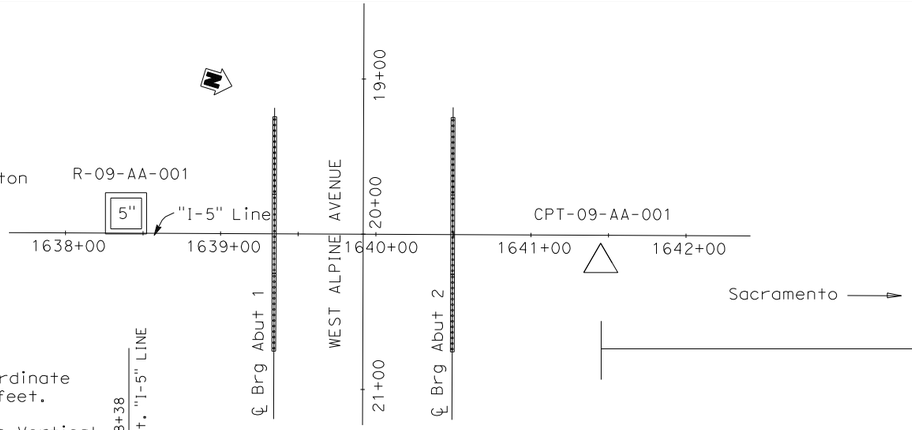
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

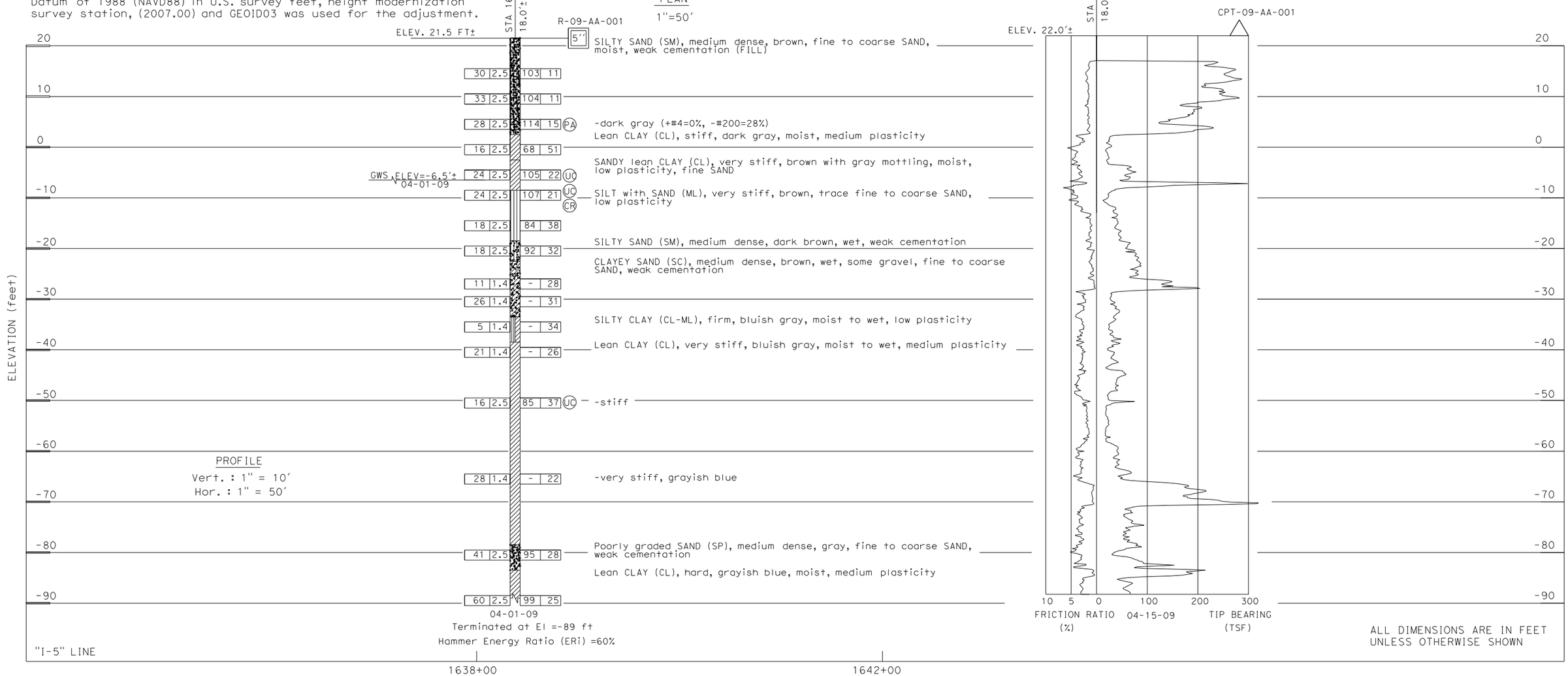
SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131



BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.

The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.



ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER

BRIDGE NO. 29-0191
POST MILES 28.97

ALPINE AVENUE UC (WIDEN)
LOG OF TEST BORINGS 1 OF 2

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 28 OF 29

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DIST.	COUNTY	ROUTE	KILOMETER POST-TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
10	SJ	5	25.5/31.9	316	332

Date Approved: September 5, 1967

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	KILOMETER POST-TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5	—	—	—

REGISTERED CIVIL ENGINEER — DATE

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

ALPINE AVENUE UC (WIDEN)

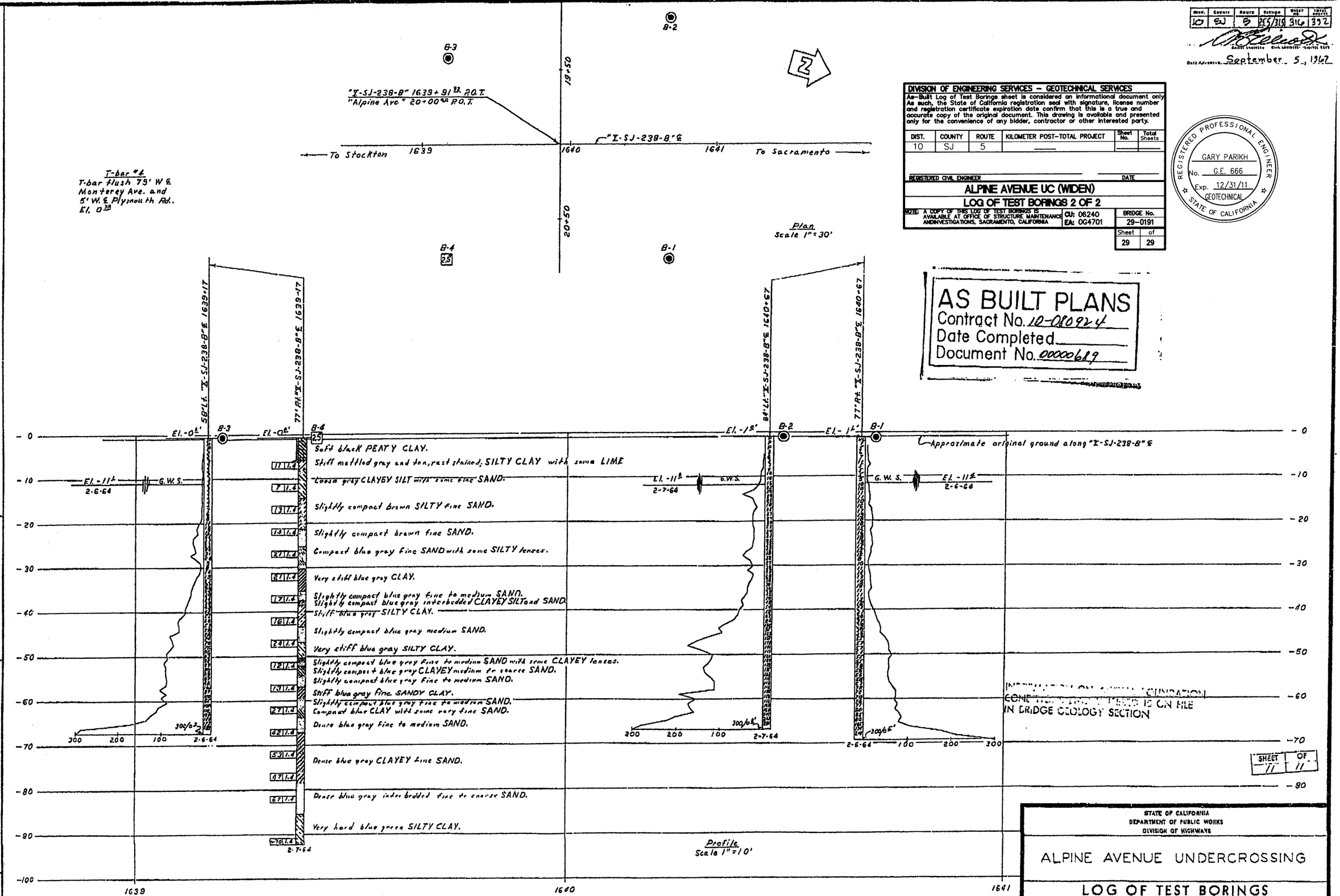
LOG OF TEST BORINGS 2 OF 2

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0G4701

BRIDGE No.	
29-0191	
Sheet	of
29	29

AS BUILT PLANS
Contract No. 10-080924
Date Completed
Document No. 00000689



LEGEND OF BORING OPERATIONS

- Penetrometer
- 2½" Cone Penetrometer
- Standard Boring (Dry)
- Rotary Boring (Wet)
- Auger Boring (Dry)
- Jet Boring
- Core Boring
- Test Pit
- Penetration Boring
- Rotary Boring
- 1" Soil Tube

LEGEND OF EARTH MATERIALS

- Gravel
- Sand
- Silt
- Clay
- Silty Clay or Clayey Silt
- Peat, Muck & Organic Matter
- Fill Material
- Igneous Rock
- Sedimentary Rock
- Metamorphic Rock
- Sandy Clay or Clayey Sand
- Sandy Silt or Silty Sand

CLASSIFICATION OF MATERIAL BASED ON STANDARD GRADE SIZE LIMITS

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

ALPINE AVENUE UNDERCROSSING

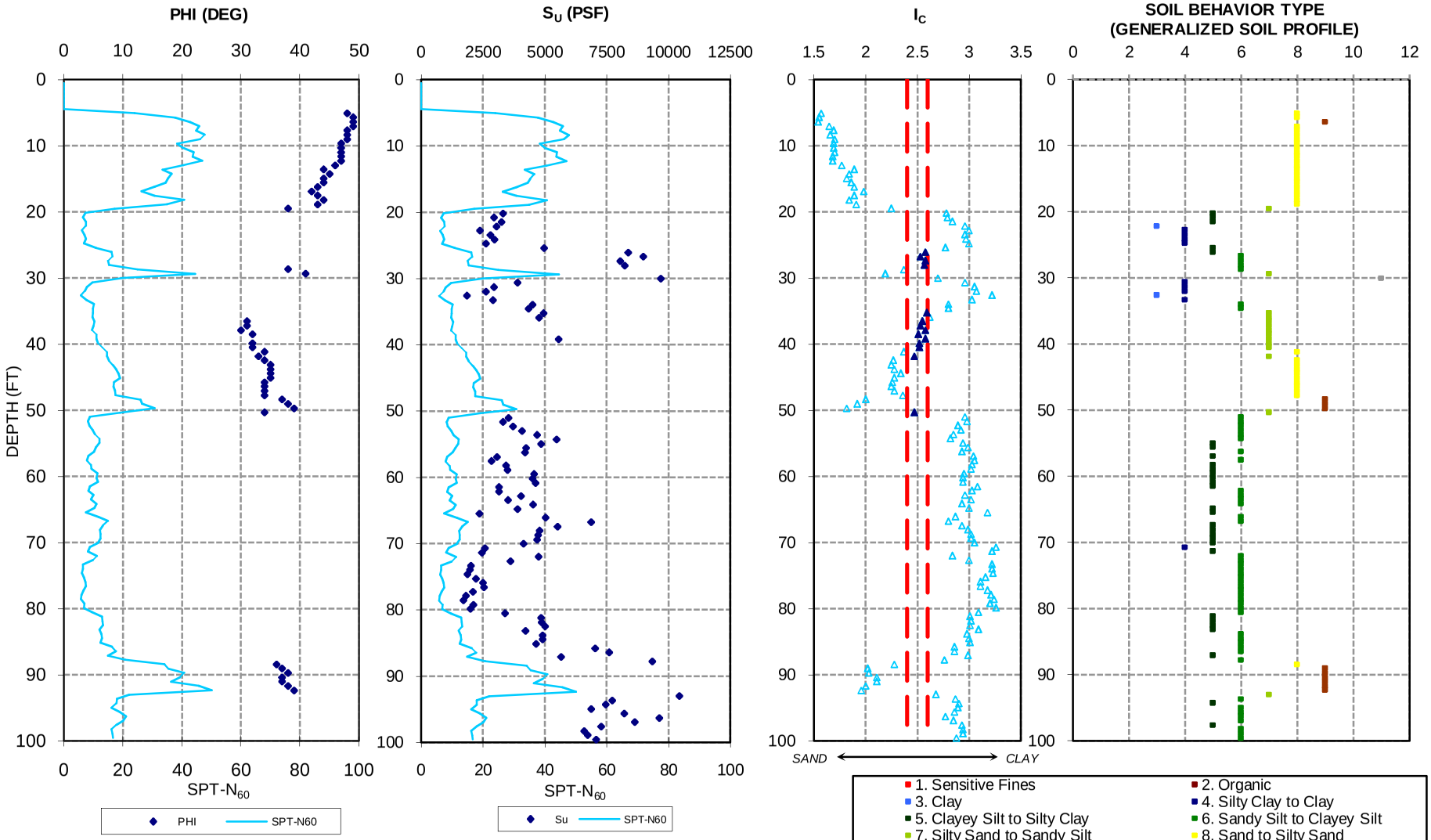
LOG OF TEST BORINGS

SCALE As Noted	BRIDGE 29-191 1/A	FILE	DRAWING 29191-8

Disregard prints bearing earlier numbers → PREL. DRAWING No. PR-29191-2

CPT DATA CORRELATION

CPT-09-AA-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

WEST ALPINE AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0191)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-WEST ALPINE AVENUE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0191)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-AA-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*Kσ*Kα*		
Depth (ft)	Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	Kσ	Kα	F.S.
5.085	8 Sand to Silty Sand	141.407	1.881	0.308		0.308	1.57	1.45889	0.99	0.129	1.570	1.46	206.30	1.000	206.297		1.00	1	
5.741	8 Sand to Silty Sand	224.703	3.414	0.347		0.347	1.55	1.42211	0.99	0.129	1.550	1.42	319.55	1.000	319.552		1.00	1	
6.398	9 Sand	256.239	3.812	0.388		0.388	1.54	1.38539	0.99	0.128	1.540	1.39	354.99	1.000	354.991		1.00	1	
7.054	8 Sand to Silty Sand	264.988	5.04	0.428		0.428	1.65	1.35135	0.99	0.128	1.650	1.35	358.09	1.003	359.287		1.00	1	
7.71	8 Sand to Silty Sand	255.672	5.12	0.467		0.467	1.69	1.31974	0.98	0.128	1.690	1.32	337.42	1.031	347.908		1.00	1	
8.366	8 Sand to Silty Sand	274.485	5.019	0.507		0.507	1.66	1.28881	0.98	0.128	1.660	1.29	353.76	1.010	357.440		1.00	1	
9.022	8 Sand to Silty Sand	261.864	5.046	0.547		0.547	1.7	1.2593	0.98	0.128	1.700	1.26	329.77	1.038	342.250		1.00	1	
9.678	8 Sand to Silty Sand	217.622	3.535	0.586		0.586	1.69	1.2318	0.98	0.127	1.690	1.23	268.07	1.031	276.400		1.00	1	
10.335	8 Sand to Silty Sand	228.774	3.619	0.626		0.626	1.69	1.20482	0.98	0.127	1.690	1.20	275.63	1.031	284.199		1.00	1	
10.991	8 Sand to Silty Sand	249.016	4.143	0.666		0.666	1.7	1.17899	0.98	0.127	1.700	1.18	293.59	1.038	304.702		1.00	1	
11.647	8 Sand to Silty Sand	250.054	3.773	0.705		0.705	1.68	1.15486	0.98	0.127	1.680	1.15	288.78	1.024	295.782		1.00	1	
12.303	8 Sand to Silty Sand	267.823	4.122	0.745		0.745	1.68	1.13111	0.97	0.127	1.680	1.13	302.94	1.024	310.286		1.00	1	
12.959	8 Sand to Silty Sand	224.108	3.802	0.785		0.785	1.77	1.10831	0.97	0.126	1.770	1.11	248.38	1.085	269.581		1.00	1	
13.615	8 Sand to Silty Sand	177.949	3.42	0.824		0.824	1.89	1.08696	0.97	0.126	1.890	1.09	193.42	1.180	228.156		1.00	1	
14.272	8 Sand to Silty Sand	196.972	3.434	0.864		0.864	1.84	1.06589	0.97	0.126	1.840	1.07	209.95	1.137	238.721		1.00	1	
14.928	8 Sand to Silty Sand	191.313	3.013	0.904		0.904	1.82	1.04563	0.97	0.126	1.820	1.05	200.04	1.122	224.350		1.00	1	
15.584	8 Sand to Silty Sand	184.844	3.005	0.944		0.944	1.86	1.02612	0.97	0.126	1.860	1.03	189.67	1.153	218.757		1.00	1	
16.24	8 Sand to Silty Sand	162.737	2.501	0.983		0.983	1.89	1.00779	0.97	0.126	1.890	1.01	164.00	1.180	193.455		1.00	1	
16.896	8 Sand to Silty Sand	135.221	2.265	1.023		1.023	1.98	0.98965	0.96	0.125	1.980	0.99	133.82	1.275	170.587		0.99	1	
17.552	8 Sand to Silty Sand	163.383	2.36	1.063		1.063	1.89	0.97216	0.96	0.125	1.890	0.97	158.83	1.180	187.357		0.98	1	
18.209	8 Sand to Silty Sand	219.89	3.44	1.102		1.102	1.84	0.95569	0.96	0.125	1.840	0.96	210.15	1.137	238.944		0.96	1	
18.865	8 Sand to Silty Sand	183.523	2.933	1.142		1.142	1.91	0.93937	0.96	0.125	1.910	0.94	172.40	1.198	206.600		0.96	1	
19.521	7 Silty Sand to Sandy Silt	78.896	1.584	1.181		1.181	2.25	0.92398	0.96	0.125	2.250	0.92	72.90	1.798	131.090	0.29	0.96	1	
20.177	5 Clayey Silt to Silty Clay	25.979	0.831	1.218		1.218	2.78	0.90984	0.96	0.124							0.96	1	
20.833	5 Clayey Silt to Silty Clay	23.156	0.628	1.256		1.256	2.79	0.89577	0.95	0.124							0.96	1	
21.49	5 Clayey Silt to Silty Clay	25.435	0.916	1.293		1.293	2.84	0.88247	0.95	0.124							0.95	1	
22.146	3 Clay	24.001	1.193	1.33		1.33	2.96	0.86957	0.95	0.124							0.94	1	
22.802	4 Silty Clay to Clay	19.17	0.796	1.367		1.367	3	0.85703	0.95	0.123							0.94	1	
23.458	4 Silty Clay to Clay	22.248	0.916	1.405		1.405	2.96	0.84453	0.95	0.123							0.93	1	
24.114	4 Silty Clay to Clay	23.519	1.048	1.443		1.443	2.97	0.83239	0.94	0.123							0.93	1	
24.77	4 Silty Clay to Clay	21.098	0.895	1.48		1.48	3	0.8209	0.94	0.123							0.92	1	
25.427	5 Clayey Silt to Silty Clay	38.623	1.49	1.518		1.518	2.77	0.80942	0.94	0.122							0.92	1	
26.083	5 Clayey Silt to Silty Clay	63.826	2.333	1.555		1.555	2.58	0.79855	0.94	0.122	2.580	0.80	50.97	3.206	163.427		0.92	1	
26.739	6 Sandy Silt to Clayey Silt	66.746	2.186	1.593		1.593	2.53	0.78768	0.94	0.122	2.530	0.79	52.57	2.925	153.769	0.42	0.91	1	
27.395	6 Sandy Silt to Clayey Silt	59.313	2.027	1.631		1.631	2.58	0.77711	0.93	0.121	2.580	0.78	46.09	3.206	147.794	0.38	0.91	1	
28.051	6 Sandy Silt to Clayey Silt	60.56	2.01	1.668		1.668	2.57	0.76709	0.93	0.121	2.570	0.77	46.45	3.148	146.237	0.37	0.90	1	
28.707	6 Sandy Silt to Clayey Silt	108.334	3.193	1.706		1.706	2.37	0.75705	0.93	0.120	2.370	0.76	82.01	2.194	179.944		0.88	1	
29.364	7 Silty Sand to Sandy Silt	209.638	6.118	1.744		1.744	2.19	0.74728	0.92	0.120	2.190	0.75	156.66	1.643	257.332		0.84	1	
30.02	11 Sitff Fine-Grained	74.321	3.904	1.787		1.787	2.7	0.73652	0.92	0.120							0.89	1	
30.676	4 Silty Clay to Clay	30.963	1.451	1.825	0.021	1.804	2.96	0.73236	0.92	0.121							0.89	1	
31.332	4 Silty Clay to Clay	23.836	1.054	1.862	0.041	1.821	3.05	0.72824	0.91	0.121							0.89	1	
31.988	4 Silty Clay to Clay	21.403	0.87	1.9	0.062	1.838	3.07	0.72416	0.91	0.122							0.89	1	
32.644	3 Clay	15.717	0.68	1.937	0.082	1.854	3.22	0.72037	0.91	0.123							0.88	1	
33.301	4 Silty Clay to Clay	23.621	0.924	1.974	0.103	1.871	3.03	0.71638	0.90	0.124							0.88	1	
33.957	6 Sandy Silt to Clayey Silt	35.732	1.052	2.012	0.123	1.889	2.8	0.7122	0.90	0.124							0.88	1	
34.613	6 Sandy Silt to Clayey Silt	34.569	0.951	2.049	0.144	1.906	2.8	0.70831	0.89	0.125							0.88	1	
35.269	7 Silty Sand to Sandy Silt	39.094	0.539	2.088	0.164	1.924	2.59	0.70423	0.89	0.125	2.590	0.70	27.53	3.266	89.917	0.15	0.88	1	1.40
35.925	7 Silty Sand to Sandy Silt	37.739	0.546	2.127	0.185	1.942	2.62	0.70019	0.88	0.126							0.88	1	
36.581	7 Silty Sand to Sandy Silt	41.696	0.532	2.165	0.205	1.96	2.55	0.6962	0.88	0.126	2.550	0.70	29.03	3.034	88.080	0.14	0.87	1	1.36
37.238	7 Silty Sand to Sandy Silt	40.568	0.435	2.204	0.226	1.978	2.53	0.69226	0.87	0.127	2.530	0.69	28.08	2.925	82.138	0.13	0.87	1	1.24
37.894	7 Silty Sand to Sandy Silt	37.767	0.44	2.243	0.246	1.997	2.58	0.68815	0.87	0.127	2.580	0.69	25.99	3.206	83.333	0.13	0.87	1	1.26
38.55	7 Silty Sand to Sandy Silt	45.393	0.534	2.281	0.267	2.015	2.51	0.68429	0.86	0.127	2.510	0.68	31.06	2.820	87.580	0.14	0.87	1	1.34
39.206	7 Silty Sand to Sandy Silt	43.942	0.645	2.32	0.287	2.033	2.58	0.68048	0.86	0.127	2.580	0.68	29.90	3.206	95.879	0.16	0.87	1	1.52
39.862	7 Silty Sand to Sandy Silt	47.168	0.609	2.358	0.307	2.051	2.52	0.67671	0.85	0.127	2.520	0.68	31.92	2.872	91.660	0.15	0.87	1	1.42
40.518	7 Silty Sand to Sandy Silt	52.735	0.792	2.397	0.328	2.069	2.52	0.67299	0.85	0.127	2.520	0.67	35.49	2.872	101.914	0.18	0.86	1	1.67
41.175	8 Sand to Silty Sand	63.758	0.689	2.437	0.348	2.088	2.37	0.6691	0.84	0.128	2.370	0.67	42.66	2.194	93.599	0.16	0.86	1	1.46
41.831	7 Silty Sand to Sandy Silt	60.566	0.909	2.475	0.369	2.107	2.47	0.66526	0.83	0.127	2.470	0.67	40.29	2.621	105.618	0.19	0.86	1	1.77
42.487	8 Sand to Silty Sand	69.654	0.542	2.515	0.389	2.126	2.27	0.66146	0.83	0.127	2.270	0.66	46.07	1.856	85.515	0.14	0.86	1	1.29
43.143	8 Sand to Silty Sand	77.014	0.636	2.555	0.41	2.145	2.25	0.6577	0.82	0.127	2.250	0.66	50.65	1.798	91.085	0.15	0.86	1	1.41
43.799	8 Sand to Silty Sand	80.643	0.802	2.595	0.43	2.164	2.28	0.65398	0.82	0.127	2.280	0.65	52.74	1.886	99.475	0.17	0.86	1	1.61
44.455	8 Sand to Silty Sand	82.168	1.067	2.634	0.451	2.183	2.34	0.65031	0.81	0.127	2.340	0.65	53.43	2.084	111.345	0.21	0.86	1	1.96
45.112	8 Sand to Silty Sand	86.472	0.96	2.674	0.471	2.203	2.28	0.64649	0.80	0.127	2.280	0.65	55.90	1.886	105.443	0.19	0.85	1	1.78
45.768	8 Sand to Silty Sand	77.995	0.651	2.714	0.492	2.222	2.26	0.6429	0.80	0.126	2.260	0.64	50.14	1.827	91.600	0.15	0.85	1	1.43
46.424	8 Sand to Silty Sand	77.133	0.605	2.753	0.512	2.241	2.25	0.63935	0.79	0.126	2.250	0.64	49.31	1.798	88.681	0.14	0.85	1	1.37
47.08	8 Sand to Silty Sand	78.811	0.721	2.793	0.533	2.26	2.28	0.63584	0.78	0.126	2.280	0.64	50.11	1.886	94.518	0.16	0.85	1	1.50
47.736	8 Sand to Silty Sand	76.798	0.912	2.833	0.553	2.279	2.36	0.63237	0.78	0.125	2.360	0.63	48.56	2.156	104.725	0.19	0.85	1	1.78
48.392	9 Sand	132.267	0.804	2.873	0.574	2.3	2	0.62857	0.77	0.125	2.000	0.63	83.14	1.300	108.081	0.20	0.82	1	1.89
49.049	9 Sand	138.839	0.626	2.914	0.594	2.32	1.92	0.625	0.76	0.124	1.920	0.63	86.77	1.208	104.847	0.19	0.82	1	1.79
49.705	9 Sand	167.398	0.652	2.955	0.615	2.34	1.82	0.62147	0.76	0.124	1.820	0.62	104.03	1.122	116.674	0.23	0.79	1	2.19
50.361	7 Silty Sand to Sandy Silt	78.84	1.464	2.993	0.635	2.358	2.47	0.61832	0.75	0.124	2.470	0.62	48.75	2.621	127.786	0.27	0.84	1	2.64
51.017	6 Sandy Silt to Clayey Silt	28.315	0.801	3.031	0.655	2.376	2.96	0.61521	0.74	0.123							0.84	1	
51.673	6 Sandy Silt to Clayey Silt	26.002	0.716	3.069	0.676	2.393	2.98	0.6123	0.74	0.123							0.84	1	
52.329	6 Sandy Silt to Clayey Silt	28.763	0.678	3.106	0.696	2.41	2.89	0.60942	0.73	0.122							0.84	1	
52.986	6 Sandy Silt to Clayey Silt	31.938	0.958	3.144	0.717	2.427	2.92	0.60656	0.72	0.122							0.84	1	
53.642	6 Sandy Silt to Clayey Silt	36.032	0.971	3.181	0.737	2.444	2.85	0.60373	0.72	0.121							0.84	1	
54.298	6 Sandy Silt to Clayey Silt	42.065	1.271	3.219	0.758	2.461	2.82	0.60093	0.71	0.121							0.84	1	
54.954	5 Clayey Silt to Silty Clay	38.714	1.492	3.257	0.778	2.478	2.94	0.59815	0.70	0.120							0.83	1	
55.61	5 Clayey Silt to Silty Clay	34.24	1.287	3.294	0.799	2.495	2.99	0.5954	0.70	0.120							0.83	1	
56.266	6 Sandy Silt to Clayey Silt	33.475	0.989	3.332	0.819	2.512	2.93	0.59267	0.69	0.119							0.83	1	
56.923	5 Clayey Silt to Silty Clay	25.418	0.712	3.369	0.84	2.53	3.04	0.58981	0.69	0.119							0.83	1	
57.579	6 Sandy Silt to Clayey Silt	23.836	0.623	3.407	0.86	2.547	3.05	0.58714	0.68	0.118							0.83	1	
58.235	5 Clayey Silt to Silty Clay	28.287	0.893	3.445	0.881	2.564	3.03	0.58448	0.67	0.118							0.83	1	
58.891	5 Clayey Silt to Silty Clay	28.684	0.868	3.482	0.901	2.581	3.02	0.58186	0.67	0.117							0.83	1	
59.547	5 Clayey Silt to Silty Clay	36.599	1.268	3.52	0.922	2.598	2.95	0.57925	0.66	0.117							0.83	1	
60.203	5 Clayey Silt to Silty Clay	36.049	1.169	3.557	0.942	2.615	2.94	0.57667	0.66	0.116							0.83	1	
60.86	5 Clayey Silt to Silty Clay	37.115	1.233	3.595	0.963	2.632	2.94	0.57411	0.65	0.116							0.82	1	
61.516	5 Clayey Silt to Silty Clay	26.529	0.834	3.633	0.983	2.649	3.08	0.57158	0.65	0.115							0.82	1	
62.172	6 Sandy Silt to Clayey Silt	25.741	0.681	3.67	1.004	2.667	3.03	0.56892	0.64	0.115							0.82	1	
62.828	6 Sandy Silt to Clayey Silt	32.16	0.928	3.708	1.024	2.684	2.96	0.56643	0.64	0.114							0.82	1	
63.484	6 Sandy Silt to Clayey Silt	28.395	0.83	3.745	1.044	2.701	3.02	0.56396	0.63	0.114							0.82	1	
64.14	6 Sandy Silt to Clayey Silt	36.191	1.082	3.783	1.065	2.718	2.93	0.56151	0.63	0.114							0.82	1	
64.797	5 Clayey Silt to Silty Clay	32.233	0.98	3.82	1.085	2.735	3	0.55909	0.62	0.113							0.82	1	
65.453	5 Clayey Silt to Silty Clay	20.53	0.58	3.858	1.106	2.752	3.18	0.55668	0.62	0.113							0.82	1	
66.109	6 Sandy Silt to Clayey Silt	39.627	1.039	3.896	1.126	2.769	2.87	0.5543	0.61	0.112							0.82	1	
66.765	6 Sandy Silt to Clayey Silt	52.991	1.638	3.933	1.147	2.786	2.8	0.55193	0.61	0.112							0.81	1	
67.421	5 Clayey Silt to Silty Clay	43.454	1.598	3.971	1.167	2.804	2.93	0.54945	0.61	0.112							0.81	1	
68.077	5 Clayey Silt to Silty Clay	38.362	1.44	4.008	1.188	2.821	2.99	0.54713	0.60	0.111							0.81	1	
68.734	5 Clayey Silt to Silty Clay	38.34	1.539	4.046	1.208	2.838	3.02	0.54482	0.60	0.111							0.81	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-WEST ALPINE AVENUE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0191)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-AA-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
69.39	5	Clayey Silt to Silty Clay	38.322	1.494	4.084	1.229	2.855	3.02	0.54254	0.60	0.111							0.81	1	
70.046	5	Clayey Silt to Silty Clay	34.864	1.297	4.121	1.249	2.872	3.05	0.54028	0.59	0.110							0.81	1	
70.702	4	Silty Clay to Clay	23.32	0.957	4.159	1.27	2.889	3.26	0.53803	0.59	0.110							0.81	1	
71.358	5	Clayey Silt to Silty Clay	22.061	0.688	4.196	1.29	2.906	3.22	0.5358	0.58	0.110							0.81	1	
72.014	6	Sandy Silt to Clayey Silt	38.935	0.746	4.234	1.311	2.923	2.84	0.53359	0.58	0.109							0.81	1	
72.671	6	Sandy Silt to Clayey Silt	30.889	0.737	4.272	1.331	2.941	3	0.53127	0.58	0.109							0.81	1	
73.327	6	Sandy Silt to Clayey Silt	17.695	0.395	4.309	1.352	2.958	3.22	0.5291	0.58	0.109							0.81	1	
73.983	6	Sandy Silt to Clayey Silt	17.219	0.371	4.347	1.372	2.975	3.22	0.52695	0.57	0.109							0.80	1	
74.639	6	Sandy Silt to Clayey Silt	16.42	0.348	4.384	1.393	2.992	3.23	0.52481	0.57	0.108							0.80	1	
75.295	6	Sandy Silt to Clayey Silt	19.096	0.398	4.422	1.413	3.009	3.16	0.52269	0.57	0.108							0.80	1	
75.951	6	Sandy Silt to Clayey Silt	21.114	0.434	4.46	1.433	3.026	3.11	0.52059	0.56	0.108							0.80	1	
76.608	6	Sandy Silt to Clayey Silt	21.432	0.448	4.497	1.454	3.043	3.11	0.5185	0.56	0.108							0.80	1	
77.264	6	Sandy Silt to Clayey Silt	18.314	0.354	4.535	1.474	3.06	3.18	0.51643	0.56	0.108							0.80	1	
77.92	6	Sandy Silt to Clayey Silt	16.091	0.272	4.572	1.495	3.077	3.21	0.51438	0.56	0.107							0.80	1	
78.576	6	Sandy Silt to Clayey Silt	15.348	0.269	4.61	1.515	3.095	3.24	0.51222	0.55	0.107							0.80	1	
79.232	6	Sandy Silt to Clayey Silt	19.011	0.399	4.648	1.536	3.112	3.2	0.5102	0.55	0.107							0.80	1	
79.888	6	Sandy Silt to Clayey Silt	17.73	0.428	4.685	1.556	3.129	3.26	0.5082	0.55	0.107							0.80	1	
80.545	6	Sandy Silt to Clayey Silt	28.298	0.82	4.723	1.577	3.146	3.09	0.50621	0.55	0.107							0.80	1	
81.201	5	Clayey Silt to Silty Clay	39.922	1.412	4.76	1.597	3.163	3.01	0.50424	0.54	0.106							0.79	1	
81.857	5	Clayey Silt to Silty Clay	40.398	1.467	4.798	1.618	3.18	3.02	0.50228	0.54	0.106							0.79	1	
82.513	5	Clayey Silt to Silty Clay	41.353	1.495	4.836	1.638	3.197	3.01	0.50034	0.54	0.106							0.79	1	
83.169	5	Clayey Silt to Silty Clay	35.675	1.363	4.873	1.659	3.214	3.09	0.49841	0.54	0.106							0.79	1	
83.825	6	Sandy Silt to Clayey Silt	40.245	1.235	4.911	1.679	3.232	2.98	0.49639	0.54	0.106							0.79	1	
84.482	6	Sandy Silt to Clayey Silt	40.358	1.3	4.948	1.7	3.249	3	0.49449	0.53	0.106							0.79	1	
85.138	6	Sandy Silt to Clayey Silt	38.595	1.217	4.986	1.72	3.266	3.01	0.49261	0.53	0.106							0.79	1	
85.794	6	Sandy Silt to Clayey Silt	55.446	1.759	5.023	1.741	3.283	2.86	0.49074	0.53	0.105							0.79	1	
86.45	6	Sandy Silt to Clayey Silt	60.413	2.08	5.061	1.761	3.3	2.86	0.48889	0.53	0.105							0.79	1	
87.106	5	Clayey Silt to Silty Clay	46.64	1.758	5.099	1.781	3.317	2.99	0.48705	0.53	0.105							0.79	1	
87.762	6	Sandy Silt to Clayey Silt	74.445	2.321	5.136	1.802	3.334	2.76	0.48522	0.52	0.105							0.79	1	
88.419	8	Sand to Silty Sand	155.321	2.119	5.176	1.822	3.354	2.28	0.48309	0.52	0.105	2.280	0.48	75.03	1.886	141.527	0.34	0.77	1	3.91
89.075	9	Sand	179.253	1.037	5.217	1.843	3.374	2.02	0.48098	0.52	0.105	2.020	0.48	86.22	1.327	114.411	0.22	0.75	1	2.49
89.731	9	Sand	205.488	1.533	5.257	1.863	3.394	2.03	0.47889	0.52	0.105	2.030	0.48	98.41	1.341	131.982	0.29	0.73	1	3.35
90.387	9	Sand	192.096	1.781	5.298	1.884	3.414	2.11	0.47681	0.52	0.105	2.110	0.48	91.59	1.473	134.932	0.31	0.74	1	3.52
91.043	9	Sand	177.615	1.45	5.339	1.904	3.435	2.11	0.47465	0.52	0.104	2.110	0.47	84.30	1.473	124.195	0.26	0.75	1	2.95
91.699	9	Sand	232.204	1.825	5.38	1.925	3.455	2	0.47261	0.52	0.104	2.000	0.47	109.74	1.300	142.665	0.35	0.71	1	4.00
92.356	9	Sand	258.167	2.048	5.42	1.945	3.475	1.96	0.47059	0.51	0.104	1.960	0.47	121.49	1.251	151.997	0.41	0.70	1	4.65
93.012	7	Silty Sand to Sandy Silt	82.61	2.092	5.459	1.966	3.493	2.68	0.46878	0.51	0.104							0.78	1	
93.668	6	Sandy Silt to Clayey Silt	61.161	2.059	5.497	1.986	3.51	2.87	0.46709	0.51	0.104							0.78	1	
94.324	5	Clayey Silt to Silty Clay	59.488	2.174	5.534	2.007	3.527	2.9	0.46541	0.51	0.104							0.78	1	
94.98	6	Sandy Silt to Clayey Silt	54.164	1.728	5.572	2.027	3.545	2.89	0.46365	0.51	0.104							0.78	1	
95.636	6	Sandy Silt to Clayey Silt	64.319	2.233	5.609	2.048	3.562	2.86	0.46199	0.51	0.104							0.78	1	
96.293	6	Sandy Silt to Clayey Silt	75.12	2.332	5.647	2.068	3.579	2.77	0.46035	0.51	0.104							0.77	1	
96.949	6	Sandy Silt to Clayey Silt	68.577	2.386	5.685	2.089	3.596	2.85	0.45872	0.50	0.104							0.77	1	
97.605	5	Clayey Silt to Silty Clay	58.706	2.191	5.722	2.109	3.613	2.93	0.4571	0.50	0.104							0.77	1	
98.261	6	Sandy Silt to Clayey Silt	52.713	1.807	5.76	2.13	3.63	2.94	0.45549	0.50	0.104							0.77	1	
98.917	6	Sandy Silt to Clayey Silt	53.654	1.912	5.797	2.15	3.647	2.94	0.45389	0.50	0.103							0.77	1	
99.573	6	Sandy Silt to Clayey Silt	56.007	1.706	5.835	2.17	3.664	2.88	0.4523	0.50	0.103							0.77	1	
100.23	6	Sandy Silt to Clayey Silt	70.839	2.45	5.872	2.191	3.682	2.83	0.45063	0.50	0.103							0.77	2	
100.886	5	Clayey Silt to Silty Clay	75.001	3.007	5.91	2.211	3.699	2.88	0.44907	0.50	0.103							0.77	3	
101.542	4	Silty Clay to Clay	48.182	2.158	5.948	2.232	3.716	3.08	0.44752	0.50	0.103							0.77	4	
102.198	4	Silty Clay to Clay	32.222	1.37	5.985	2.252	3.733	3.23	0.44598	0.49	0.103							0.77	5	
102.854	6	Sandy Silt to Clayey Silt	52.849	1.657	6.023	2.273	3.75	2.93	0.44444	0.49	0.103							0.77	6	
103.511	6	Sandy Silt to Clayey Silt	78.487	2.656	6.06	2.293	3.767	2.81	0.44292	0.49	0.103							0.77	7	
104.167	6	Sandy Silt to Clayey Silt	110.001	3.368	6.098	2.314	3.784	2.67	0.44141	0.49	0.103							0.77	8	
104.823	6	Sandy Silt to Clayey Silt	101.99	3.255	6.136	2.334	3.801	2.7	0.43991	0.49	0.103							0.77	9	
105.479	9	Sand	170.958	1.541	6.176	2.355	3.822	2.18	0.43807	0.49	0.103	2.180	0.44	74.89	1.619	121.262	0.25	0.76	10	2.85
106.135	9	Sand	179.208	1.121	6.217	2.375	3.842	2.09	0.43633	0.49	0.103	2.090	0.44	78.19	1.437	112.360	0.21	0.76	11	2.46
106.791	8	Sand to Silty Sand	103.333	1.148	6.257	2.396	3.861	2.44	0.4347	0.49	0.103	2.440	0.43	44.92	2.483	111.539	0.21	0.76	12	2.43
107.448	6	Sandy Silt to Clayey Silt	47.247	1.322	6.294	2.416	3.878	2.96	0.43324	0.49	0.103							0.76	13	
108.104	6	Sandy Silt to Clayey Silt	63.264	1.985	6.332	2.437	3.895	2.87	0.4318	0.49	0.103							0.76	14	
108.76	6	Sandy Silt to Clayey Silt	54.822	1.726	6.37	2.457	3.912	2.92	0.43036	0.48	0.103							0.76	15	
109.416	6	Sandy Silt to Clayey Silt	54.612	1.654	6.407	2.478	3.93	2.91	0.42885	0.48	0.102							0.76	16	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Grain Size Classification

Grain size classification test (ASTM Test Method D 420) was performed on selected sample of granular soil to aid in the classification. The result is presented on Plate B-2, "Grain Size Distribution Curves".

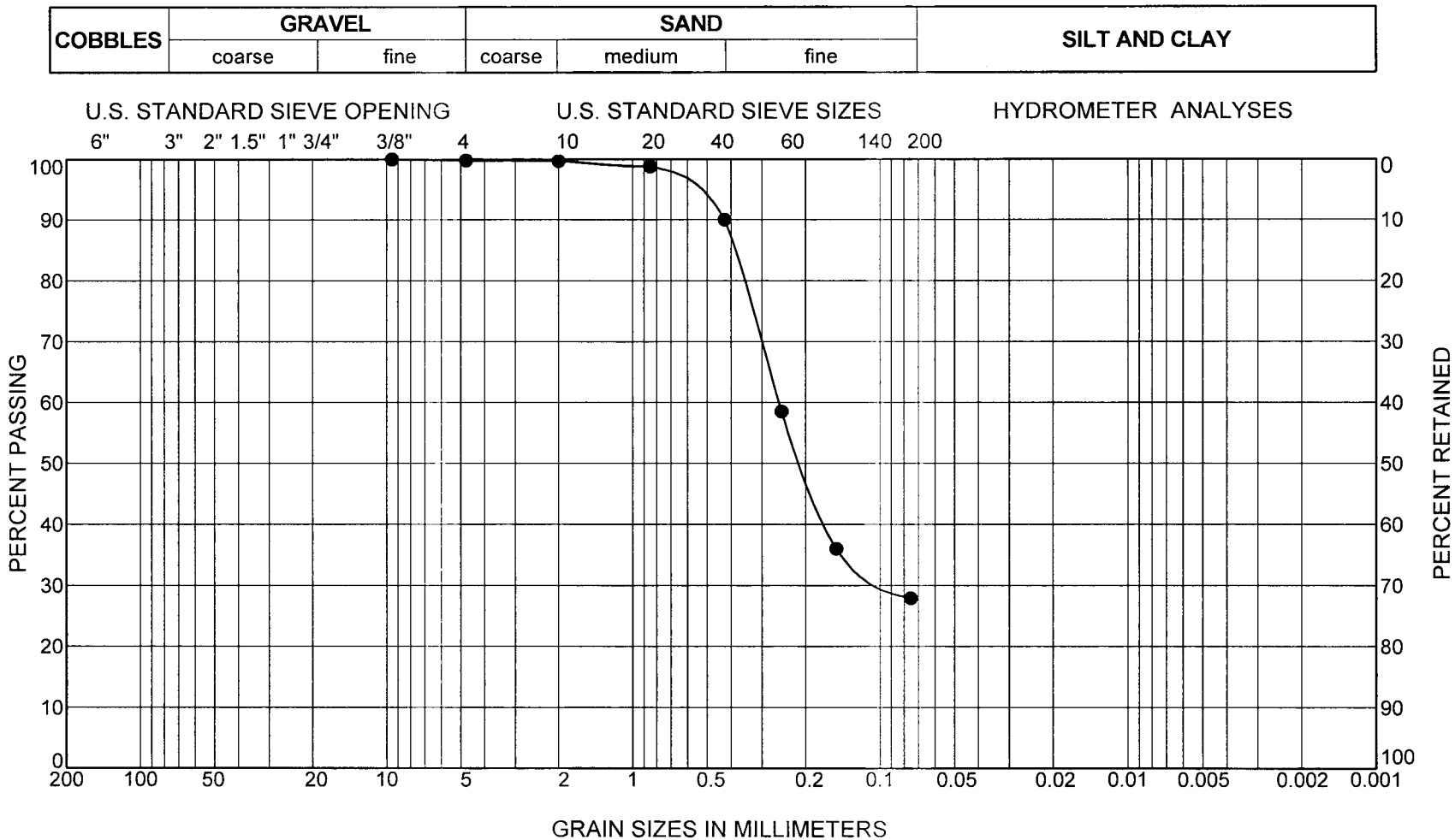
Unconfined Compression Test

Strength tests were performed on selected undisturbed samples using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-4A through B-4C.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-3.

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
R09-AA-001	MC-3	16.0	●			SILTY SAND (SM)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

JOB NO: 206127.BR (WAB)

Alpine Avenue UC (Widen)
Stockton, California

PLATE NO: B-2

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/29/2009
Date Submitted 04/24/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR Site ID : R-09-AA-001#6@31.0'
Thank you for your business.

* For future reference to this analysis please use SUN # 55554-111856.

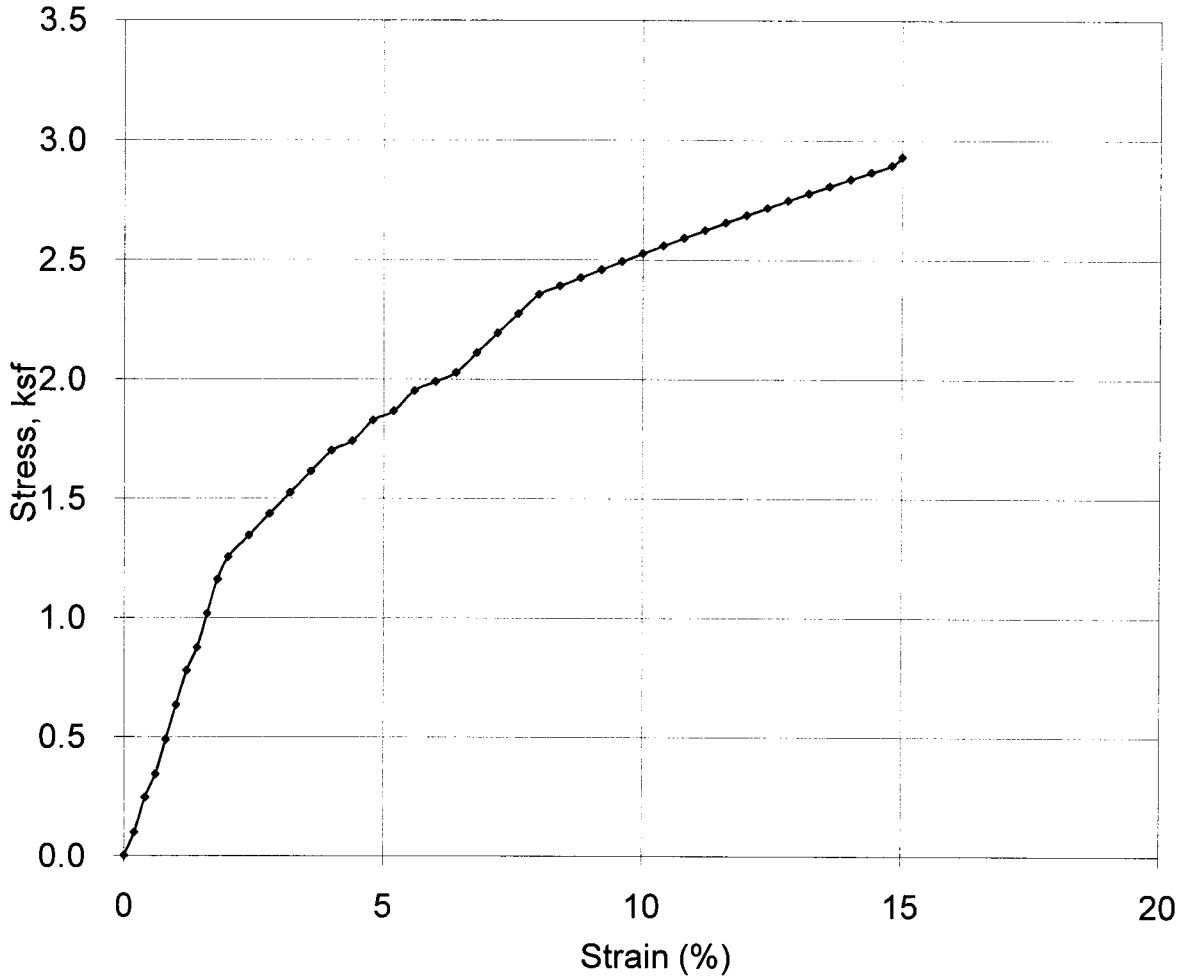
EVALUATION FOR SOIL CORROSION

Soil pH	6.83	
Minimum Resistivity	1.47 ohm-cm (x1000)	
Chloride	60.8 ppm	00.00608 %
Sulfate	47.6 ppm	00.00476 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 1 2009
By

UNCONFINED COMPRESSION TEST



Boring No.: R-09-AA-001

Sample No. : MC-5

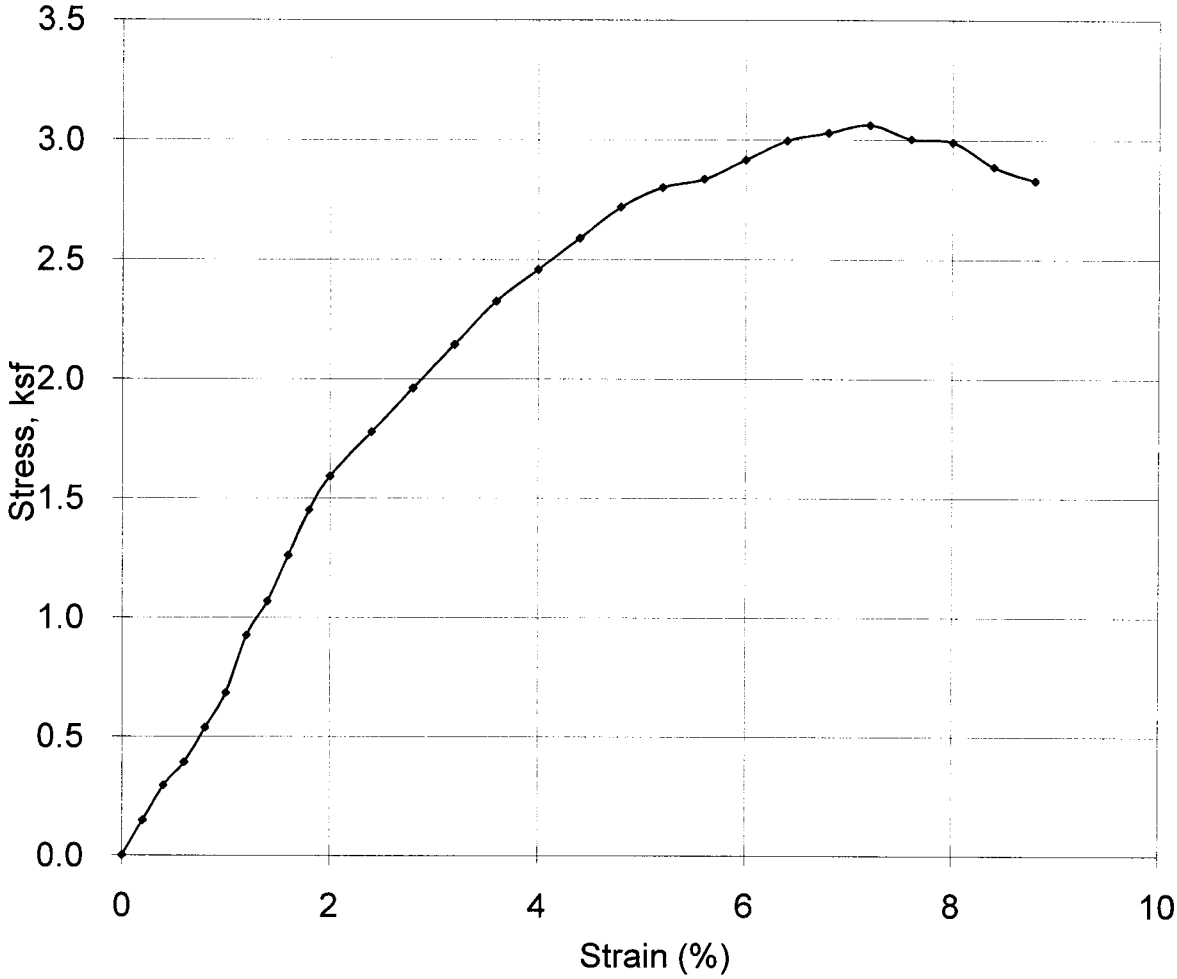
Depth (feet): 26

Material Description:
Sandy Lean Clay

Unconfined Compressive Strength: 2.9 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**ALPINE AVENUE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-4A

UNCONFINED COMPRESSION TEST



Boring No.: R-09-AA-001

Sample No. : MC-6B

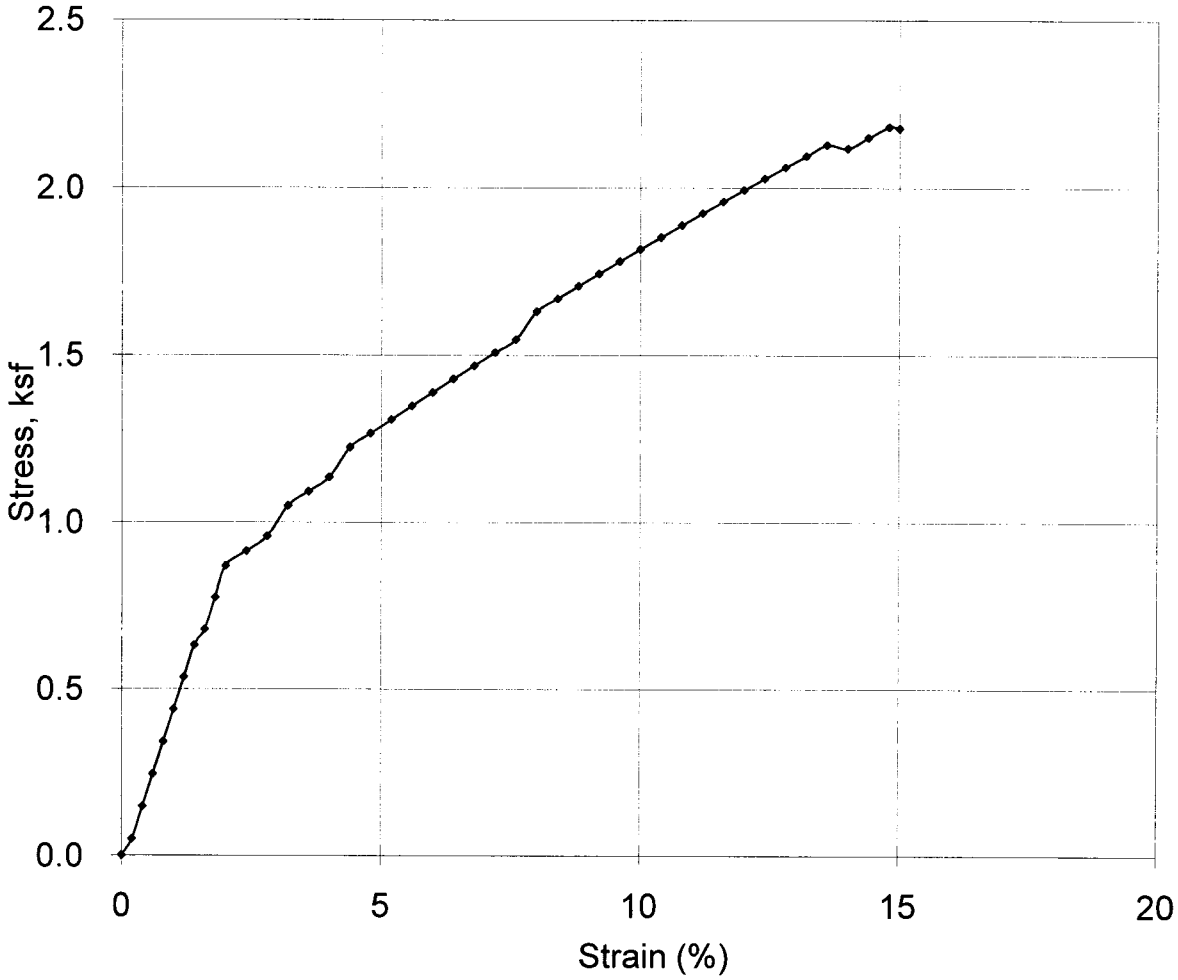
Depth (feet): 31

Material Description:
Silt with Sand

Unconfined Compressive Strength: 3.0 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**ALPINE AVENUE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-4B

UNCONFINED COMPRESSION TEST



Boring No.: R-09-AA-001

Sample No. : MC-13

Depth (feet): 71

Material Description:
Silty Clay

Unconfined Compressive Strength: 2.2 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**ALPINE AVENUE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-4C

APPENDIX C

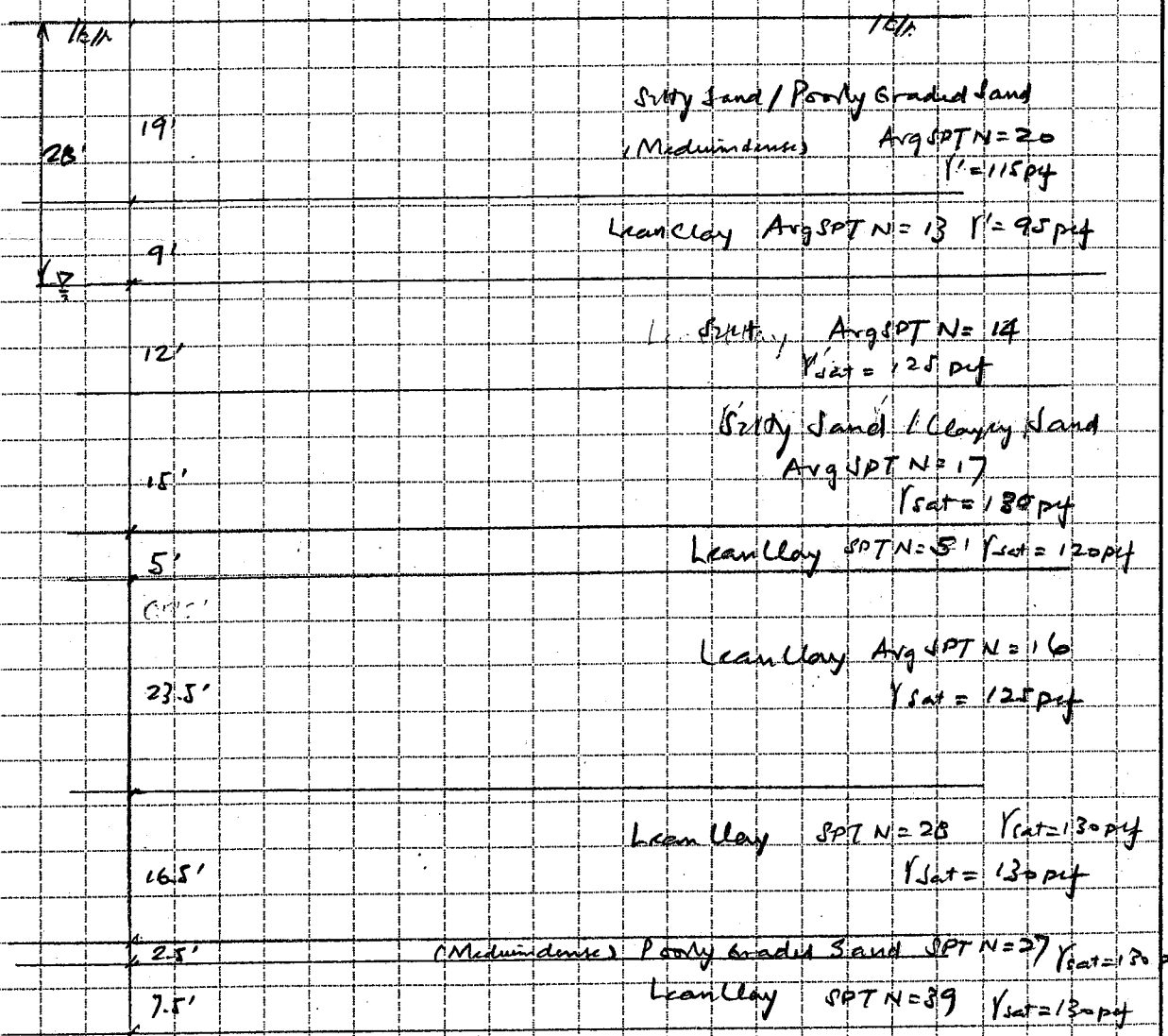
Evaluation of Liquefaction Potential

PARIKH

Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement
SHEET NO ____ OF ____
CALCULATED BY A Lam DATE 8/2009
CHECKED BY ____ DATE ____
SCALE ____

Evaluation of Liquefaction Potential ← Alpine Area (Boring A-09-AA-001)



LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-AA-001 (Alpine Avenue UC) (Widen)***

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

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} (g)= 0.2
FAULT M_w = 7.0

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 28

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=($CRR_{7.0}$/CSR)*MSF*Kσ*Kα*

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE ($CRR_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	19.0	1	20	SPT	2185.0	2185.0	0.91	0.12	23	0.96	1	0.85	1.2	1.15	25.90	28%	34.04		0.97	1	
2	28.0	2	13	SPT	3040.0	3040.0	0.86		15	0.81	1	0.80	1.2	1.15	13.43	55%			0.88	1	
3	40.0	2	14	SPT	4540.0	3850.0	0.75		16	0.72	1	0.80	1.2	1.15	12.85	55%			0.82	1	
4	55.0	1	17	SPT	6490.0	4862.5	0.61	0.11	20	0.64	1	0.80	1.2	1.15	13.89	20%	18.61	0.20	0.77	1	1.71
5	60.0	2	5	SPT	7090.0	5150.0	0.58		6	0.62	1	0.80	1.2	1.15	3.97	55%			0.75	1	
6	83.5	2	16	SPT	10145.0	6736.3	0.49		18	0.54	1	0.80	1.2	1.15	11.11	55%			0.69	1	
7	100.0	2	28	SPT	12290.0	7850.0	0.47		32	0.50	1	0.80	1.2	1.15	18.00	55%			0.66	1	
8	102.5	1	27	SPT	12615.0	8018.8	0.47	0.10	31	0.50	1	0.80	1.2	1.15	17.18	15%	20.50	0.22	0.66	1	1.83
9	110.0	2	39	SPT	13590.0	8525.0	0.46		45	0.48	1	0.80	1.2	1.15	24.06	55%			0.65	1	

Vertical Pile Capacity Calculations/Lateral Pile Capacity Analyses

Rajappan & Meyer
CONSULTING ENGINEERS INC

Project Name: *I-5 North Stockton Corridor Improvement*
Bridge Name: *Alpine Ave UC (Widen)*
Br No: *29-0191 R/L*
Date: *April 7, 2010*

FOUNDATION DESIGN INFORMATION FROM BRIDGE DESIGNER TO GEOTECHNICAL ENGINEER

ALPINE AVE UC (WIDEN)

Table 1. Foundation Design Data Sheet

Location	Design Method	Pile Type	Finish Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		No. of Piles Per Support
					B	L	
Abut 1	WSD	Class 140 (Mod) Alt X	12.60±	9.40±	2.50	51.00	14
Abut 2	WSD	Class 140 (Mod) Alt X	12.80±	9.60±	2.50	51.00	14

Note: Finish Grade is at the front face of abutment diaphragm

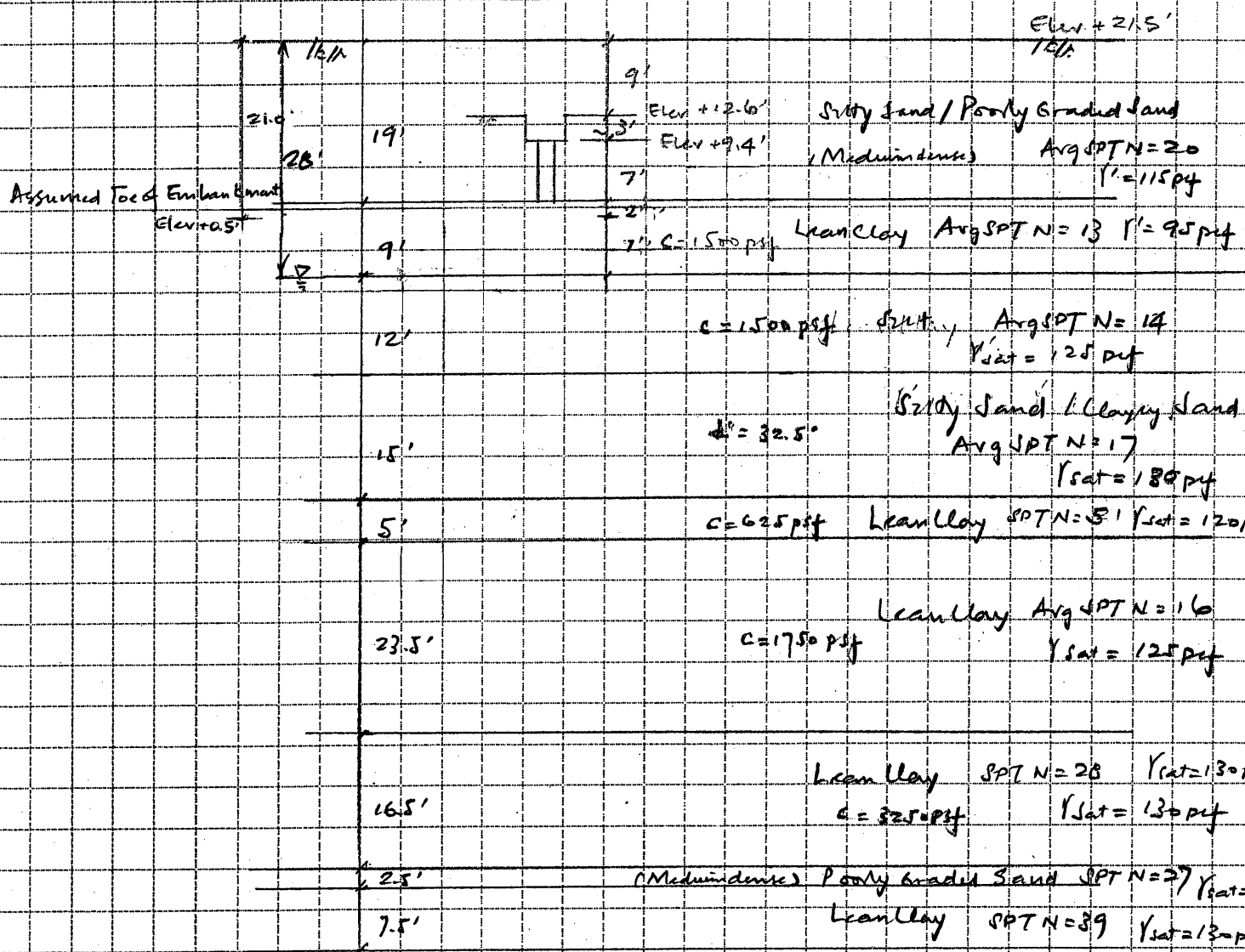
Table 2. Foundation Design Loads

Location	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent	Compression		Tension		Compression		Tension	
	Per Support	Max per Pile	Per Support	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile
Abut 1	1932	138	1368	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1932	138	1368	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 3. Lateral Load on Piles

Per Support	Service Limit (kips/pile)	Max per Pile
Abut 1	Negligible	28
Abut 2	Negligible	28

Pile Capacity Analysis - West Alpaugh Ave UC (Boring B-09-001) Abutment 1
14" □ precast concrete pile 138 tons service load



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Alpine Ave. UC (Widen)*** ***Abut 1 - 70 tons service load (140 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/06/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.17 — 1- GRAVELS, SANDS AND NONPLASTIC SILTS — (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 2 (1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 1 (1=CONCRETE ,2=STEEL) — SOIL TYPES — GRAVEL, SAND, NSILT Ref Boring : A-09-AA-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) : 4.50 SQ. FT.

DRIVEN/DRILLED = 1 (1=DRIVEN ,2=DRILLED) — CLAY, PSILT — AREA OF THE PILE TIP 1.35 SQ. FT.

Class 140 Alt. "X" - 14" square precast prestressed c

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	115	345	173	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	10.0		1	115	805	748	SAND	DENSE	0	38	0.00	17.15	17.15	5.75	5.75
10.0	12.0		2	95	190	1245	CLAY	STIFF	1500	0	0.80	5.40	22.55	5.40	11.15
12.0	19.0		2	95	665	1673	CLAY	STIFF	1500	0	0.80	18.90	41.45	18.90	30.05
19.0	25.0		2	63	378	2194	CLAY	STIFF	1500	0	0.80	16.20	57.65	16.20	46.25
25.0	31.0		2	63	378	2572	CLAY	STIFF	1500	0	0.80	16.20	73.85	16.20	62.45
31.0	36.0		1	68	340	2931	SAND	MED DEN	0	35	0.00	24.94	98.79	8.36	70.80
36.0	41.0		1	68	340	3271	SAND	MED DEN	0	35	0.00	24.76	123.55	8.29	79.09
41.0	46.0		1	68	340	3611	SAND	MED DEN	0	35	0.00	18.30	141.85	6.13	85.22
46.0	51.0		2	58	290	3926	CLAY	MEDIUM	625	0	1.00	7.03	148.88	7.03	92.26
51.0	56.5		2	63	346.5	4244	CLAY	STIFF	1750	0	0.95	20.57	169.45 ←	20.57	112.83
56.5	61.0		2	63	283.5	4559	CLAY	STIFF	1750	0	0.95	16.83	186.29	16.83	129.66
61.0	66.0		2	63	315	4859	CLAY	STIFF	1750	0	0.95	18.70	204.99	18.70	148.37
66.0	71.0		2	63	315	5174	CLAY	STIFF	1750	0	0.95	18.70	223.69	18.70	167.07
71.0	74.5		2	63	220.5	5441	CLAY	STIFF	1750	0	0.95	13.09	236.78	13.09	180.16
74.5	79.5		2	68	340	5722	CLAY	V.STIFF	3250	0	0.40	14.63	251.41	14.63	194.79
79.5	84.5		2	68	340	6062	CLAY	V.STIFF	3250	0	0.40	14.63	266.03	14.63	209.41
84.5	91.0		2	68	442	6453	CLAY	V.STIFF	3250	0	0.40	19.01	285.05	19.01	228.42
91.0	93.5		1	68	170	6759	SAND	MED DEN	0	35	0.00	12.41	297.46	4.16	232.58
93.5	101.0		2	68	510	7099	CLAY	V.STIFF	4000	0	0.40	27.00	324.46	27.00	259.58

Abutment 1 --> Pile cut-off elevation is +9.40 feet. Finished grade elevation is +12.60 feet

Pre-drill through the exsiting embankment to toe of embankment. Toe of existing embankment assumed to be Elev. +0.5 feet (assumed to the depth of 12.0 feet)

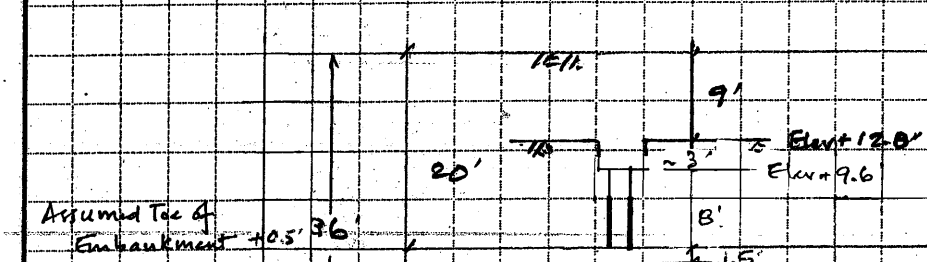
Pile Length @ Abutment 1 = 56.5' - 3.0' = 53.5'

Pile tip at 53.5' below pile cut-off (pile tip @ Elev. -44.0', pile cut-off @ Elev. +9.40'), Ult. Pile Capacity =169.5 tons (Friction)-22.6 tons (Predrilled)=146.9 tons > 280K/140T (Service Limit State), OK

I-5 Alpine Ave. Abut 1 Class 140 Alt X (4-06-10)

Sheet 2 of 5

Pile Capacity Analysis - Alpine Ave Ue (Widen) (CPT-09-001) Abutment 2
14" □ precast concrete pile. 137 tons Service load



Silty Sand $\phi' = 36°$ $\gamma' = 115$ pcf

5' $c = 2500$ psf Lean Clay $\gamma' = 115$ pcf

5' $c = 4000$ psf Lean Clay $\gamma' = 120$ pcf

6.5' $c = 2000$ psf Silt $\gamma' = 115$ pcf $\gamma_{sat} = 135$ pcf

14' $\phi' = 32.5°$ Silty Sand $\gamma_{sat} = 130$ pcf

21.5' $c = 2750$ psf Lean Clay $\gamma_{sat} = 130$ pcf

8.5' $c = 1750$ psf Lean Clay $\gamma_{sat} = 130$ pcf

7.5' $c = 4000$ psf Lean Clay $\gamma_{sat} = 130$ pcf

4.5' $\phi' = 35.5°$ Sand $\gamma_{sat} = 134$ pcf

17 $c = 4000$ psf Lean Clay $\gamma_{sat} = 130$ pcf

17.5'

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Alpine Ave. UC (Widen)*** ***Abut 2 - 70 tons service load (140 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/06/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.17 — 1- GRAVELS, SANDS (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 2 — AND NONPLASTIC SILTS

(1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 1 — SOIL TYPES

(1=CONCRETE ,2=STEEL) — GRAVEL, SAND, NSILT Ref Boring : CPT-09-AA-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

DRIVEN/DRILLED = 1 — CLAY, PSILT — AREA OF THE PILE TIP 1.35 SQ. FT.

(1=DRIVEN ,2=DRILLED)

Class 140 Alt. "X" - 14" square precast prestressed

DEPTH FROM	TO	SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0 BT2 FTG	1	125	375	188	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	11.0	1	125	1000	875	SAND	DENSE	0	36	0.00	21.11	21.11	7.07	7.07
11.0	12.5	2	120	180	1465	CLAY	V.STIFF	2500	0	0.40	3.38	24.48	3.38	10.45
12.5	16.0	2	120	420	1765	CLAY	V.STIFF	2500	0	0.40	7.88	32.36	7.88	18.32
16.0	21.0	2	120	600	2275	CLAY	V.STIFF	4000	0	0.25	11.25	43.61	11.25	29.57
21.0	27.0	2	115	690	2920	CLAY	STIFF	2000	0	0.65	17.55	61.16	17.55	47.12
27.0	27.5	2	63	31.5	3281	CLAY	STIFF	2000	0	0.65	1.46	62.62	1.46	48.58
27.5	32.5	1	68	340	3467	SAND	LOOSE	0	32.5	0.00	19.69	82.31	6.60	55.18
32.5	37.5	1	68	340	3807	SAND	MED DEN	0	32.5	0.00	19.74	102.05	6.61	61.79
37.5	41.5	1	68	272	4113	SAND	MED DEN	0	32.5	0.00	15.78	117.83	5.29	67.08
41.5	46.5	2	68	340	4419	CLAY	V.STIFF	2750	0	0.40	12.38	130.20	12.38	79.45
46.5	51.5	2	68	340	4759	CLAY	V.STIFF	2750	0	0.40	12.38	142.58	12.38	91.83
51.5	60.0	2	68	578	5218	CLAY	V.STIFF	2750	0	0.40	21.04	163.62	21.04	112.87
60.0	63.0	2	68	204	5609	CLAY	V.STIFF	2750	0	0.40	7.43	171.04 ←	7.43	120.29
63.0	65.0	2	68	136	5779	CLAY	STIFF	1750	0	0.75	5.91	176.95	5.91	126.20
65.0	71.5	2	68	442	6068	CLAY	STIFF	1750	0	0.75	19.20	196.14	19.20	145.39
71.5	75.5	2	68	272	6425	CLAY	V.STIFF	4000	0	0.40	14.40	210.54	14.40	159.79
75.5	79.0	2	68	238	6680	CLAY	V.STIFF	4000	0	0.40	12.60	223.14	12.60	172.39
79.0	83.5	1	73	328.5	6963	SAND	MED DEN	0	35.5	0.00	23.85	247.00	7.99	180.38
83.5	89.5	2	68	408	7331	CLAY	V.STIFF	4000	0	0.40	21.60	268.60	21.60	201.98
89.5	95.5	2	68	408	7739	CLAY	V.STIFF	4000	0	0.40	21.60	290.20	21.60	223.58
95.5	101.0	2	68	374	8130	CLAY	V.STIFF	4000	0	0.40	19.80	310.00	19.80	243.38

Abutment 2 --> Pile cut-off elevation is +9.60 feet. Finished grade elevation is +12.80 feet

Pre-drill through the exsiting embankment to toe of embankment. Toe of existing embankment assumed to be Elev. +0.5 feet (assumed to the depth of 12.5 feet)

Pile Length @ Abutment 1 = 63.0' - 3.0' = 60.0'

Pile tip at 60.0' below pile cut-off (pile tip @ Elev. -50.5', pile cut-off @ Elev. +9.60'), Ult. Pile Capacity =171.0 tons (Friction)-24.5 tons (Predrilled)=146.5 tons > 280K/140T (Service Limit State), OK

I-5 Alpine Ave. Abut 2 Class 140 Alt X (4-06-10)

Sheet 4 of 5

ALPINE AVE UC (WIDEN)

(Sheet 5 of 5)

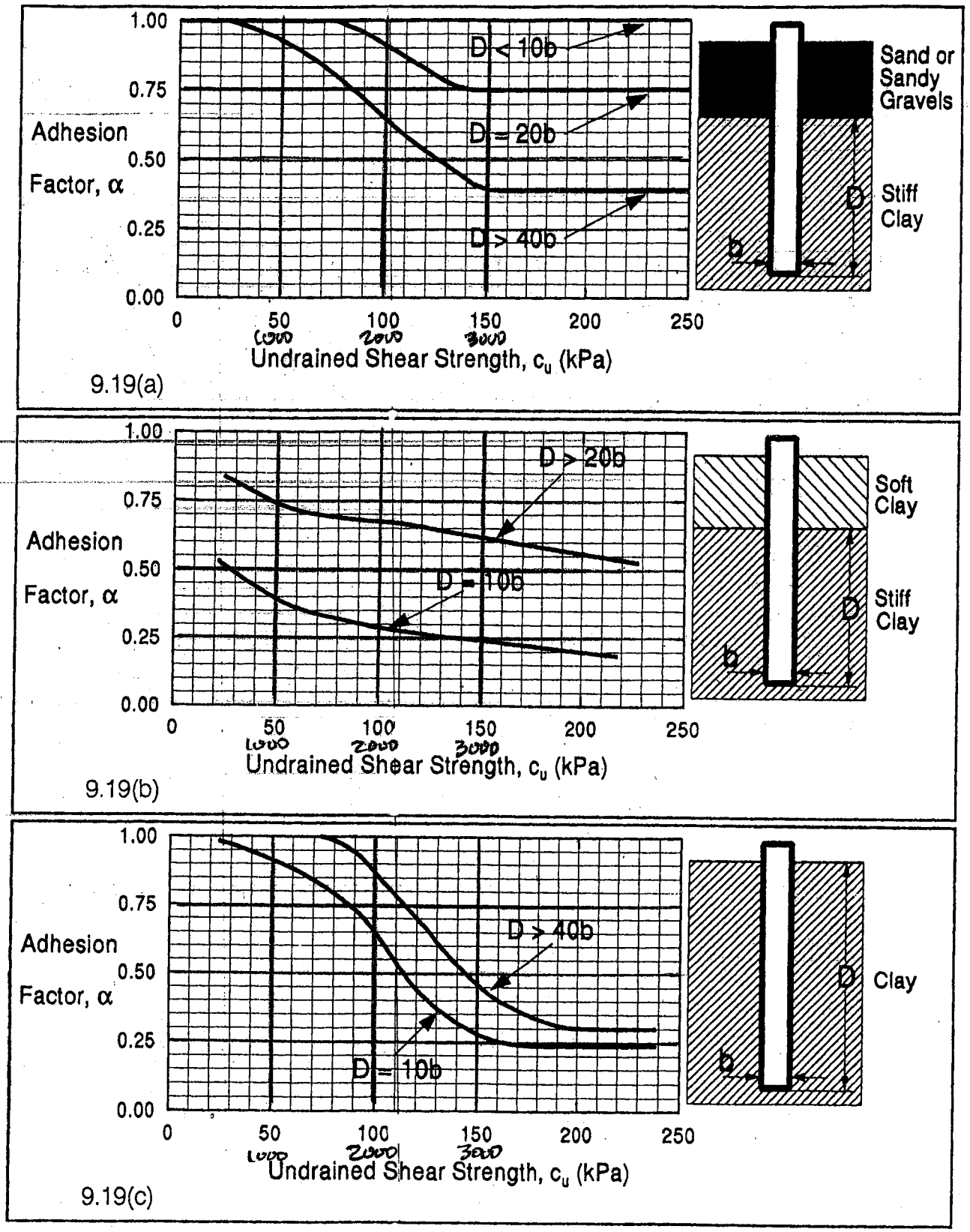
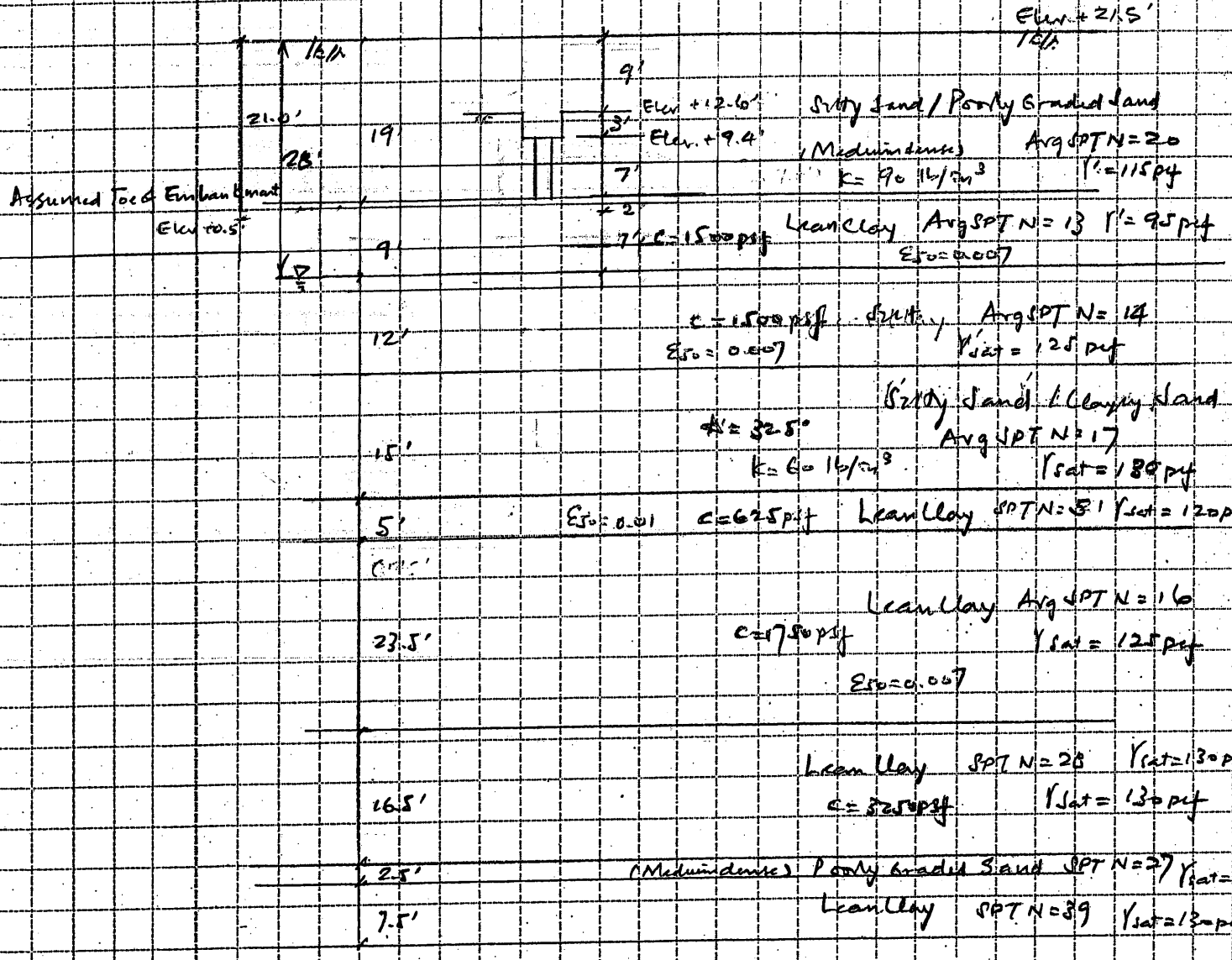


Figure 9.19 Adhesion Factors for Driven Piles in Clay (after Tomlinson, 1980)

Lateral Pile Capacity Analysis - Alpaw Arc LLC (Boring R-09-001) Abutment 1
14"□ precast concrete pile 138 tons service load



```
LPILEP5
I-5 Stockton-Alpine-Abut-1
1       1       0       0       0       0
100     2       -36     600     0
0       14      3112    194     3000000
600     14      3112    194     3000000
6       12      12      0       2
4       -36     84      90      90
3       84      192     0       0
3       192     336     0       0
4       336     516     60      60
3       516     576     0       0
3       576     858     0       0
-36     0.069
84      0.069
84      0.055
192     0.055
192     0.036
336     0.036
336     0.039
516     0.039
516     0.033
576     0.033
576     0.036
858     0.036
-36     0       35      0       0
84      0       35      0       0
84      10.4    0       0.007   0
192     10.4    0       0.007   0
192     10.4    0       0.007   0
336     10.4    0       0.007   0
336     0       32      0       0
516     0       32      0       0
516     4.34    0       0.01    0
576     4.34    0       0.01    0
576     12.15   0       0.007   0
858     12.15   0       0.007   0
0       0.6     1
1248    0.6     1
0       1       1
1
1       28000   0       128000
0
1       1       0
100     1E-5    100
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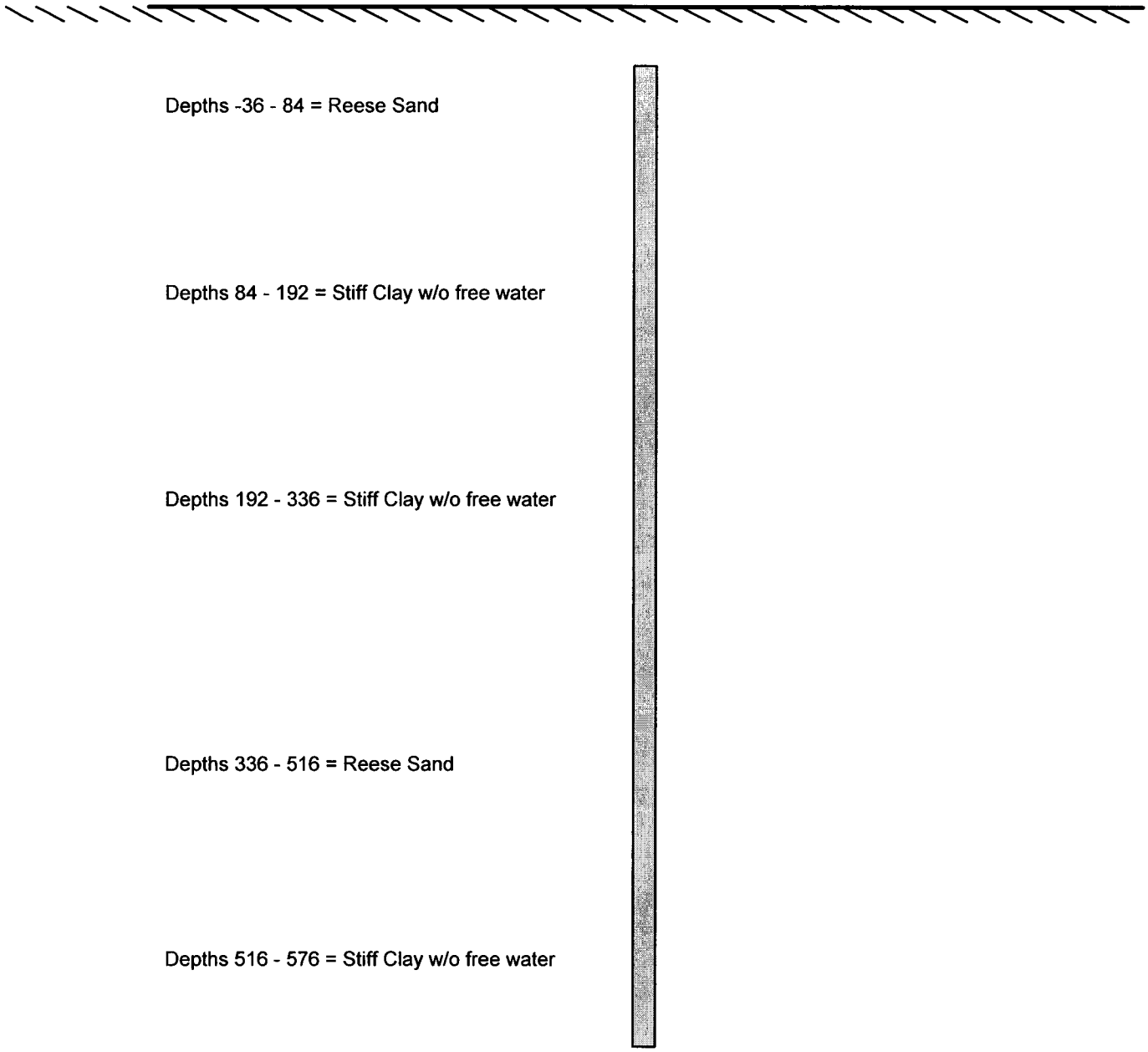
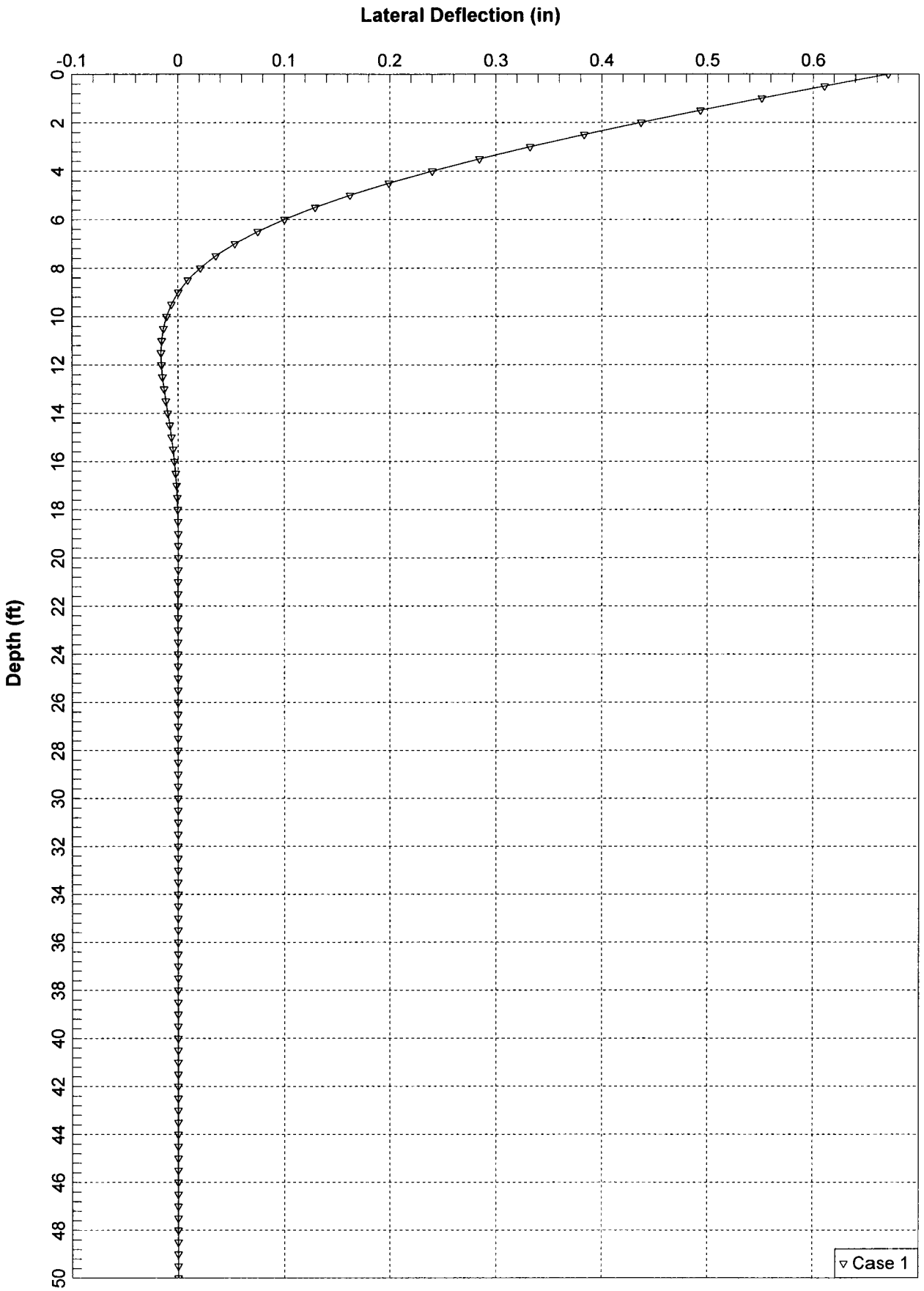
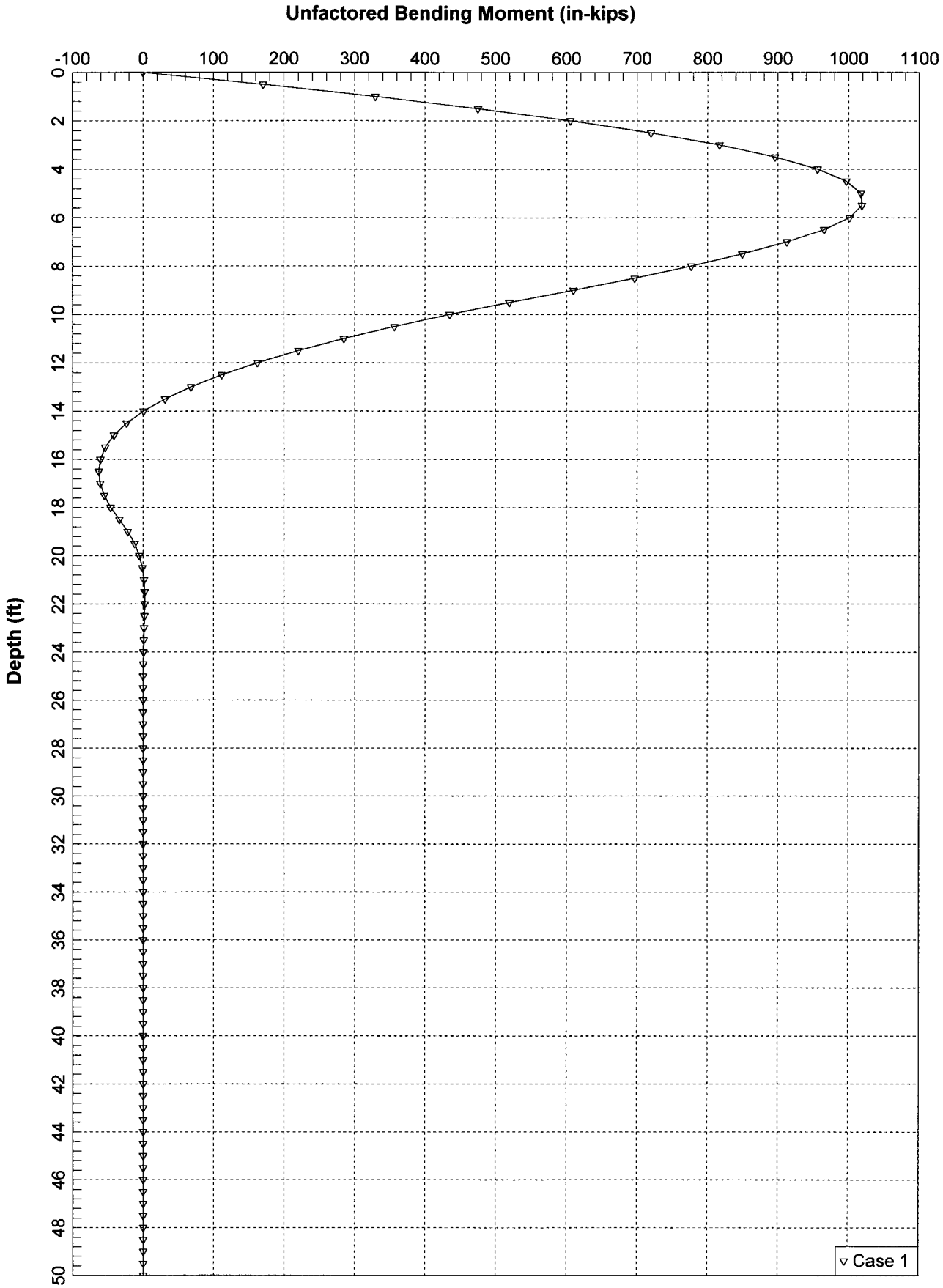
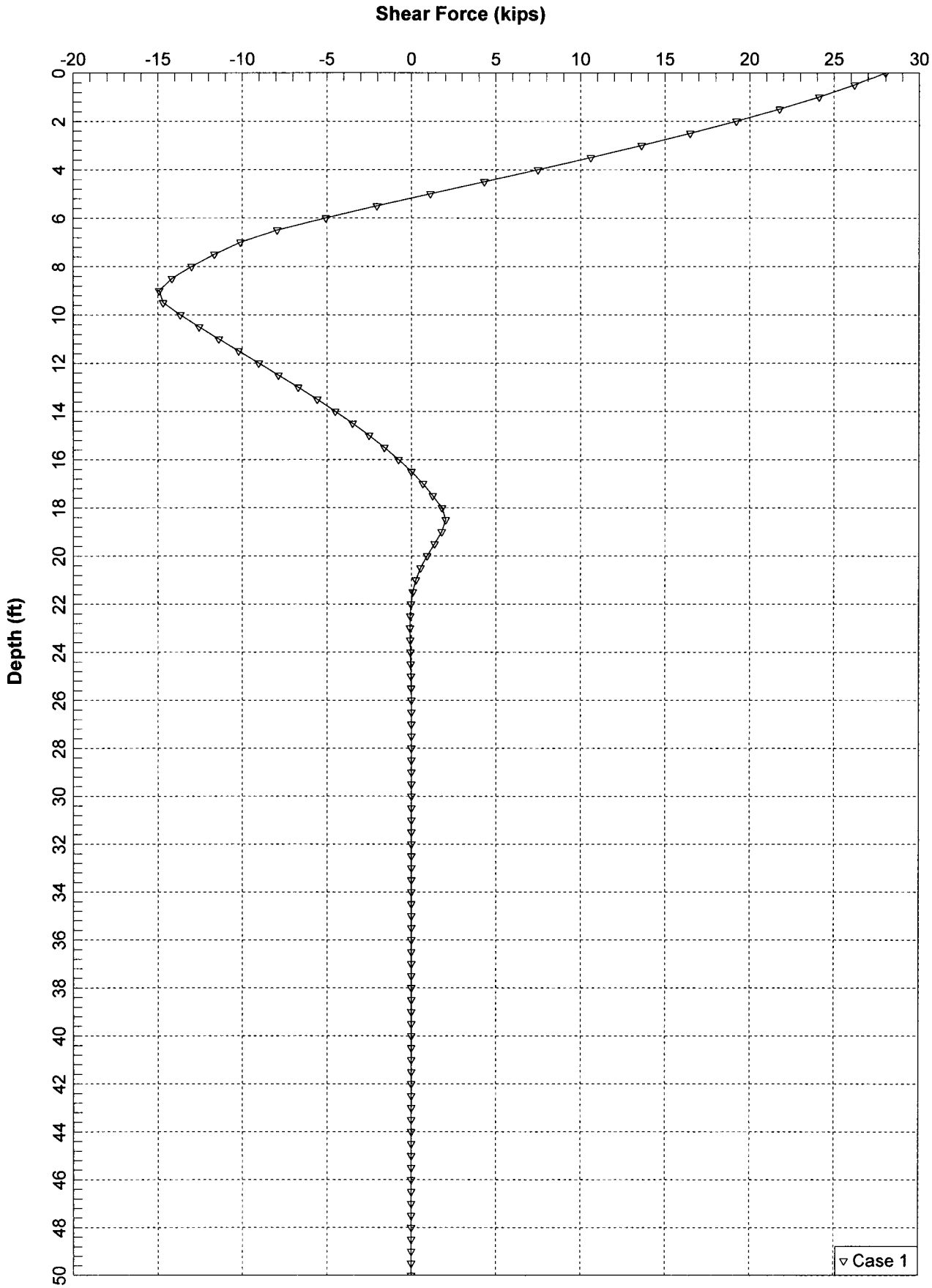

Depths -36 - 84 = Reese Sand
Depths 84 - 192 = Stiff Clay w/o free water
Depths 192 - 336 = Stiff Clay w/o free water
Depths 336 - 516 = Reese Sand
Depths 516 - 576 = Stiff Clay w/o free water
Depths 576 - 858 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1

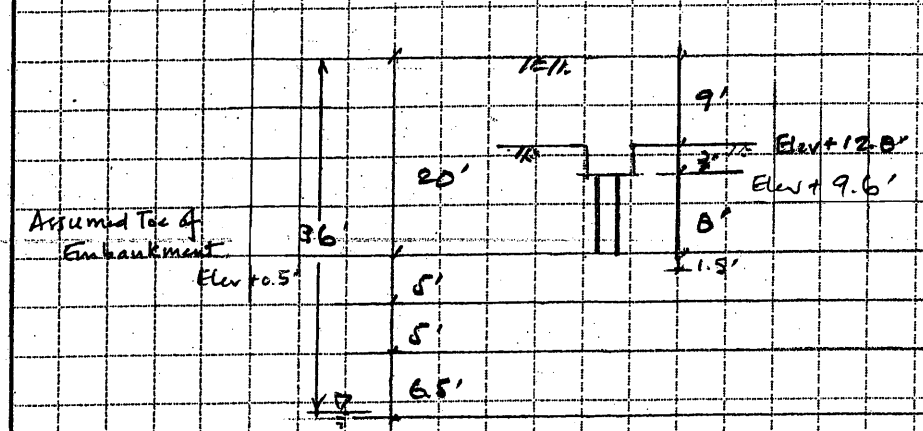

Unfactored Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1

PARIKH
Practicing in the Geosciences

JOB Alpine Ave Ue (Warden) 206127.BR
SHEET NO OF
CALCULATED BY A Lam DATE 4/2010
CHECKED BY DATE
SCALE

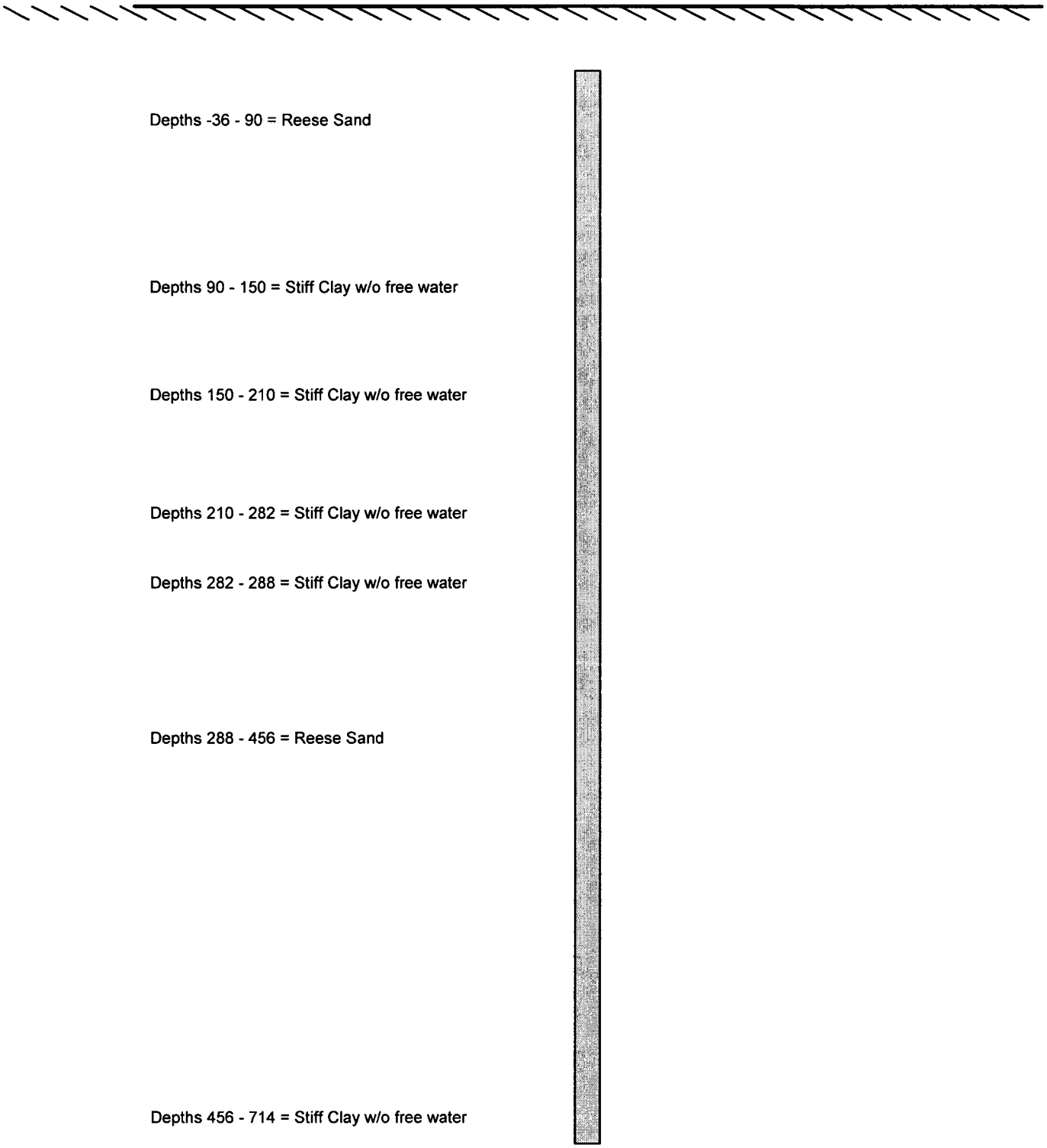
Lateral Pile Capacity Analysis - Alpine Ave Ue (Warden) (CDT-09-001) Abutment 2
14" □ precast concrete pile. 138 tons Service load

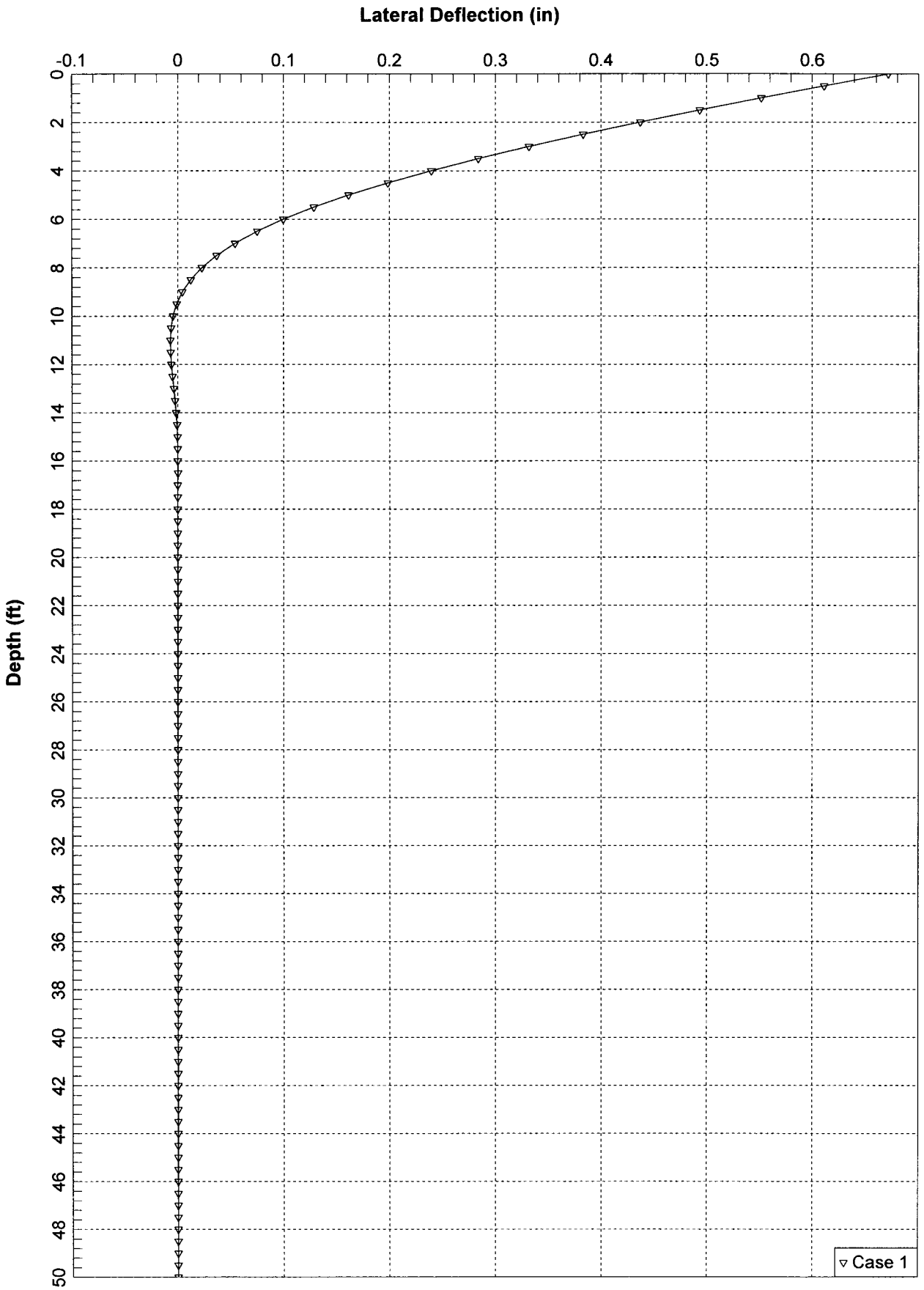


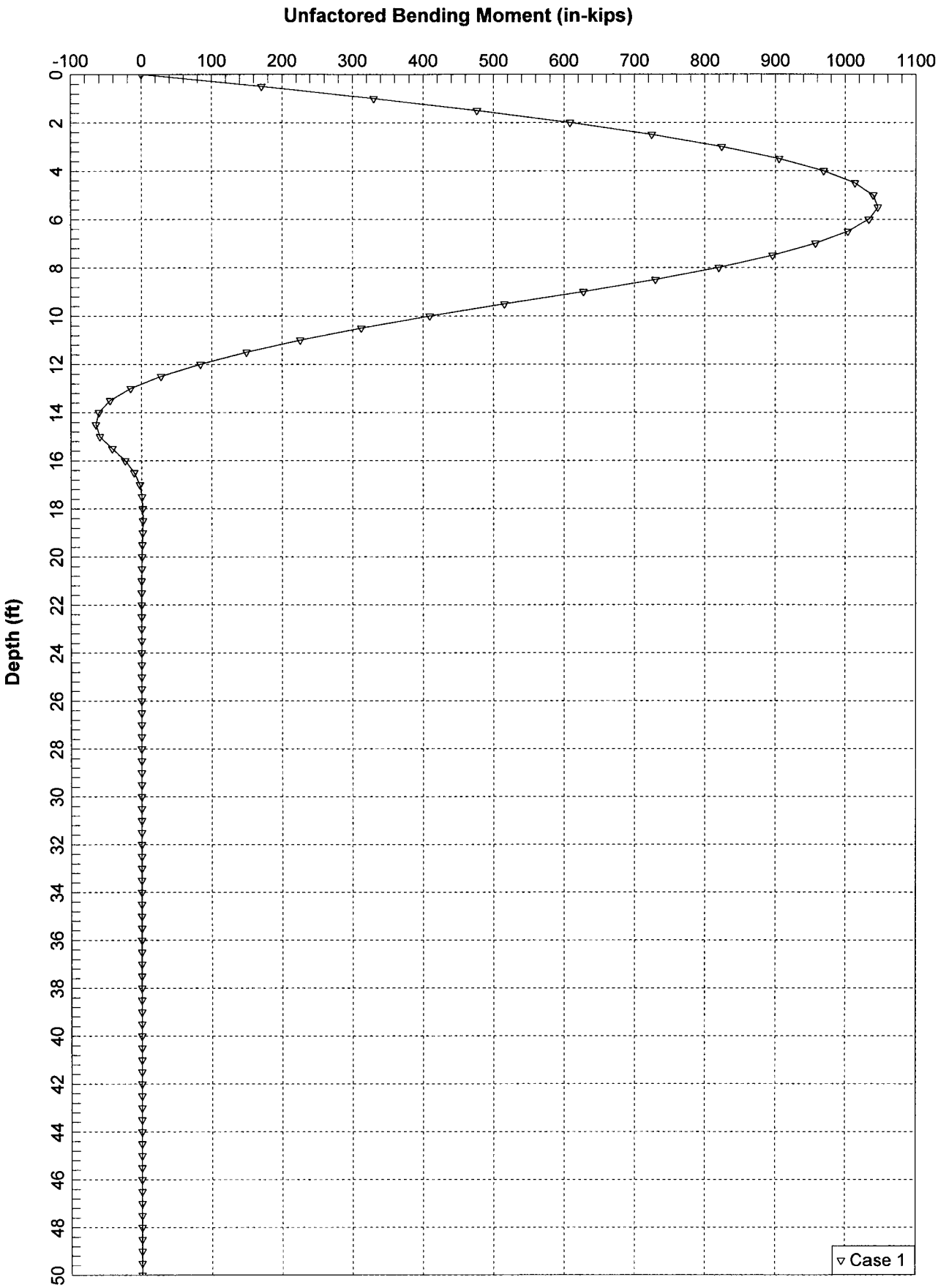
Silty Sand
$\phi' = 36°$
$k = 225$ lb/in^3

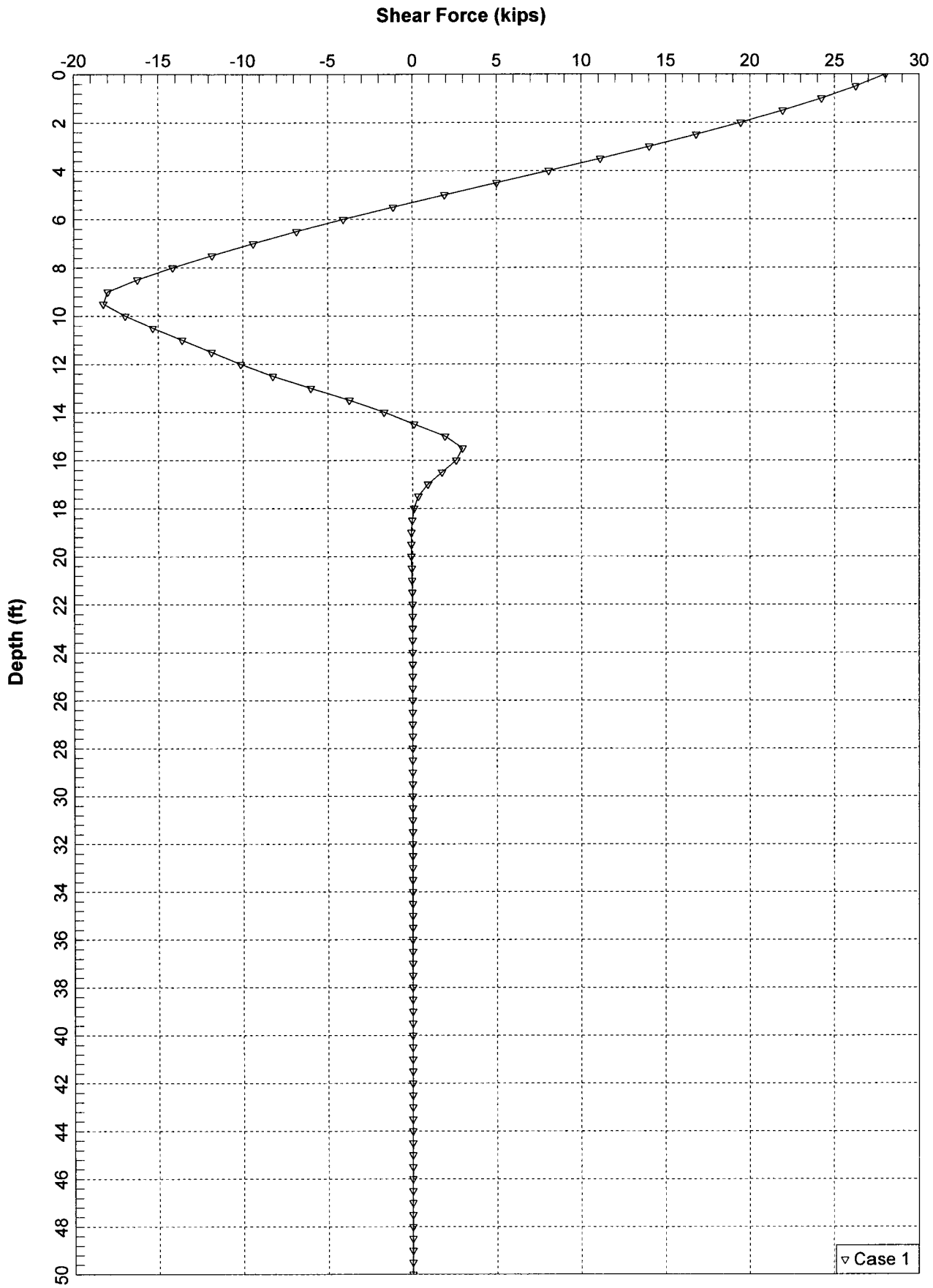
Depth	Soil
5'	$c = 2500$ psf Lean Clay $\varepsilon_{50} = 0.005$
5'	$c = 4000$ psf Lean Clay $\varepsilon_{50} = 0.005$
6.5'	$c = 2000$ psf Silt $\varepsilon_{50} = 0.007$
14'	$\phi' = 32.5°$ Silty Sand $k = 60$ lb/in^3
21.5'	$c = 2750$ psf Lean Clay $\varepsilon_{50} = 0.005$
8.5'	$c = 1750$ psf Lean Clay
7.5'	$c = 4000$ psf Lean Clay
4.5'	$\phi = 35.5°$ Sand
17	$c = 4000$ psf Lean Clay
17.5'	

```
LPILEP5
I-5 Stockton-Alpine-Abut-2
1       1       0       0       0       0
100     2       -36     600     0
0       14      3112    194     3000000
600     14      3112    194     3000000
7       14      14      0       2
4       -36     90      225     225
3       90      150     0       0
3       150     210     0       0
3       210     282     0       0
3       282     288     0       0
4       288     456     60      60
3       456     714     0       0
-36     0.067
90      0.067
90      0.067
150     0.067
150     0.069
210     0.069
210     0.067
282     0.067
282     0.036
288     0.036
288     0.039
456     0.039
456     0.039
714     0.039
-36     0       35      0       0
90      0       35      0       0
90      17.36   0       0.005   0
150     17.36   0       0.005   0
150     27.77   0       0.005   0
210     27.77   0       0.005   0
210     13.88   0       0.007   0
282     13.88   0       0.007   0
282     13.88   0       0.007   0
288     13.88   0       0.007   0
288     0       32      0       0
456     0       32      0       0
456     19.09   0       0.005   0
714     19.09   0       0.005   0
0       0.6     1
1206    0.6     1
0       1       1
1
1       28000   0       128000
0
1       1       0
100     1E-5    100
```


Depths -36 - 90 = Reese Sand
Depths 90 - 150 = Stiff Clay w/o free water
Depths 150 - 210 = Stiff Clay w/o free water
Depths 210 - 282 = Stiff Clay w/o free water
Depths 282 - 288 = Stiff Clay w/o free water
Depths 288 - 456 = Reese Sand
Depths 456 - 714 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1


Unfactored Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1

Lateral Pile Capacity Analysis - Alpine Ave UC (Widen)

Abutment 1

Depth of Second Point of Counterflexure = 18.5 feet

Design pile tip elevation = 9.85' - 18.5'

= -8.65' Say -9.0 feet

Abutment 2

Depth of Second Point of Counterflexure = 16 feet

Design pile tip elevation = 10.05' - 16.0'

= -5.95' Say -6.0 feet

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Alpine Avenue UC (Widen)

Boring R-09-AA-001 (Abutment 1) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
*21.5 to 2.5	0 to 19	Sand (medium dense)	$\phi = 35°$; c= 0 psf	4, Sand	90	-	0.069	-
2.5 to -6.5	19 to 28	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.055	-
-6.5 to -18.5	28 to 40	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.036	Assumed GWL at 28 feet deep below existing ground surface for design.
-18.5 to -33.5	40 to 55	Sand (medium dense)	$\phi = 32°$; c= 0 psf	4, Sand	60	-	0.039	-
-33.5 to -38.5	55 to 60	Clay (soft to firm)	$\phi = 0°$; c=625 psf	3, Clay	100	0.01	0.033	-
-38.5 to -62.0	60 to 83.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.036	-
-62.0 to -78.5	83.5 to 100	Clay (very stiff)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-78.5 to -81.0	100 to 102.5	Sand (medium dense)	$\phi = 31°$; c= 0 psf	4, Sand	60	-	0.039	-
-81.0 to -88.5	102.5 to 110	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

*Finish ground elevation of +12.80 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Alpine Avenue UC (Widen)

Boring CPT-09-AA-001 (Abutment 2) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
22.0 to 2.0	0 to 20	Sand (dense)	$\phi = 35°$; c= 0 psf	4, Sand	225	-	0.067	-
2.0 to -3.0	20 to 25	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.067	-
-3.0 to -8.0	25 to 30	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.069	-
-3.0 to -9.0	30 to 36	Clay (stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.067	-
-9.0 to -9.5	36 to 36.5	Clay (stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.036	Assumed GWL at 36 feet deep below existing ground surface for design.
-9.5 to -23.5	36.5 to 50.5	Sand (medium dense)	$\phi = 32.5°$; c= 0 psf	4, Sand	60	-	0.039	-
-23.5 to -45.0	50.5 to 72	Clay (very stiff)	$\phi = 0°$; c=2750 psf	3, Clay	1000	0.005	0.039	-
-45.0 to -53.5	72 to 80.5	Clay (stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-53.5 to -61.0	80.5 to 88	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-61.0 to -65.5	88 to 92.5	Sand (dense)	$\phi = 35.5°$; c= 0 psf	4, Sand	125	-	0.039	-
-65.5 to -83.0	92.5 to 110	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

*Finish ground elevation of +12.80 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection __ 65% PS&E Unchecked Details _X PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 7/8/2010 **Structure Name*:** Alpine Avenue UC **Br No*:** 29-0191 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	_____	_____	_____	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 4th review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated June 11, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** Alpine Avenue UC **Br No*:** 29-0191 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	6/07/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	9	Please provide the Nominal Driving Resistance in Table 5.	Comment is incorporated. "Nominal Driving Resistance" is provided in Table 5.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Alpine Avenue UC_ **Br No*:** 29-0191 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.co	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 EA: 0G4701 Project Name: I-5 North Stockton Widening OSFP Liaison: John Fujimoto Phone: 916-227-8757 e-mail: john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection X 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	Reviewer Name: Brandon Badeker Functional Unit: Geotechnical Services Cost Center: 59-323 Phone Number: 916-227-1046 e-mail: brandon_badeker@dot.ca.gov Date of Review: 12/16/2009 Structure Name*: Alpine Avenue UC Br No*: 29-0191 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
3	FR	8	Tables 2 and 3, Although the pile tip appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	
4	FR	9	Table 4: Pile data table shows two different design tip elevations for the two abutments. It is recommended to keep the same pile length for ease of pile fabrication.	Comment is incorporated. Same pile length is used in the design of specified pile tip elevation.	
5	FR	LOTB	As is stated in the Soil Legend, the sample number or	Comment is incorporated.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

			unconfined compressive strength is generally not located to the left of the graphic on the LOTB.		
6	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #3 above.	
7	FR	NA	C2 – Approved subject to OSFP Verification	N/A	

FOUNDATION REPORT
TELEGRAPH AVENUE UC (WIDEN)
(BRIDGE NO. 29-0192)
STOCKTON, CALIFORNIA
10-SJ-05, PM 29.19 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

PARIKH CONSULTANTS, INC.
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June 11, 2010

Job No. 206127.BR

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Laboratory Test ..... Plate B-1
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Evaluation of Liquefaction Potential
Vertical Pile Capacity Calculations/Lateral Pile Capacity Analyses
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APPENDIX D

Office of Special Funded Projects Comment & Response Form (with PARIKH Consultants, Inc. Responses)

FOUNDATION REPORT
TELEGRAPH AVENUE UC (WIDEN)
(BRIDGE NO. 29-0192)
STOCKTON, CALIFORNIA
10-SJ-05 PM 29.19 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Telegraph Avenue Undercrossing (Widen) (Bridge No. 29-0192)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Telegraph Avenue Undercrossing (UC) (Widen) (Bridge No. 29-0192).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the subsurface soil conditions during construction nor is it practical to

determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are single-span structures. Based on the as-built general plan and foundation plan of the Telegraph Avenue UC, the structures were constructed in 1970 and widened to the outside in 1996. The existing left and right bridges are approximately 100 feet long. The left bridge (Bridge No. 29-0192L) is approximately 75 feet wide and the right bridge (Bridge No. 29-0192R) is approximately 77 feet wide.

The abutments of the existing left and right bridge are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class I 45 tons) based on the as-built data. The specified tip elevation is -45 feet (NAVD 29).

Proposed Construction

The widening of the Telegraph Avenue UC is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. There will be no outside widening for the Telegraph Avenue UC.

According to the designer, the proposed Telegraph Avenue UC (Widen) will match the existing span with supports at Abutments 1 and 2. The widen structure will be a cast-in-place prestressed concrete box girder with approximate length of 100 feet measured along the "I-5" Line. The width of the inside widening will be 48 feet. The abutments will incorporate diaphragm type abutments.

Based on our discussions with the designer, the proposed bridge abutments are to be supported on Caltrans Standard Precast Prestressed Class 140 Alternate "X" (Mod) (14-inch square) piles. Our recommendations presented in this report are based on the above information. Any major

deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The Telegraph Ave. UC is located at the intersection of Interstate 5 (I-5) and Telegraph Ave. It is approximately 0.4 miles south of Calaveras River. The Project Location Map, Plate 1, shows the location of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the Telegraph Avenue UC (February 1964) have been used as a reference (Ref. 1). Based on the as-built LOTB, Borings B-3, B-6 and B-8 were drilled and Cone Penetration Test (CPT) at B-1, B-4, B-5, B-7 and B-9 were performed by Caltrans in 1964. The soil borings were drilled to the approximate depths between 71.5 feet and 82.5 feet and to the bottom elevation between approximately -71 feet and -82 feet. The CPTs were pushed to the approximate depths between 60 feet and 80 feet and to the bottom elevation between approximately -60 feet and -80 feet.

Field Exploration in 2009

Additional field exploration was conducted in April 2009. Boring R-09-TA-001 was drilled on the south side and CPT-09-TA-001 was performed on the north side of Telegraph Avenue UC. The soil boring was drilled to the depth of 111.5 feet and the CPT was pushed to the approximate depth of 109.5 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit

weight, direct shear, Plasticity Index and grain size classification of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built Borings and CPT, the subsurface soils generally consist of very soft to hard clay/silt, underlain by loose to dense sands, underlain by stiff to very stiff clays (with pocket/lens of soft organic silt in Boring B-6), underlain by loose to medium dense sands.

Based on Boring R-09-TA-001 and CPT-09-TA-001, the subsurface soils generally consist of dense silty/poorly graded sand (embankment fill), underlain by firm to hard clays/silt with intermittent layers of medium dense sands.

Groundwater was recorded at approximate depths between 9 feet and 12 feet and between Elevation -8.5 feet and -11 feet in February 1964. Groundwater was measured at Elevation -5 feet (the depth of 27 feet below existing ground surface) during drilling for R-09-TA-001 in April 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of medium dense sands, underlain by firm to hard clays/silt with intermittent layers of medium dense sands. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking.

Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring R-09-TA-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-TA-001 attached, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure, different foundation systems have been evaluated with respect to their design capacity and construction considerations. Caltrans Class 140 Alt. X (Mod) Piles are recommended to be the foundation system for the abutments of the proposed bridge structure. These piles may be designed for service load

of 70 tons at the abutments to the indicated pile tip elevations. Pertinent foundation design information provided by Rajappan & Meyer Consulting Engineers, Inc. (R&M), including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2: FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 140 Alt. (Mod)	14.10	10.80±	2.50	51.00	1.00	12
Abut 2	WSD	Class 140 Alt. (Mod)	13.90	10.60±	2.50	51.00	1.00	12

TABLE 3: FOUNDATION DESIGN LOADS

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	1,580	132	1,055	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1,580	132	1,055	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

* Entire support remains in overall compression

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads", based upon WSD is presented in Table 3 above.

The evaluation of Load Demands on the piles, based upon WSD is presented in Table 4 below. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Table 4 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	Class 140 Alt. X (Mod)	10.80±	1,580	1,055	140	280	-44.5 (a), -7.0 (b)	-49.3
Abut 2	Class 140 Alt. X (Mod)	10.60±	1,580	1,055	140	280	-49.5 (a), -4.5 (b)	-49.5

Notes: (i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

TABLE 5: PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.(1) (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 140 Alt. X (Mod)	280	0	-44.5 (a), -7.0 (c)	-49.3	280
Abut 2	Class 140 Alt. X (Mod)	280	0	-49.5 (a), -4.5 (c)	-49.5	280

Notes
(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression, (b) Tension, (c) Lateral Load.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iii) Based on Caltrans review comment, it was recommended to keep the same pile length (specified tip elevation) for "ease of pile fabrication".

The estimation of the driven pile capacity in clay is based on the adhesion factor per FHWA manual (Tomlinson 1980). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 5, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles should not be spaced closer than 3 times the pile diameter measured center-to-center. For piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter,

there is no group effect for vertical pile capacity.

It is anticipated that the pile capacity will develop after driving as a result of soil "freeze" and dissipation of excess pore water pressures. The final pile capacity can be verified and the gain of pile capacity after initial driving may be evaluated based on "re-driving" after 24-hour (min.) set up. Caltrans Standard Specifications (Section 49-1.08) may be used as the pile driving acceptance criteria. As an option, Pile Driving Analyzer (PDA) may be used to evaluate the pile capacity of the driven piles when unanticipated conditions arise. Typical applications of the PDA include capacity evaluation (for both during driving and re-driving) and integrity testing for piles that have experienced hard driving.

It is prudent to make the contractor aware of these conditions so that appropriate steps can be taken to comply with the standards and maintain the integrity of the piles.

Predrilling to the bottom of the existing water line is required to minimize the vibration impact on the existing Cal Water Service water lines. The bottom of predrill elevation is -3.0 feet according to R&M. Based on the available boring/CPT information, dense sands were encountered in the embankment fill. It is recommended to follow Caltrans standard specifications (Section 49-1.06) (Ref. 7) for oversize predrilling through the existing embankment to the bottom of predrill elevation described above.

Due to the hard driving condition that may be encountered during pile driving, pile driving should be allowed to terminate short of the specified tip elevation provided the following conditions are satisfied:

- Other requirements including tension and lateral demands are met;
- Pile attaining its capacity and refusal within 5 feet above the specified tip elevation.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. Lateral load analyses for the piles were performed using LPILE Program (v. 5). "Generalized Vertical Soil Profile and Strength Parameters" based on R-09-TA-001 and CPT-09-TA-001 are included in Appendix C. To account for group

effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D, respectively.

Generally, abutment piles are designed for service loads only, and batter piles are used for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed

rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for new structures. Settlement is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Boring R-09-TA-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
R-09-TA-001	26.0	2760	6.9	21.6	35.2

Based on the corrosion test results, the native subsurface soils in Boring R-09-TA-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.8 Construction Observation

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

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Telegraph Ave. UC (Widen) Foundation Report (6-2010).rpt

REFERENCES

1. Telegraph Ave. UC As-built Log of Test Borings (February 1964).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

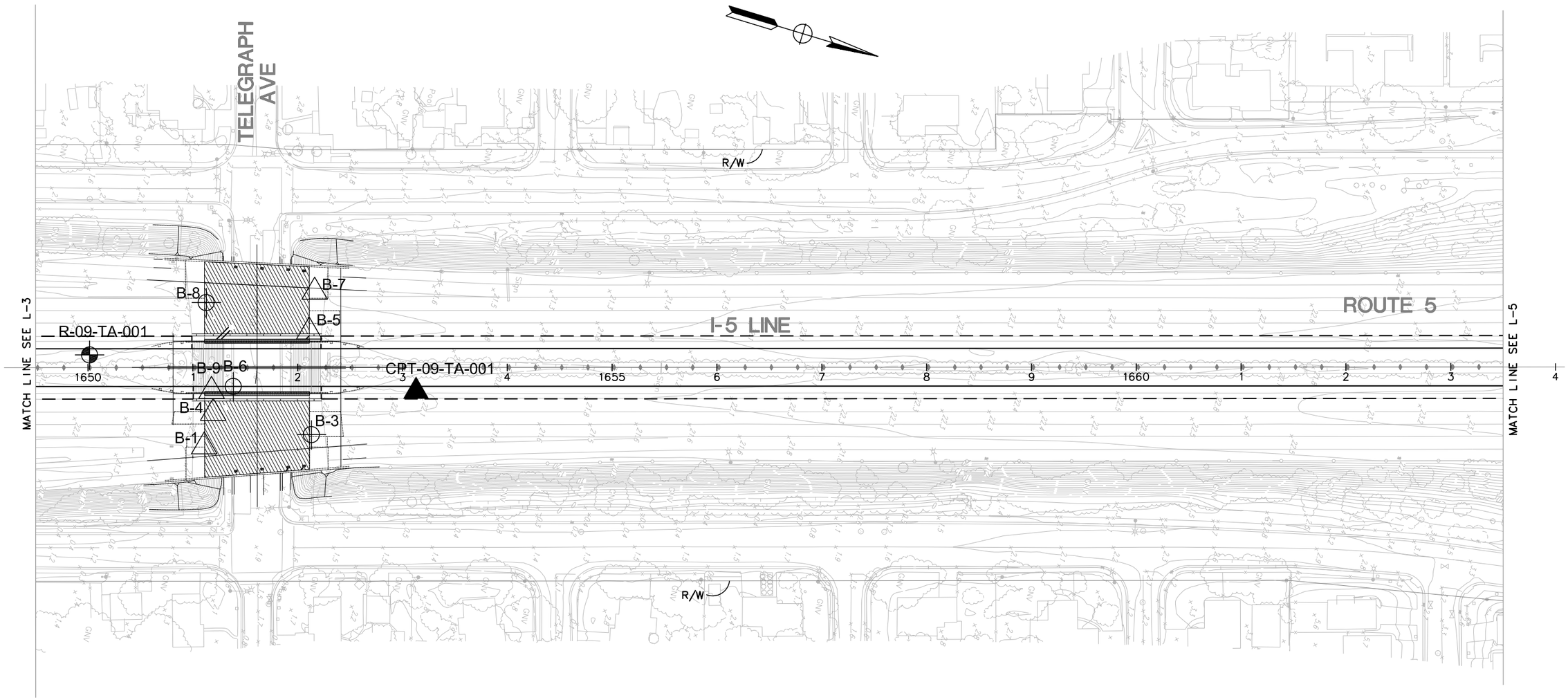
6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

7. State of California Department of Transportation Standard Specifications May 2006.

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

TELEGRAPH AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0192)	PLATE NO.: 1



LEGEND

A-09-TA-001
Approximate Boring Location

B-1
Approximate CPT location performed in 1964

B-8
Approximate Boring location drilled in 1964

CPT-09-TA-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

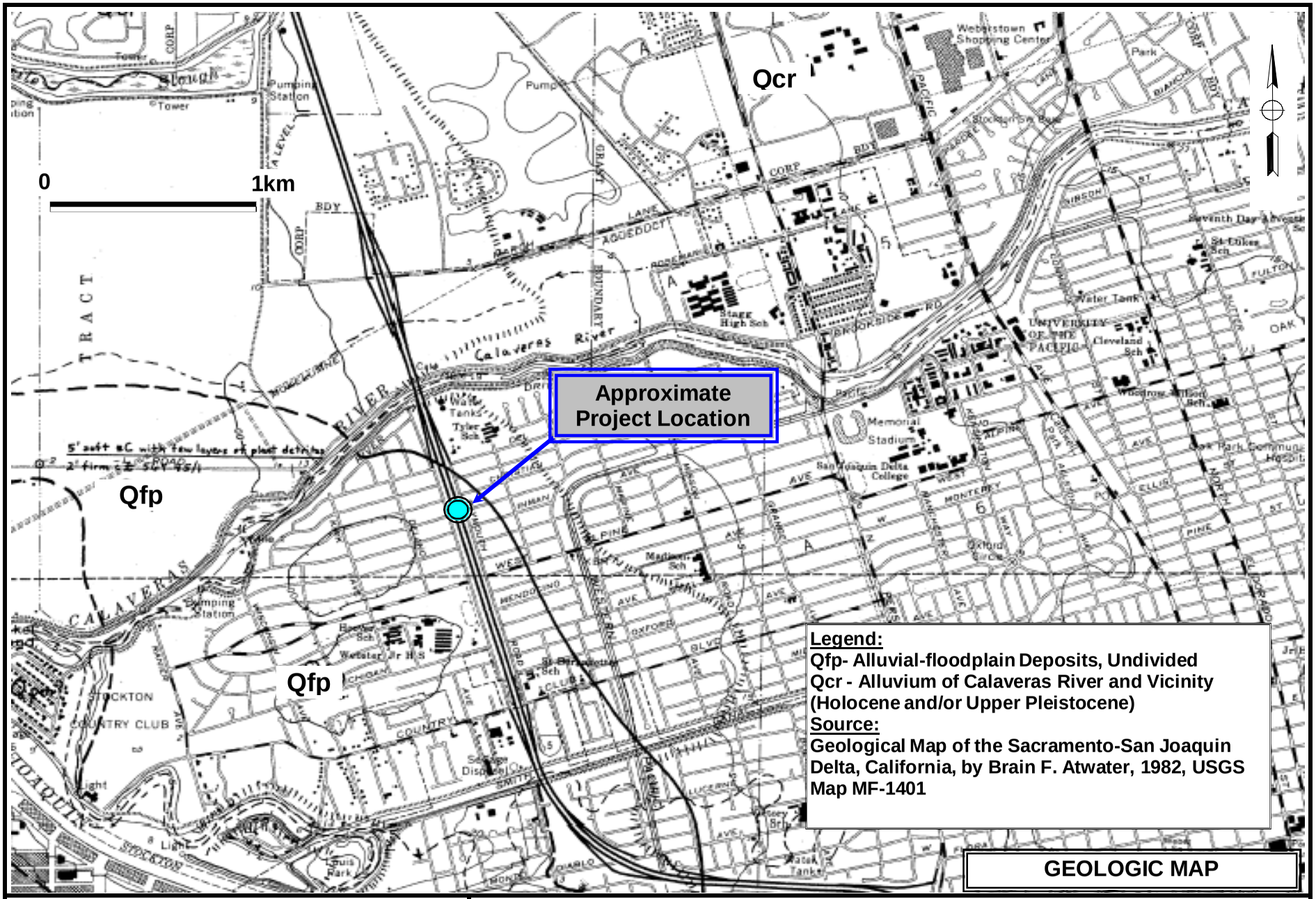
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

TELEGRAPH AVENUE UC (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

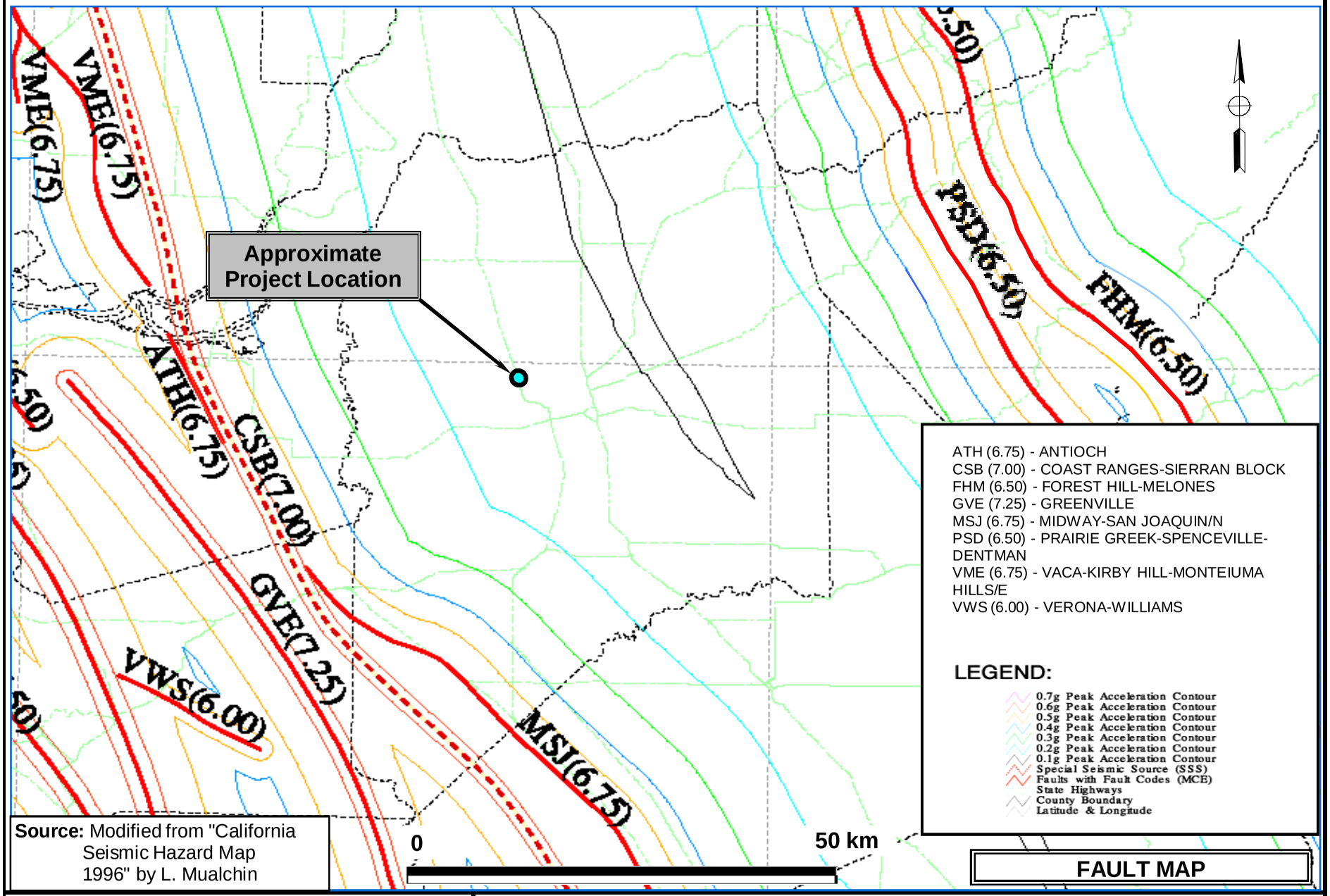
PLATE NO: 2



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

TELEGRAPH AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0192)	PLATE NO.: 3



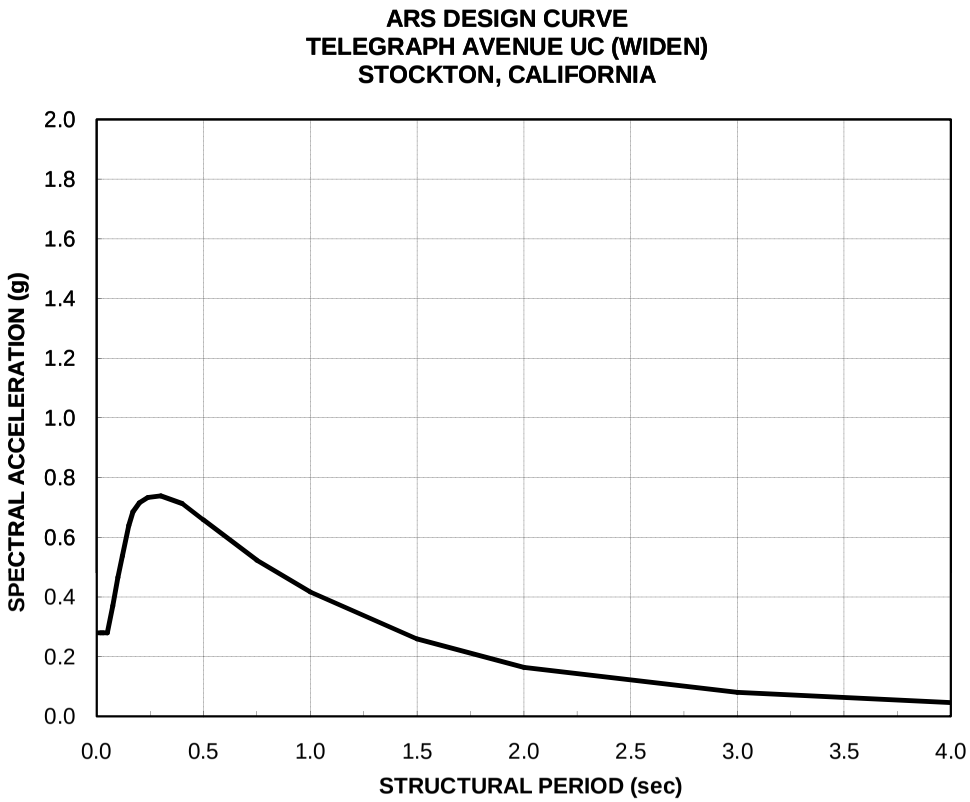
Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

TELEGRAPH AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0192)	PLATE NO.: 4



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

TELEGRAPH AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0192)

PLATE NO.: 5

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

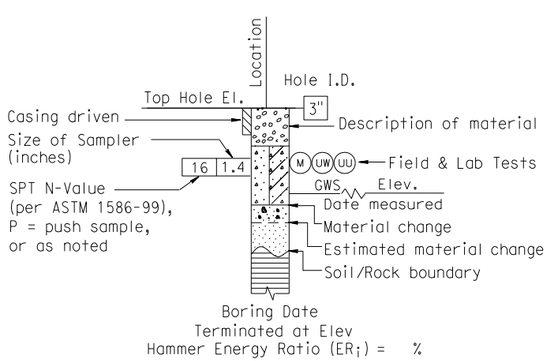
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

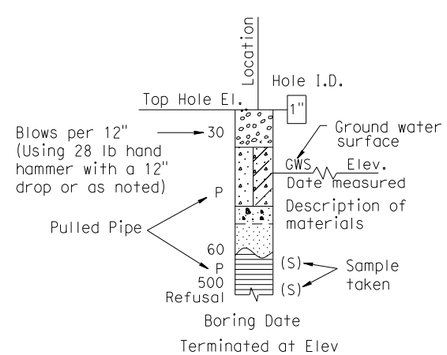
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

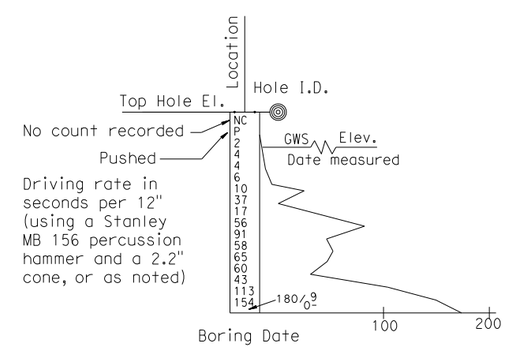
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



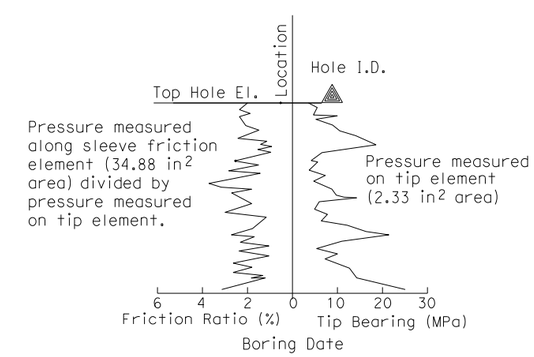
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM	BRIDGE NO. 29-0192
SIGN OFF DATE	CHECKED BY D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER	POST MILES 29.19

TELEGRAPH AVENUE UC (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

REVISION DATES

SHEET 25 OF 28

...\29-0192-z-ltb01.dgn 6/8/2010 9:58:44 AM

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 5333)
CP	Compaction Curve (CTM 216)
CR	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content-% (ASTM D 2974)
P	Permeability (CTM 220)
PA	Particle Size Analysis (ASTM D 422)
PI	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (CTM 217)
SG	Specific Gravity (AASHTO T 100)
SL	Shrinkage Limit (ASTM D 427)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 4767)
VS	Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0192
POST MILES 29.19

TELEGRAPH AVENUE UC (WIDEN)
SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/18/10 04/23/10 06/11/10

SHEET 26 OF 28

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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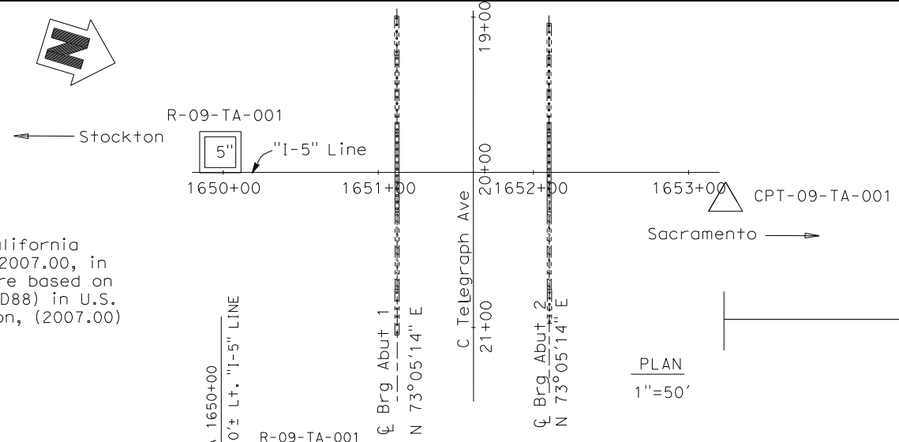
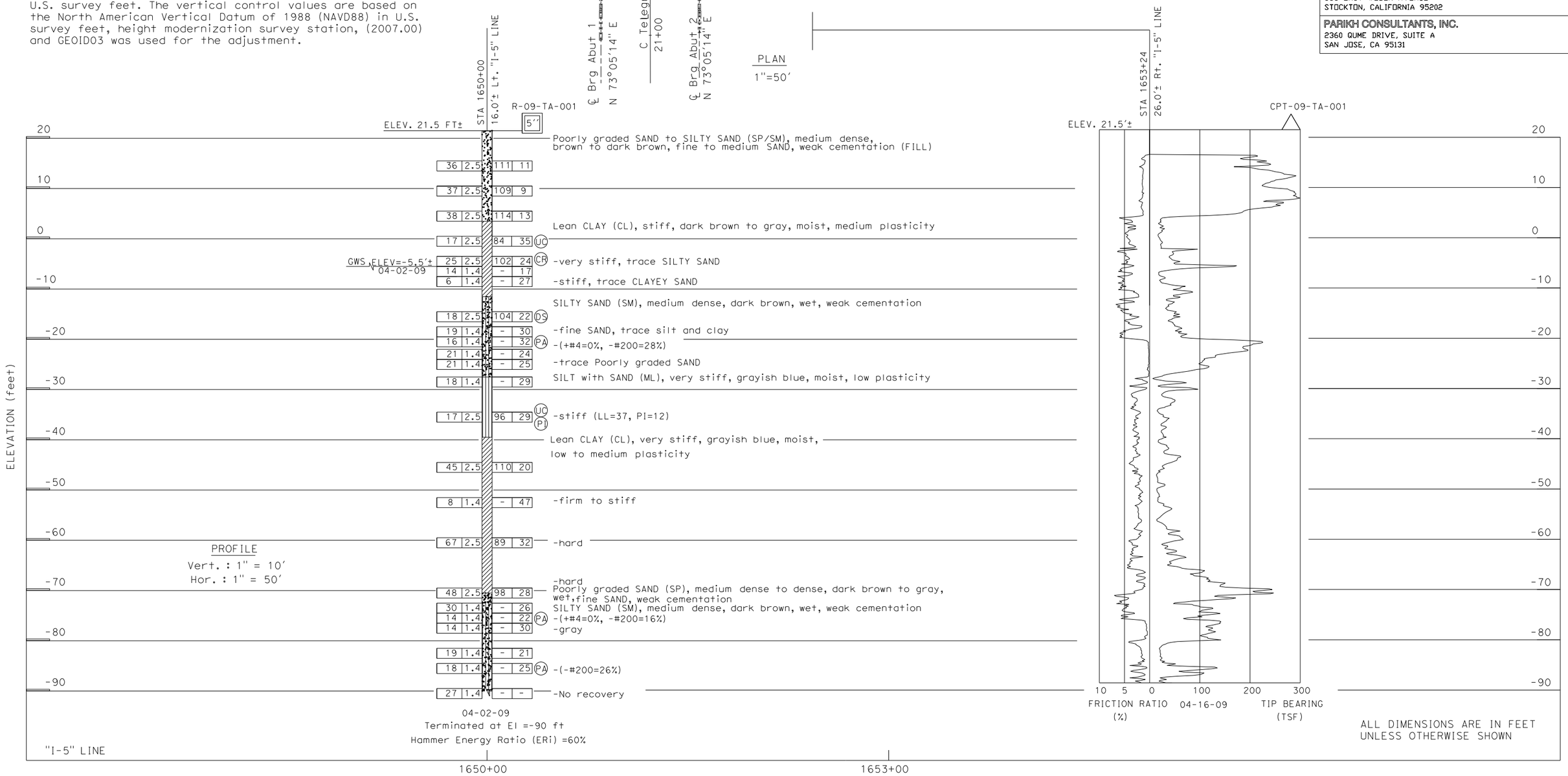
SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet. The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.





DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER

BRIDGE NO. 29-0192
POST MILES 29.19

TELEGRAPH AVENUE UC (WIDEN)
LOG OF TEST BORINGS 1 OF 2

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 27 OF 28

DIST.	COUNTY	ROUTE	POST MILES - TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5	28.3/29.6	173	173

OFFICE OF ENGINEERING GEOLOGY - DIV. OF NEW TECH., MATLS. & RESEARCH

R.W. Fox 9-26-94
CERTIFIED ENGINEERING GEOLOGIST - PROJECT GEOLOGIST PLANS APPROVAL DATE

TELEGRAPH AVE. UNDERCROSSING

LOG OF TEST BORINGS

NOTE: THIS LOG OF TEST BORINGS IS AVAILABLE ON MICROFILM AT OFFICE OF STRUCTURES DESIGN SACRAMENTO, CALIFORNIA

CU: 10207
EA: 4120U1

BRIDGE No. 29-192R/L

REGISTERED GEOLOGIST
R.W. FOX
No. 78
Exp. 8-30-94
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

DIST.	COUNTY	ROUTE	KILOMETER POST-TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5			

September 5, 1997

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

REGISTERED CIVIL ENGINEER DATE

TELEGRAPH AVENUE UC (WIDEN)

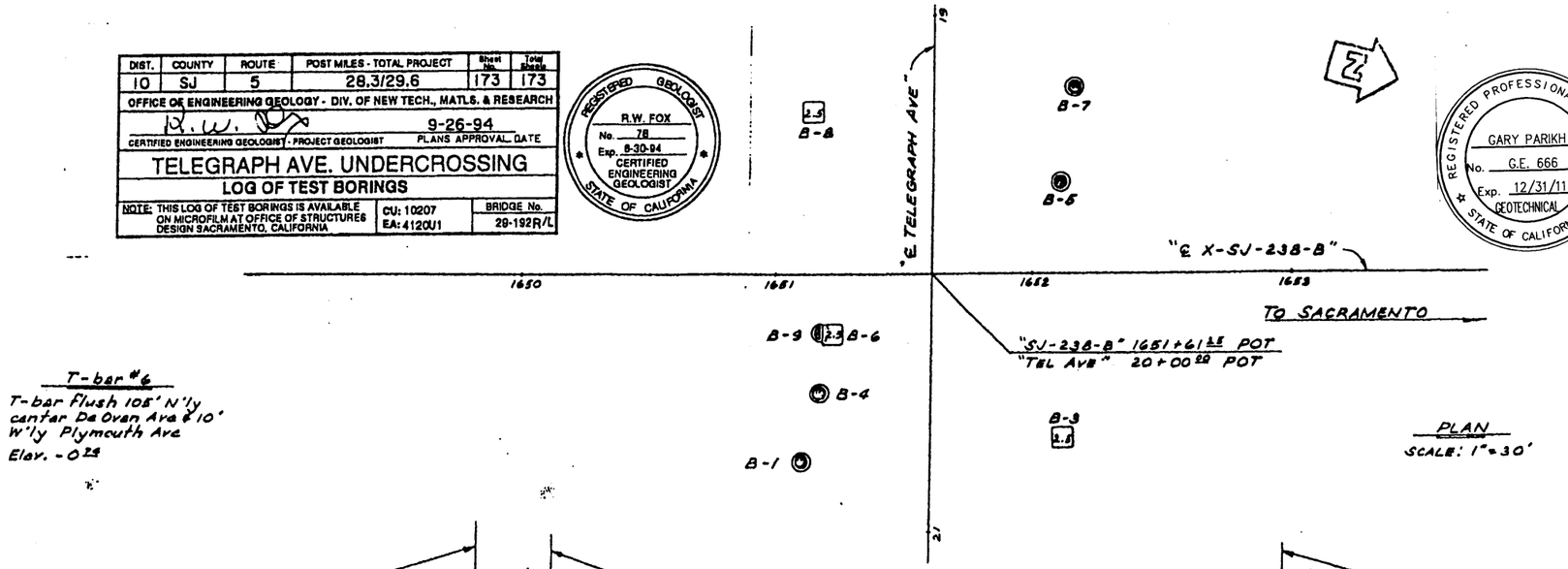
LOG OF TEST BORINGS 2 OF 2

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0G4701

BRIDGE No. 29-0192

Sheet 28 of 28



PLAN
SCALE: 1"=30'

NOTE: Cone penetrometer boring "B-2" between B-6 & B-9 (not shown) pushed from EL. -12.2 to EL. -96 (old wall).
Rotary boring "B-6" intersected wall at EL. -76.

NO CHANGES

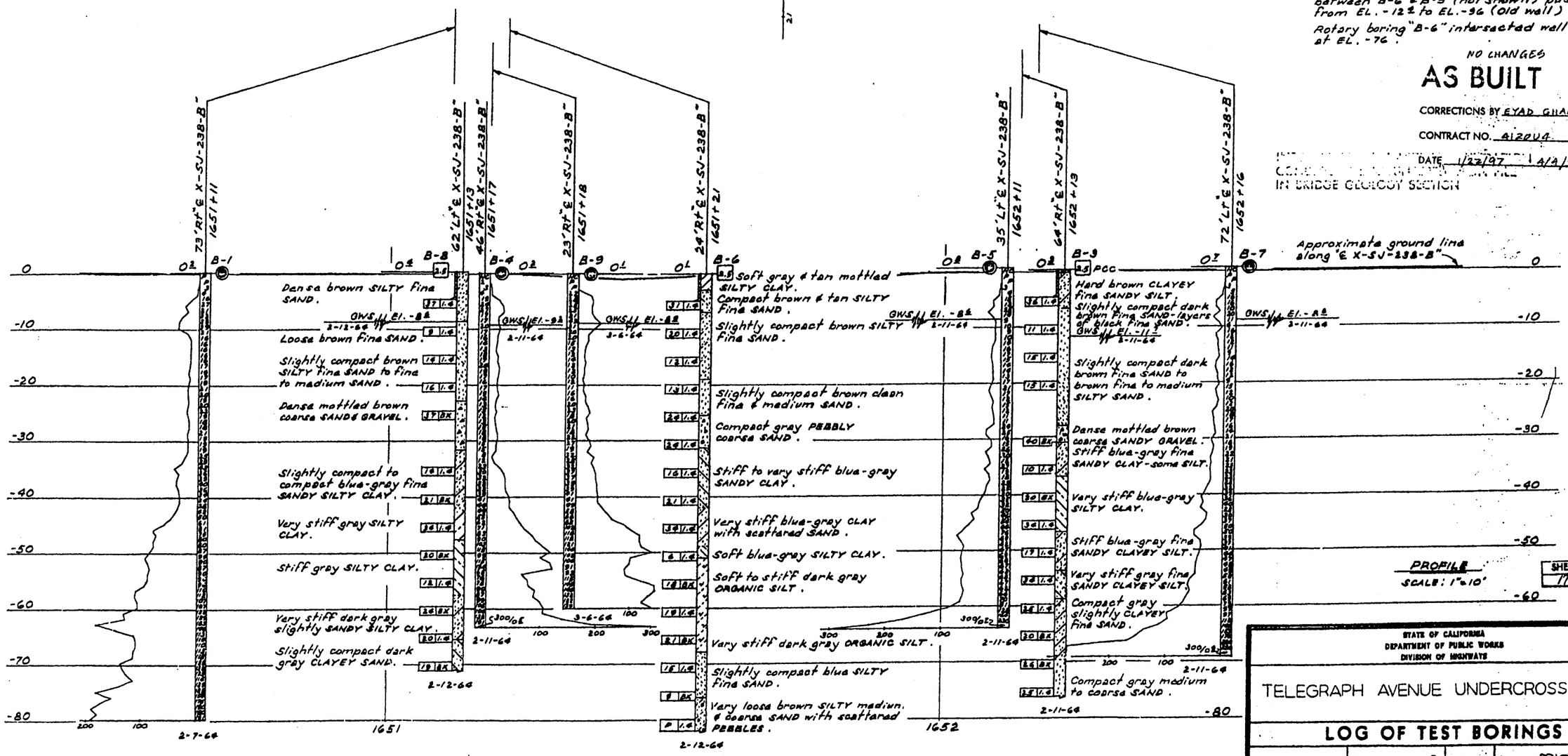
AS BUILT

CORRECTIONS BY EYAD GHANI / T. LAM

CONTRACT NO. 4120U4

DATE 1/22/97 4/4/97

CONTRACT RECORD FILE
IN BRIDGE GEOLOGY SECTION



PROFILE
SCALE: 1"=10'

SHEET	OF
17	17

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

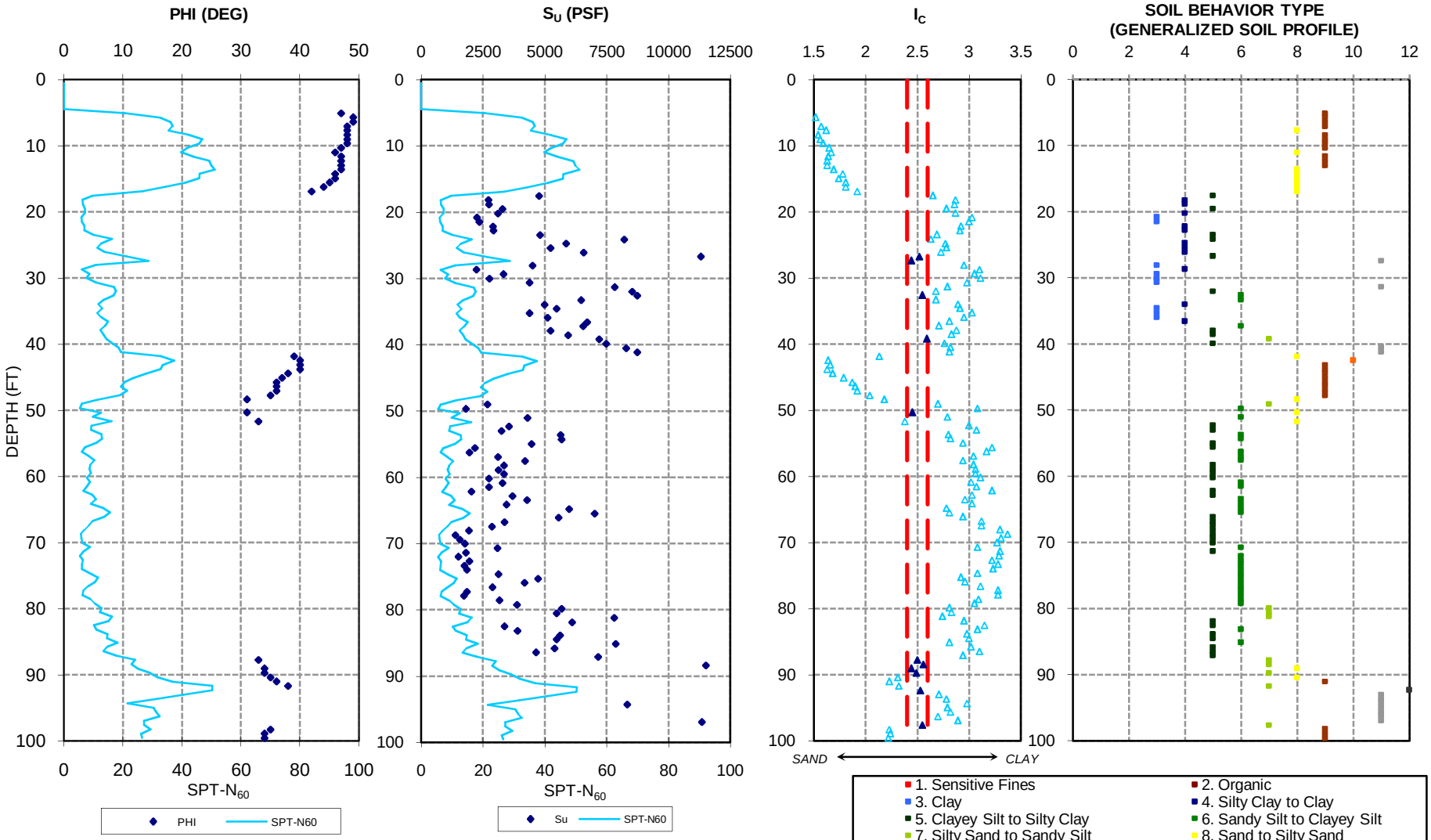
TELEGRAPH AVENUE UNDERCROSSING

LOG OF TEST BORINGS

SCALE AS Noted | BRIDGE 29-192R/L | FILE | DRAWING 29192-8

CPT DATA CORRELATION

CPT-09-TA-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

TELEGRAPH AVENUE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0192)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-TELEGRAPH AVENUE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0192)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-TA-004***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*CRR $_{7.5}$*						*F.S.=(CRR $_{7.5}$/(CSR)*MSF*K $_\sigma$*K $_\alpha$*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
5.085	9	Sand	125.926	1.186	0.316		0.316	1.48	1.45119	0.99	0.129	1.480	1.45	182.74	1.000	182.742		1.00	1	
5.741	9	Sand	196.628	2.525	0.356		0.356	1.52	1.41388	0.99	0.129	1.520	1.41	278.01	1.000	278.009		1.00	1	
6.398	9	Sand	226.273	2.153	0.397		0.397	1.4	1.37758	0.99	0.128	1.400	1.38	311.71	1.000	311.710		1.00	1	
7.054	9	Sand	218.384	3.022	0.438		0.438	1.57	1.3431	0.99	0.128	1.570	1.34	293.31	1.000	293.312		1.00	1	
7.71	8	Sand to Silty Sand	206.723	3.027	0.477		0.477	1.62	1.31187	0.98	0.128	1.620	1.31	271.19	1.000	271.193		1.00	1	
8.366	9	Sand	251.42	3.232	0.518		0.518	1.54	1.28056	0.98	0.128	1.540	1.28	321.96	1.000	321.958		1.00	1	
9.022	9	Sand	279.052	3.8	0.559		0.559	1.56	1.25071	0.98	0.128	1.560	1.25	349.01	1.000	349.013		1.00	1	
9.678	9	Sand	270.606	3.729	0.6		0.6	1.59	1.22222	0.98	0.127	1.590	1.22	330.74	1.000	330.741		1.00	1	
10.335	9	Sand	242.363	3.52	0.64		0.64	1.65	1.19565	0.98	0.127	1.650	1.20	289.78	1.003	290.749		1.00	1	
10.991	8	Sand to Silty Sand	227.889	3.299	0.68		0.68	1.67	1.17021	0.98	0.127	1.670	1.17	266.68	1.017	271.312		1.00	1	
11.647	9	Sand	254.756	3.465	0.721		0.721	1.64	1.14524	0.98	0.127	1.640	1.15	291.76	1.000	291.756		1.00	1	
12.303	9	Sand	285.281	3.983	0.762		0.762	1.63	1.1213	0.97	0.127	1.630	1.12	319.89	1.000	319.887		1.00	1	
12.959	9	Sand	290.228	3.882	0.802		0.802	1.63	1.0989	0.97	0.126	1.630	1.10	318.93	1.000	318.932		1.00	1	
13.615	8	Sand to Silty Sand	290.298	4.531	0.842		0.842	1.69	1.07738	0.97	0.126	1.690	1.08	312.76	1.031	322.481		1.00	1	
14.272	8	Sand to Silty Sand	253.752	4.419	0.882		0.882	1.78	1.05668	0.97	0.126	1.780	1.06	268.13	1.092	292.894		1.00	1	
14.928	8	Sand to Silty Sand	256.149	3.934	0.921		0.921	1.74	1.03725	0.97	0.126	1.740	1.04	265.69	1.065	282.909		1.00	1	
15.584	8	Sand to Silty Sand	224.341	3.627	0.961		0.961	1.81	1.01805	0.97	0.126	1.810	1.02	228.39	1.114	254.429		1.00	1	
16.24	8	Sand to Silty Sand	187.909	2.549	1.001		1.001	1.81	0.99955	0.97	0.126	1.810	1.00	187.82	1.114	209.237		1.00	1	
16.896	8	Sand to Silty Sand	139.957	1.95	1.04		1.04	1.92	0.98214	0.96	0.125	1.920	0.98	137.46	1.208	166.087		0.99	1	
17.552	5	Clayey Silt to Silty Clay	36.77	1.358	1.078		1.078	2.65	0.96576	0.96	0.125							0.99	1	
18.209	4	Silty Clay to Clay	21.461	0.826	1.116		1.116	2.87	0.94991	0.96	0.125							0.98	1	
18.865	4	Silty Clay to Clay	21.745	0.798	1.153		1.153	2.86	0.93498	0.96	0.125							0.97	1	
19.521	5	Clayey Silt to Silty Clay	25.774	0.846	1.191		1.191	2.78	0.92012	0.96	0.125							0.97	1	
20.177	4	Silty Clay to Clay	24.486	0.998	1.228		1.228	2.87	0.9061	0.96	0.124							0.96	1	
20.833	3	Clay	18.164	0.838	1.265		1.265	3.03	0.89249	0.95	0.124							0.95	1	
21.49	3	Clay	19.065	0.833	1.301		1.301	3	0.87965	0.95	0.124							0.95	1	
22.146	4	Silty Clay to Clay	23.149	0.927	1.339		1.339	2.92	0.86648	0.95	0.124							0.94	1	
22.802	4	Silty Clay to Clay	23.192	0.879	1.377		1.377	2.91	0.85371	0.95	0.123							0.94	1	
23.458	5	Clayey Silt to Silty Clay	37.354	1.151	1.414		1.414	2.69	0.84162	0.95	0.123							0.93	1	
24.114	5	Clayey Silt to Silty Clay	62.873	2.792	1.452		1.452	2.63	0.82956	0.94	0.123							0.93	1	
24.77	4	Silty Clay to Clay	45.369	2.213	1.489		1.489	2.77	0.81815	0.94	0.123							0.92	1	
25.427	4	Silty Clay to Clay	40.707	1.754	1.527		1.527	2.78	0.80675	0.94	0.122							0.92	1	
26.083	4	Silty Clay to Clay	50.528	2.285	1.565		1.565	2.73	0.79566	0.94	0.122							0.91	1	
26.739	5	Clayey Silt to Silty Clay	85.923	3.369	1.602		1.602	2.52	0.78515	0.94	0.122	2.520	0.79	67.46	2.872	193.727		0.91	1	
27.395	11	Sitff Fine-Grained	120.936	4.997	1.645		1.645	2.44	0.77329	0.93	0.121	2.440	0.77	93.52	2.483	232.218		0.91	1	
28.051	3	Clay	35.143	2.014	1.682		1.682	2.95	0.76336	0.93	0.121							0.90	1	
28.707	4	Silty Clay to Clay	18.279	0.738	1.719		1.719	3.1	0.75368	0.93	0.120							0.90	1	
29.364	3	Clay	26.408	1.445	1.756		1.756	3.05	0.74425	0.92	0.120							0.89	1	
30.02	3	Clay	22.286	1.179	1.792		1.792	3.11	0.73529	0.92	0.120							0.89	1	
30.676	3	Clay	34.477	1.942	1.829		1.829	2.98	0.72631	0.92	0.119							0.89	1	
31.332	11	Sitff Fine-Grained	60.269	3.299	1.872		1.872	2.79	0.71615	0.91	0.119							0.88	1	
31.988	5	Clayey Silt to Silty Clay	65.832	2.74	1.909		1.909	2.68	0.70762	0.91	0.118							0.88	1	
32.644	6	Sandy Silt to Clayey Silt	67.409	1.775	1.947		1.947	2.55	0.69908	0.91	0.118	2.550	0.70	47.12	3.034	142.986	0.35	0.88	1	
33.301	6	Sandy Silt to Clayey Silt	50.381	1.445	1.984	0.009	1.975	2.68	0.69291	0.90	0.118							0.87	1	
33.957	4	Silty Clay to Clay	39.249	1.708	2.022	0.03	1.992	2.89	0.68922	0.90	0.118							0.87	1	
34.613	3	Clay	43.136	2.181	2.059	0.05	2.008	2.91	0.68579	0.89	0.119							0.87	1	
35.269	3	Clay	34.974	2.109	2.095	0.071	2.025	3.03	0.68217	0.89	0.120							0.87	1	
35.925	3	Clay	40.275	2.131	2.132	0.091	2.041	2.95	0.6788	0.88	0.120							0.87	1	
36.581	4	Silty Clay to Clay	52.581	2.361	2.169	0.111	2.058	2.81	0.67526	0.88	0.120							0.87	1	
37.238	6	Sandy Silt to Clayey Silt	51.183	1.555	2.207	0.132	2.075	2.71	0.67176	0.87	0.121							0.86	1	
37.894	5	Clayey Silt to Silty Clay	41.471	1.74	2.244	0.152	2.092	2.88	0.66829	0.87	0.121							0.86	1	
38.55	5	Clayey Silt to Silty Clay	46.75	1.86	2.282	0.173	2.109	2.83	0.66485	0.86	0.121							0.86	1	
39.206	7	Silty Sand to Sandy Silt	56.348	1.208	2.321	0.193	2.127	2.59	0.66126	0.86	0.122	2.590	0.66	37.26	3.266	121.694	0.25	0.86	1	2.43
39.862	5	Clayey Silt to Silty Clay	58.357	2.393	2.358	0.214	2.144	2.76	0.65789	0.85	0.122							0.86	1	
40.518	11	Sitff Fine-Grained	64.653	3.508	2.401	0.234	2.167	2.82	0.6534	0.85	0.122							0.86	1	
41.175	11	Sitff Fine-Grained	67.912	3.731	2.444	0.255	2.189	2.81	0.64916	0.84	0.122							0.85	1	
41.831	8	Sand to Silty Sand	157.592	2.265	2.484	0.275	2.208	2.13	0.64554	0.83	0.122	2.130	0.65	101.73	1.512	153.794	0.42	0.82	1	4.09
42.487	10	Gravelly Sand to Sand	215.949	0.64	2.525	0.296	2.23	1.64	0.6414	0.83	0.122	1.640	0.64	138.51	1.000	138.510	0.33	0.78	1	3.20
43.143	9	Sand	191.987	0.466	2.566	0.316	2.25	1.66	0.63768	0.82	0.122	1.660	0.64	122.43	1.010	123.700	0.26	0.79	1	2.51
43.799	9	Sand	190.398	0.365	2.607	0.337	2.27	1.63	0.63401	0.82	0.122	1.630	0.63	120.71	1.000	120.713	0.24	0.79	1	2.39
44.455	9	Sand	158.782	0.233	2.647	0.357	2.29	1.68	0.63037	0.81	0.122	1.680	0.63	100.09	1.024	102.520	0.18	0.80	1	1.77
45.112	9	Sand	128.858	0.213	2.688	0.378	2.311	1.79	0.6266	0.80	0.121	1.790	0.63	80.74	1.099	88.772	0.15	0.82	1	1.43
45.768	9	Sand	109.263	0.185	2.729	0.398	2.331	1.87	0.62305	0.80	0.121	1.870	0.62	68.08	1.162	79.094	0.13	0.83	1	1.24
46.424	9	Sand	102.182	0.168	2.77	0.419	2.351	1.9	0.61954	0.79	0.121	1.900	0.62	63.31	1.189	75.261	0.12	0.84	1	1.18
47.08	9	Sand	112.288	0.3	2.81	0.439	2.371	1.92	0.61607	0.78	0.121	1.920	0.62	69.18	1.208	83.586	0.13	0.83	1	1.33
47.736	9	Sand	95.532	0.341	2.851	0.46	2.392	2.04	0.61247	0.78	0.120	2.040	0.61	58.51	1.356	79.334	0.13	0.84	1	1.25
48.392	8	Sand to Silty Sand	54.208	0.081	2.891	0.48	2.411	2.18	0.60925	0.77	0.120	2.180	0.61	33.03	1.619	53.475	0.09	0.84	1	0.94
49.049	7	Silty Sand to Sandy Silt	22.952	0.091	2.929	0.5	2.429	2.7	0.60623	0.76	0.120							0.84	1	
49.705	6	Sandy Silt to Clayey Silt	16.335	0.223	2.967	0.521	2.446	3.08	0.6034	0.76	0.119							0.84	1	
50.361	8	Sand to Silty Sand	52.428	0.411	3.007	0.541	2.465	2.45	0.60027	0.75	0.119	2.450	0.60	31.47	2.528	79.566	0.13	0.83	1	1.27
51.017	6	Sandy Silt to Clayey Silt	35.193	0.597	3.044	0.562	2.482	2.79	0.5975	0.74	0.118							0.83	1	
51.673	8	Sand to Silty Sand	70.616	0.682	3.084	0.582	2.502	2.38	0.59427	0.74	0.118	2.380	0.59	41.97	2.233	93.691	0.16	0.83	1	1.58
52.329	5	Clayey Silt to Silty Clay	29.449	0.866	3.122	0.603	2.519	3	0.59156	0.73	0.117							0.83	1	
52.986	5	Clayey Silt to Silty Clay	27.189	0.937	3.159	0.623	2.536	3.07	0.58887	0.72	0.117							0.83	1	
53.642	6	Sandy Silt to Clayey Silt	44.922	1.177	3.197	0.644	2.553	2.8	0.5862	0.72	0.117							0.83	1	
54.298	6	Sandy Silt to Clayey Silt	44.747	1.276	3.234	0.664	2.57	2.82	0.58355	0.71	0.116							0.83	1	
54.954	5	Clayey Silt to Silty Clay	35.64	1.126	3.272	0.685	2.587	2.94	0.58093	0.70	0.116							0.83	1	
55.61	5	Clayey Silt to Silty Clay	19.041	0.62	3.31	0.705	2.604	3.22	0.57834	0.70	0.115							0.83	1	
56.266	6	Sandy Silt to Clayey Silt	17.373	0.368	3.347	0.726	2.622	3.17	0.57561	0.69	0.115							0.82	1	
56.923	6	Sandy Silt to Clayey Silt	25.737	0.688	3.385	0.746	2.639	3.04	0.57307	0.69	0.114							0.82	1	
57.579	6	Sandy Silt to Clayey Silt	33.779	0.939	3.422	0.767	2.656	2.94	0.57054	0.68	0.114							0.82	1	
58.235	5	Clayey Silt to Silty Clay	27.587	0.812	3.46	0.787	2.673	3.04	0.56804	0.67	0.113							0.82	1	
58.891	5	Clayey Silt to Silty Clay	25.873	0.745	3.498	0.808	2.69	3.06	0.56555	0.67	0.113							0.82	1	
59.547	5	Clayey Silt to Silty Clay	27.762	0.882	3.535	0.828	2.707	3.06	0.56309	0.66	0.113							0.82	1	
60.203	5	Clayey Silt to Silty Clay	23.192	0.639	3.573	0.848	2.724	3.11	0.56065	0.66	0.112							0.82	1	
60.86	6	Sandy Silt to Clayey Silt	27.156	0.695	3.61	0.869	2.741	3.02	0.55823	0.65	0.112							0.82	1	
61.516	6	Sandy Silt to Clayey Silt	23.64	0.532	3.648	0.889	2.758	3.07	0.55584	0.65	0.111							0.82	1	
62.172	5	Clayey Silt to Silty Clay	18.093	0.463	3.686	0.91	2.776	3.22	0.55332	0.64	0.111							0.82	1	
62.828	5	Clayey Silt to Silty Clay	29.968	0.924	3.723	0.93	2.793	3.03	0.55096	0.64	0.110							0.81	1	
63.484	6	Sandy Silt to Clayey Silt	34.428	1.019	3.761	0.951	2.81	2.96	0.54863	0.63	0.110							0.81	1	
64.14	6	Sandy Silt to Clayey Silt	27.942	0.785	3.798	0.971	2.827	3.03	0.54631	0.63	0.110							0.81	1	
64.797	6	Sandy Silt to Clayey Silt	46.985	1.094	3.836	0.992	2.844	2.78	0.54402	0.62	0.109							0.81	1	
65.453	6	Sandy Silt to Clayey Silt	55.049	1.746	3.873	1.012	2.861	2.81	0.54174	0.62	0.109							0.81	1	
66.109	5	Clayey Silt to Silty Clay	44.288	1.648	3.911	1.033	2.878	2.94	0.53948	0.61	0.109							0.81	1	
66.765	5	Clayey Silt to Silty Clay	28.352	1.028	3.949	1.053	2.895	3.12	0.53724	0.61	0.108							0.81	1	
67.421	5	Clayey Silt to Silty Clay	24.672	0.668	3.986	1.074	2.913	3.12	0.53489	0.61	0.108							0.81	1	
68.077	5	Clayey Silt to Silty Clay	17.946	0.518	4.024	1.094	2.93	3.3	0.53269	0.60	0.108							0.81	1	
68.734	5	Clayey Silt to Silty Clay	13.927	0.304	4.061	1.115	2.947	3.37	0.5305	0.60	0.107							0.81	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-TELEGRAPH AVENUE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0192)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-TA-004***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
69.39	5	Clayey Silt to Silty Clay	15.052	0.302	4.099	1.135	2.964	3.31	0.52834	0.60	0.107							0.80	1	
70.046	5	Clayey Silt to Silty Clay	16.444	0.356	4.137	1.156	2.981	3.27	0.52619	0.59	0.107							0.80	1	
70.702	6	Sandy Silt to Clayey Silt	26.08	0.671	4.174	1.176	2.998	3.08	0.52406	0.59	0.106							0.80	1	
71.358	5	Clayey Silt to Silty Clay	16.817	0.413	4.212	1.197	3.015	3.3	0.52195	0.58	0.106							0.80	1	
72.014	6	Sandy Silt to Clayey Silt	14.151	0.246	4.249	1.217	3.032	3.29	0.51985	0.58	0.106							0.80	1	
72.671	6	Sandy Silt to Clayey Silt	17.356	0.356	4.287	1.237	3.05	3.22	0.51765	0.58	0.106							0.80	1	
73.327	6	Sandy Silt to Clayey Silt	16.106	0.34	4.325	1.258	3.067	3.28	0.51558	0.58	0.105							0.80	1	
73.983	6	Sandy Silt to Clayey Silt	16.209	0.315	4.362	1.278	3.084	3.23	0.51354	0.57	0.105							0.80	1	
74.639	6	Sandy Silt to Clayey Silt	25.883	0.658	4.4	1.299	3.101	3.08	0.51151	0.57	0.105							0.80	1	
75.295	6	Sandy Silt to Clayey Silt	37.78	0.934	4.437	1.319	3.118	2.92	0.5095	0.57	0.105							0.80	1	
75.951	6	Sandy Silt to Clayey Silt	33.8	0.814	4.475	1.34	3.135	2.96	0.5075	0.56	0.105							0.80	1	
76.608	6	Sandy Silt to Clayey Silt	24.273	0.578	4.513	1.36	3.152	3.11	0.50551	0.56	0.104							0.79	1	
77.264	6	Sandy Silt to Clayey Silt	17.034	0.384	4.55	1.381	3.169	3.28	0.50355	0.56	0.104							0.79	1	
77.92	6	Sandy Silt to Clayey Silt	16.242	0.305	4.588	1.401	3.187	3.28	0.50148	0.56	0.104							0.79	1	
78.576	6	Sandy Silt to Clayey Silt	26.577	0.662	4.625	1.422	3.204	3.09	0.49955	0.55	0.104							0.79	1	
79.232	6	Sandy Silt to Clayey Silt	31.895	0.943	4.663	1.442	3.221	3.05	0.49762	0.55	0.104							0.79	1	
79.888	7	Silty Sand to Sandy Silt	44.561	0.903	4.702	1.463	3.239	2.81	0.49561	0.55	0.104							0.79	1	
80.545	7	Silty Sand to Sandy Silt	42.35	0.872	4.74	1.483	3.257	2.83	0.49361	0.55	0.103							0.79	1	
81.201	7	Silty Sand to Sandy Silt	59.27	1.433	4.779	1.504	3.275	2.74	0.49162	0.54	0.103							0.79	1	
81.857	5	Clayey Silt to Silty Clay	48.765	1.785	4.816	1.524	3.292	2.95	0.48976	0.54	0.103							0.79	1	
82.513	5	Clayey Silt to Silty Clay	28.788	0.962	4.854	1.545	3.31	3.15	0.4878	0.54	0.103							0.79	1	
83.169	6	Sandy Silt to Clayey Silt	33.178	1.011	4.892	1.565	3.327	3.08	0.48597	0.54	0.103							0.79	1	
83.825	5	Clayey Silt to Silty Clay	46.478	1.628	4.929	1.585	3.344	2.98	0.48415	0.54	0.103							0.79	1	
84.482	5	Clayey Silt to Silty Clay	45.653	1.619	4.967	1.606	3.361	3	0.48235	0.53	0.103							0.78	1	
85.138	6	Sandy Silt to Clayey Silt	63.703	1.818	5.004	1.626	3.378	2.81	0.48056	0.53	0.102							0.78	1	
85.794	5	Clayey Silt to Silty Clay	45.348	1.717	5.042	1.647	3.395	3.02	0.47878	0.53	0.102							0.78	1	
86.45	5	Clayey Silt to Silty Clay	39.304	1.629	5.08	1.667	3.412	3.1	0.47702	0.53	0.102							0.78	1	
87.106	5	Clayey Silt to Silty Clay	58.259	2.326	5.117	1.688	3.429	2.94	0.47526	0.53	0.102							0.78	1	
87.762	7	Silty Sand to Sandy Silt	98.808	1.606	5.156	1.708	3.448	2.5	0.47332	0.52	0.102	2.500	0.47	46.77	2.768	129.474	0.28	0.78	1	3.29
88.419	7	Silty Sand to Sandy Silt	91.394	1.657	5.194	1.729	3.466	2.56	0.4715	0.52	0.102	2.560	0.47	43.09	3.091	133.178	0.30	0.78	1	3.51
89.075	8	Sand to Silty Sand	106.632	1.474	5.234	1.749	3.485	2.44	0.46958	0.52	0.102	2.440	0.47	50.07	2.483	124.337	0.26	0.78	1	3.03
89.731	7	Silty Sand to Sandy Silt	119.582	2.413	5.273	1.77	3.503	2.49	0.46779	0.52	0.102	2.490	0.47	55.94	2.718	152.063	0.41	0.78	1	4.77
90.387	8	Sand to Silty Sand	142.294	1.822	5.312	1.79	3.522	2.31	0.4659	0.52	0.102	2.310	0.47	66.30	1.981	131.347	0.29	0.78	1	3.41
91.043	9	Sand	170.902	2.103	5.353	1.811	3.543	2.23	0.46384	0.52	0.102	2.230	0.46	79.27	1.743	138.207	0.33	0.76	1	3.82
91.699	7	Silty Sand to Sandy Silt	224.319	5.206	5.392	1.831	3.561	2.32	0.46209	0.52	0.101	2.320	0.46	103.66	2.015	208.820		0.73	1	
92.356	12	Sand to Clayey Sand	203.463	8.387	5.432	1.852	3.58	2.53	0.46025	0.51	0.101	2.530	0.46	93.64	2.925	273.887		0.77	1	
93.012	11	Sitff Fine-Grained	149.468	7.965	5.474	1.872	3.602	2.71	0.45814	0.51	0.101							0.77	1	
93.668	11	Sitff Fine-Grained	112.626	5.557	5.517	1.893	3.625	2.78	0.45596	0.51	0.101							0.77	1	
94.324	11	Sitff Fine-Grained	67.901	3.548	5.56	1.913	3.647	2.98	0.45389	0.51	0.101							0.77	1	
94.98	11	Sitff Fine-Grained	107.374	5.127	5.603	1.934	3.669	2.79	0.45184	0.51	0.101							0.77	1	
95.636	11	Sitff Fine-Grained	108.772	5.684	5.646	1.954	3.692	2.82	0.44971	0.51	0.101							0.77	1	
96.293	11	Sitff Fine-Grained	120.494	4.839	5.688	1.974	3.714	2.7	0.4477	0.51	0.101							0.77	1	
96.949	11	Sitff Fine-Grained	90.695	4.768	5.731	1.995	3.736	2.89	0.44571	0.50	0.101							0.77	1	
97.605	7	Silty Sand to Sandy Silt	108.28	2.17	5.77	2.015	3.754	2.55	0.44409	0.50	0.101	2.550	0.44	48.09	3.034	145.903	0.37	0.77	1	4.38
98.261	9	Sand	135.966	1.034	5.811	2.036	3.775	2.23	0.44221	0.50	0.100	2.230	0.44	60.13	1.743	104.827	0.19	0.77	1	2.22
98.917	9	Sand	120.346	0.743	5.851	2.056	3.795	2.24	0.44044	0.50	0.100	2.240	0.44	53.01	1.770	93.845	0.16	0.77	1	1.86
99.573	9	Sand	123.245	0.714	5.892	2.077	3.815	2.22	0.43868	0.50	0.100	2.220	0.44	54.07	1.717	92.840	0.15	0.77	1	1.84
100.23	9	Sand	118.348	0.523	5.933	2.097	3.835	2.19	0.43694	0.50	0.100	2.190	0.44	51.71	1.643	84.942	0.14	0.76	2	1.63
100.886	9	Sand	108.717	0.86	5.974	2.118	3.856	2.34	0.43513	0.50	0.100	2.340	0.44	47.31	2.084	98.574	0.17	0.76	3	2.01
101.542	7	Silty Sand to Sandy Silt	106.681	1.874	6.012	2.138	3.874	2.54	0.43358	0.50	0.100	2.540	0.43	46.26	2.979	137.792	0.32	0.76	4	3.86
102.198	5	Clayey Silt to Silty Clay	37.955	1.355	6.05	2.159	3.891	3.15	0.43214	0.49	0.100							0.76	5	
102.854	6	Sandy Silt to Clayey Silt	22.242	0.51	6.087	2.179	3.908	3.3	0.4307	0.49	0.100							0.76	6	
103.511	6	Sandy Silt to Clayey Silt	18.748	0.344	6.125	2.2	3.925	3.32	0.42927	0.49	0.100							0.76	7	
104.167	6	Sandy Silt to Clayey Silt	19.321	0.333	6.163	2.22	3.942	3.28	0.42785	0.49	0.100							0.76	8	
104.823	6	Sandy Silt to Clayey Silt	20.381	0.352	6.2	2.241	3.96	3.25	0.42636	0.49	0.100							0.76	9	
105.479	6	Sandy Silt to Clayey Silt	22.177	0.301	6.238	2.261	3.977	3.16	0.42496	0.49	0.100							0.76	10	
106.135	6	Sandy Silt to Clayey Silt	35.089	0.645	6.275	2.282	3.994	3.02	0.42357	0.49	0.100							0.76	11	
106.791	8	Sand to Silty Sand	83.013	0.994	6.315	2.302	4.013	2.56	0.42202	0.49	0.100	2.560	0.42	35.03	3.091	108.272	0.20	0.76	12	2.37
107.448	7	Silty Sand to Sandy Silt	100.189	1.721	6.354	2.322	4.031	2.57	0.42057	0.49	0.100	2.570	0.42	42.14	3.148	132.644	0.30	0.76	13	3.56
108.104	7	Silty Sand to Sandy Silt	87.932	1.569	6.392	2.343	4.049	2.63	0.41913	0.49	0.100							0.76	14	
108.76	7	Silty Sand to Sandy Silt	59.319	0.843	6.431	2.363	4.067	2.75	0.4177	0.48	0.100							0.76	15	
109.416	6	Sandy Silt to Clayey Silt	24.159	0.502	6.469	2.384	4.085	3.28	0.41627	0.48	0.099							0.75	16	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. The result was used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The result of the test is presented on Plate B-2, "Plasticity Chart".

Grain Size Classification

Grain size classification tests (ASTM Test Method D 420) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plate B-3, "Grain Size Distribution Curves".

Unconfined Compression Tests

Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-6A and B-6B.

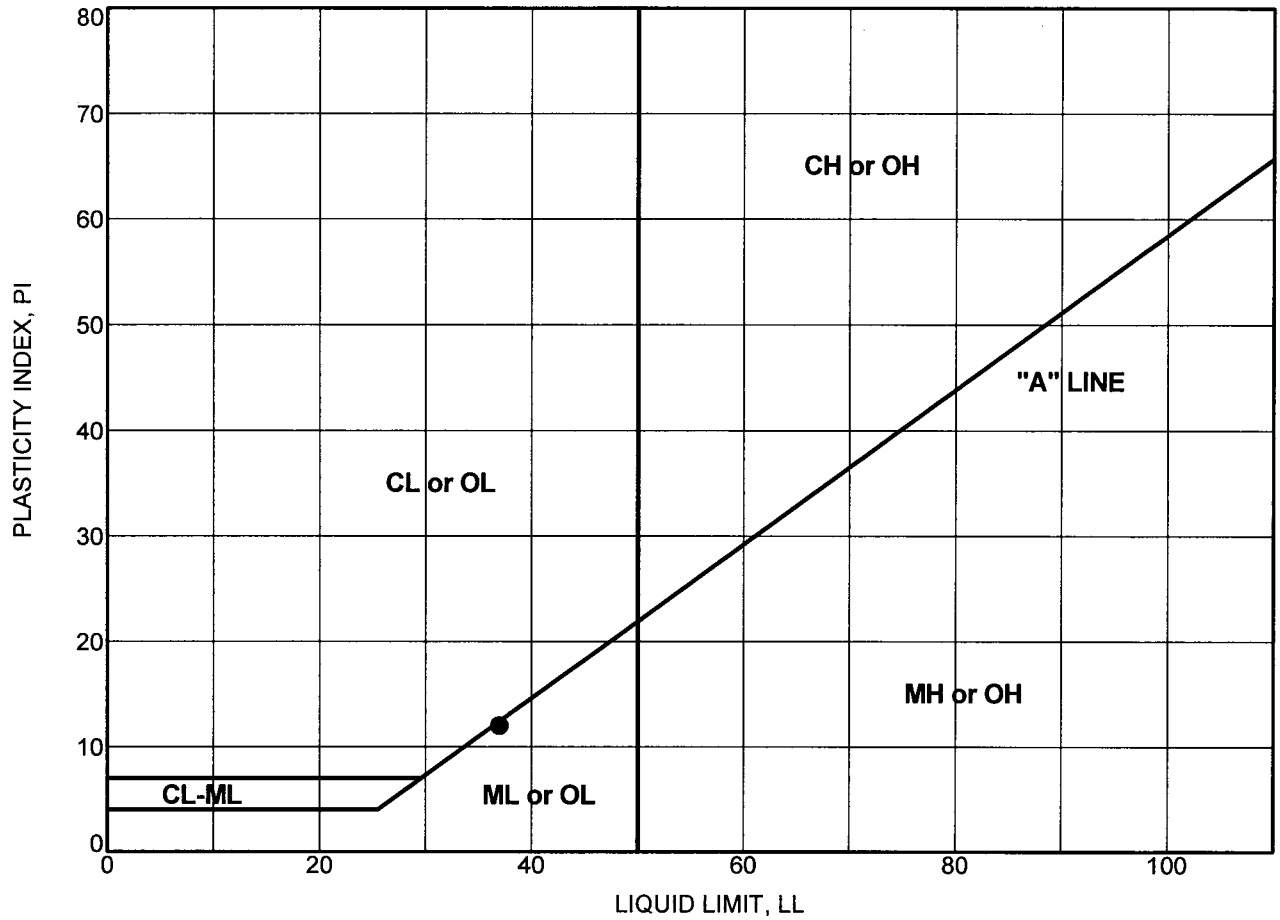
Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-4.

LABORATORY TESTS
(Continued)

Direct Shear Tests

Direct Shear test was performed on selected relatively undisturbed sample to determine the shear strength of a soil material in direct shear. The test was performed according to ASTM Test Method D 3080 by Cooper . The test result is presented on Plate B-5.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
R-09-001	MC-14	56.0	●	29	37	25	12	SILT (ML)

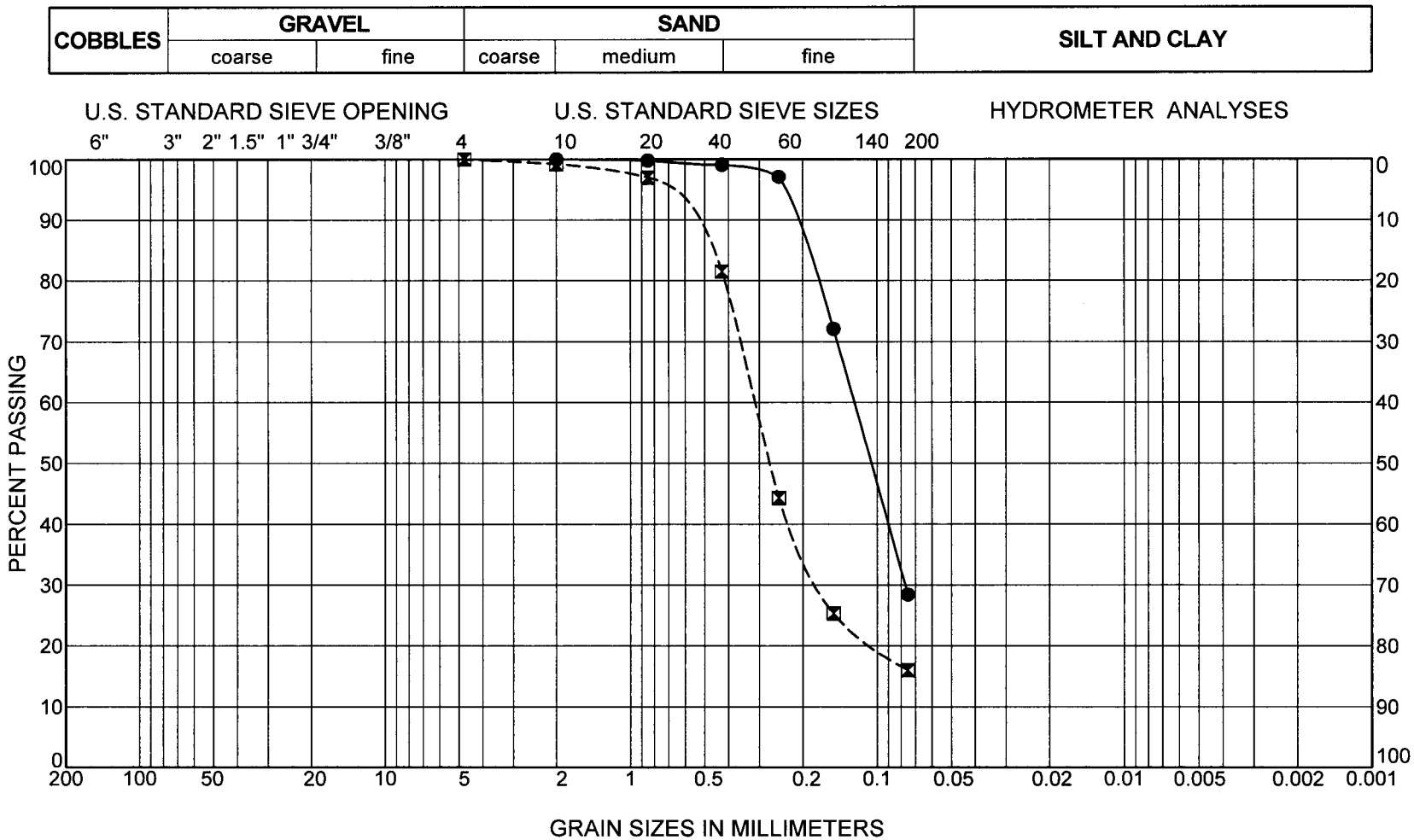
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Telegraph Avenue UC (Widen)
Stockton, California

JOB NO: 206127.BR (TAB)

PLATE NO: B-2

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
R-09-001	SPT-10	40.5	●			SILTY SAND (SM)
R-09-001	SPT-20	96.0	⊠			SILTY SAND (SM)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

JOB NO: 206127.BR (TAB)

Telegraph Avenue UC (Widen)
Stockton, California

PLATE NO: B-3

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/29/2009
Date Submitted 04/24/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR Site ID : R-09-TA-001#5@26.0'
Thank you for your business.

* For future reference to this analysis please use SUN # 55554-111854.

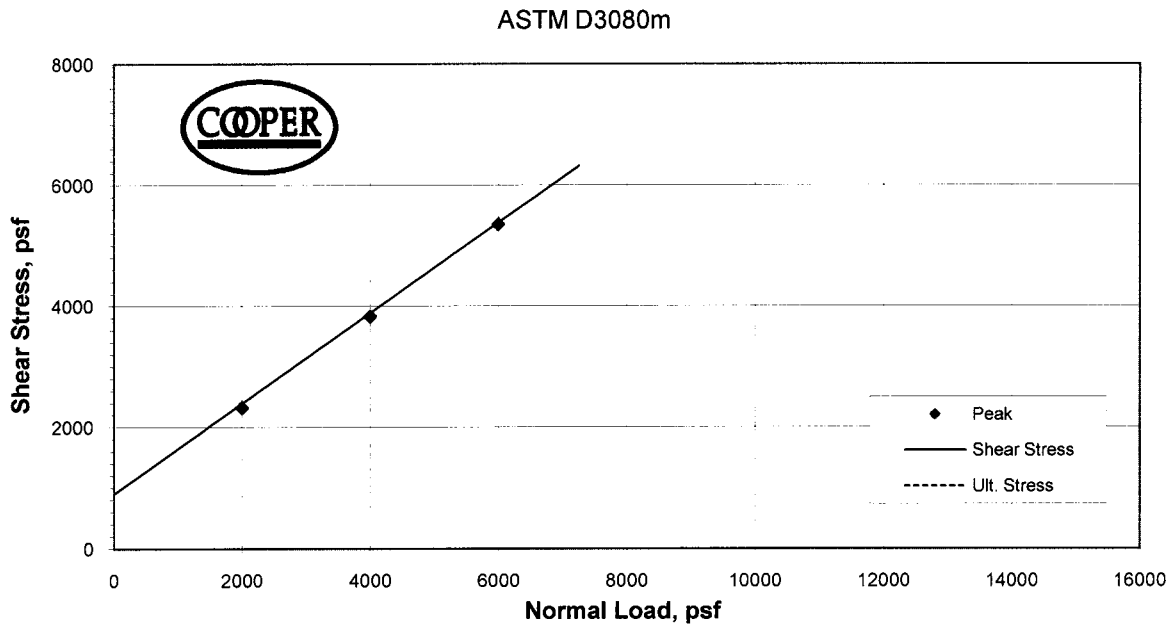
EVALUATION FOR SOIL CORROSION

Soil pH	6.86		
Minimum Resistivity	2.76 ohm-cm (x1000)		
Chloride	21.6 ppm	00.00216	%
Sulfate	35.2 ppm	00.00352	%

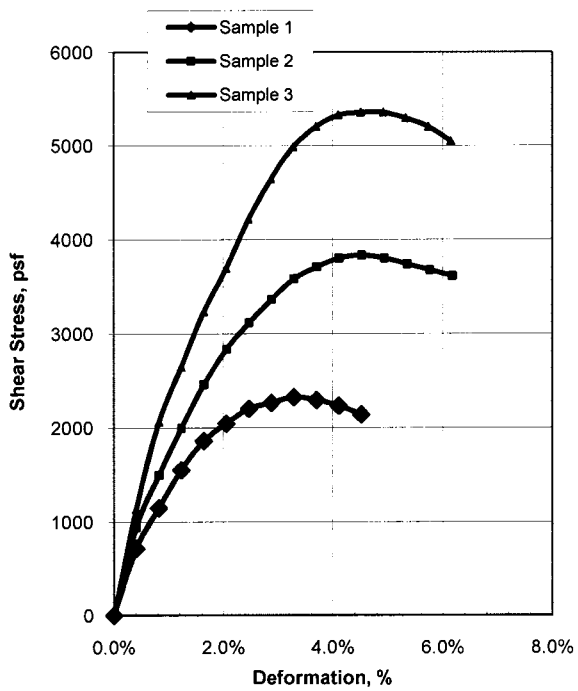
METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 1 2009
By

Direct Shear



P. Phi (degrees)	36.8	Ult. Phi (degrees)	
P. Cohesion(psf)	900	Ult. Cohesion (psf)	



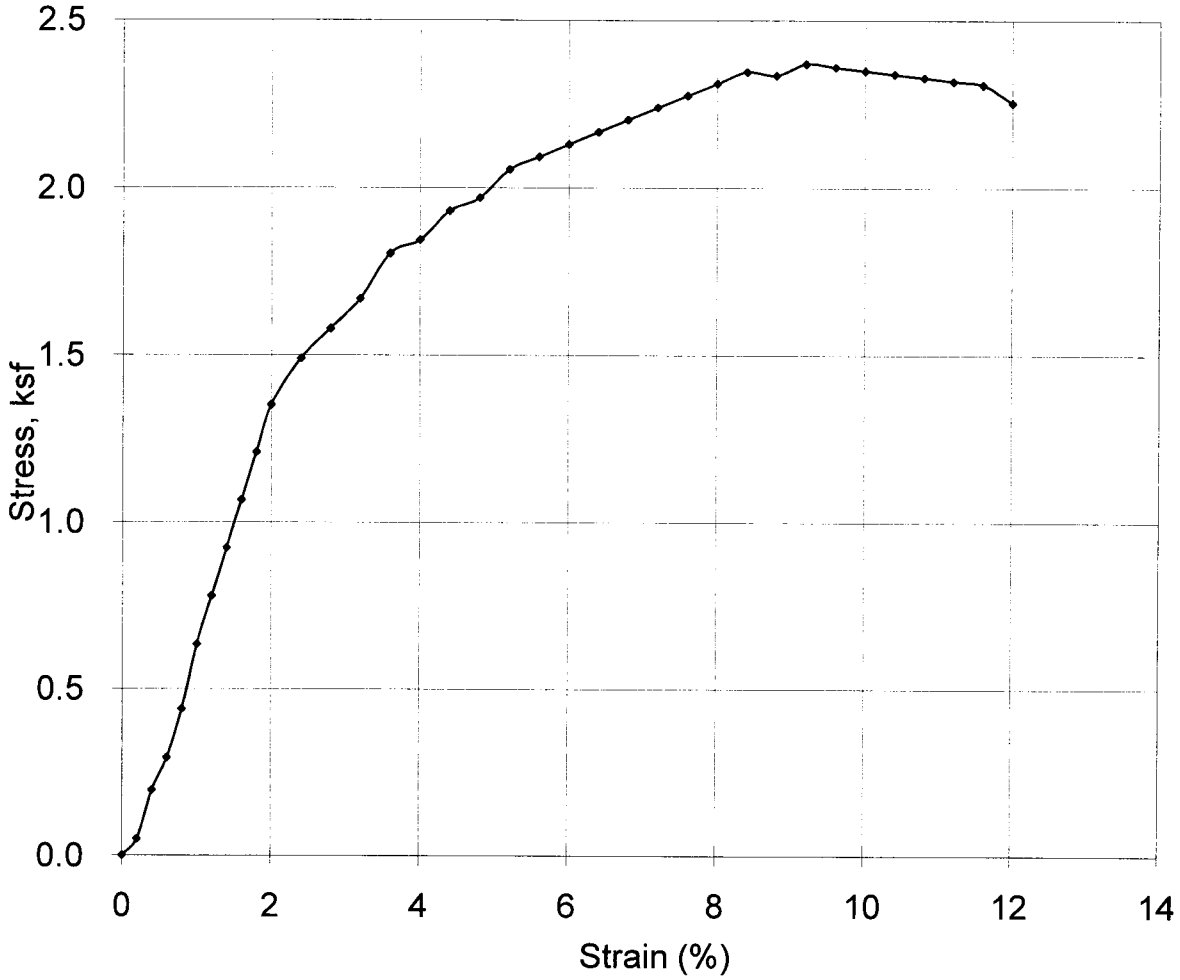
	Sample Data: Initial			
	1	2	3	4
Moisture %	22.0	22.0	22.2	
Dry Dens., pcf	103.9	105.6	103.5	
Void Ratio	0.622	0.596	0.629	
Saturation %	95.5	99.8	95.5	
Diameter	2.43	2.43	2.44	
Height	1.01	1.01	1.03	
	Sample Data: At Test			
Moisture %	22.0	20.5	21.1	
Dry Dens., pcf	105.5	108.5	107.3	
Void Ratio	0.598	0.553	0.571	
Saturation %	99.2	100.0	99.7	
Diameter	2.43	2.43	2.44	
Height	1.00	0.98	0.99	
Normal Stress, psf	2000	4000	6000	
Shear Stress, psf	2329	3835	5358	
Strengths picked at Ult. Stress, psf	3.3%	4.5%	4.9%	
Strain Rate, in/min	0.020	0.020	0.020	

CTL #	157-272
Client:	Parikh Consultants, Inc.
Project	206127.GDR
Tested By:	MD
Reduced By:	RU
Date:	4/20/2009

Specimen #	Boring:	Sample:	Depth, ft:	Visual Soil Classification
1	R-09-TA-001	#8	36.0	Dark Bluish Gray Silty SAND
2	R-09-TA-001	#8	36.0	Dark Bluish Gray Silty SAND
3	R-09-TA-001	#8	36.0	Dark Bluish Gray Silty SAND
4				

Remarks: *DS-CU* A fully undrained condition may not be attained in this test.

UNCONFINED COMPRESSION TEST



Boring No.: R-09-TA-001

Sample No. : MC-4

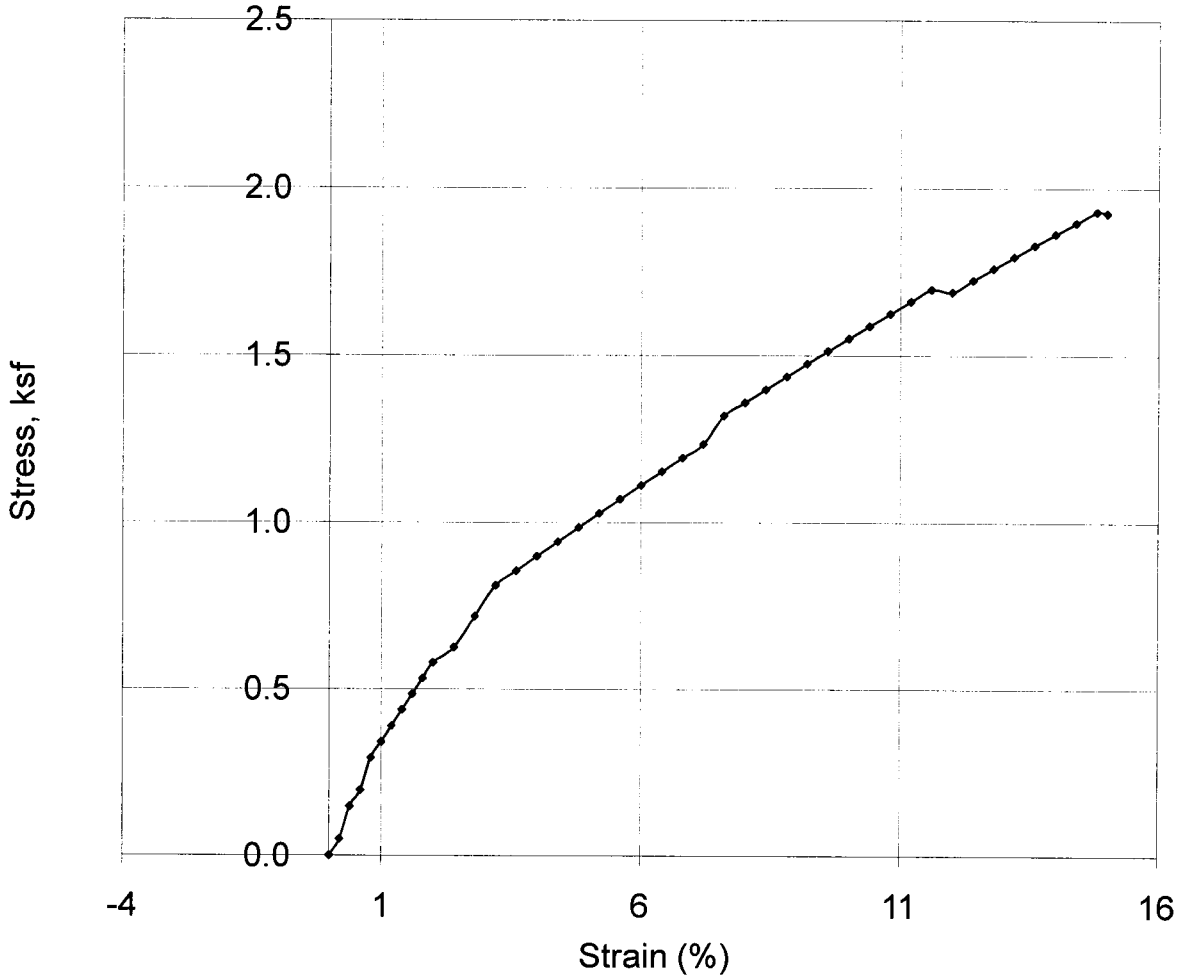
Depth (feet): 21

Material Description:
Organic Clay

Unconfined Compressive Strength: 2.3 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**TELEGRAPH AVENUE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-6A

UNCONFINED COMPRESSION TEST



Boring No.: R-09-TA-001

Sample No. : MC-14

Depth (feet): 56

Material Description:
Silt with Sand

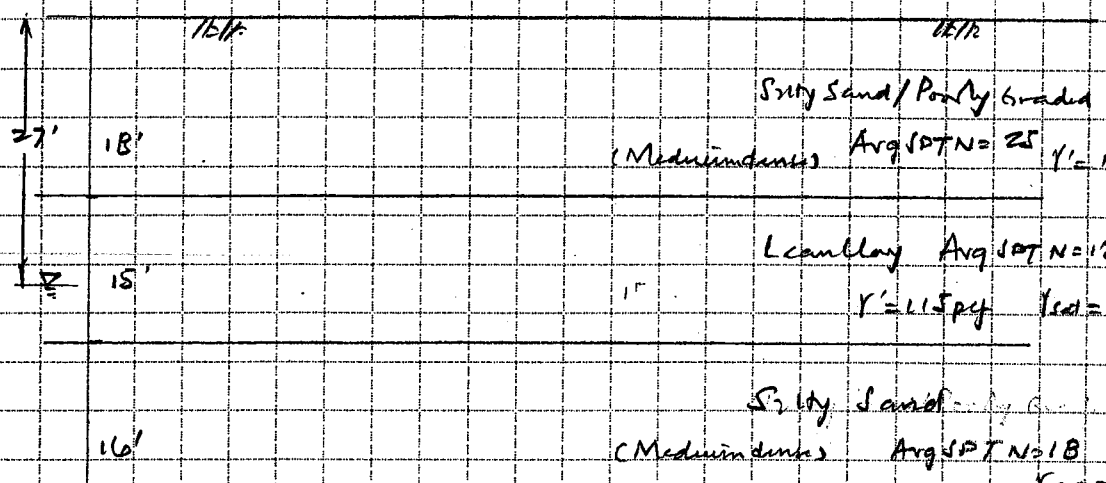
Unconfined Compressive Strength: 1.9 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**TELEGRAPH AVENUE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-6B

APPENDIX C

Evaluation of Liquefaction Potential

Evaluation of Liquefaction Potential - Telegraph Avenue (Boring R-09-TA-001)



LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***R-09-TA-001 (Telegraph Avenue UC)(Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 27

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w *=* 0.7

MSF = 1.19
(Min. FS = 1.1 based on SPT)
F.S.=(CRR$_{7.0}$*/CSR)*MSF*K*σ**K*α

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	18.0	1	25	SPT	2160.0	2160.0	0.92	0.12	29	0.96	1	0.85	1.2	1.15	32.56	15%	36.62		0.98	1	
2	33.0	2	12	SPT	3975.0	3600.0	0.82		14	0.75	1	0.80	1.2	1.15	11.39	55%			0.84	1	
3	49.0	1	18	SPT	6055.0	4680.0	0.66	0.11	21	0.65	1	0.80	1.2	1.15	14.99	28%	21.62	0.24	0.77	1	1.96
4	61.0	2	15	SPT	7615.0	5490.0	0.57		17	0.60	1	0.80	1.2	1.15	11.53	55%			0.74	1	
5	69.0	2	30	SPT	8655.0	6030.0	0.53		35	0.58	1	0.80	1.2	1.15	22.01	55%			0.72	1	
6	76.0	2	8	SPT	9530.0	6467.5	0.51		9	0.56	1	0.80	1.2	1.15	5.67	55%			0.70	1	
7	92.0	2	39	SPT	11610.0	7547.5	0.48		45	0.51	1	0.80	1.2	1.15	25.57	55%			0.67	1	
8	111.5	1	21	SPT	14145.0	8863.8	0.46	0.09	24	0.48	1	0.80	1.2	1.15	12.71	16%	16.16	0.17	0.64	1	1.38

Vertical Pile Capacity Calculations/Lateral Pile Capacity Analyses

Rajappan & Meyer
CONSULTING ENGINEERS INC

Project Name: ***I-5 North Stockton Corridor Improvement***
Bridge Name: ***Telegraph Ave UC (Widen)***
Br No: ***29-0192 R/L***
Date: ***April 15, 2010***

FOUNDATION DESIGN INFORMATION FROM BRIDGE DESIGNER TO GEOTECHNICAL ENGINEER

TELEGRAPH AVE UC (WIDEN)

Table 1. Foundation Design Data Sheet

Location	Design Method	Pile Type	Finish Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		No. of Piles Per Support
					B	L	
Abut 1	WSD	Class 140 Alt X (Mod)	14.10±	10.80±	2.50	51.00	12
Abut 2	WSD	Class 140 Alt X (Mod)	13.90±	10.60±	2.50	51.00	12

Note: Finish Grade is at the front face of abutment diaphragm

Table 2. Foundation Design Loads

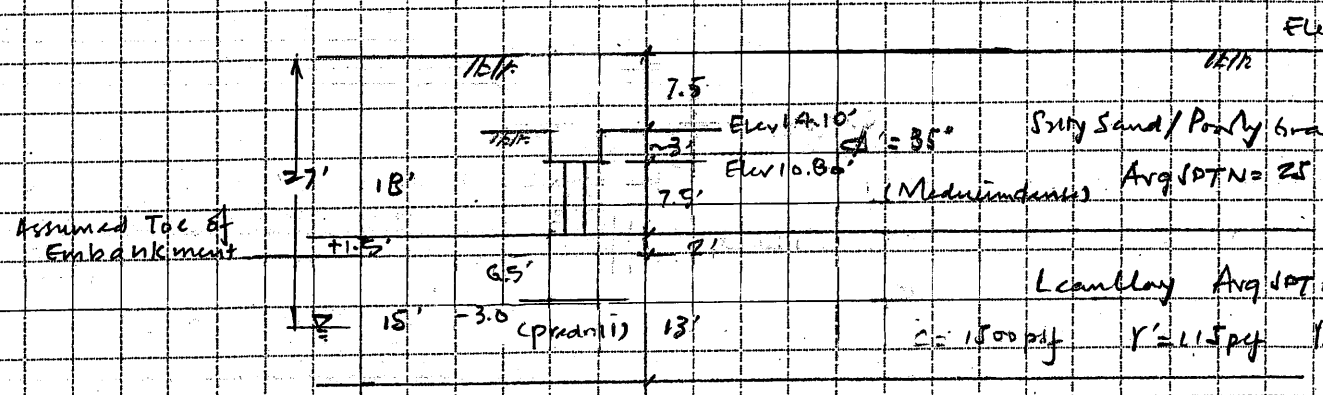
Location	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent	Compression		Tension		Compression		Tension	
	Per Support	Max per Pile	Per Support	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile
Abut 1	1580	132	1055	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1580	132	1055	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 3. Lateral Load on Piles

Per Support	Service Limit (kips/pile)	Max per Pile
Abut 1	Neglig ble	25
Abut 2	Neglig ble	25

Pile capacity Analysis - Telegraph Ave UC (Bridge R-09-001) Abutment 1

14"□ precast concrete pile - 138 tons service load



Silty Sand / Poorly Graded Sand
$\phi' = 35°$
Avg SPT N = 25
(Medium dense)
$\gamma' = 120$ pcf

Lean Clay Avg SPT N = 12
c = 1500 psf $\gamma' = 115$ pcf $\gamma_{sat} = 130$ pcf

Silty Sand
16'
(Medium dense) Avg SPT N = 18
$\phi' = 37°$
$\gamma_{sat} = 130$ pcf

12'
Silt with Sand Avg SPT N = 15
c = 750 psf $\gamma_{sat} = 125$ pcf

8'
Lean Clay SPT N = 30 $\gamma_{sat} = 130$ pcf
c = 3500 psf

7'
c = 1000 psf Lean Clay SPT N = 8 $\gamma_{sat} = 125$ pcf

Lean Clay Avg SPT N = 38
16'
c = 4000 psf $\gamma_{sat} = 125$ pcf

Poorly Graded Sand Avg SPT N = 21
19.5'
(Medium dense) $\gamma_{sat} = 130$ pcf

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Telegraph Ave. UC (Widen)*** ***Abut 1 - 70 tons service load (140 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 03/31/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.17 — 1- GRAVELS, SANDS AND NONPLASTIC SILTS — (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 2 (1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 1 (1=CONCRETE ,2=STEEL) — SOIL TYPES: GRAVEL, SAND, NSILT Ref Boring : A-09-TA-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

DRIVEN/DRILLED = 1 (1=DRIVEN ,2=DRILLED) — CLAY, PSILT — AREA OF THE PILE TIP 1.35 SQ. FT.

Class 140 Alt. "X" - 14" square precast prestressed c

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	120	360	180	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	10.5		1	120	900	810	SAND	DENSE	0	36	0.00	18.36	18.36	6.15	6.15
10.5	17.0		2	115	747.5	1634	CLAY	STIFF	1500	0	0.80	17.55	35.91	17.55	23.70
17.0	19.5		2	115	287.5	2151	CLAY	STIFF	1500	0	0.80	6.75	42.66	6.75	30.45
19.5	25.5		2	68	408	2499	CLAY	STIFF	1500	0	0.80	16.20	58.86	16.20	46.65
25.5	30.5		1	68	340	2873	SAND	LOOSE	0	32	0.00	21.93	80.79	7.35	54.00
30.5	35.5		1	68	340	3213	SAND	LOOSE	0	32	0.00	21.81	102.60	7.31	61.30
35.5	41.5		1	68	408	3587	SAND	LOOSE	0	32	0.00	26.37	128.98	8.83	70.14
41.5	45.5		2	63	252	3917	CLAY	STIFF	1750	0	0.75	11.81	140.79	11.81	81.95
45.5	49.5		2	63	252	4169	CLAY	STIFF	1750	0	0.75	11.81	152.60	11.81	93.76
49.5	53.5		2	63	252	4421	CLAY	STIFF	1750	0	0.75	11.81	164.41	11.81	105.58
53.5	58.0		2	68	306	4700	CLAY	V.STIFF	3500	0	0.55	19.49	183.90 ←	19.49	125.07
58.0	61.5		2	68	238	4972	CLAY	V.STIFF	3500	0	0.55	15.16	199.06	15.16	140.23
61.5	65.0		2	63	220.5	5201	CLAY	MEDIUM	1000	0	0.85	6.69	205.76	6.69	146.92
65.0	68.5		2	63	220.5	5422	CLAY	MEDIUM	1000	0	0.85	6.69	212.45	6.69	153.61
68.5	72.5		2	63	252	5658	CLAY	V.STIFF	4000	0	0.20	7.20	219.65	7.20	160.81
72.5	76.5		2	63	252	5910	CLAY	V.STIFF	4000	0	0.20	7.20	226.85	7.20	168.01
76.5	80.5		2	63	252	6162	CLAY	V.STIFF	4000	0	0.20	7.20	234.05	7.20	175.21
80.5	84.5		2	63	252	6414	CLAY	V.STIFF	4000	0	0.20	7.20	241.25	7.20	182.41
84.5	90.5		1	68	408	6744	SAND	MED DEN	0	34	0.00	28.16	269.42	9.44	191.85
90.5	96.5		1	68	408	7152	SAND	MED DEN	0	34	0.00	28.02	297.44	9.39	201.24
96.5	104.0		1	68	510	7611	SAND	MED DEN	0	34	0.00	34.84	332.27	11.67	212.91

Abutment 1 --> Pile cut-off elevation is +10.8 feet. Finished grade elevation is +14.10 feet

Pre-drill through the exsiting embankment to bottom of pipe. Elevation of bottom of pipe is Elev. -3.0 feet (assumed to the depth of 17.0 feet)

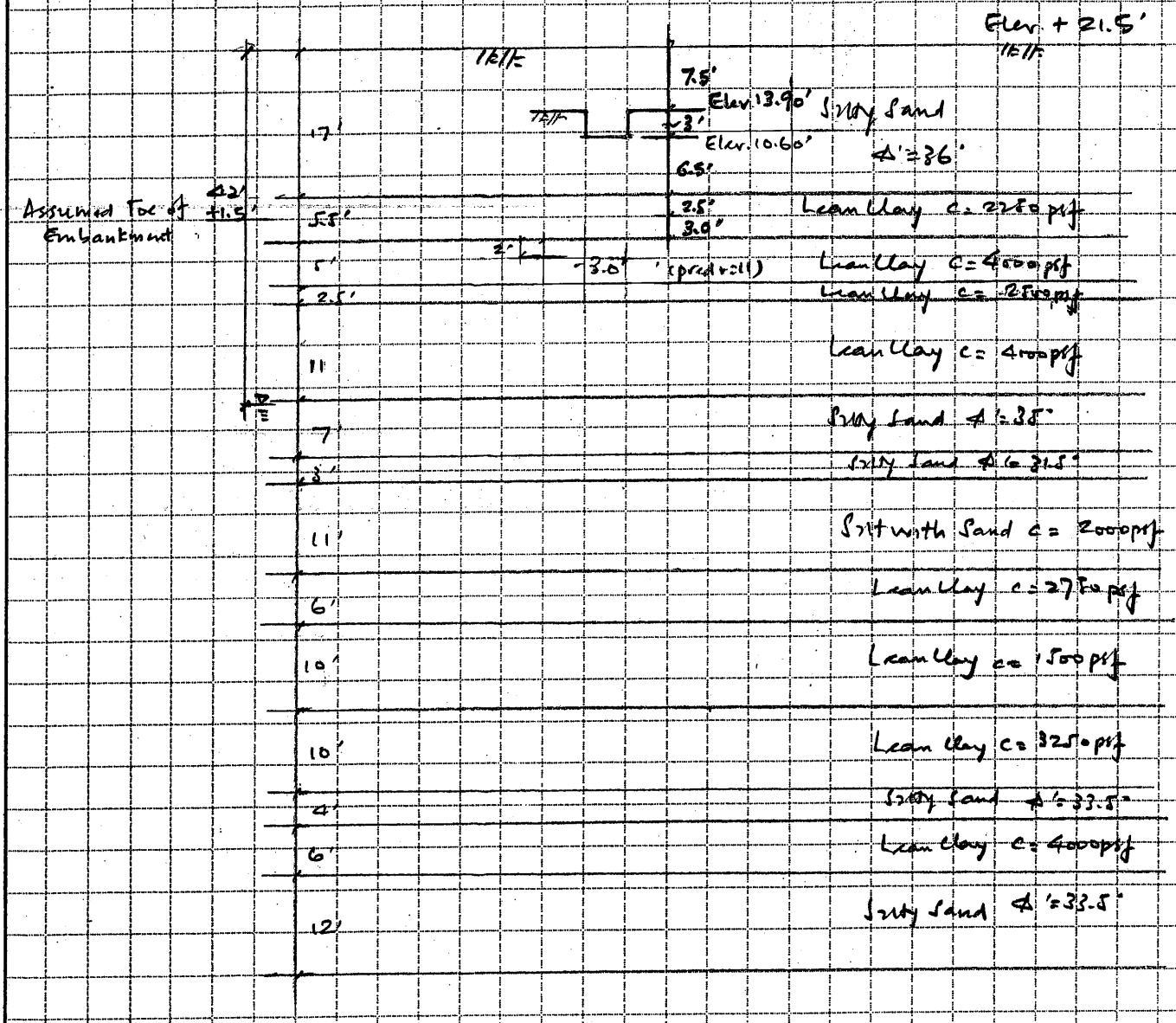
Pile Length @ Abutment 1 = 58.0' - 3.0' = 55.0'

Pile tip at 55.0' below pile cut-off (pile tip @ Elev. -44.5', pile cut-off @ Elev. +10.8'), Ult. Pile Capacity =183.9 tons (Friction)-35.9 tons (Predrilled)=148.0 tons > 280K/140T (Service Limit State), OK

I-5 Telegraph Ave. Abut 1 Class 140 Alt X (3-31-10)

Sheet 2 of 5

Pile Capacity Analysis · Telegraph Ave UC (Warden) (CPT-09-001) Abutment 2 14" □ precast concrete pile 138 tons service load



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Telegraph Ave. UC (Widen)*** ***Abut 2 - 70 tons service load (140 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 03/31/10

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS INCREAMENTS

1- GRAVELS, SANDS (FT)= 5

AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : CPT-09-TA-001 (PARIKH 2009)

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) : 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 140 Alt. "X" - 14" square precast prestressed c

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	120	360	180	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	9.5		1	120	780	750	SAND	DENSE	0	36	0.00	14.77	14.77	4.95	4.95
9.5	12.0		2	115	287.5	1284	CLAY	V.STIFF	2250	0	0.55	6.96	21.73	6.96	11.91
12.0	15.0		2	115	345	1600	CLAY	V.STIFF	2250	0	0.55	8.35	30.08	8.35	20.26
15.0	17.0		2	120	240	1893	CLAY	V.STIFF	4000	0	0.25	4.50	34.58	4.50	24.76
17.0	22.5		2	115	632.5	2329	CLAY	V.STIFF	2500	0	0.40	12.38	46.96	12.38	37.14
22.5	28.0		2	120	660	2975	CLAY	V.STIFF	4000	0	0.25	12.38	59.33	12.38	49.51
28.0	33.5		2	120	660	3635	CLAY	V.STIFF	4000	0	0.25	12.38	71.71	12.38	61.89
33.5	34.5		1	120	120	4025	SAND	MED DEN	0	35	0.00	3.77	75.48	1.26	63.15
34.5	40.5		1	68	408	4289	SAND	MED DEN	0	35	0.00	22.52	98.00	7.54	70.69
40.5	43.5		1	68	204	4595	SAND	MED DEN	0	33	0.00	10.45	108.45	3.50	74.19
43.5	49.0		2	68	374	4884	CLAY	STIFF	2000	0	0.90	22.28	130.72	22.28	96.47
49.0	54.5		2	68	374	5258	CLAY	STIFF	2000	0	0.90	22.28	153.00	22.28	118.74
54.5	57.5		2	68	204	5547	CLAY	V.STIFF	2750	0	0.60	11.14	164.14	11.14	129.88
57.5	60.5		2	68	204	5751	CLAY	V.STIFF	2750	0	0.60	11.14	175.27	11.14	141.02
60.5	63.0		2	68	170	5938	CLAY	STIFF	1500	0	0.90	7.59	182.87 ←	7.59	148.61
63.0	70.5		2	68	510	6278	CLAY	STIFF	1500	0	0.90	22.78	205.65	22.78	171.39
70.5	75.5		2	68	340	6703	CLAY	V.STIFF	3250	0	0.40	14.63	220.27	14.63	186.02
75.5	80.5		2	68	340	7043	CLAY	V.STIFF	3250	0	0.40	14.63	234.90	14.63	200.64
80.5	84.5		1	68	272	7349	SAND	MED DEN	0	34	0.00	14.64	249.54	4.91	205.55
84.5	90.5		2	68	408	7689	CLAY	V.STIFF	4000	0	0.40	21.60	271.14	21.60	227.15
90.5	102.5		1	68	816	8301	SAND	MED DEN	0	34	0.00	43.47	314.61	14.56	241.71

Abutment 2 --> Pile cut-off elevation is +10.60 feet. Finished grade elevation is +13.90 feet

Pre-drill through the exsiting embankment to bottom of pipe. Elevation of bottom of pipe is Elev. -3.0 feet (assumed to the depth of 17.0 feet)

Pile Length @ Abutment 2 = 63.0' - 3.0' = 60.0'

Pile tip at 60.0' below pile cut-off (pile tip @ Elev. -49.5', pile cut-off @ Elev. +10.60'), Ult. Pile Capacity =182.9 tons (Friction)-34.6 tons (Predrilled)=148.3 tons > 280K/140T (Service Limit State), OK

I-5 Telegraph Ave. Abut 2 Class 140 Alt X (3-31-10)

Sheet 4 of 5

TELEGRAPH AVE UC (WIDEN)
Sheet 5 of 5

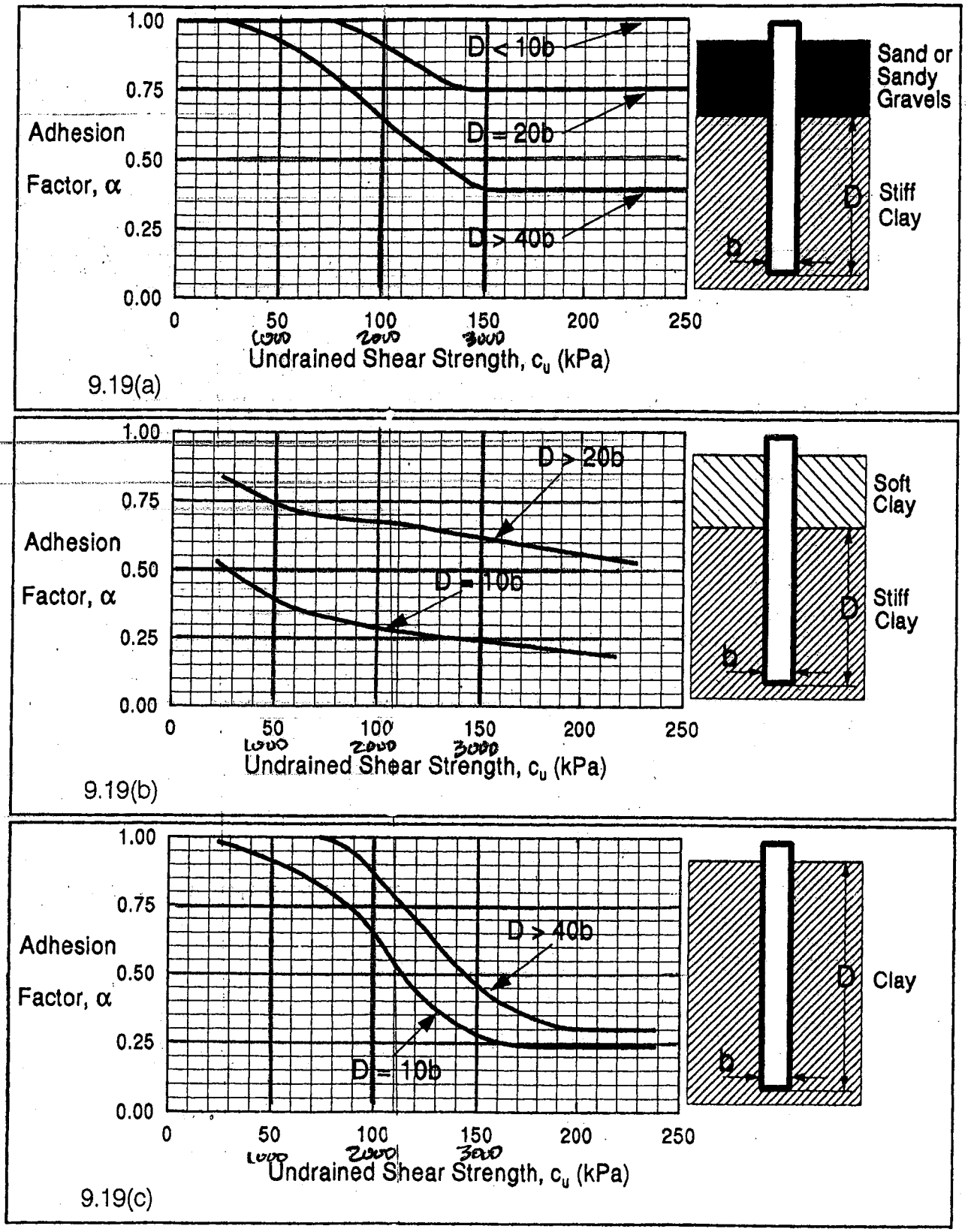
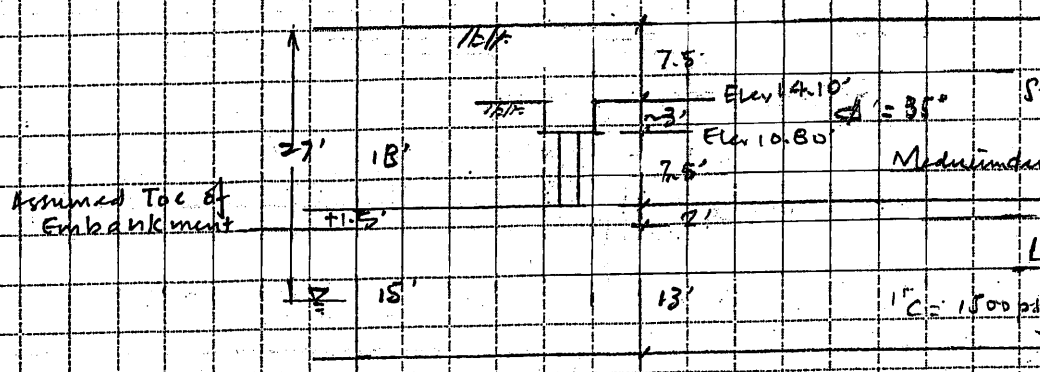


Figure 9.19 Adhesion Factors for Driven Piles in Clay (after Tomlinson, 1980)

Lateral Pile capacity Analysis - Telegraph Ave UC (Bonny R-09-001) Abutment 1

14"□ precast concrete pile - 133 tons service load



Silty Sand / Poorly Graded Sand
(Medium dense) Avg SPT N = 25 $\gamma' = 120$ pcf
$\phi' = 35°$ $K = 90$ lb/in^3

Lean Clay Avg SPT N = 12
$c = 1500$ psf $\gamma' = 115$ pcf $\gamma_{sat} = 130$ pcf
$\varepsilon_{50} = 0.007$

16' Silty Sand
(Medium dense) Avg SPT N = 18
$\phi' = 37°$ $K = 60$ lb/in^3 $\gamma_{sat} = 130$ pcf

12' Silt with Sand Avg SPT N = 15
$\varepsilon_{50} = 0.007$ $c = 1750$ psf $\gamma_{sat} = 125$ pcf

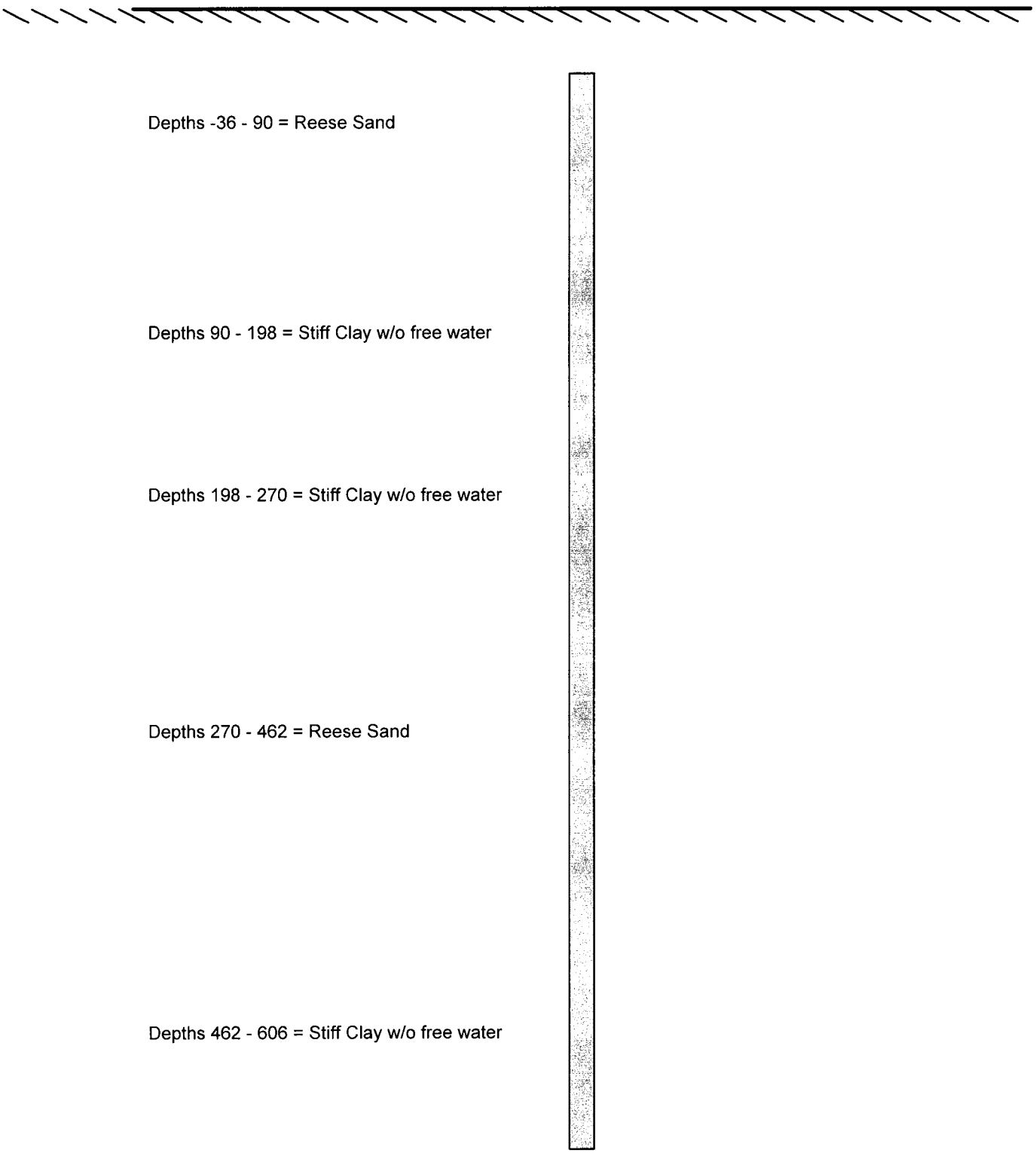
8' Lean Clay SPT N = 30 $\gamma_{sat} = 130$ pcf
$c = 3500$ psf $\varepsilon_{50} = 0.005$

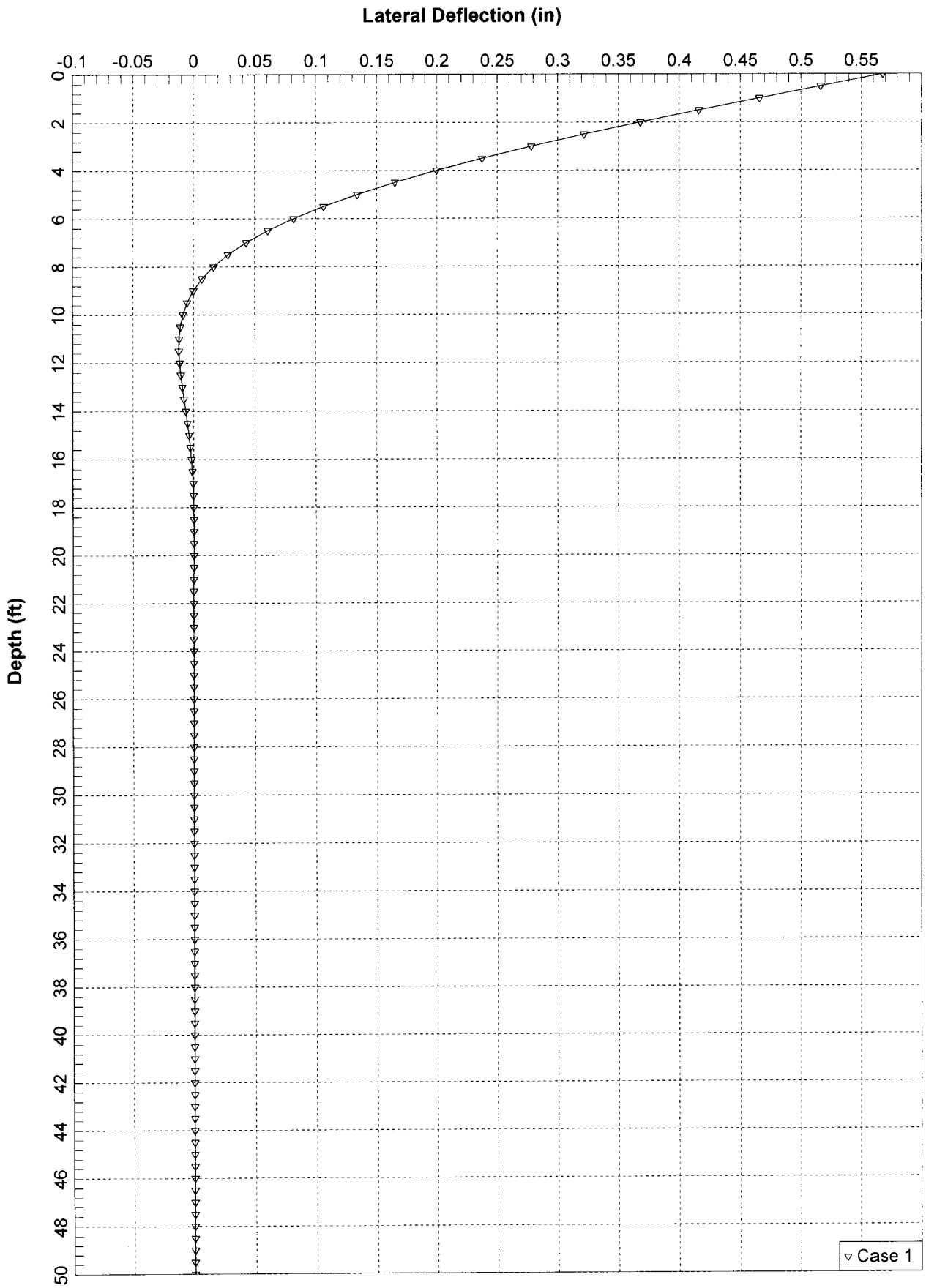
7' $c = 1000$ psf Lean Clay SPT N = 8 $\gamma_{sat} = 125$ pcf

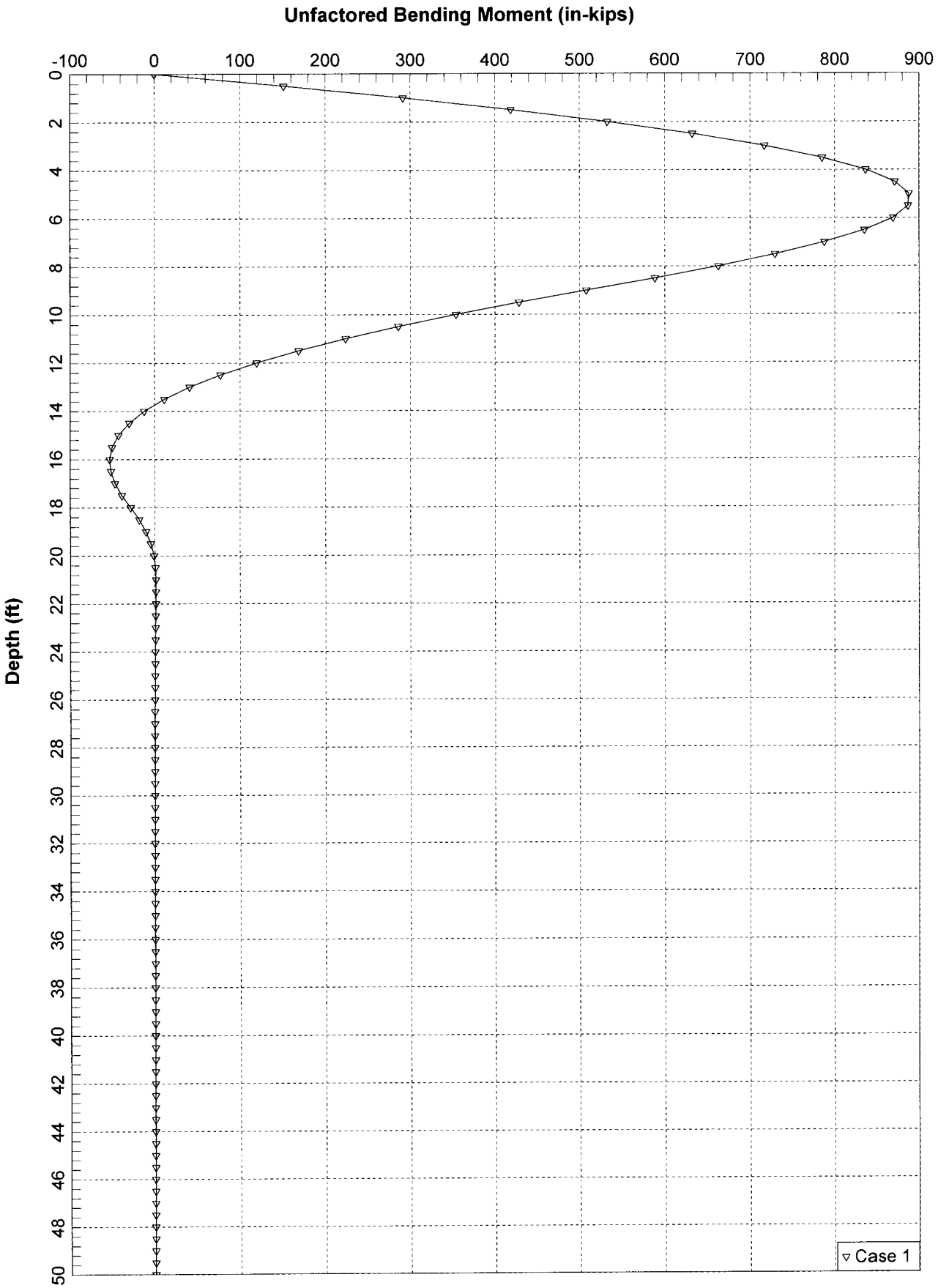
16' Lean Clay Avg SPT N = 38
$c = 4000$ psf $\gamma_{sat} = 125$ pcf

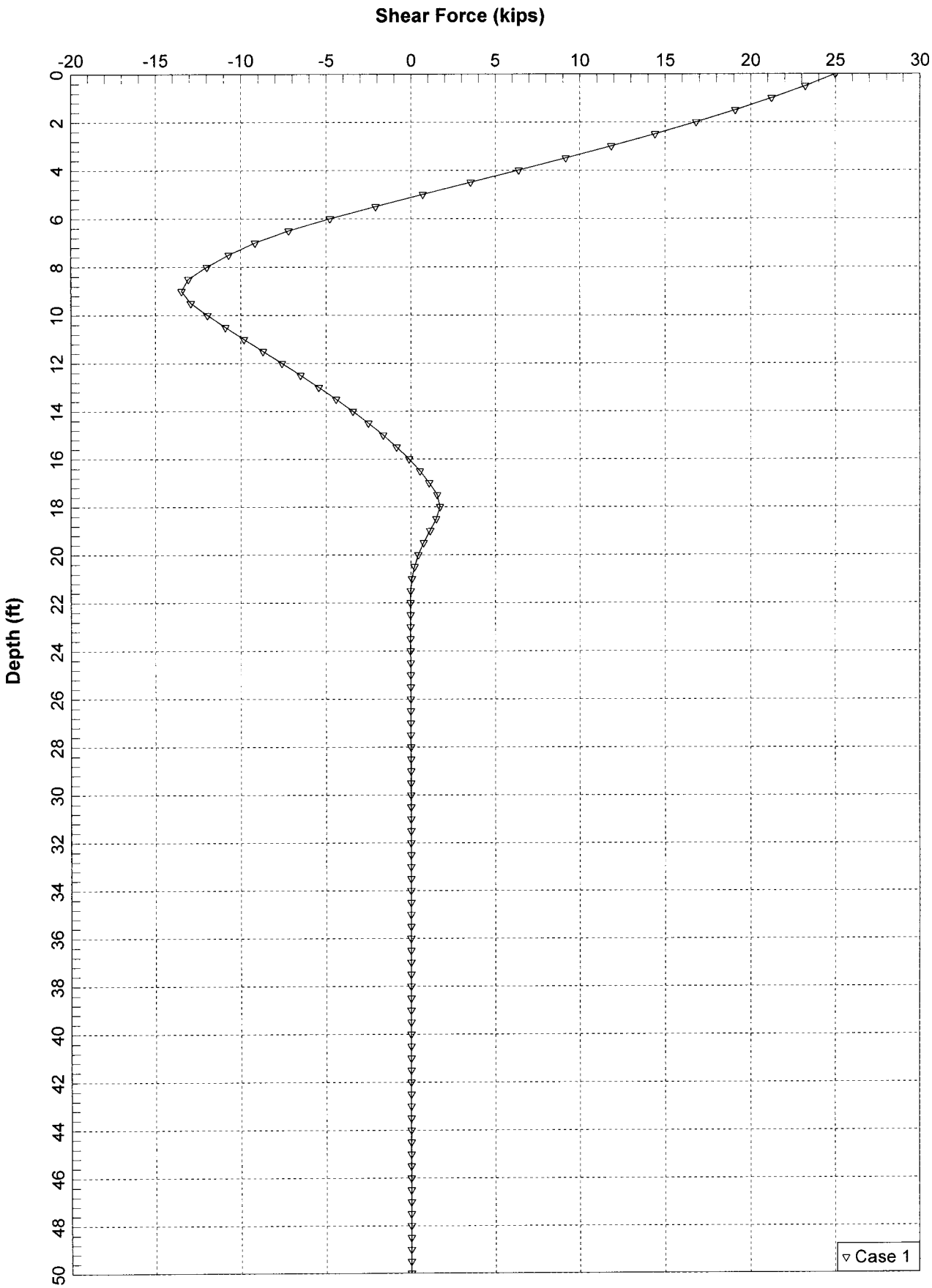
19.5' Poorly Graded Sand Avg SPT N = 21
(Medium dense) $\gamma_{sat} = 130$ pcf

```
LPILEP5
I-5 Stockton-Telegraph-Abut-1
1       1       0       0       0       0
100     2       -36     600     0
0       14      3112    194     3000000
600     14      3112    194     3000000
6       12      12      0       2
4       -36     90      90      90
3       90      198     0       0
3       198     270     0       0
4       270     462     60      60
3       462     606     0       0
3       606     702     0       0
-36     0.069
90      0.069
90      0.067
198     0.067
198     0.039
270     0.039
270     0.039
462     0.039
462     0.036
606     0.036
606     0.039
702     0.039
-36     0       35      0       0
90      0       35      0       0
90      10.4    0       0.007   0
198     10.4    0       0.007   0
198     10.4    0       0.007   0
270     10.4    0       0.007   0
270     0       32      0       0
462     0       32      0       0
462     12.15   0       0.007   0
606     12.15   0       0.007   0
606     24.3    0       0.005   0
702     24.3    0       0.005   0
0       0.6     1
1248    0.6     1
0       1       1
1
1       25000   0       125000
0
1       1       0
100     1E-5    100
```


Depths -36 - 90 = Reese Sand
Depths 90 - 198 = Stiff Clay w/o free water
Depths 198 - 270 = Stiff Clay w/o free water
Depths 270 - 462 = Reese Sand
Depths 462 - 606 = Stiff Clay w/o free water
Depths 606 - 702 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
0.55
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1


Unfactored Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
Case 1

PARIKH

Practicing in the Geosciences

JOB I-5 North Stockton – Telegraph Ave UC (Wilden) 2061217.88

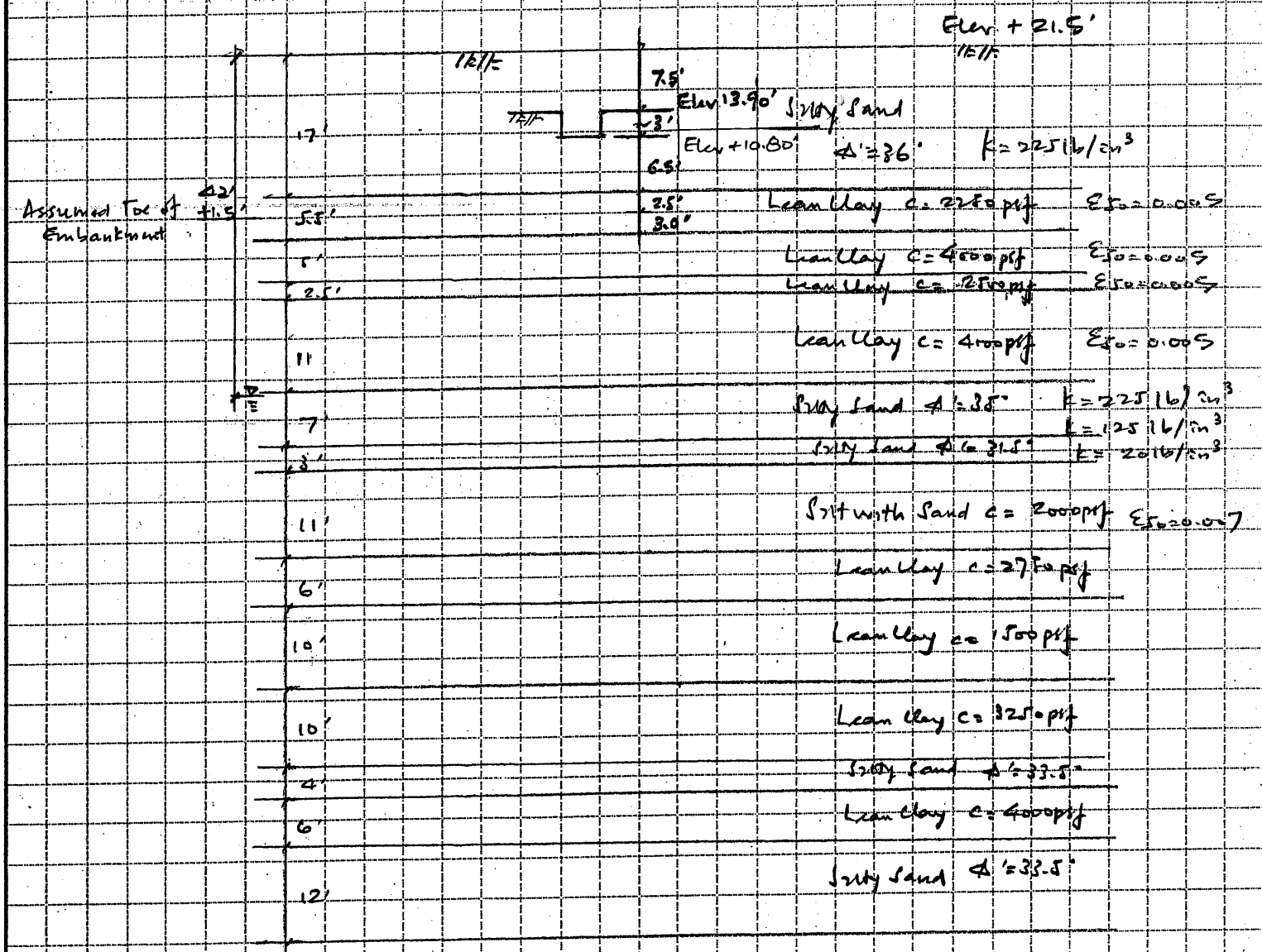
SHEET NO ____ OF ____

CALCULATED BY A Lam DATE 3/2010

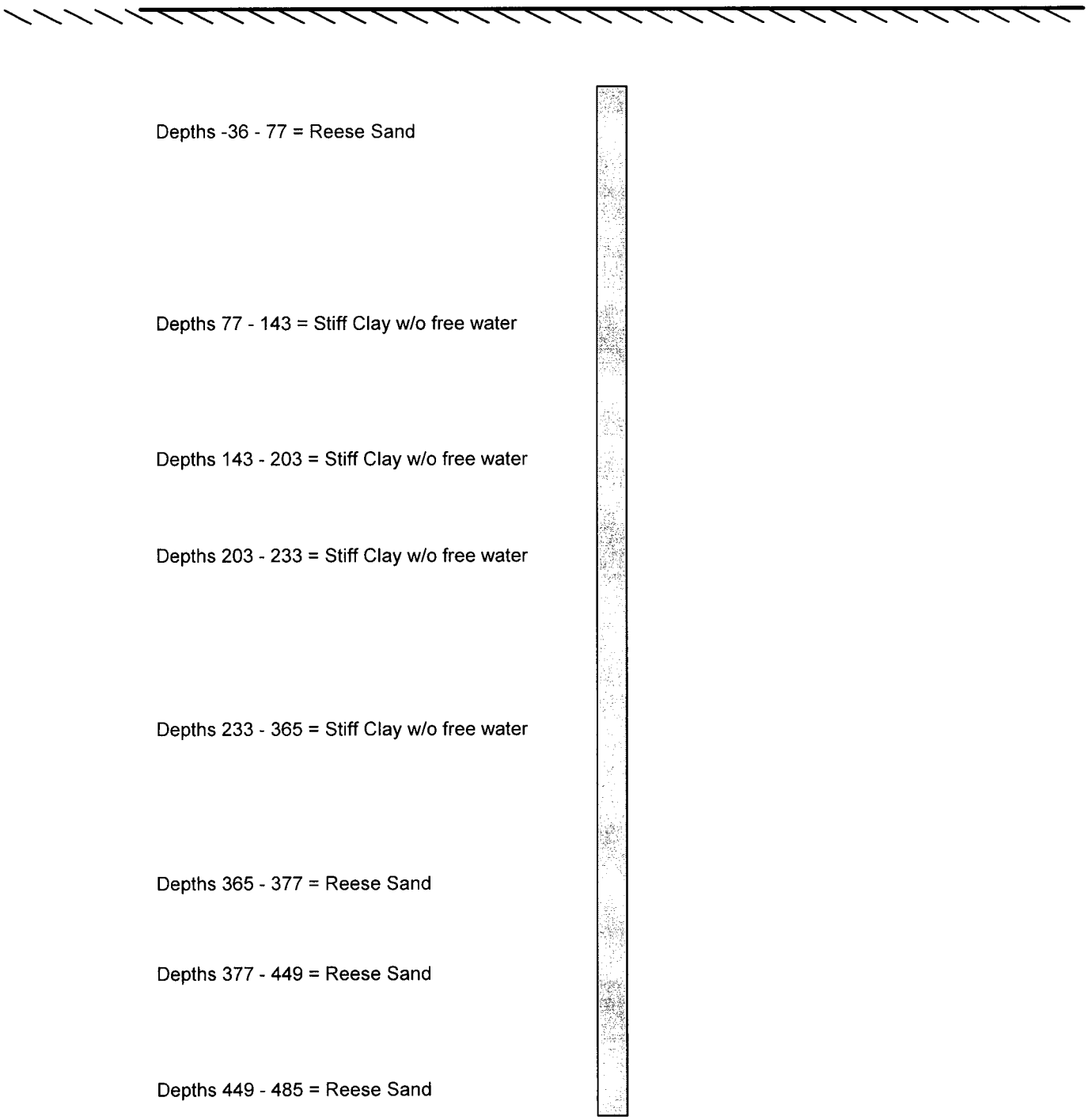
CHECKED BY ____ DATE ____

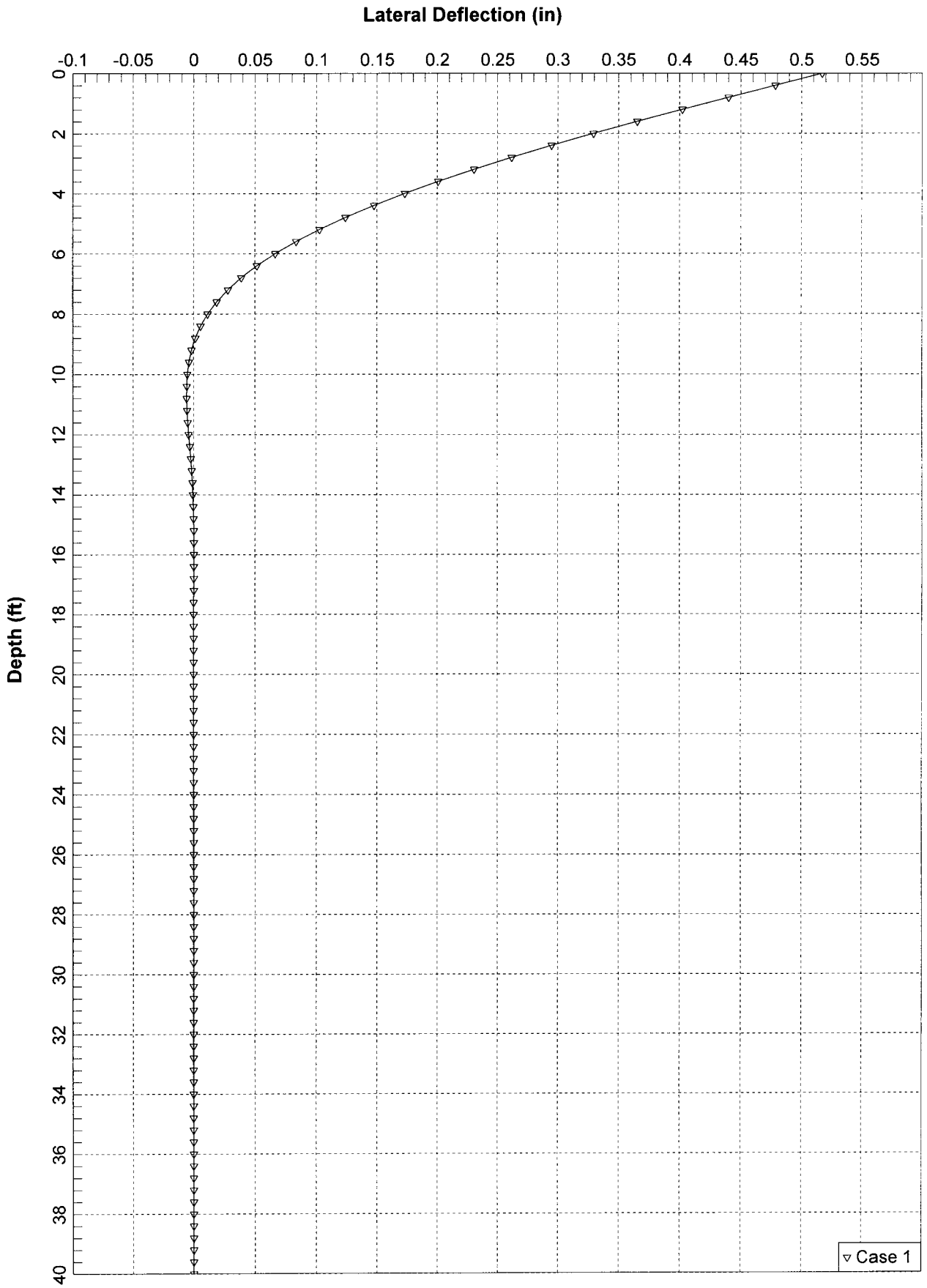
SCALE ____

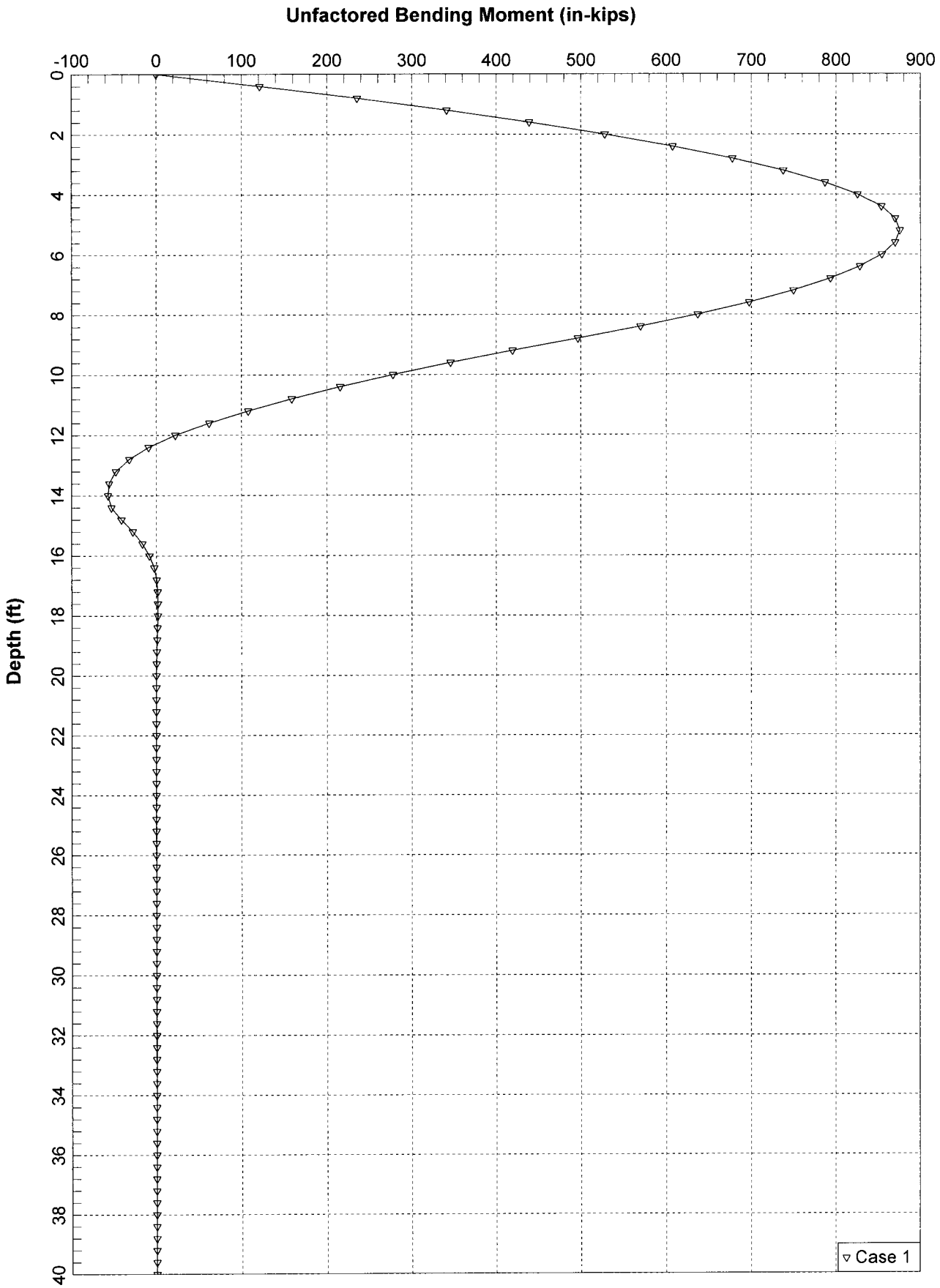
Lateral Pile Capacity Analyses Telegraph Ave UC (Wilden) (CPT-09-001) Abutment 2 14" □ precast concrete pile 133 tons service load

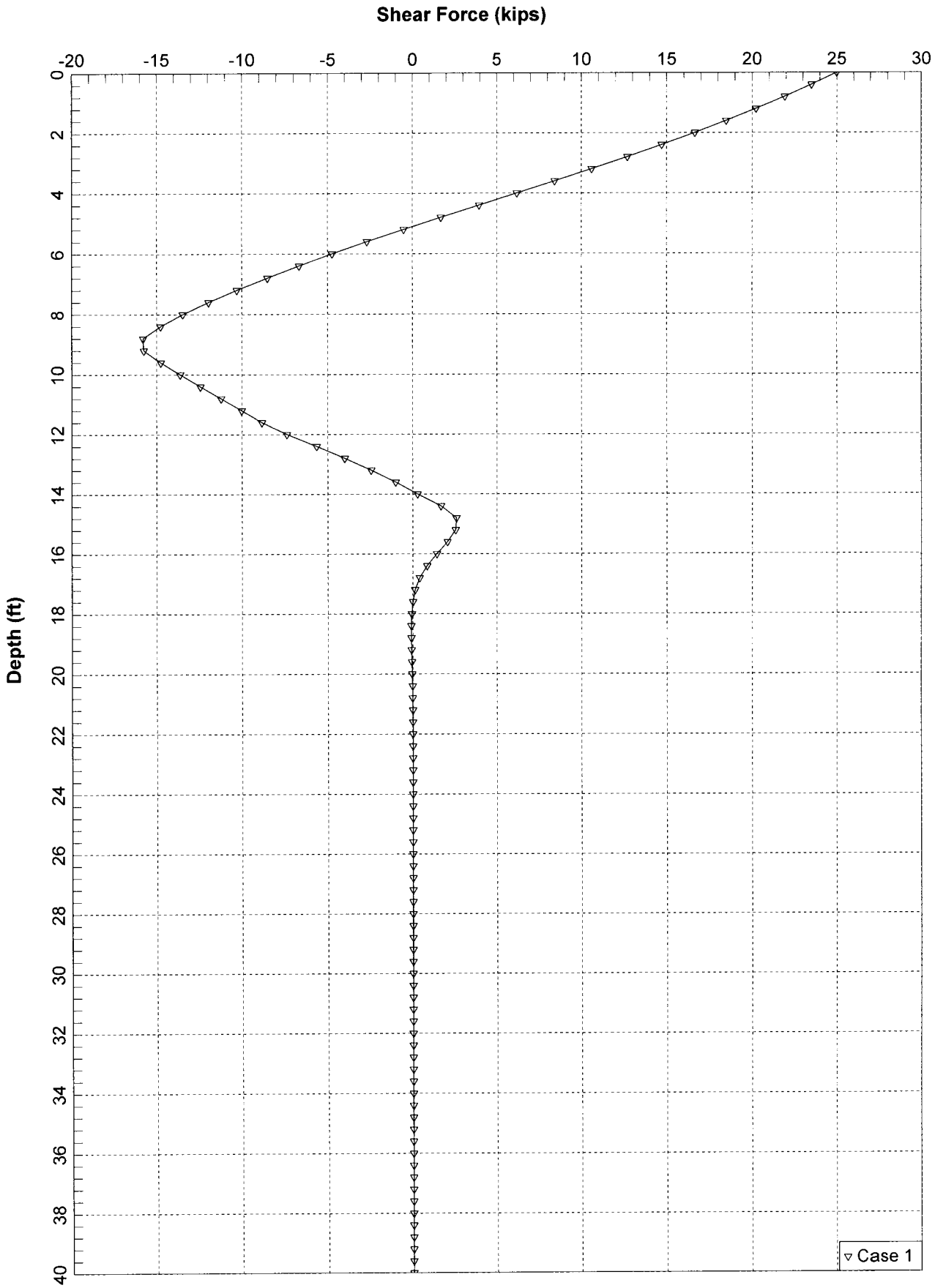


```
LPILEP5
I-5 Stockton Telegraph-Abut-2
1       1       0       0       0       0
100     2       -36     480     0
0       14      3112    194     3000000
480     14      3112    194     3000000
9       18      18      0       2
4       -36     77      225     225
3       77      143     0       0
3       143     203     0       0
3       203     233     0       0
3       233     365     0       0
4       365     377     225     225
4       377     449     125     125
4       449     485     20      20
3       485     617     0       0
-36     0.069
77      0.069
77      0.067
143     0.067
143     0.069
203     0.069
203     0.067
233     0.067
233     0.067
365     0.067
365     0.069
377     0.069
377     0.042
449     0.042
449     0.039
485     0.039
485     0.039
617     0.039
-36     0       36      0       0
77      0       36      0       0
77      15.62   0       0.005   0
143     15.62   0       0.005   0
143     27.77   0       0.005   0
203     27.77   0       0.005   0
203     17.36   0       0.005   0
233     17.36   0       0.005   0
233     27.77   0       0.005   0
365     27.77   0       0.005   0
365     0       35      0       0
377     0       35      0       0
377     0       35      0       0
449     0       35      0       0
449     0       31.5    0       0
485     0       31.5    0       0
485     13.88   0       0.007   0
617     13.88   0       0.007   0
0       0.6     1
1206    0.6     1
0       1       1
1
1       25000   0       125000
0
1       1       0
100     1E-5    100
```


Depths -36 - 77 = Reese Sand
Depths 77 - 143 = Stiff Clay w/o free water
Depths 143 - 203 = Stiff Clay w/o free water
Depths 203 - 233 = Stiff Clay w/o free water
Depths 233 - 365 = Stiff Clay w/o free water
Depths 365 - 377 = Reese Sand
Depths 377 - 449 = Reese Sand
Depths 449 - 485 = Reese Sand
Depths 485 - 617 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
0.55
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Unfactored Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

Lateral Pile Capacity Analysis - Telegraph Ave UC (Widen)

Abutment 1

Depth of Second Point of Counterflexure = 18 feet

Design pile top elevation = 11.35' - 18'

= -6.65' Say -7.0 feet

Abutment 2

Depth of Second Point of Counterflexure = 15.5 feet

Design pile top elevation = 11.15' - 15.5'

= -4.35' Say -4.5 feet

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Telegraph Avenue UC (Widen)

Boring R-09-TA-001 (Abutment 1) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
21.5* to 3.5	0 to 18	Sand (medium dense)	$\phi = 35°$; c= 0 psf	4, Sand	90	-	0.069	-
3.5 to -5.5	18 to 27	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.067	-
-5.5 to -11.5	27 to 33	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.039	Assumed GWL at 27 feet deep below existing ground surface for design.
-11.5 to -27.5	33 to 49	Sand (medium dense)	$\phi = 32°$; c= 0 psf	4, Sand	60	-	0.039	-
-27.5 to -39.5	49 to 61	Clay (stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.036	-
-39.5 to -47.5	61 to 69	Clay (very stiff)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-47.5 to -54.5	69 to 76	Clay (soft to firm)	$\phi = 0°$; c=1000 psf	3, Clay	100	0.01	0.036	-
-54.5 to -70.5	76 to 92	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.036	-
-70.5 to -90.0	92 to 111.5	Sand (medium dense)	$\phi = 31°$; c= 0 psf	4, Sand	60	-	0.039	-

*Finish ground elevation of +14.10 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Telegraph Avenue UC (Widen)

CPT-09-TA-001 (Abutment 2) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
21.5* to 4.5	0 to 17	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.069	-
4.5 to -1.0	17 to 22.5	Clay (very stiff)	$\phi = 0°$; c=2250 psf	3, Clay	1000	0.005	0.067	-
-1.0 to -6.0	22.5 to 27.5	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.069	
-6.0 to -8.5	27.5 to 30	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.067	-
-8.5 to -19.5	30 to 41	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.069	-
-19.5 to -20.5	41 to 42	Sand (dense)	$\phi = 35°$; c= 0 psf	4, Sand	225	-	0.069	-
-20.5 to -26.5	42 to 48	Sand (dense)	$\phi = 35°$; c= 0 psf	4, Sand	125	-	0.042	Assumed GWL at 42 feet deep below existing ground surface for design.
-26.5 to -29.5	48 to 51	Sand (loose)	$\phi = 31.5°$; c= 0 psf	4, Sand	20	-	0.039	
-29.5 to -40.5	51 to 62	Clay (stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.039	-
-40.5 to -46.5	62 to 68	Clay (very stiff)	$\phi = 0°$; c=2750 psf	3, Clay	1000	0.005	0.039	-
-46.5 to -56.5	68 to 78	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.039	-
-56.5 to -66.5	78 to 88	Clay (very stiff)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-66.5 to -70.5	88 to 92	Sand (medium dense)	$\phi = 33.5°$; c= 0 psf	4, Sand	60	-	0.039	-
-70.5 to -76.5	92 to 98	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-76.5 to -88.5	98 to 110	Sand (medium dense)	$\phi = 33.5°$; c= 0 psf	4, Sand	60	-	0.039	-

*Finish ground elevation of +13.90 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection __ 65% PS&E Unchecked Details _X PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 7/8/2010 **Structure Name*:** Telegraph Avenue UC **Br No*:** 29-0192 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	_____	_____	_____	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 4th review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated June 11, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** Telegraph Avenue UC **Br No*:** 29-0192 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	6/07/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	9	Please provide the Nominal Driving Resistance in Table 5.	Comment is incorporated. "Nominal Driving Resistance" is provided in Table 5.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Telegraph Avenue UC **Br No*:** 29-0192 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.co	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection X 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 12/16/2009 **Structure Name*:** Telegraph Avenue UC **Br No*:** 29-0192 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/15/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	3	Under Section 5. As-Built LOTB, there is no information regarding the borings performed in 1997 for the bridge widening.	According to all the as-built LOTB that we received from Rajappan & Meyer, there is no available information for boring performed in 1997 for the bridge widening.	
3	FR	6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
4	FR	8	Tables 2 and 3, Although the pile tip appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Submittal Data (Reviewer to complete)

Dist-EA: 10-0G4701 Reviewer: BHB Str Name*: Telegraph Avenue UC

Date of Review: 12/16/09 Functional Unit: 59-323 Br No*. 29-0192 *=if applicable

5	FR	9	Table 4: Pile data table shows two different design tip elevations for the two abutments. It is recommended to keep the same pile length for ease of pile fabrication.	Comment is incorporated. Same pile length is used in the design of specified pile tip elevation.	
6	FR	12, S9.6	The first sentence of the second paragraph states that the corrosion tests were for Alpine Avenue, this should be corrected to Michigan Avenue.	Comment noted. The subsurface soils are referred to the boring instead of its vicinity to the bridge structure. An excerpt of the report is attached.	
7	FR	LOTB	As is stated in the Soil Legend, the sample number or unconfined compressive strength is generally not located to the left of the graphic on the LOTB.	Comment is incorporated.	
8	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #4 above.	
9	FR	LOTB	No As-Built LOTB for the borings performed in 1997.	According to all the as-built LOTB that we received from Rajappan & Meyer, there is no as-built LOTB for the borings performed in 1997.	
10	FR	NA	C2 – Approved subject to OSFP Verification	NA	

FOUNDATION REPORT
CALAVERAS RIVER BRIDGE (WIDEN)
(BRIDGE NO. 29-0174)
STOCKTON, CALIFORNIA
10-SJ-05, PM 29.56 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

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June 11, 2010 Job No. 206127.BR

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Log of Test Borings (Br. No. 29-0174)
As-built Log of Test Borings (Br. No. 29-0174 R/L)
CPT Correlation Data (CPT-09-CR-001)

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Laboratory Test ..... Plate B-1
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APPENDIX C

Evaluation of Liquefaction Potential
Vertical Pile Capacity Calculations
Geotechnical Parameters for LPILE (v. 5) Analyses

APPENDIX D

Calaveras River Bridge (Widen) General Plan

APPENDIX E

Office of Special Funded Projects Comment & Response Form (with PARIKH Consultants, Inc. Responses)

FOUNDATION REPORT
CALAVERAS RIVER BRIDGE (WIDEN)
(BRIDGE NO. 29-0174)
STOCKTON, CALIFORNIA
10-SJ-05 PM 29.56 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Calaveras River Bridge (Widen) (Bridge No. 29-0174)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Calaveras River Bridge (Widen) (Bridge No. 29-0174).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the subsurface soil conditions during construction nor is it practical to

determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are eleven-span structures. Based on the as-built general plan and foundation plan of the Calaveras River Bridge, the structures were constructed in 1968. The existing left and right bridges (Bridge No. 29-0174 L/R) are approximately 520 feet long and 60 feet wide. The left and right bridges have not been widened since their construction in 1968.

The abutments of the existing left and right bridge are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class I 45 tons) based on the as-built data.

Proposed Construction

The widening of the Calaveras River Bridge is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. There will be no outside widening for the Calaveras River Bridge.

According to Rajappan & Meyer Consulting Engineers Inc., the proposed Calaveras River Bridge (Widen) will match the existing span with supports at abutments and bents. The widen portion over the Calaveras River will be a standard PC/PS concrete "I" girder supported on drop bent cap. The widen portion over River Drive and Brookside Road will be a CIP concrete T-beam structure supported on integral bent caps. The total structure will have an approximate length of 520 feet measured along the "I-5" Line. The width of the inside widening will be 48 feet and 7 inches. The abutments will incorporate diaphragm type abutments supported on piles.

The construction operation of Abutment 1 (south abutment), Abutment 12 (north abutment), and Bents 2, 3, 10 and 11 will be completely located outside of the top of the levees, and Bents 4, 5 and

9 will be located in the levees. Bents 6, 7 and 8 will be located within the river and will be constructed with the use of temporary construction trestle. The temporary trestle will extend approximately from Bent 4 to Bent 9 and be supported on piles within the river bed and levees.

Based on our discussions with the designer, the proposed bridge abutments and Bents 2 and 11 will be supported on Caltrans Standard Precast Prestressed Class 200 Alternate "X" (14-inch square) piles. Bents 3 through 10 will be supported on 24-inch diameter Cast-In-Drilled-Hole (CIDH) concrete piles. Retrofit Bent 5a will be supported on 60-inch diameter CIDH concrete piles.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The "Calaveras River Bridge" is located at the intersection of Interstate 5 (I-5) and Calaveras River. It is approximately 0.4 miles south of March Lane Undercrossing (UC). The Project Location Map, Plate 1, shows the location of the site.

There are existing "levees" between River Drive and the river and between Brookside Road and the river. Based on the "Elevation" in the "Calaveras River Bridge (Widen) – General Plan" prepared by Biggs Cardosa Associates (a copy is include in Appendix D), the gradient of the landside slope is 2(H): 1(V) and the gradient of the waterside slope is 2(H): 1(V) and 3(H): 1(v) (there is a berm in the middle of the waterside slope). The toe of the waterside slope (near Bent 5) is eroded for the levee on the southern side.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the Calaveras River Bridge (August 1963) have been used as a reference (Ref. 1). Based on the as-built LOTB, Borings B-1, B-2, B-5, B-7 and D-22 were drilled and Cone Penetration Test (CPT) at B-3, B-4, B-6, B-8 through B-10 were performed by Caltrans in 1963. The soil borings were drilled to the approximate depths between 49 feet and 116 feet and to the bottom elevation between approximately -44 feet and -99.5 feet. The CPTs

were pushed to the approximate depths between 46 feet and 82 feet and to the bottom elevation between approximately -44.5 feet and -64.5 feet.

Field Exploration in 2009

Additional field exploration was conducted in April 2009. Boring R-09-CR-001 was drilled on the south side and CPT-09-CR-001 was performed on the north side of Calaveras River. The soil boring was drilled to the depth of 111.5 feet and the CPT was pushed to the approximate depth of 99.5 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content and dry unit weight of the soil samples are presented in the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built Borings and CPT, the subsurface soils generally (except Boring B-5) consist of very soft to soft clay/silt (levee fill) and loose to medium dense silty sand, underlain by stiff to very stiff clay/silt with pockets/lenses of loose to dense silty sand.

Based on Boring R-09-CR-001 and CPT-09-CR-001, the subsurface soils generally consist of interbedded layers of soft to very stiff lean clays with occasional pocket/lens/layer of medium

dense silty sand/poorly graded sand, underlain by loose to medium dense sands.

Groundwater was recorded between approximate Elevation -0.5 feet and -5 feet in August 1963. Groundwater was measured at Elevation -6 feet (the depth of 12 feet below existing ground surface) during drilling for R-09-CR-001 in April 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquak	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N*(Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of interbedded layers of soft to very stiff lean clays with occasional pocket/lens/layer of medium dense sands, underlain by loose to medium dense sands. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring R-09-CR-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. Based on the results of the evaluation of liquefaction, the factor of safety for Layers 12 and 13 (that are over 100 feet deep) are below 1.1 in Boring A-09-CR-001 and may have potential to liquefy. It should be noted that Seed and Idriss Simplified Method applies only to the depth of 50 feet, the simplified procedure may have reached the limit below that depth. By comparing with the CPT liquefaction analysis data for CPT-09-CR-001 included in this report and the evaluation of liquefaction potential based on the soil profiles developed from the soil borings for the other bridge structures widening, the existence of the loose to medium dense sands in Layers 12 and 13 in Boring A-09-CR-001 appears to be localized and in form of pocket/lens. It is our opinion that the impact of the localized liquefaction potential on the project site should be insignificant. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure, different foundation systems have been evaluated with respect to their design capacity and construction considerations. Caltrans standard Class 200 Alternative "X" Piles are recommended to be the foundation system for the abutments and Bents 2 and 11 of the proposed bridge structure. 30-inch diameter CIDH piles are recommended for the interior piles of Bents 3 through 10, 36-inch diameter CIDH piles are recommended for the exterior piles of Bents 3 through 10 and 60-inch CIDH are recommended for retrofit Bent 5a for the foundation system for the proposed bridge. These piles may be designed for service load of 100 tons at the abutments and Bent 2 and 11 to the indicated pile tip elevations. Pertinent foundation design information provided by Biggs Cardosa Associates (Structural designer), including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2A: FOUNDATION DESIGN DATA (INTERIOR PILES)

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 200	19.3	17.3	2.5	46.96	1.00	7
Bent 2	LRFD	Class 200	6.1	2.1	3	46.96	1.00	8
Bent 3	LRFD	2'-6" Dia. CIDH	5.4	4.4	N/A	N/A	1.00	8
Bent 4	LRFD	2'-6" Dia. CIDH	17.2	17.2	N/A	N/A	1.00	8
Bent 5	LRFD	2'-6" Dia. CIDH	10.0	8.4	N/A	N/A	1.00	8
Bent 5a	LRFD	5'-0" Dia. CIDH	10.0	10.0	N/A	N/A	1.00	2
Bent 6	LRFD	2'-6" Dia. CIDH	-5.2	-10.0	N/A	N/A	1.00	8
Bent 7	LRFD	2'-6" Dia. CIDH	-8.5	-13.6	N/A	N/A	1.00	8

Bent 8	LRFD	2'-6" Dia. CIDH	-4.0	-9.0	N/A	N/A	1.00	8
Bent 9	LRFD	2'-6" Dia. CIDH	18.5	18.5	N/A	N/A	1.00	8
Bent 10	LRFD	2'-6" Dia. CIDH	6.5	5.5	N/A	N/A	1.00	8
Bent 11	LRFD	Class 200	5.1	1.1	3	48.02	1.00	8
Abut 12	WSD	Class 200	20.1	18.1	2.5	48.02	1.00	7

TABLE 2B: FOUNDATION DESIGN DATA (EXTERIOR PILES)

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 200	19.3	17.3	2.5	46.96	1.00	7
Bent 2	LRFD	Class 200	6.1	2.1	3	46.96	1.00	8
Bent 3	LRFD	3'-0" Dia. CIDH	5.4	4.4	N/A	N/A	1.00	8
Bent 4	LRFD	3'-0" Dia. CIDH	17.2	17.2	N/A	N/A	1.00	8
Bent 5	LRFD	3'-0" Dia. CIDH	10.0	8.4	N/A	N/A	1.00	8
Bent 5a	LRFD	5'-0" Dia. CIDH	10.0	10.0	N/A	N/A	1.00	2
Bent 6	LRFD	3'-0" Dia. CIDH	-5.2	-10.0	N/A	N/A	1.00	8
Bent 7	LRFD	3'-0" Dia. CIDH	-8.5	-13.6	N/A	N/A	1.00	8
Bent 8	LRFD	3'-0" Dia. CIDH	-4.0	-9.0	N/A	N/A	1.00	8
Bent 9	LRFD	3'-0" Dia. CIDH	18.5	18.5	N/A	N/A	1.00	8
Bent 10	LRFD	3'-0" Dia. CIDH	6.5	5.5	N/A	N/A	1.00	8
Bent 11	LRFD	Class 200	5.1	1.1	3	48.02	1.00	8
Abut 12	WSD	Class 200	20.1	18.1	2.5	48.02	1.00	7

TABLE 3A: FOUNDATION DESIGN LOADS (INTERIOR PILES)

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	490	70	340	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	1000	NA	710	1410	240	0	0	N/A	260	-90	0
Bent 3	930	NA	660	1360	233	0	0	N/A	210	-50	0
Bent 4	1240	NA	800	1600	253	0	0	N/A	240	-60	0
Bent 5	1030	NA	740	1460	241	0	0	N/A	240	-30	0
Bent 5a	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1100	560	0	0
Bent 6	1020	NA	760	1470	247	0	0	N/A	210	0	0
Bent 7	1040	NA	770	1490	252	0	0	N/A	210	0	0
Bent 8	1050	NA	760	1470	247	0	0	N/A	210	0	0
Bent 9	1230	NA	800	1590	243	0	0	N/A	240	-50	0
Bent 10	980	N/A	710	1450	250	0	0	N/A	200	-20	0
Bent 11	1040	NA	750	1460	260	0	0	N/A	200	-90	0
Abut 12	510	80	360	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

TABLE 3B: FOUNDATION DESIGN LOADS (EXTERIOR PILES)

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	490	70	340	N/A	NA	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	1000	N/A	710	1410	240	0	0	N/A	260	-90	0
Bent 3	930	N/A	660	1360	327	0	0	N/A	210	-50	0
Bent 4	1240	N/A	800	1600	378	0	0	N/A	240	-60	0
Bent 5	1030	N/A	740	1460	361	0	0	N/A	240	-30	0
Bent 5a	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1100	560	0	0
Bent 6	1020	N/A	760	1470	343	0	0	N/A	210	0	0
Bent 7	1040	N/A	770	1490	340	0	0	N/A	210	0	0
Bent 8	1050	N/A	760	1470	342	0	0	N/A	210	0	0
Bent 9	1230	N/A	800	1590	373	0	0	N/A	240	-50	0
Bent 10	980	N/A	710	1450	371	0	0	N/A	200	-20	0
Bent 11	1040	N/A	750	1460	260	0	0	N/A	200	-90	0
Abut 12	510	80	360	N/A	NA	N/A	N/A	N/A	N/A	N/A	N/A

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations and Load and Resistance Factor Design (LRFD) is used for the bent foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads". The Bent foundations are evaluated for the foundation design data and loading conditions using AASHTO LRFD Bridge Design Specifications–4th Edition, with interims through December 2008 and current Caltrans Amendments.

The evaluation of Load Demands on the piles, based upon WSD and LRFD are presented in Table 3 above. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Table 4 and Table 5 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)	Nominal Driving Resistance (kips)
			Total	Permanent					
Abut 1	Class 200 Alt "X"	17.3	490	340	70	140	-26.5 (a)	-26.5	140
Abut 12	Class 200 Alt "X"	18.1	310	360	80	160	-37.5 (a)	-37.5	160

Notes:

(i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively.

(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral.

TABLE 5A: FOUDNATION RECOMMENDATION FOR BENTS (INTERIOR PILES)

Location	Pile Type	Cut-off Elev.	Service-I Limit State Load (kips) per Support	Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev.[(i)] (ft)	Specified Tip Elev. (ft)[(ii)]	Nominal Driving Resistance Required (kips)(iii)
					Strength Limit		Extreme Event				
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1.0)	Tension (φ=1.0)			
Bent 2	Class 200 Alt "X"	2.1	710	1	240	0	260	90	-58.0 (a), -41.5(c), -25.0 (d)	-58.0	350
Bent 3	2'-6" Dia. CIDH	4.4	660	1	233	0	210	50	-58.0 (a), -41.5(c), -25.5 (d)	-58.0	N/A
Bent 4	2'-6" Dia. CIDH	17.2	800	1	253	0	240	60	-57.5 (a), -38.0(c), -26.5 (d)	-57.5	N/A
Bent 5	2'-6" Dia. CIDH	8.4	740	1	241	0	240	30	-62.0 (a), -44.0(c), -21.5 (d)	-62.0	N/A
Bent 5a	5'-0" Dia. CIDH	10.0	N/A	1	N/A	0	560	0	-42.5 (c)	-42.5	N/A
Bent 6	2'-6" Dia. CIDH	-10.0	760	1	247	0	210	0	-70.0 (a), -59.0(c)	-70.0	N/A
Bent 7	2'-6" Dia. CIDH	-13.6	770	1	252	0	210	0	-64.5 (a), -50.5(c)	-64.5	N/A
Bent 8	2'-6" Dia. CIDH	-9.0	760	1	247	0	210	0	-60.0 (a), -45.5(c)	-60.0	N/A
Bent 9	2'-6" Dia. CIDH	18.5	800	1	243	0	240	50	-45.5 (a), -31.5(c), -11.0 (d)	-45.5	N/A
Bent 10	2'-6" Dia. CIDH	5.5	710	1	250	0	200	20	-54.5 (a), -36.5(c), -11.0 (d)	-54.5	N/A
Bent 11	Class 200 Alt "X"	1.1	750	1	260	0	260	90	-71.0 (a), -56.0(c), -24.0 (d)	-71.0	380

Notes

(i) Design tip elevations are controlled by (a) Compression (Strength Limit), (b) Tension (Strength Limit), (c) Compression (Extreme Event), (d) Tension (Extreme Event), (e) Lateral Load, (f) Settlement respectively.

(ii) The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.

(iii) The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the penetrated soil layers, if any, which do not contribute to the design resistance.

TABLE 5B: FOUDNATION RECOMMENDATION FOR BENTS (EXTERIOR PILES)

Location	Pile Type	Cut-off Elev.	Service-I Limit State Load (kips) per Support	Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev.[1] (ft)	Specified Tip Elev. (ft)[ii]	Nominal Driving Resistance Required (kips)(iii)
					Strength Limit		Extreme Event				
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1.0)	Tension (φ=1.0)			
Bent 2	Class 200 Alt "X"	2.1	710	1	240	0	260	90	-58.0 (a), -41.5(c), -25.0 (d)	-58.0	350
Bent 3	3'-0" Dia. CIDH	4.4	660	1	327	0	210	50	-61.0 (a), -35.5(c), -17.5 (d)	-61.0	N/A
Bent 4	3'-0" Dia. CIDH	17.2	800	1	378	0	240	60	-66.0 (a), -38.0(c), -21.5 (d)	-66.0	N/A
Bent 5	3'-0" Dia. CIDH	8.4	740	1	361	0	240	30	-69.0 (a), -38.0(c), -15.5 (d)	-69.0	N/A
Bent 5a	5'-0" Dia. CIDH	10.0	N/A	1	N/A	0	560	0	-42.5 (c)	-42.5	N/A
Bent 6	3'-0" Dia. CIDH	-10.0	760	1	343	0	210	0	-76.0 (a), -50.0(c)	-76.0	N/A
Bent 7	3'-0" Dia. CIDH	-13.6	770	1	340	0	210	0	-70.5 (a), -45.5(c)	-70.5	N/A
Bent 8	3'-0" Dia. CIDH	-9.0	760	1	342	0	210	0	-65.0 (a), -39.0(c)	-65.0	N/A
Bent 9	3'-0" Dia. CIDH	18.5	800	1	373	0	240	50	-56.5 (a), -25.5(c), -9.5 (d)	-56.5	N/A
Bent 10	3'-0" Dia. CIDH	5.5	710	1	371	0	200	20	-61.5 (a), -31.5(c), -10.0 (d)	-61.5	N/A
Bent 11	Class 200 Alt "X"	1.1	750	1	260	0	260	90	-71.0 (a), -56.0(c), -24.0 (d)	-71.0	380

Notes

(i) Design tip elevations are controlled by (a) Compression (Strength Limit), (b) Tension (Strength Limit), (c) Compression (Extreme Event), (d) Tension (Extreme Event), (e) Lateral Load, (f) Settlement respectively.

(ii) The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.

(iii) The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the penetrated soil layers, if any, which do not contribute to the design resistance.

TABLE 6A: PILE DATA TABLE (INTERIOR PILES)

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
		Compression	Tension			
Abut 1	Class 200 Alt "X"	140	0	-26.5 (a)	-26.5	140
Bent 2	Class 200 Alt "X"	350	90	-58.0 (a), -25.0 (b)	-58.0	350
Bent 3	2'-6" Dia. CIDH	340	50	-58.0 (a), -25.5 (b)	-58.0	N/A
Bent 4	2'-6" Dia. CIDH	370	60	-57.5 (a), -26.5 (b)	-57.5	N/A
Bent 5	2'-6" Dia. CIDH	350	30	-62.0 (a), -21.5 (b)	-62.0	N/A
Bent 5a	5'-0" Dia. CIDH	560	0	-42.5 (a)	-42.5	N/A
Bent 6	2'-6" Dia. CIDH	360	0	-70.0 (a)	-70.0	N/A
Bent 7	2'-6" Dia. CIDH	360	0	-64.5 (a)	-64.5	N/A
Bent 8	2'-6" Dia. CIDH	360	0	-60.0 (a)	-60.0	N/A

Bent 9	2'-6" Dia. CIDH	350	50	-45.5 (a), -11.0 (b)	-45.5	N/A
Bent 10	2'-6" Dia. CIDH	360	20	-54.5 (a), -11.0 (b)	-54.5	N/A
Bent 11	Class 200 Alt "X"	380	90	-71.0 (a), -24.0 (b)	-71.0	380
Abut 12	Class 200 Alt "X"	160	0	-37.5 (a)	-37.5	160

Notes

(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression (c) Lateral Load.
(ii) Design tip elevations for the ***Bents*** are controlled by (a) Compression, (b) Tension, (c) Lateral Load.
(iii) The design tip elevation due to the lateral loads are estimated by Biggs Cardosa Associates and not included in the pile data table above.
(iv) The specified tip elevation shall not be raised above the design tip elevations for tension and lateral load.

TABLE 6B: PILE DATA TABLE (EXTERIOR PILES)

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.(1) (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
		Compression	Tension			
Abut 1	Class 200 Alt "X"	140	0	-26.5 (a)	-26.5	140
Bent 2	Class 200 Alt "X"	350	90	-58.0 (a), -25.0 (b)	-58.0	350
Bent 3	3'-0" Dia. CIDH	470	50	-61.0 (a), -17.5 (b)	-61.0	N/A
Bent 4	3'-0" Dia. CIDH	540	60	-66.0 (a), -21.5 (b)	-66.0	N/A
Bent 5	3'-0" Dia. CIDH	520	30	-69.0 (a), -15.5 (b)	-69.0	N/A
Bent 5a	5'-0" Dia. CIDH	560	0	-42.5 (a)	-42.5	N/A
Bent 6	3'-0" Dia. CIDH	490	0	-76.0 (a)	-76.0	N/A
Bent 7	3'-0" Dia. CIDH	490	0	-70.5 (a)	-70.5	N/A
Bent 8	3'-0" Dia. CIDH	490	0	-65.0 (a)	-65.0	N/A
Bent 9	3'-0" Dia. CIDH	540	50	-56.5 (a), -9.5 (b)	-56.5	N/A
Bent 10	3'-0" Dia. CIDH	530	20	-61.5 (a), -10.0 (b)	-61.5	N/A
Bent 11	Class 200 Alt "X"	380	90	-71.0 (a), -24.0 (b)	-71.0	380
Abut 12	Class 200 Alt "X"	160	0	-37.5 (a)	-37.5	160

Notes

(iv) Design tip elevations for the ***Abutments*** are controlled by (a) Compression (c) Lateral Load.
(v) Design tip elevations for the ***Bents*** are controlled by (a) Compression, (b) Tension, (c) Lateral Load.
(vi) The design tip elevation due to the lateral loads are estimated by Biggs Cardosa Associates and not included in the pile data table above.
(iv) The specified tip elevation shall not be raised above the design tip elevations for tension and lateral load.

The estimation of the driven pile capacity in clay is based on the adhesion factor per FHWA manual (Tomlinson 1980). The estimation of the CIDH concrete pile capacity in clay and sand are based on the adhesion factors per FHWA manual (O'Neil and Reese 1999). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 6,

the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles should not be spaced closer than 3 times the pile diameter measured center-to-center. For piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter, there is no group effect for vertical pile capacity.

Caltrans standard specification for "Cast-in-Place Concrete Piling" should be used for the construction of CIDH piles. Groundwater is expected during pile construction. Due to presence of sand/gravel layers, and groundwater, raveling or caving may be expected, which may require additional drilling and cleaning effort and may increase the concrete volume for the piles. The CIDH holes are not expected to remain open without implementation of appropriate measures. Temporary steel casing and using slurry displacement method of construction should be used at all times to maintain the integrity of the piles. This should be consistent with any other special conditions required by the Regulatory Agency. Caltrans Standard Specifications and SSPs should be used for construction and quality assurance procedures. All pile excavations should be observed by the Geotechnical Engineer prior to the placement of reinforcement and concrete so that if conditions differ from those anticipated, appropriate recommendations can be made.

It is prudent to make the contractor aware of these conditions so that appropriate steps can be taken to comply with the standard and maintain the integrity of the pile foundations.

Predrilling to the bottom of the existing water line is required to minimize the vibration impact on the existing Cal Water Service water lines near Bent 2. The bottom of predrill elevation is 0.0 feet according to BCA. It is recommended to follow Caltrans standard specifications (Section 49-1.06) (Ref. 7) for oversize predrilling through the existing ground to the bottom of predrill elevation described above.

9.2.1 Scour

Based on the hydraulic study, the bent piles extension will have future scour depths between 1.58 feet and 5.11 feet at Bent 5 through Bent 8. These scour depths should be accounted for in the finish ground elevations in the pile capacity calculations.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. It is our understanding that the structural designer will perform lateral load analyses for the piles using LPILE Program (v. 5). "Generalized Vertical Soil Profile and Strength Parameters" based on R-09-CR-001 and CPT-09-CR-001 are included in Appendix C. To account for group effect, p-y multiplier of 0.6 is recommended for Abutment 1, Bent 2, Bent 11 and Abutment 12 with the pile spacing of 3D. For Bents 3 through 10, p-y multipliers of 0.66 in the longitudinal direction and 0.44 in the transverse direction are recommended for Bents 3 through 10 with the pile spacing of 1.83 times the pile diameter.

Generally, abutment piles are designed for service loads only, and batter piles are used for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A

minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.

Traffic Load Surcharge — The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for the widening of the existing structures. Settlement of the existing embankment is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Boring R-09-CR-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 7: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
R-09-CR-001	26.0	1880	7.2	25.3	2.4

Based on the corrosion test results, the native subsurface soils in Boring R-09-CR-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Potential Impact on Existing Levees and Construction Consideration

In our opinion, the pile foundation of the proposed structure widening should not impose significant additional horizontal or vertical loads onto the levees.

The slope stability and consolidation settlement of the existing levee may be impacted by the proposed project in the following ways.

a) Soil reaction of the pile may exert lateral load onto the levee. The extent of this load is localized and small in comparison with the overall length of the levee. This loading should not be any different than the existing piles at the project site. The impact of the foundation piles on the slope stability of the existing levees is negligible.

b) Cast In-Drilled-Hole concrete piles were selected to eliminate the vibration (that would otherwise have resulted from pile driving) and impact on the stability of the levees.

c) The widened structure should not impose vertical load on the levee since it is supported on pile foundation. Hence, there should be no consolidation settlement of the existing levee due to the proposed structure. Based on the available boring information, the subsurface soils of the existing levees consist of soft to firm lean clays. Some localized settlement may result from construction equipments on the levees during construction. This impact should be minimized with the implementation of precautionary measures recommended below.

Construction Precautionary Measures and Construction Monitoring

Construction on the levees shall abide by Army Corps of Engineers and/or Department of Water Resources construction and backfill requirements.

The following precautionary measures and construction monitoring can be considered during construction to minimize the potential impact due to construction on the existing levees:

a) To minimize vibration due to the installation of temporary piles, steel pipe piles can be spun into the levees to support the temporary trestle, or temporary CIDH piles with steel pipe piles placed inside the CIDH can be used.

b) The maximum allowable load on the levee from the construction equipment should be in conformance with the regulatory agency's requirements. Normally a 200 to 250 pound per square foot (psf) traffic surcharge loading condition is used in the slope stability analyses, which generally represents maintenance equipment loads. This transient loading should not be exceeded by the total (dead + live) load of the construction equipment during construction. Parking and leaving equipment for a long term should be avoided. Also materials should not be stored at the levees.

c) No equipment shall be allowed within 10 feet from the edge of the levee regardless of its weight. In order to further reduce the loading on the levees, timber planks or steel plates may also be considered to reduce point loads.

d) Contractor should be required to set horizontal and vertical monuments in levee section, which will be periodically checked for horizontal or vertical movements.

e) Construction monitoring may be considered to evaluate the settlement and movement in the levees. However, the level of sophisticated instrumentation may have to be further discussed, if it is deemed necessary.

In summary, it is our opinion that the impact on the existing levees due to the proposed project should be insignificant. However, precautionary measures should be taken and construction monitoring should be implemented during construction to minimize any potential impact on the integrity of the existing levees.

The above opinions are based on our understanding of the project and limited to the portion of the levee to be crossed by the proposed structure. Evaluation of the existing levee corridor has not been performed since it is beyond the scope of our work for this project.

9.8 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.9 Construction Observation

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

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,
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STATE OF CALIFORNIA

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Calaveras River Bridge Foundation Report (6-2010).rpt

REFERENCES

1. Calaveras River Bridge As-built Log of Test Borings (August 1963).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

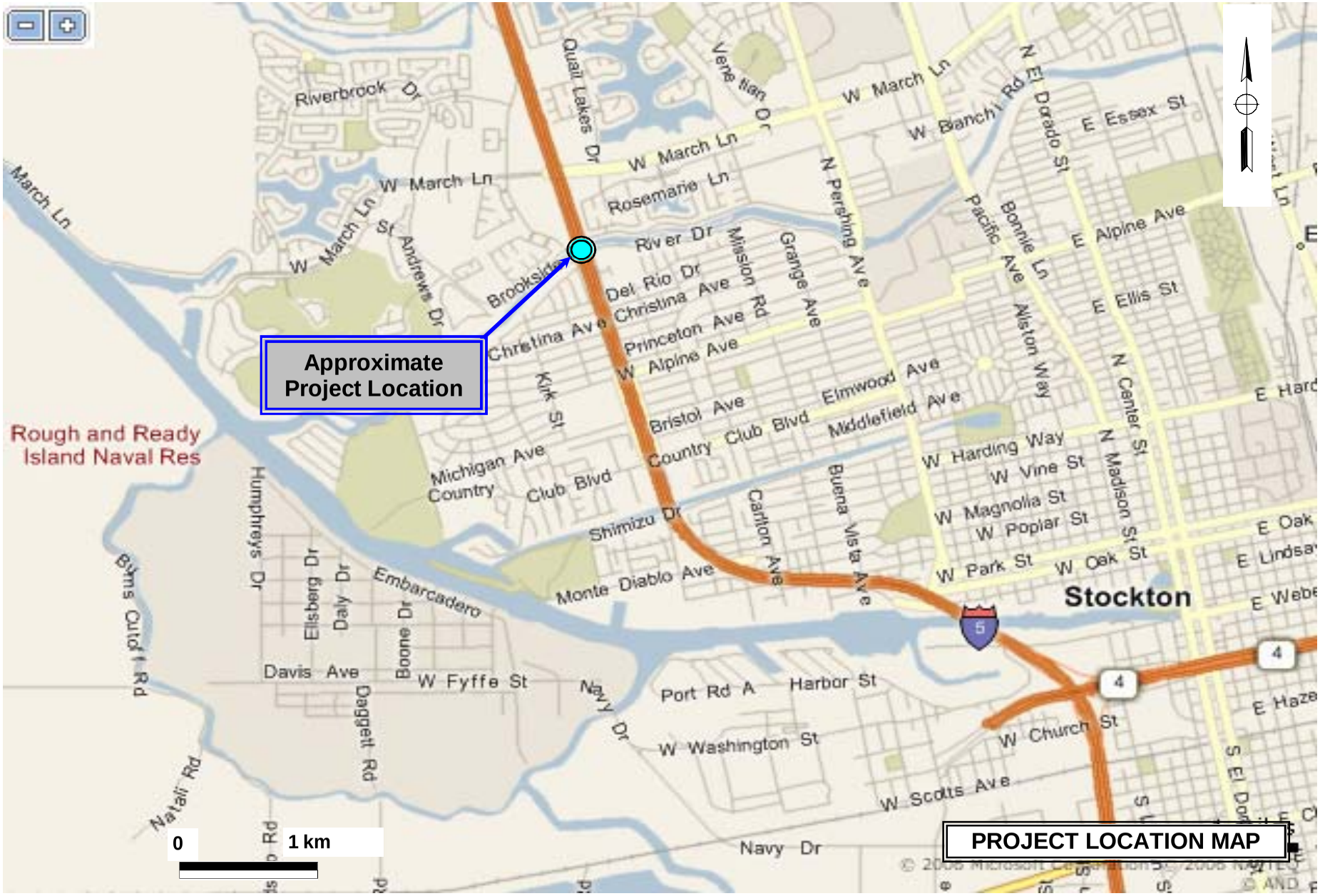
3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

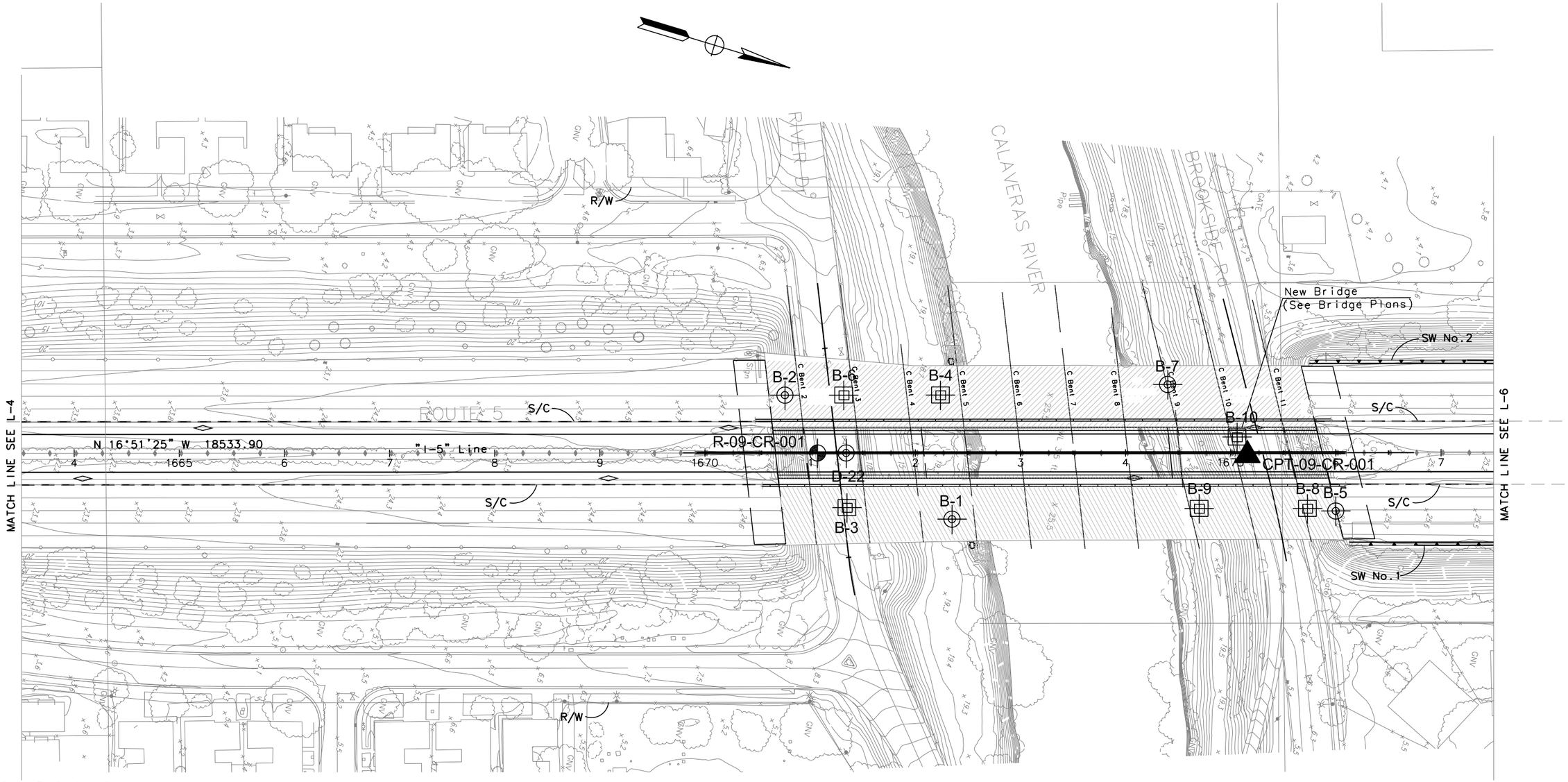
7. State of California Department of Transportation Standard Specifications May 2006.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

CALAVERAS RIVER BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0174)	PLATE NO.: 1



LEGEND

A-09-CR-001
Approximate Boring Location

B-6
Approximate CPT location performed in 1963

B-2
Approximate Boring location drilled in 1963

CPT-09-CR-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

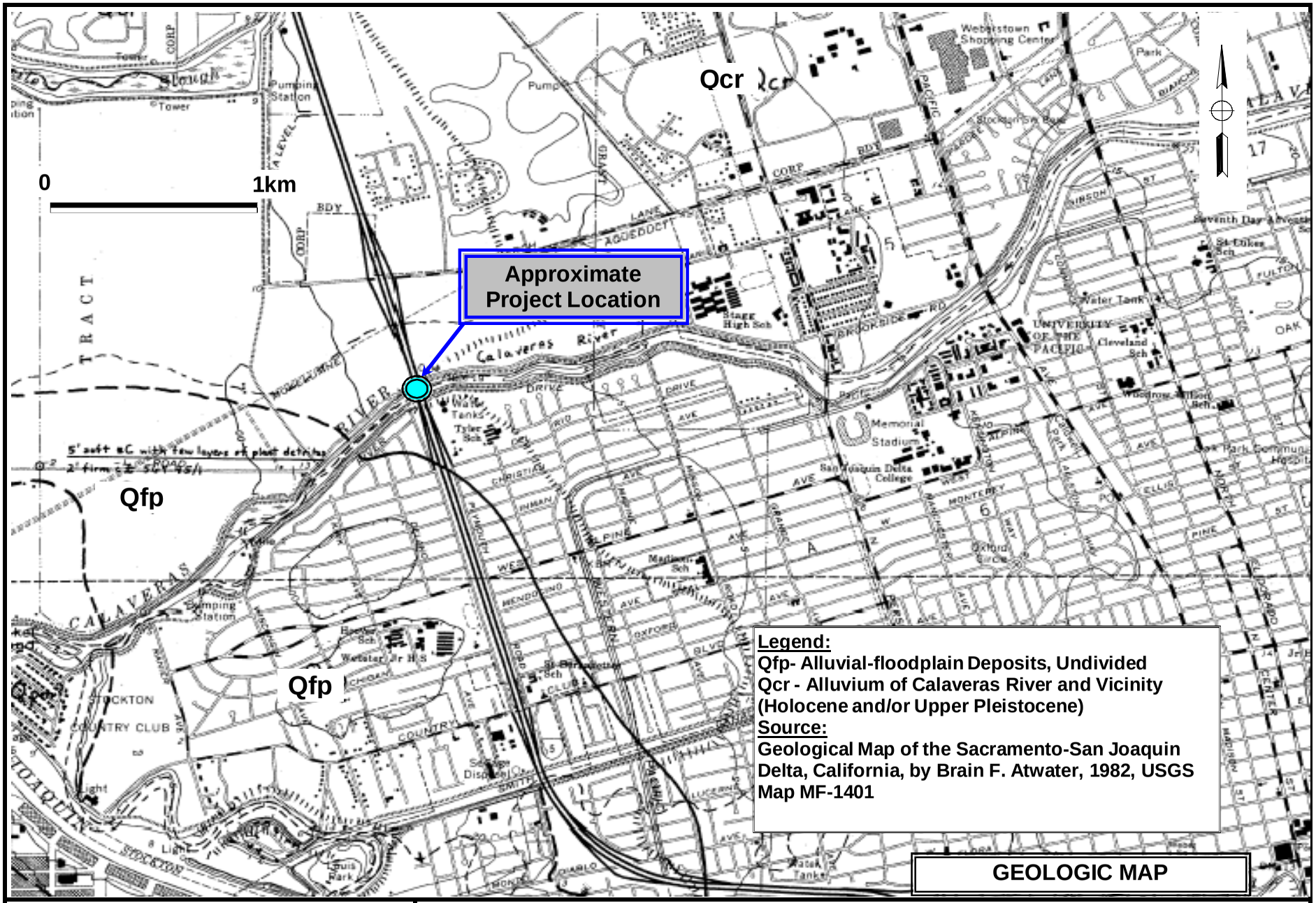
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

CALAVERAS RIVER BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

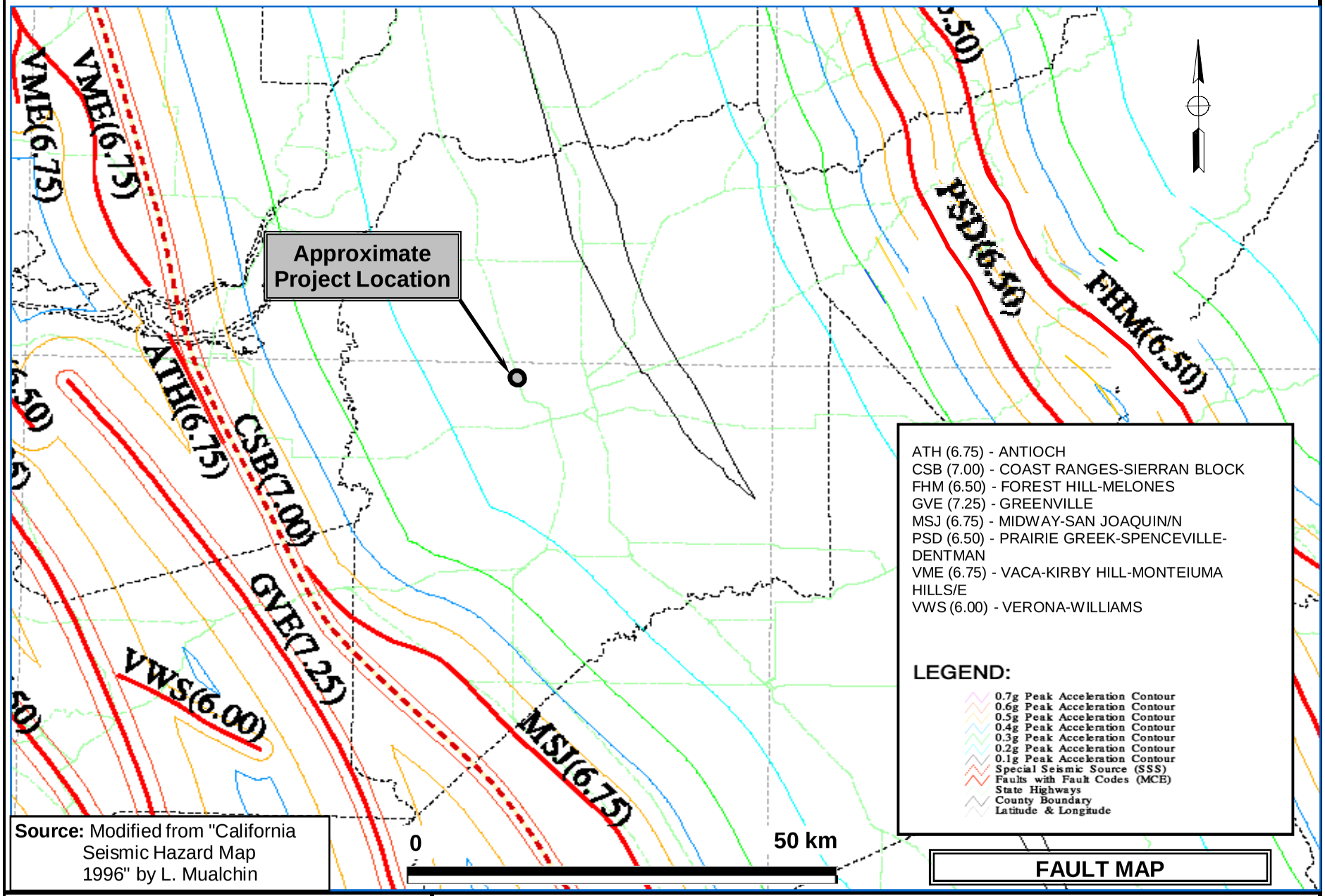
PLATE NO: 2



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

CALAVERAS RIVER BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0174)	PLATE NO.: 3



Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

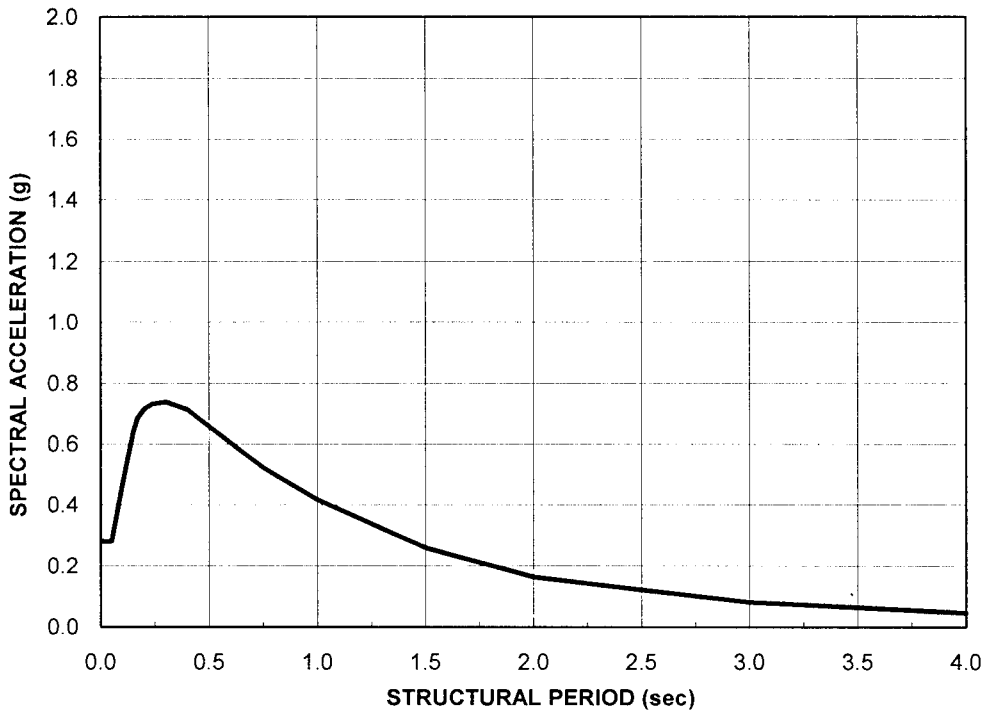
FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

CALAVERAS RIVER BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0174) | **PLATE NO.: 4**

ARS DESIGN CURVE
CALAVERAS RIVER BRIDGE (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 5-inch diameter rotary wash drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

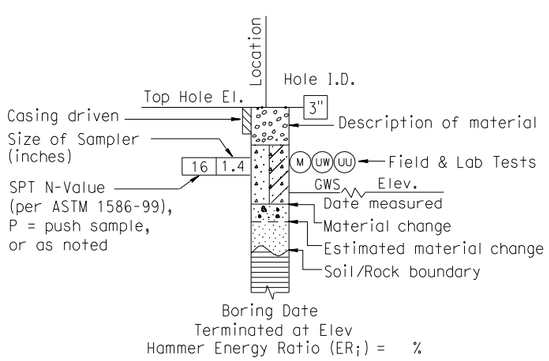
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

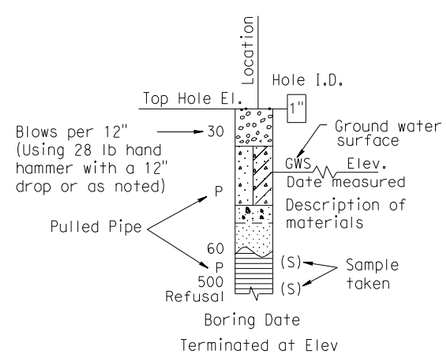
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

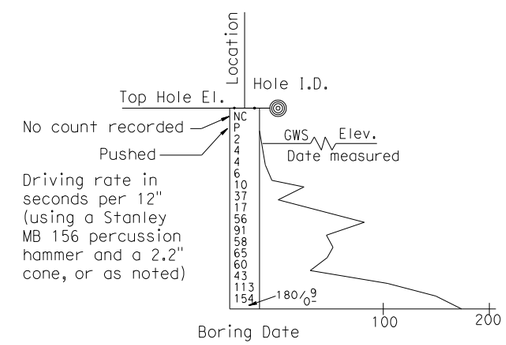
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



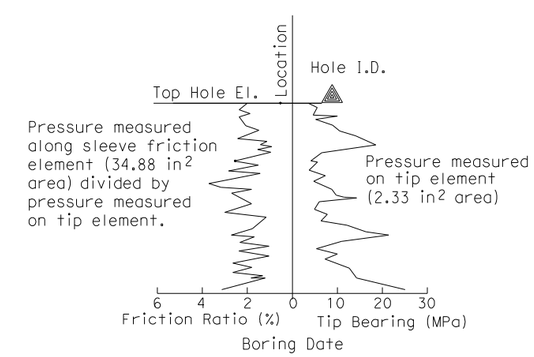
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER
SIGN OFF DATE	CHECKED BY	D. WANG	DATE: MARCH 2009 - APRIL 2009		

BRIDGE NO. 29-0174 — POST MILES 29.56

CALAVERAS RIVER BRIDGE (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 57 OF 60

...\29-0174-z-ltb01.dgn 6/8/2010 9:18:22 AM

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 5333)
CP	Compaction Curve (CTM 216)
CR	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content-% (ASTM D 2974)
P	Permeability (CTM 220)
PA	Particle Size Analysis (ASTM D 422)
PI	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (CTM 217)
SG	Specific Gravity (AASHTO T 100)
SL	Shrinkage Limit (ASTM D 427)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 4767)
VS	Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

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STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0174
POST MILES 29.56

CALAVERAS RIVER BRIDGE (WIDEN)
SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 58 OF 60

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

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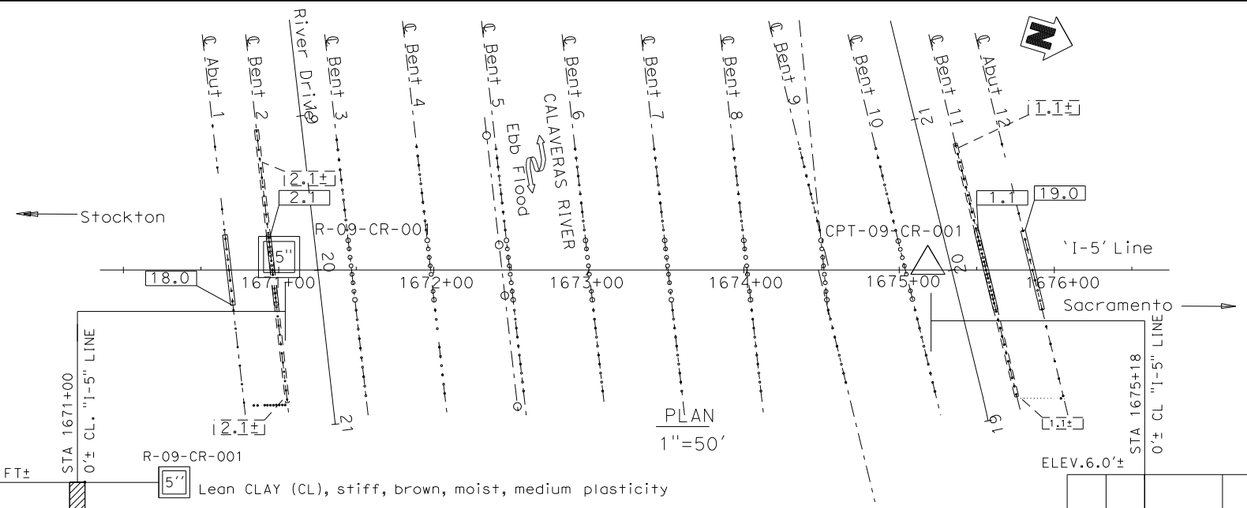
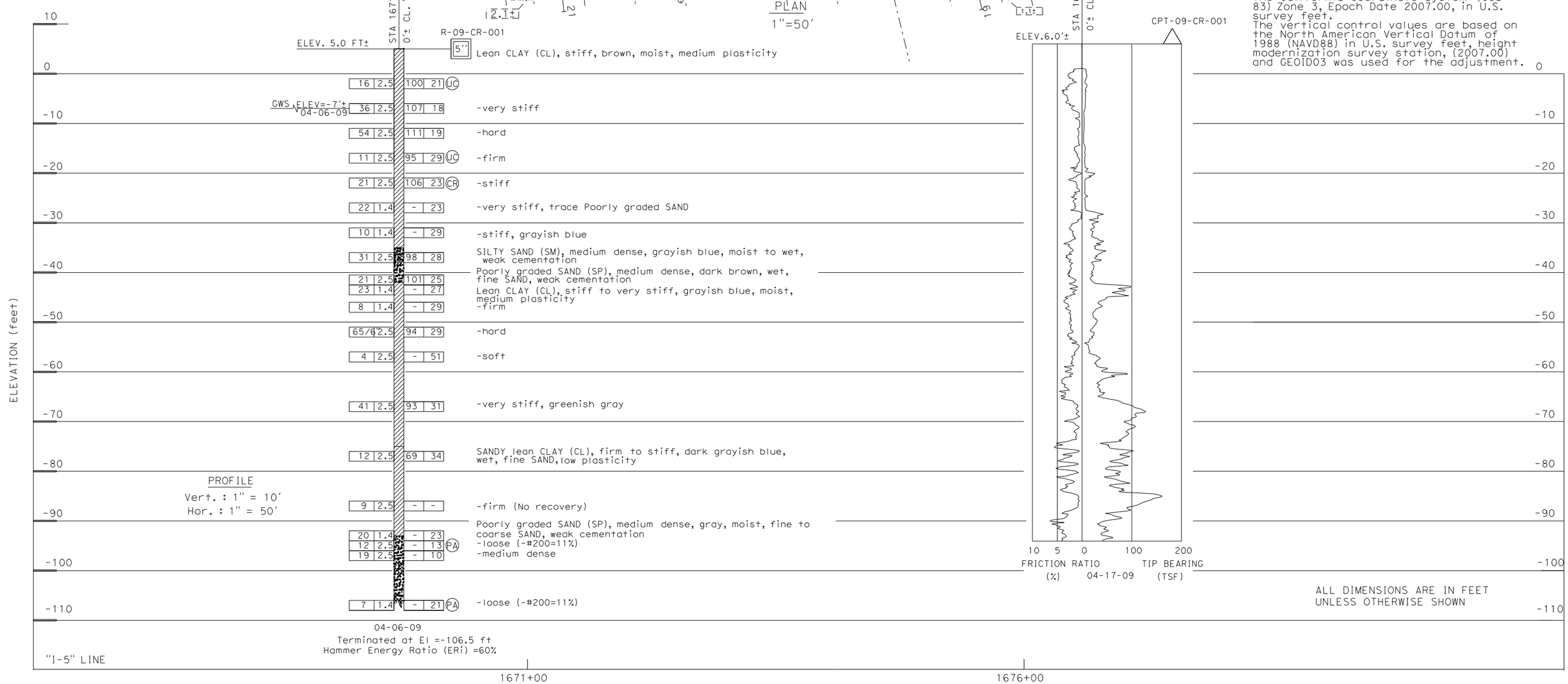
SAN JOSE, CA 95131



BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.
The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.



ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	A. LAM	BRIDGE NO. 29-0174
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PROJECT ENGINEER	POST MILES 29.56

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

CALAVERAS RIVER BRIDGE (WIDEN)
LOG OF TEST BORINGS 1 OF 2

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES → 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 59 OF 60

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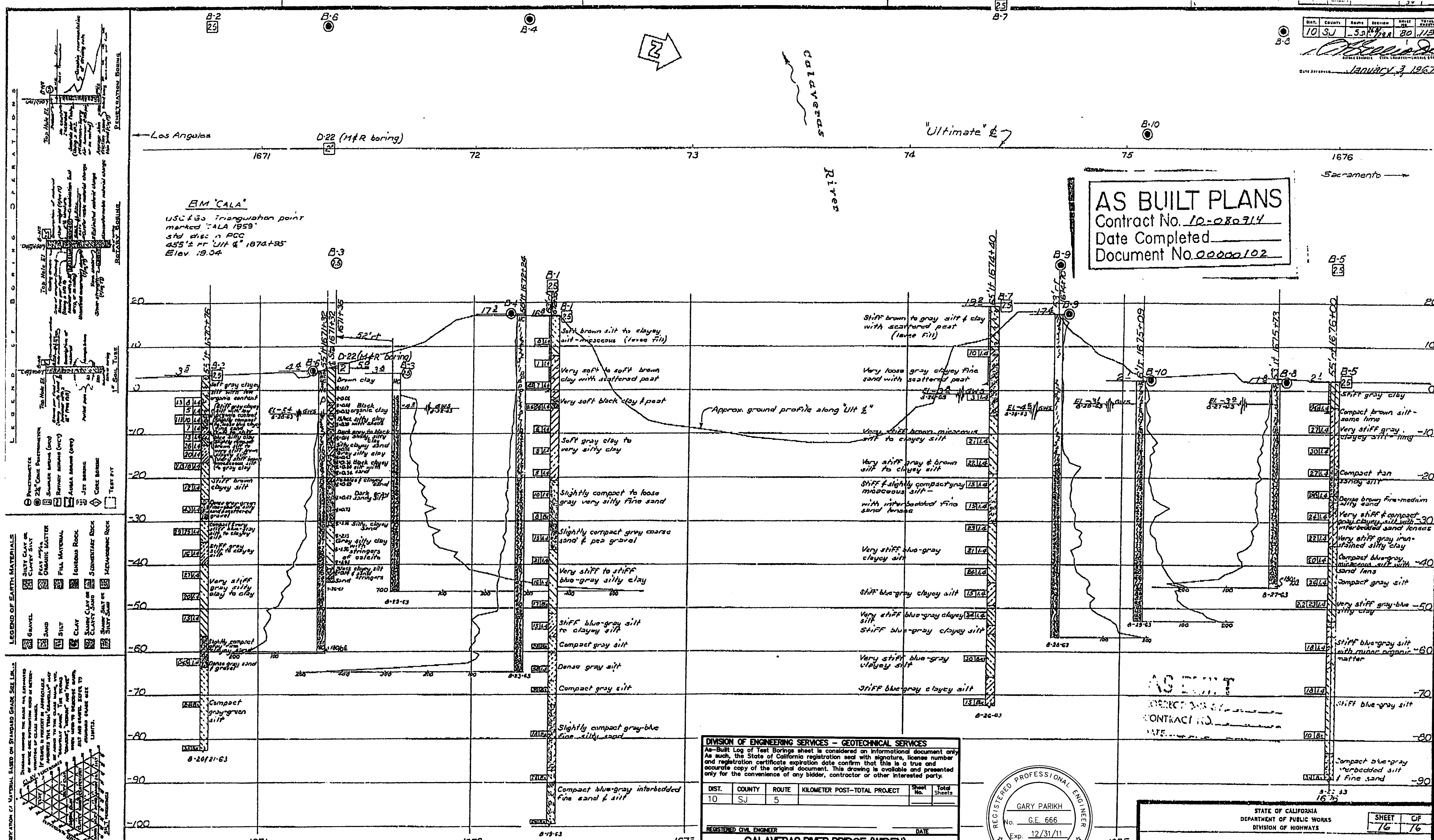


DIST.	COUNTY	ROUTE	SECTION	SHEET NO.	TOTAL SHEETS
10	SJ	5		80	113

Date Approved January 3, 1967

AS BUILT PLANS
Contract No. 10-080914
Date Completed
Document No. 00000102

AS BUILT
CORRECTIONS BY
CONTRACT NO.
DATE

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES

As–Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5			

REGISTERED CIVIL ENGINEER — DATE

CALAVERAS RIVER BRIDGE (WIDEN)

LOG OF TEST BORINGS 2 OF 2

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0G4701

BRIDGE No. 29–0174
Sheet 60 of 60

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

SHEET 76 OF 76

CALAVERAS RIVER BRIDGE

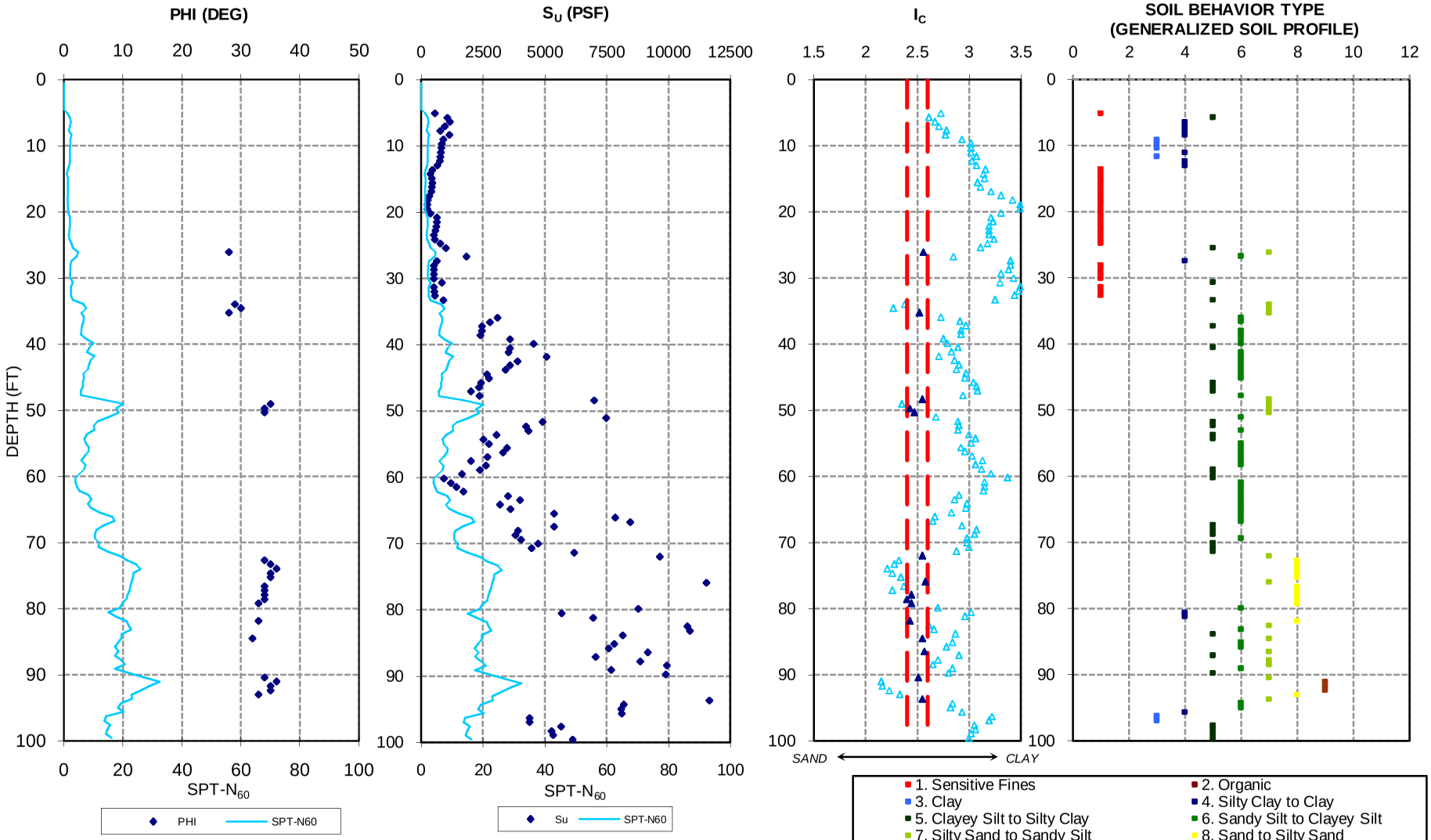
LOG OF TEST BORINGS

SCALE Horiz: 1" = 20' Vert: 1" = 10' | BRIDGE 29-174 R/L | FILE | DRAWING 29174-33

10-080914

CPT DATA CORRELATION

CPT-09-CR-005



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

CALAVERAS RIVER BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0174)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-CALAVERAS RIVER BRIDGE (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0174)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-CR-005***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*Kσ*Kα*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	Kσ	Kα	F.S.
5.085	1	Sensitive Fines	4.45	0.076	0.283		0.283	2.73	1.48348	0.99	0.129							1.00	1	
5.741	5	Clayey Silt to Silty Clay	8.13	0.179	0.321		0.321	2.61	1.44642	0.99	0.129							1.00	1	
6.398	4	Silty Clay to Clay	8.992	0.247	0.358		0.358	2.67	1.41207	0.99	0.128							1.00	1	
7.054	4	Silty Clay to Clay	7.469	0.16	0.396		0.396	2.71	1.37845	0.99	0.128							1.00	1	
7.71	4	Silty Clay to Clay	6.186	0.115	0.434		0.434	2.78	1.34639	0.98	0.128							1.00	1	
8.366	4	Silty Clay to Clay	8.877	0.244	0.471		0.471	2.77	1.31658	0.98	0.128							1.00	1	
9.022	3	Clay	7.141	0.239	0.508		0.508	2.93	1.28806	0.98	0.128							1.00	1	
9.678	3	Clay	6.661	0.264	0.544		0.544	3.02	1.26147	0.98	0.127							1.00	1	
10.335	3	Clay	6.437	0.219	0.581		0.581	3.02	1.23526	0.98	0.127							1.00	1	
10.991	4	Silty Clay to Clay	6.437	0.194	0.618		0.618	3.02	1.21012	0.98	0.127							1.00	1	
11.647	3	Clay	6.263	0.196	0.655		0.655	3.07	1.18598	0.98	0.127							1.00	1	
12.303	4	Silty Clay to Clay	6.132	0.141	0.693		0.693	3.03	1.16218	0.97	0.127							1.00	1	
12.959	4	Silty Clay to Clay	5.569	0.116	0.73		0.73	3.07	1.1399	0.97	0.126							1.00	1	
13.615	1	Sensitive Fines	3.882	0.049	0.767		0.767	3.16	1.11845	0.97	0.126							1.00	1	
14.272	1	Sensitive Fines	3.27	0.02	0.803		0.803	3.14	1.09835	0.97	0.126							1.00	1	
14.928	1	Sensitive Fines	3.843	0.036	0.84		0.84	3.15	1.07843	0.97	0.126							1.00	1	
15.584	1	Sensitive Fines	4.045	0.023	0.876		0.876	3.08	1.05973	0.97	0.126							1.00	1	
16.24	1	Sensitive Fines	3.783	0.02	0.913		0.913	3.11	1.04117	0.97	0.126							1.00	1	
16.896	1	Sensitive Fines	3.647	0.027	0.95		0.95	3.21	1.02326	0.96	0.125							1.00	1	
17.552	1	Sensitive Fines	3.243	0.025	0.986		0.986	3.31	1.0064	0.96	0.125							1.00	1	
18.209	1	Sensitive Fines	2.779	0.02	1.023		1.023	3.42	0.98965	0.96	0.125							1.00	1	
18.865	1	Sensitive Fines	2.713	0.023	1.059		1.059	3.49	0.97388	0.96	0.125							0.99	1	
19.521	1	Sensitive Fines	2.719	0.02	1.096		1.096	3.49	0.95819	0.96	0.125							0.98	1	
20.177	1	Sensitive Fines	3.685	0.026	1.132		1.132	3.31	0.9434	0.96	0.124							0.98	1	
20.833	1	Sensitive Fines	5.705	0.084	1.169		1.169	3.21	0.92866	0.95	0.124							0.97	1	
21.49	1	Sensitive Fines	5.64	0.085	1.205		1.205	3.23	0.91476	0.95	0.124							0.96	1	
22.146	1	Sensitive Fines	5.525	0.061	1.242		1.242	3.19	0.9009	0.95	0.124							0.96	1	
22.802	1	Sensitive Fines	5.258	0.047	1.278		1.278	3.19	0.88781	0.95	0.123							0.95	1	
23.458	1	Sensitive Fines	4.805	0.028	1.315		1.315	3.19	0.87475	0.95	0.123							0.95	1	
24.114	1	Sensitive Fines	5.045	0.047	1.352		1.352	3.24	0.86207	0.94	0.123							0.94	1	
24.77	1	Sensitive Fines	6.814	0.096	1.388		1.388	3.18	0.85008	0.94	0.123							0.94	1	
25.427	5	Clayey Silt to Silty Clay	8.741	0.13	1.426		1.426	3.11	0.83778	0.94	0.122							0.93	1	
26.083	7	Silty Sand to Sandy Silt	18.923	0.089	1.464		1.464	2.56	0.82583	0.94	0.122	2.560	0.83	15.63	3.091	48.296	0.09	0.93	1	
26.739	6	Sandy Silt to Clayey Silt	15.03	0.186	1.502		1.502	2.85	0.81421	0.94	0.122							0.92	1	
27.395	4	Silty Clay to Clay	6.197	0.131	1.54		1.54	3.4	0.80292	0.93	0.121							0.92	1	
28.051	1	Sensitive Fines	5.072	0.063	1.576		1.576	3.4	0.79251	0.93	0.121							0.91	1	
28.707	1	Sensitive Fines	5.148	0.058	1.613		1.613	3.38	0.78208	0.93	0.120							0.91	1	
29.364	1	Sensitive Fines	5.149	0.037	1.649		1.649	3.31	0.7722	0.92	0.120							0.90	1	
30.02	1	Sensitive Fines	5.198	0.068	1.686		1.685	3.43	0.76256	0.92	0.120							0.90	1	
30.676	5	Clayey Silt to Silty Clay	7.524	0.136	1.723	0.021	1.702	3.3	0.7581	0.92	0.121							0.90	1	
31.332	1	Sensitive Fines	5.149	0.089	1.76	0.041	1.719	3.49	0.75368	0.91	0.122							0.90	1	
31.988	1	Sensitive Fines	5.334	0.093	1.796	0.062	1.735	3.48	0.74957	0.91	0.122							0.90	1	
32.644	1	Sensitive Fines	5.536	0.086	1.833	0.082	1.751	3.44	0.74551	0.91	0.123							0.89	1	
33.301	5	Clayey Silt to Silty Clay	8.195	0.128	1.871	0.103	1.768	3.25	0.74124	0.90	0.124							0.89	1	
33.957	7	Silty Sand to Sandy Silt	28.696	0.071	1.909	0.123	1.786	2.38	0.73677	0.90	0.125	2.380	0.74	21.14	2.233	47.202	0.09	0.89	1	0.85
34.613	7	Silty Sand to Sandy Silt	34.276	0.061	1.948	0.144	1.804	2.27	0.73236	0.89	0.125	2.270	0.73	25.10	1.856	46.592	0.09	0.89	1	0.84
35.269	7	Silty Sand to Sandy Silt	24.568	0.097	1.986	0.164	1.822	2.52	0.72799	0.89	0.126	2.520	0.73	17.89	2.872	51.360	0.09	0.89	1	0.88
35.925	6	Sandy Silt to Clayey Silt	24.787	0.318	2.024	0.185	1.839	2.73	0.72392	0.88	0.126							0.89	1	
36.581	6	Sandy Silt to Clayey Silt	22.314	0.514	2.062	0.205	1.857	2.91	0.71966	0.88	0.127							0.88	1	
37.238	5	Clayey Silt to Silty Clay	19.884	0.485	2.099	0.226	1.874	2.97	0.71568	0.87	0.127							0.88	1	
37.894	6	Sandy Silt to Clayey Silt	19.688	0.397	2.137	0.246	1.891	2.92	0.71174	0.87	0.128							0.88	1	
38.55	6	Sandy Silt to Clayey Silt	19.142	0.35	2.174	0.267	1.908	2.92	0.70785	0.86	0.128							0.88	1	
39.206	6	Sandy Silt to Clayey Silt	27.577	0.489	2.212	0.287	1.925	2.75	0.704	0.86	0.128							0.88	1	
39.862	6	Sandy Silt to Clayey Silt	35.444	0.994	2.25	0.307	1.942	2.79	0.70019	0.85	0.128							0.88	1	
40.518	5	Clayey Silt to Silty Clay	28.762	0.828	2.287	0.328	1.959	2.89	0.69642	0.85	0.128							0.87	1	
41.175	6	Sandy Silt to Clayey Silt	27.189	0.62	2.325	0.348	1.976	2.83	0.6927	0.84	0.129							0.87	1	
41.831	6	Sandy Silt to Clayey Silt	38.534	0.883	2.362	0.369	1.994	2.71	0.68879	0.83	0.128							0.87	1	
42.487	6	Sandy Silt to Clayey Silt	30.623	0.87	2.4	0.389	2.011	2.86	0.68514	0.83	0.128							0.87	1	
43.143	6	Sandy Silt to Clayey Silt	28.237	0.811	2.438	0.41	2.028	2.9	0.68154	0.82	0.128							0.87	1	
43.799	6	Sandy Silt to Clayey Silt	27.134	0.657	2.475	0.43	2.045	2.88	0.67797	0.82	0.128							0.87	1	
44.455	6	Sandy Silt to Clayey Silt	21.052	0.503	2.513	0.451	2.062	2.97	0.67443	0.81	0.128							0.87	1	
45.112	6	Sandy Silt to Clayey Silt	21.855	0.521	2.55	0.471	2.079	2.96	0.67094	0.80	0.128							0.86	1	
45.768	5	Clayey Silt to Silty Clay	19.676	0.515	2.588	0.492	2.096	3.04	0.66748	0.80	0.128							0.86	1	
46.424	5	Clayey Silt to Silty Clay	19.453	0.538	2.626	0.512	2.113	3.07	0.66405	0.79	0.128							0.86	1	
47.08	5	Clayey Silt to Silty Clay	16.854	0.37	2.663	0.533	2.131	3.08	0.66046	0.78	0.127							0.86	1	
47.736	6	Sandy Silt to Clayey Silt	18.798	0.299	2.701	0.553	2.148	2.94	0.65711	0.78	0.127							0.86	1	
48.392	7	Silty Sand to Sandy Silt	54.039	0.926	2.739	0.574	2.166	2.55	0.65359	0.77	0.126	2.550	0.65	35.32	3.034	107.168	0.19	0.86	1	1.83
49.049	7	Silty Sand to Sandy Silt	88.157	1.341	2.778	0.594	2.184	2.35	0.65012	0.76	0.126	2.350	0.65	57.31	2.120	121.482	0.25	0.86	1	2.33
49.705	7	Silty Sand to Sandy Silt	75.518	1.219	2.817	0.615	2.202	2.43	0.64668	0.76	0.126	2.430	0.65	48.84	2.439	119.112	0.24	0.85	1	2.25
50.361	7	Silty Sand to Sandy Silt	77.341	1.492	2.855	0.635	2.22	2.47	0.64327	0.75	0.125	2.470	0.64	49.75	2.621	130.414	0.29	0.85	1	2.73
51.017	6	Sandy Silt to Clayey Silt	58.118	1.701	2.893	0.655	2.237	2.68	0.64009	0.74	0.125							0.85	1	
51.673	5	Clayey Silt to Silty Clay	38.867	1.386	2.93	0.676	2.255	2.89	0.63676	0.74	0.124							0.85	1	
52.329	5	Clayey Silt to Silty Clay	33.751	1.056	2.968	0.696	2.272	2.9	0.63364	0.73	0.124							0.85	1	
52.986	6	Sandy Silt to Clayey Silt	34.472	1.045	3.006	0.717	2.289	2.89	0.63055	0.72	0.123							0.85	1	
53.642	5	Clayey Silt to Silty Clay	24.634	0.69	3.043	0.737	2.306	3	0.6275	0.72	0.123							0.85	1	
54.298	5	Clayey Silt to Silty Clay	20.851	0.534	3.081	0.758	2.323	3.06	0.62447	0.71	0.122							0.84	1	
54.954	6	Sandy Silt to Clayey Silt	22.532	0.55	3.118	0.778	2.34	3.02	0.62147	0.70	0.122							0.84	1	
55.61	6	Sandy Silt to Clayey Silt	27.839	0.671	3.156	0.799	2.357	2.92	0.6185	0.70	0.121							0.84	1	
56.266	6	Sandy Silt to Clayey Silt	26.725	0.682	3.194	0.819	2.374	2.96	0.61556	0.69	0.121							0.84	1	
56.923	6	Sandy Silt to Clayey Silt	22.253	0.527	3.231	0.84	2.392	3.03	0.61247	0.69	0.120							0.84	1	
57.579	6	Sandy Silt to Clayey Silt	17.345	0.383	3.269	0.86	2.409	3.13	0.60959	0.68	0.120							0.84	1	
58.235	6	Sandy Silt to Clayey Silt	21.92	0.552	3.306	0.881	2.426	3.06	0.60673	0.67	0.119							0.84	1	
58.891	5	Clayey Silt to Silty Clay	20.463	0.547	3.344	0.901	2.443	3.12	0.6039	0.67	0.119							0.84	1	
59.547	5	Clayey Silt to Silty Clay	14.676	0.299	3.382	0.922	2.46	3.21	0.60109	0.66	0.118							0.84	1	
60.203	5	Clayey Silt to Silty Clay	9.669	0.123	3.419	0.942	2.477	3.37	0.59831	0.66	0.118							0.83	1	
60.86	6	Sandy Silt to Clayey Silt	11.454	0.085	3.457	0.963	2.494	3.15	0.59556	0.65	0.118							0.83	1	
61.516	6	Sandy Silt to Clayey Silt	12.759	0.149	3.494	0.983	2.511	3.15	0.59283	0.65	0.117							0.83	1	
62.172	6	Sandy Silt to Clayey Silt	14.943	0.241	3.532	1.004	2.528	3.14	0.59013	0.64	0.117							0.83	1	
62.828	6	Sandy Silt to Clayey Silt	27.855	0.555	3.57	1.024	2.546	2.9	0.58729	0.64	0.116							0.83	1	
63.484	6	Sandy Silt to Clayey Silt	31.66	0.654	3.607	1.044	2.563	2.86	0.58464	0.63	0.116							0.83	1	
64.14	6	Sandy Silt to Clayey Silt	25.868	0.578	3.645	1.065	2.58	2.98	0.58201	0.63	0.115							0.83	1	
64.797	6	Sandy Silt to Clayey Silt	29.307	0.783	3.682	1.085	2.597	2.97	0.5794	0.62	0.115							0.83	1	
65.453	6	Sandy Silt to Clayey Silt	41.985	1.136	3.72	1.106	2.614	2.83	0.57682	0.62	0.115							0.83	1	
66.109	6	Sandy Silt to Clayey Silt	61.71	1.5	3.758	1.126	2.631	2.67	0.57426	0.61	0.114							0.82	1	
66.765	6	Sandy Silt to Clayey Silt	65.701	1.604	3.795	1.147	2.648	2.65	0.57173	0.61	0.114							0.82	1	
67.421	5	Clayey Silt to Silty Clay	43.562	1.667	3.833	1.167	2.665	2.93	0.56921	0.61	0.113							0.82	1	
68.077	5	Clayey Silt to Silty Clay	32.96	1.335	3.87	1.188	2.683	3.07	0.56657	0.60	0.113							0.82	1	
68.734	5	Clayey Silt to Silty Clay	32.075	1.16	3.908	1.208	2.7	3.05	0.5641	0.60	0.113							0.82	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-CALAVERAS RIVER BRIDGE (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0174)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-CR-005***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
69.39	6	Sandy Silt to Clayey Silt	33.38	1.014	3.946	1.229	2.717	2.98	0.56165	0.60	0.112							0.82	1	
70.046	5	Clayey Silt to Silty Clay	38.021	1.404	3.983	1.249	2.734	2.98	0.55923	0.59	0.112							0.82	1	
70.702	5	Clayey Silt to Silty Clay	36.426	1.355	4.021	1.27	2.751	3	0.55682	0.59	0.112							0.82	1	
71.358	5	Clayey Silt to Silty Clay	49.89	1.749	4.058	1.29	2.768	2.88	0.55444	0.58	0.111							0.82	1	
72.014	7	Silty Sand to Sandy Silt	76.424	1.422	4.097	1.311	2.786	2.55	0.55193	0.58	0.111	2.550	0.55	42.18	3.034	127.986	0.27	0.81	1	2.95
72.671	8	Sand to Silty Sand	94.916	0.973	4.137	1.331	2.806	2.32	0.54918	0.58	0.111	2.320	0.55	52.13	2.015	105.011	0.19	0.81	1	2.02
73.327	8	Sand to Silty Sand	111.993	1.225	4.176	1.352	2.825	2.28	0.54658	0.58	0.111	2.280	0.55	61.21	1.886	115.459	0.22	0.81	1	2.41
73.983	8	Sand to Silty Sand	121.64	1.134	4.216	1.372	2.844	2.21	0.54402	0.57	0.110	2.210	0.54	66.17	1.692	111.941	0.21	0.81	1	2.28
74.639	8	Sand to Silty Sand	108.041	0.988	4.256	1.393	2.863	2.26	0.54147	0.57	0.110	2.260	0.54	58.50	1.827	106.869	0.19	0.81	1	2.10
75.295	8	Sand to Silty Sand	103.11	1.255	4.295	1.413	2.882	2.34	0.53895	0.57	0.110	2.340	0.54	55.57	2.084	115.797	0.22	0.81	1	2.44
75.951	7	Silty Sand to Sandy Silt	90.946	2.269	4.334	1.433	2.901	2.58	0.53645	0.56	0.109	2.580	0.54	48.79	3.206	156.438	0.44	0.81	1	4.75
76.608	8	Sand to Silty Sand	98.502	1.235	4.374	1.454	2.92	2.37	0.53398	0.56	0.109	2.370	0.53	52.60	2.194	115.403	0.22	0.81	1	2.43
77.264	8	Sand to Silty Sand	101.882	0.799	4.413	1.474	2.939	2.26	0.53153	0.56	0.109	2.260	0.53	54.15	1.827	98.926	0.17	0.81	1	1.86
77.92	8	Sand to Silty Sand	91.432	1.294	4.453	1.495	2.958	2.44	0.5291	0.56	0.109	2.440	0.53	48.38	2.483	120.126	0.24	0.81	1	2.65
78.576	8	Sand to Silty Sand	91.732	1.109	4.493	1.515	2.977	2.4	0.52669	0.55	0.109	2.400	0.53	48.31	2.312	111.721	0.21	0.80	1	2.30
79.232	8	Sand to Silty Sand	83.341	1.01	4.532	1.536	2.997	2.44	0.52418	0.55	0.108	2.440	0.52	43.69	2.483	108.478	0.20	0.80	1	2.19
79.888	6	Sandy Silt to Clayey Silt	70.347	1.798	4.57	1.556	3.014	2.7	0.52207	0.55	0.108							0.80	1	
80.545	4	Silty Clay to Clay	46.74	2.182	4.608	1.577	3.031	3.02	0.51997	0.55	0.108							0.80	1	
81.201	4	Silty Clay to Clay	56.753	2.729	4.645	1.597	3.048	2.96	0.51789	0.54	0.108							0.80	1	
81.857	8	Sand to Silty Sand	89.958	1.127	4.685	1.618	3.067	2.43	0.51558	0.54	0.108	2.430	0.52	46.38	2.439	113.124	0.21	0.80	1	2.38
82.513	7	Silty Sand to Sandy Silt	85.381	1.977	4.724	1.638	3.085	2.61	0.51342	0.54	0.107							0.80	1	
83.169	6	Sandy Silt to Clayey Silt	86.298	2.417	4.761	1.659	3.102	2.66	0.51139	0.54	0.107							0.80	1	
83.825	5	Clayey Silt to Silty Clay	65.931	2.716	4.799	1.679	3.12	2.87	0.50926	0.54	0.107							0.80	1	
84.482	7	Silty Sand to Sandy Silt	78.548	1.22	4.837	1.7	3.138	2.55	0.50715	0.53	0.107	2.550	0.51	39.84	3.034	120.869	0.24	0.80	1	2.72
85.138	6	Sandy Silt to Clayey Silt	63.201	2.201	4.875	1.72	3.155	2.84	0.50517	0.53	0.107							0.79	1	
85.794	6	Sandy Silt to Clayey Silt	61.683	1.613	4.913	1.741	3.172	2.78	0.5032	0.53	0.107							0.79	1	
86.45	7	Silty Sand to Sandy Silt	73.727	1.11	4.951	1.761	3.19	2.57	0.50114	0.53	0.107	2.570	0.50	36.95	3.148	116.309	0.23	0.79	1	2.53
87.106	5	Clayey Silt to Silty Clay	57.861	2.131	4.989	1.781	3.207	2.9	0.49921	0.53	0.106							0.79	1	
87.762	7	Silty Sand to Sandy Silt	71.51	1.698	5.027	1.802	3.225	2.7	0.49718	0.52	0.106							0.79	1	
88.419	7	Silty Sand to Sandy Silt	79.344	1.795	5.066	1.822	3.244	2.65	0.49505	0.52	0.106							0.79	1	
89.075	6	Sandy Silt to Clayey Silt	60.083	1.997	5.104	1.843	3.261	2.84	0.49316	0.52	0.106							0.79	1	
89.731	5	Clayey Silt to Silty Clay	76.522	3.099	5.141	1.863	3.278	2.8	0.49129	0.52	0.106							0.79	1	
90.387	7	Silty Sand to Sandy Silt	110.47	2.387	5.18	1.884	3.296	2.51	0.48932	0.52	0.106	2.510	0.49	54.06	2.820	152.410	0.41	0.79	1	4.61
91.043	9	Sand	154.999	1.295	5.221	1.904	3.316	2.15	0.48716	0.52	0.106	2.150	0.49	75.51	1.553	117.249	0.23	0.77	1	2.59
91.699	9	Sand	139.269	0.995	5.261	1.925	3.337	2.16	0.4849	0.52	0.106	2.160	0.48	67.53	1.574	106.313	0.19	0.78	1	2.17
92.356	9	Sand	121.094	0.914	5.302	1.945	3.357	2.23	0.48277	0.51	0.105	2.230	0.48	58.46	1.743	101.925	0.18	0.78	1	2.02
93.012	8	Sand to Silty Sand	102.302	0.87	5.342	1.966	3.376	2.33	0.48077	0.51	0.105	2.330	0.48	49.18	2.049	100.765	0.18	0.78	1	1.98
93.668	7	Silty Sand to Sandy Silt	92.666	1.685	5.38	1.986	3.394	2.55	0.47889	0.51	0.105	2.550	0.48	44.38	3.034	134.648	0.31	0.78	1	3.48
94.324	6	Sandy Silt to Clayey Silt	66.465	2.19	5.418	2.007	3.411	2.84	0.47712	0.51	0.105							0.78	1	
94.98	6	Sandy Silt to Clayey Silt	64.058	2.02	5.456	2.027	3.428	2.82	0.47537	0.51	0.105							0.78	1	
95.636	4	Silty Clay to Clay	65.658	3.016	5.493	2.048	3.446	2.93	0.47353	0.51	0.105							0.78	1	
96.293	3	Clay	38.348	2.137	5.53	2.068	3.462	3.22	0.4719	0.51	0.105							0.78	1	
96.949	3	Clay	37.993	1.909	5.566	2.089	3.478	3.19	0.47029	0.50	0.105							0.78	1	
97.605	5	Clayey Silt to Silty Clay	47.537	2.068	5.604	2.109	3.495	3.05	0.46858	0.50	0.105							0.78	1	
98.261	5	Clayey Silt to Silty Clay	44.747	1.76	5.641	2.13	3.512	3.06	0.46689	0.50	0.105							0.78	1	
98.917	5	Clayey Silt to Silty Clay	44.338	1.549	5.679	2.15	3.529	3.02	0.46521	0.50	0.105							0.78	1	
99.573	5	Clayey Silt to Silty Clay	50.551	1.968	5.717	2.17	3.546	3	0.46355	0.50	0.105							0.78	1	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Unconfined Compression Test

Strength tests were performed on selected undisturbed samples using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-3A through B-3C.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-2.

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/06/2009
Date Submitted 05/01/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR I-5 NORTH Site ID : R-09-CR-001#5@26.0'
Thank you for your business.

* For future reference to this analysis please use SUN # 55616-112074.

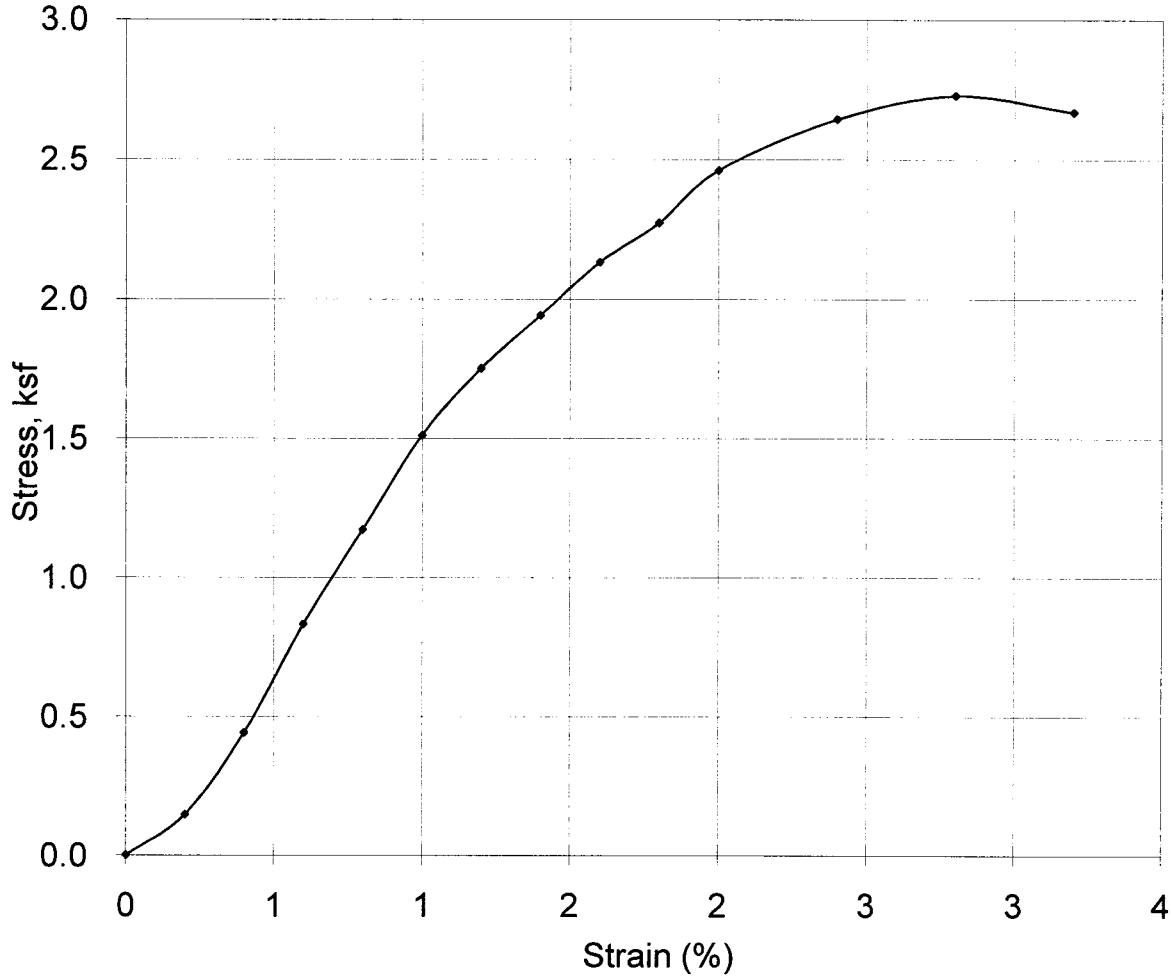
EVALUATION FOR SOIL CORROSION

Soil pH	7.18		
Minimum Resistivity	1.88 ohm-cm (x1000)		
Chloride	25.3 ppm	00.00253	%
Sulfate	2.4 ppm	00.00024	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 8 2009
By

UNCONFINED COMPRESSION TEST



Boring No.: R-09-CR-001

Sample No. : MC-1

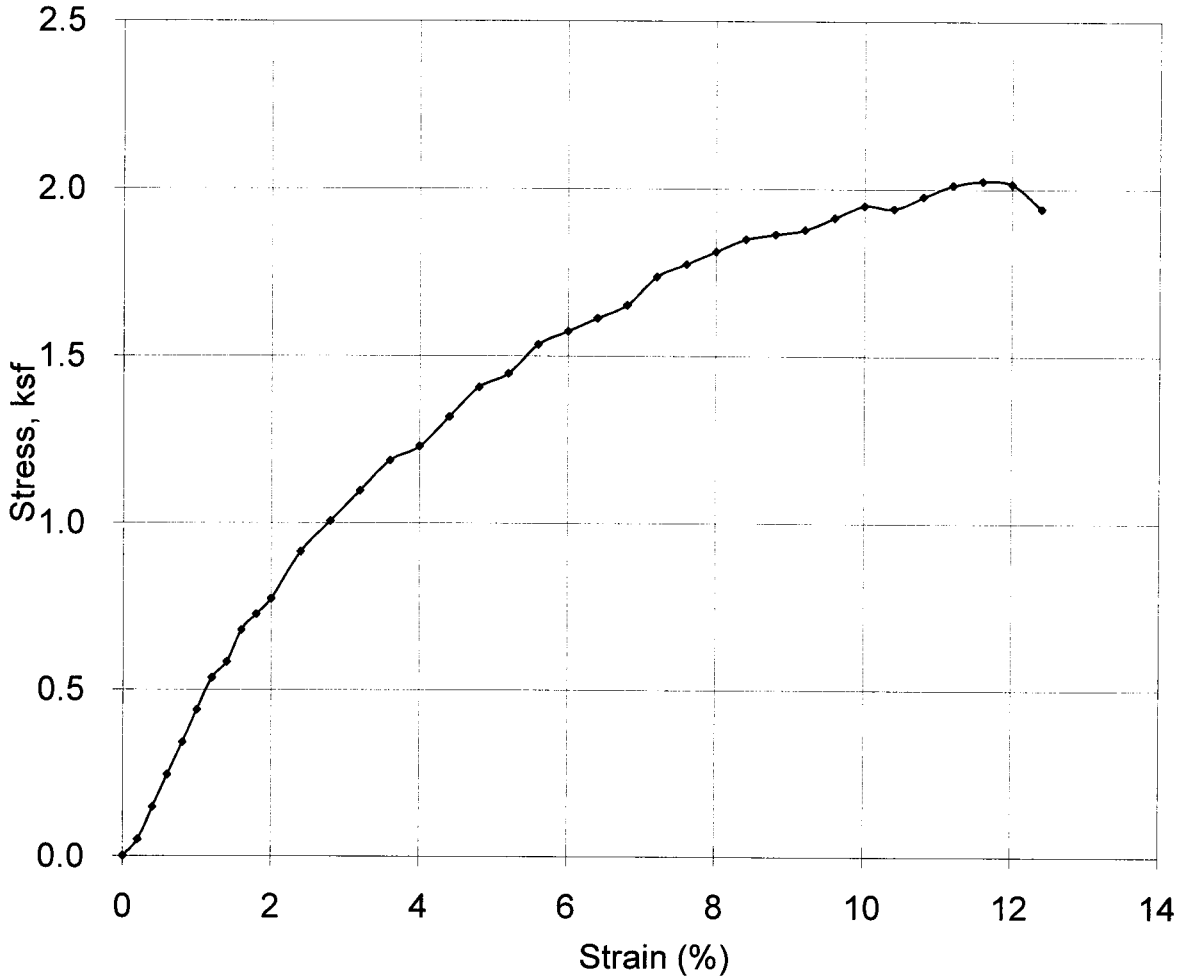
Depth (feet): 6

Material Description:
Lean Clay

Unconfined Compressive Strength: 2.7 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**CALAVERAS RIVER BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-3A

UNCONFINED COMPRESSION TEST



Boring No.: R-09-CR-001

Sample No. : MC-4

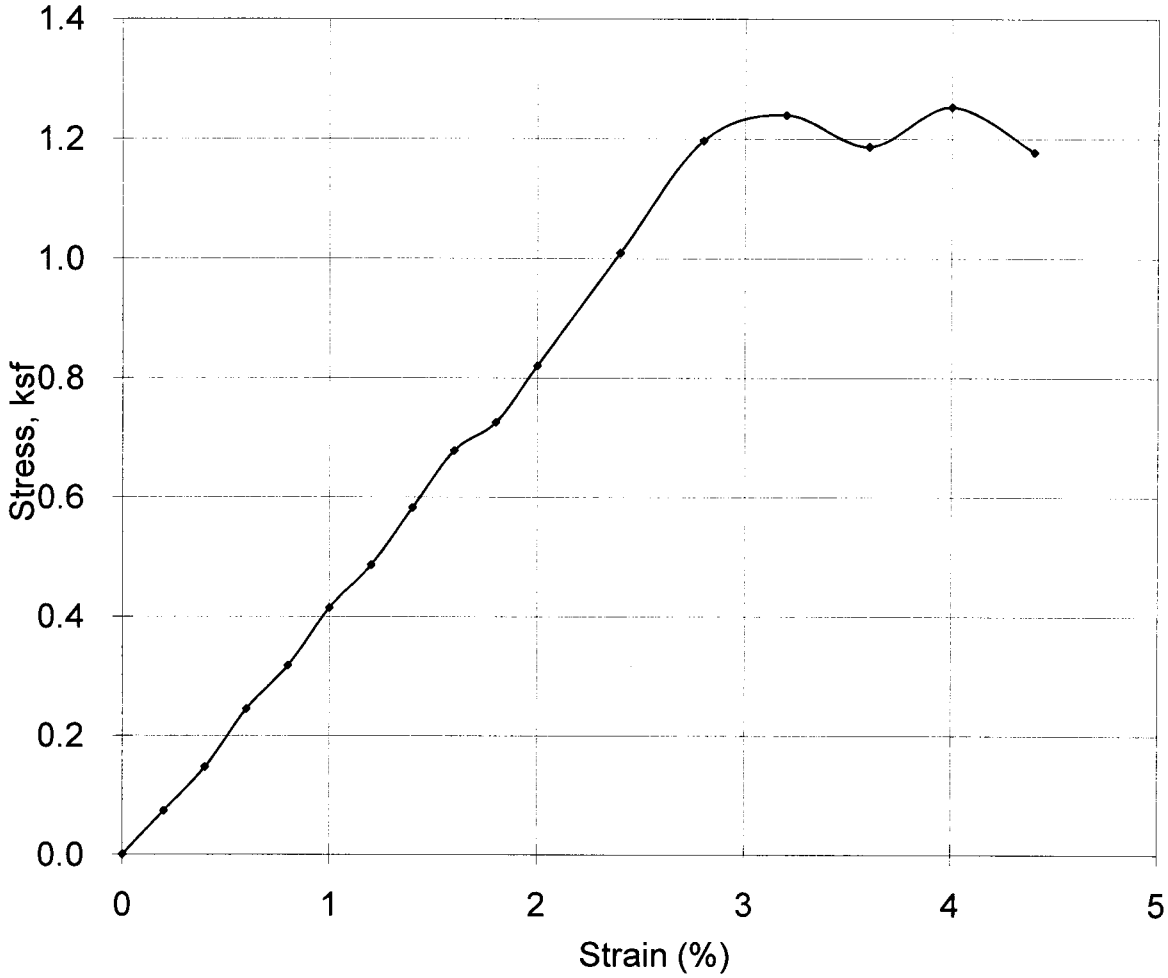
Depth (feet): 21

Material Description:
Lean Clay

Unconfined Compressive Strength: 2.0 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**CALAVERAS RIVER BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-3B

UNCONFINED COMPRESSION TEST



Boring No.: R-09-CR-001

Sample No. : MC-8

Depth (feet): 41

Material Description:
Silty Sand

Unconfined Compressive Strength: 1.3 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**CALAVERAS RIVER BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-3C

APPENDIX C

Evaluation of Liquefaction Potential

PARIKH

Practicing in the Geosciences

JOB
SHEET NO OF
CALCULATED BY A Lam DATE 4/2009
CHECKED BY DATE
SCALE

Evaluation of Liquefaction Potential - Calaveras River Bridge (Boring A-09-CR-001)

12'		
8.5'	Lean Clay SPT N=10	$\gamma' = 115$ pcf
10'	Lean Clay Avg SPT N=30	$\gamma' = 120$ pcf, $\gamma_{sat} = 130$ pcf
14.5'	Lean Clay Avg SPT N=15	$\gamma_{sat} = 130$ pcf
5.5'	Lean Clay SPT N=10	$\gamma_{sat} = 125$ pcf
10'	Lean Clay Avg SPT N=22	$\gamma_{sat} = 130$ pcf
4.5'	Lean Clay SPT N=8	$\gamma_{sat} = 125$ pcf
5'	Lean Clay SPT N=50/6"	$\gamma_{sat} = 130$ pcf
8'	Lean Clay SPT N=6	$\gamma_{sat} = 125$ pcf
10'	Lean Clay SPT N=27	$\gamma_{sat} = 130$ pcf
10'	Lean Clay SPT N=12	$\gamma_{sat} = 125$ pcf
7.5'	Lean Clay SPT N=7	$\gamma_{sat} = 125$ pcf
16.5'	Poorly Graded Sand (Medium dense) Avg SPT N=17	$\gamma_{sat} = 130$ pcf
6.5'	Poorly Graded Sand (Loose) SPT N=6	$\gamma_{sat} = 130$ pcf

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-CR-001 (Calaveras River Bridge) (Widen)***

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

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} (g)= 0.2
FAULT M_w = 7.0

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 12

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=($CRR_{7.0}$/CSR)*MSF*Kσ*Kα*

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE ($CRR_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	8.5	2	10	SPT	977.5	977.5	0.96		12	1.43	1	0.75	1.2	1.15	17.08	55%			1.00	1	
2	18.5	2	30	SPT	2242.5	1836.3	0.92		35	1.04	1	0.80	1.2	1.15	39.88	55%			1.00	1	
3	33.0	2	15	SPT	4127.5	2815.0	0.82		17	0.84	1	0.80	1.2	1.15	16.11	55%			0.90	1	
4	38.5	2	10	SPT	4815.0	3158.8	0.76		12	0.80	1	0.80	1.2	1.15	10.14	55%			0.87	1	
5	48.5	1	22	SPT	6115.0	3833.8	0.67	0.14	25	0.72	1	0.80	1.2	1.15	20.24	15%	23.71	0.27	0.82	1	1.90
6	53.0	2	8	SPT	6677.5	4115.0	0.63		9	0.70	1	0.80	1.2	1.15	7.10	55%			0.81	1	
7	58.0	2	50	SPT	7327.5	4452.5	0.59		58	0.67	1	0.80	1.2	1.15	42.69	55%			0.79	1	
8	66.0	2	6	SPT	8327.5	4952.5	0.55		7	0.64	1	0.80	1.2	1.15	4.86	55%			0.76	1	
9	76.0	2	27	SPT	9627.5	5627.5	0.51		31	0.60	1	0.80	1.2	1.15	20.50	55%			0.73	1	
10	86.0	2	12	SPT	10877.5	6252.5	0.49		14	0.57	1	0.80	1.2	1.15	8.65	55%			0.71	1	
11	93.5	2	7	SPT	11815.0	6721.3	0.48		8	0.55	1	0.80	1.2	1.15	4.86	55%			0.70	1	
12	105.0	1	17	SPT	13310.0	7497.5	0.46	0.11	20	0.52	1	0.80	1.2	1.15	11.18	11%	12.69	0.14	0.67	1	1.03 <- LIQ!!
13	111.5	1	6	SPT	14155.0	7936.3	0.46	0.11	7	0.50	1	0.80	1.2	1.15	3.84	11%	5.15	0.07	0.66	1	0.54 <- LIQ!!

Liquefaction is unlikely to occur for soils at the depth of more than 50 feet below ground surface

Vertical Pile Capacity Calculations

PROJECT: Calaveras River Bridge (Widen)
BRIDGE NO: 29-0174 R/L

FOUNDATION DESIGN DATA

FOR INTERIOR PILES ONLY

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size (ft)		Permissible Settlement (in.).	No. of Piles per Support Per Bridge
					B	L		
Abutment 1	WSD	Class 200 Alt 'X'	19.3	17.3	2.5	46.96	1	7
Bent 2	LRFD	Class 200 Alt 'X'	6.1	2.1	3	46.96	1	8
Bent 3	LRFD	2'-6" Dia CIDH	5.4	4.4	N/A	N/A	1	8
Bent 4	LRFD	2'-6" Dia CIDH	17.2	17.2	N/A	N/A	1	8
Bent 5	LRFD	2'-6" Dia CIDH	10.0	8.4	N/A	N/A	1	8
Bent 5a	LRFD	5'-0" Dia CIDH	10.0	10.0	N/A	N/A	1	2
Bent 6	LRFD	2'-6" Dia CIDH	-5.2	-10.0	N/A	N/A	1	8
Bent 7	LRFD	2'-6" Dia CIDH	-8.5	-13.6	N/A	N/A	1	8
Bent 8	LRFD	2'-6" Dia CIDH	-4.0	-9.0	N/A	N/A	1	8
Bent 9	LRFD	2'-6" Dia CIDH	18.5	18.5	N/A	N/A	1	8
Bent 10	LRFD	2'-6" Dia CIDH	6.5	5.5	N/A	N/A	1	8
Bent 11	LRFD	Class 200 Alt 'X'	5.1	1.1	3	48.02	1	8
Abutment 12	WSD	Class 200 Alt 'X'	20.1	18.1	2.5	48.02	1	7

Notes: Pile cut-off elev for Bents 4 and 9 are approximate FG elevations. Pile cut-off elev for Bents 3 and 10 are 1' below approximate elevations. Pile cut-off elev for Bents 5, 6, 7, and 8 are at the bottom of estimated local scour within the river. Pile cut-off at Abut 1 an reflect the bottom of abutment diaphragm elevation. Pile cut-off at Bent 2 and Bent 11 reflect the bottom of footing elevation.

FOUNDATION LOADS

FOR INTERIOR PILES ONLY

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abutment 1	490	70	340	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	1000	N/A	710	1410	240	0	0	N/A	260	0	-90
Bent 3	930	N/A	660	1360	233	0	0	N/A	210	0	-50
Bent 4	1240	N/A	800	1600	253	0	0	N/A	240	0	-60
Bent 5	1030	N/A	740	1460	241	0	0	N/A	240	0	-30
Bent 5a	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1100	560	0	0
Bent 6	1020	N/A	760	1470	247	0	0	N/A	210	0	0
Bent 7	1040	N/A	770	1490	252	0	0	N/A	210	0	0
Bent 8	1050	N/A	760	1470	247	0	0	N/A	210	0	0
Bent 9	1230	N/A	800	1590	243	0	0	N/A	240	0	-50
Bent 10	980	N/A	710	1450	250	0	0	N/A	200	0	-20
Bent 11	1040	N/A	750	1460	260	0	0	N/A	260	0	-90
Abutment 12	510	80	360	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes: Total Load for Service-I Limit State @ Abutments 1 & 12 = DL + LL (HL93)

Total Load for Extreme Limit State @ Bent 5a = 1.0 DL only (collapse prevention only)

Strength Limit State = Governing LRFD Strength Limit State.

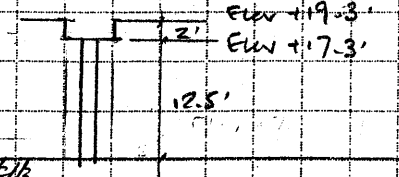
Extreme Limit State = DL + Seismic

LEGEND:

Vertical Pile Capacity Analysis - Calaveras River Bridge UC (Boring A-09-005) Abut 1

Class 200 Alt "X" 14" square precast concrete

70 tons Service load



Assumed Toe of Embankment (Elev +5.0')

1.2'

8.5' — Lean Clay SPT N=10 γ'=115 pcf, c=1250 psf

10' — Lean Clay Avg SPT N=30 γ'=120 pcf, c=3500 psf, γ_{sat}=130 pcf

14.5' — Lean Clay Avg SPT N=15 c=1500 psf, γ_{sat}=130 pcf

5.5' — c=1250 psf Lean Clay SPT N=10 γ_{sat}=125 pcf

10' — c=3500 psf Lean Clay Avg SPT N=22 γ_{sat}=130 pcf

4.5' — c=1000 psf Lean Clay SPT N=8 γ_{sat}=125 pcf

5' — c=4000 psf Lean Clay SPT N=50/6" γ_{sat}=130 pcf

8' — c=750 psf Lean Clay SPT N=6 γ_{sat}=125 pcf

10' — c=3000 psf Lean Clay SPT N=27 γ_{sat}=130 pcf

10' — c=1500 psf Lean Clay SPT N=12 γ_{sat}=125 pcf

7.5' — c=875 psf Lean Clay SPT N=7 γ_{sat}=125 pcf

18.5' — ϕ'=36° Poorly Graded Sand (Medium dense) Avg SPT N=17 γ_{sat}=130 pcf

6.5' — ϕ'=28° Poorly Graded Sand (Loose) SPT N=6 γ_{sat}=130 pcf

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Calaveras River Bridge (Widen)*** ***Abut 1 - 70 tons service load***

ALLOWABLE FRICTIONAL PILE CAPACTY 04/16/10

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS INCREAMENTS

1- GRAVELS, SANDS (FT)= 5

AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : A-09-001 (PARIKH 2009)

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 200 Alt. "X" - 14" square precast prestressed

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	BT2 FTG	1	125	250	125	SAND		0		0.00	0.00	0.00	0.00	0.00
2.0	8.0		1	125	750	625	SAND	DENSE	0	37	0.00	11.68	11.68	3.91	3.91
8.0	14.5		1	125	812.5	1406	SAND	MED DEN	0	35	0.00	27.08	38.75	9.07	12.98
14.5	18.5		2	125	500	2063	CLAY	STIFF	1250	0	0.85	9.56	48.31	9.56	22.54
18.5	23.0		2	120	540	2583	CLAY	STIFF	1250	0	0.85	10.76	59.07	10.76	33.30
23.0	26.5		2	68	238	2972	CLAY	V.STIFF	3500	0	0.25	6.89	65.96	6.89	40.19
26.5	33.0		2	68	442	3074	CLAY	V.STIFF	3500	0	0.25	12.80	78.76	12.80	52.99
33.0	38.0		2	68	340	3703	CLAY	STIFF	1500	0	0.85	14.34	93.10	14.34	67.33
38.0	45.5		2	68	510	4128	CLAY	STIFF	1500	0	0.85	21.52	114.62 ←	21.52	88.85
45.5	47.5		2	68	136	4451	CLAY	STIFF	1500	0	0.90	6.08	120.69	5.34	94.19
47.5	53.0		2	63	346.5	4556	CLAY	STIFF	1250	0	0.95	14.70	135.39	14.70	108.89
53.0	58.0		2	68	340	5035	CLAY	V.STIFF	2500	0	0.50	14.06	149.45	14.06	122.95
58.0	63.0		2	68	340	5375	CLAY	V.STIFF	2500	0	0.50	14.06	163.51	8.44	131.39
63.0	67.5		2	63	283.5	5687	CLAY	MEDIUM	1000	0	1.00	10.13	173.64	13.67	145.06
67.5	72.5		2	68	340	5999	CLAY	V.STIFF	4000	0	0.30	13.50	187.14	16.88	161.93
72.5	76.5		2	63	252	6295	CLAY	MEDIUM	750	0	1.00	6.75	193.89	13.50	175.43
76.5	80.5		2	63	252	5331	CLAY	MEDIUM	750	0	1.00	6.75	200.64	6.75	182.18
80.5	85.5		2	68	340	5715	CLAY	V.STIFF	3000	0	1.00	8.44	209.08	15.19	197.37
85.5	90.5		2	68	340	6843	CLAY	V.STIFF	3000	0	0.45	15.19	224.26	15.19	212.56
90.5	95.5		2	63	315	7170	CLAY	V.STIFF	3000	0	0.45	15.19	239.45	16.88	229.43
95.5	105.5		2	63	630	7668	CLAY	STIFF	1500	0	1.00	33.75	273.20	33.75	263.18
105.5	113.0		2	63	472.5	7904	CLAY	STIFF	1500	0	1.00	25.31	298.51	25.31	288.49
113.0	124.5		1	68	782	8689	SAND		0		1.00	0.00	298.51	0.00	288.49
124.5	131.0		1	68	442	8991	SAND	MED DEN	0	33	0.00	24.91	323.42	8.34	296.84

Abut 1 - Pile cut-off elevation is +17.30 feet. Assumed finished grade elevation is +19.30 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +5.0 feet

70 kips/pile (Service). Required pile capacity = 70 kips /0.5 = 140 kips (70 tons)

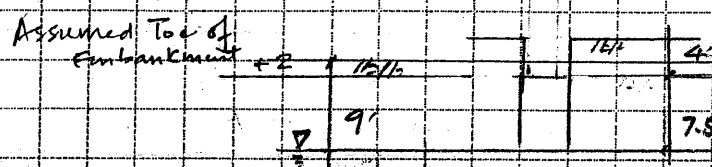
Pile Length @ Abutment 1 = 45.5' - 2.0' = 43.5 '

Pile tip at approxiamtely 43.5' below pile cut-off(pile tip@ Elev. -26.5', pile cut-off @ Elev. +17.3'),available Ult. Pile Capacity=114.6 tons (friction)-38.8 tons(Predrilled)=75.8 tons >required capacity (140 k/70 tons), OK

I-5 Calaveras River Abut 1 Class 200 Alt X (4-16-10)

Sheet 2 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-2) Bent 2
Class 200 Alt "X" Nominal Resistance 350 kips



9' Clayey Silt Avg SPT N = 7 c = 875 psf
γ' = 115 pcf

6' γsat = 125 pcf Clayey Silt Avg SPT N = 10 c = 1250 psf

5' γsat = 125 pcf Clay Avg SPT N = 13 c = 1500 psf

9' γsat = 130 pcf Clayey Silt Avg SPT N = 15 c = 1750 psf

5' Dr = 90% φ' = 36° Gravel SPT N = 43 γsat = 130 pcf

5' γsat = 130 pcf Silt SPT N = 15 c = 1750 psf

6' γsat = 130 pcf Clayey Silt SPT N = 16 c = 1750 psf

10' Clay Avg SPT N = 29 γsat = 130 pcf c = 3500 psf

6.5' γsat = 130 pcf Clay SPT N = 15 c = 1750 psf

7' φ' = 33.5° Clayey Sand SPT N = 30 γsat = 130 pcf

19.5' Silt Avg SPT N = 28 c = 3250 psf γsat = 130 pcf

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Calaveras River Bridge (Widen)*** ***Bent 2-350k(175 tons)Strength Limit State*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 06/07/10

Design per FHWA Manual — 206127.BR

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS — INCREAMENTS

1- GRAVELS, SANDS AND NONPLASTIC SILTS (FT)= 5

2- CLAYS AND PLASTIC SILTS SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : Boring B-2

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) : 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 200 Alt. "X" - 14" square precast prestressed c

DEPTH FROM	TO	SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	6.0 Predrillto+0.(	1	125	750	375	SAND		0		0.00	0.00	0.00	0.00	0.00
6.0	11.5	2	125	687.5	1094	CLAY	MEDIUM	875	0	0.95	10.29	10.29	10.29	10.29
11.5	17.5	2	63	378	1627	CLAY	STIFF	1250	0	0.85	14.34	24.63	14.34	24.63
17.5	22.5	2	63	315	1973	CLAY	STIFF	1500	0	0.80	13.50	38.13	13.50	38.13
22.5	27.0	2	68	306	2284	CLAY	STIFF	1750	0	0.75	13.29	51.42	13.29	51.42 ←
27.0	31.5	2	68	306	2590	CLAY	STIFF	1750	0	0.75	13.29	64.71	13.29	64.71
31.5	36.5	1	68	340	2607	SAND	V. DENSE	0	40	0.00	34.48	99.19	11.55	76.26
36.5	41.5	2	68	340	3253	CLAY	STIFF	1750	0	0.85	16.73	115.93	16.73	92.99
41.5	47.5	2	68	408	3627	CLAY	STIFF	1750	0	0.85	20.08	136.01 ←	20.08	113.08
47.5	52.5	2	68	340	4001	CLAY	V.STIFF	3500	0	0.25	9.84	145.85	9.84	122.92
52.5	57.5	2	68	340	4001	CLAY	V.STIFF	3500	0	0.25	9.84	155.69	9.84	132.76
57.5	64.0	2	68	442	4732	CLAY	STIFF	1750	0	0.95	24.31	180.01 ←	24.31	157.08
64.0	65.0	1	68	68	4987	SAND	MED DEN	0	35	0.00	4.76	184.77	1.60	158.67
65.0	71.0	1	68	408	5225	SAND	MED DEN	0	35	0.00	28.48	213.25	9.54	168.21
71.0	76.0	2	68	340	5599	CLAY	V.STIFF	3250	0	0.35	12.80	226.05	12.80	181.01
76.0	81.0	2	68	340	5939	CLAY	V.STIFF	3250	0	0.35	12.80	238.84	12.80	193.81
81.0	86.0	2	68	340	5123	CLAY	V.STIFF	3250	0	0.35	12.80	251.64	12.80	206.60
86.0	90.5	2	68	306	5174	CLAY	V.STIFF	3250	0	0.35	11.52	263.16	11.52	218.12

Bent 2 -- Pile cut-off elevation is +2.10 feet. Assumed finished grade elevation is +6.10 feet

Pre-drill through the exsiting embankment to bottom of pipe. Bottom of pipe assumed to be at Elev. +0.0 feet

240 kips/pile (Strength Limit State). Required pile capacity (comp)= 240 kips /0.7 = 350 kips (175 tons). 260 kips/pile (Extreme Limit State). Required pile capacity (tens) = 90 kips (45 tons)

Pile Length @ Bent 2 = 64.0' - 4.0' = 60.0 '

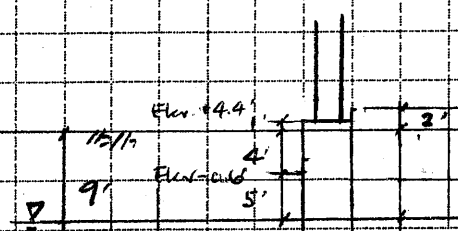
Pile tip at 60.0' (comp)& 27.0' (tens) below pile cut-off (pile tip @ Elev. -58.0'(comp) & -25.0' (tens), pile cut-off @ Elev. +2.1'), Ult. Pile Capacity =180.0 T(Friction)> required pile capacity (350 k/175 tons), OK

Sheet 4 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-2) Bent 3 (Int)

36" ø CIDH concrete pile

Nominal Resistance: 340 kips



Elev. +3.5'

Assume

Depth	Description	Properties
9'	Clayey Silt	Avg SPT N = 7, c = 875 psf, γ' = 115 pcf
6'	Clayey Silt	γsat = 125 pcf, Avg SPT N = 10, c = 1250 psf
5'	Clay	γsat = 125 pcf, Avg SPT N = 13, c = 1500 psf
9'	Clayey Silt	γsat = 130 pcf, Avg SPT N = 15, c = 1750 psf
5'	Gravel	Dr = 90%, φ' = 36°, SPT N = 43, γsat = 130 pcf
5'	Silt	γsat = 130 pcf, SPT N = 15, c = 1750 psf
6'	Clayey Silt	γsat = 130 pcf, SPT N = 16, c = 1750 psf
10'	Clay	γsat = 130 pcf, Avg SPT N = 29, c = 2500 psf
6.5'	Clay	γsat = 130 pcf, SPT N = 15, c = 1750 psf
7'	Clayey Sand	φ' = 33.5°, SPT N = 30, γsat = 130 pcf
19.5'	Silt	γsat = 130 pcf, Avg SPT N = 28, c = 3250 psf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

PILE DIA, B (ft) = 2.50

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

I-5 Calaveras River Bridge (Widen) Bent 3
206127.CRB

Bent 3 Interior Piles (Based on Boring B-2)

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY 04/06/10
206127.BR

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 7.85 SQ. FT.
AREA OF THE PILE TIP 4.91 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	6.0	2	CLAY	115	690	345.0	7	MEDIUM	0.55	875			0.00	0.00	0.00	0.00
6.0	11.0	2	CLAY	115	575	977.5	7	MEDIUM	0.55	875			9.44	9.44	6.61	6.61
11.0	17.0	2	CLAY	63	378	1454.0	10	STIFF	0.55	1250			16.19	25.64	11.33	17.94
17.0	22.0	2	CLAY	63	315	1800.5	13	STIFF	0.55	1625			17.54	43.18	12.28	30.22
22.0	31.0	2	CLAY	68	612	2264.0	15	STIFF	0.55	1875			36.43	79.60	25.50	55.72 ←
31.0	36.0	1	SAND	68	340	2740.0	43	V. DENSE			40	0.72	38.64	118.25	19.32	75.04
36.0	41.0	2	CLAY	68	340	3080.0	15	STIFF	0.55	1875			20.24	138.48	14.17	89.21
41.0	47.0	2	CLAY	68	408	3454.0	16	STIFF	0.55	2000			25.91	164.39 ←	18.13	107.34
47.0	51.0	2	CLAY	68	272	3794.0	29	V.STIFF	0.52	3625			29.52	193.91	20.67	128.01
51.0	55.0	2	CLAY	68	272	4066.0	29	V.STIFF	0.52	3625			29.52	223.44	20.67	148.68
55.0	57.0	2	CLAY	68	136	3998.0	29	V.STIFF	0.52	3625			14.76	238.20	20.67	169.34
57.0	63.5	2	CLAY	68	442	4151.0	15	STIFF	0.55	1875			26.31	264.51 ←	20.67	190.01
63.5	70.5	1	SAND	68	476	5018.0	30	DENSE			41	0.39	54.46	318.96	10.33	200.34
70.5	90.0	2	CLAY	68	1326	5919.0	28	V.STIFF	0.53	3500			140.64	459.60	18.42	218.76

Bent 3 Pile cut-off elevation is +4.4 feet. Finished grade elevation is +5.4 feet

Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71

Strength Limit (comp): 233 kips /(0.7*0.71) ≈ 470 kips (235 tons). Extreme Limit Comp): 210 kips/(1.0*0.71) ≈300 kips (150 tons). Extreme Limit (Tens): 50kips/(1.0*0.71) ≈ 75 kips (37.5 tons)

Pile Length @ Bent 3 = 63.5'- 1.0' = 62.5' (Strength Limit Comp). Pile Length = 47.0'-1.0' = 46.0' (Extreme Lmit Comp). Pile Length = 31.0' - 1.0' = 30.0' (Extreme Limit Tension)

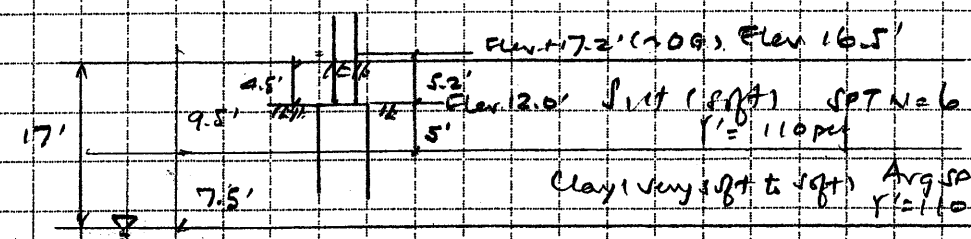
Pile tip at 62.5'(Strength Comp) below pile cut-off (pile tip @ Elev. -58.0'(Strength Comp), -41.5' (Extreme Comp) -25.5'(Extreme Tens), pile cut-off @ Elev. +4.4') Ult. Pile Capacity=264.5 tons(Friction)> 470k/235 tons, OK

Sheet 6 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-1) Bent 4 (Int)

30" ϕ CIDH Concrete pile

Nominal Resistance = 370 tons



Assume

Silt (soft) SPT N = 6 C = 750 psf
γ' = 110 pcf

Clay (very soft to soft) Avg SPT N = 5 C = 625 psf
γ' = 110 pcf

Clay (very soft to soft) Avg SPT N = 7
C = 875 psf
γsat = 130 pcf

21'

Silty Sand (loose to medium dense) Avg SPT N = 9
ϕ' = 33° γsat = 135 pcf

10'

Sand (medium dense) SPT N = 15 ϕ' = 34°
γsat = 135 pcf

6.5'

Silty Clay (stiff to very stiff)
Avg SPT N = 18 C = 2000 psf
γsat = 130 pcf

15'

γsat = 125 pcf Clayey Silt (stiff) SPT N = 15 C = 1500 psf

6'

Silt (very stiff) Avg SPT N = 33
γsat = 130 pcf C = 3500 psf

10

γsat = 130 pcf Silt (very stiff) SPT N = 30 C = 3250 psf

6'

Silty Sand (medium dense) SPT N = 18
γsat = 130 pcf ϕ' = 35°

15'

Sand Avg SPT N = 43
γsat = 135 pcf ϕ' = 36°

12.5'

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 4
206127.CRB

4/06/10
206127.BR

ALLOWABLE PILE CAPACTY

Bent 4 Interior Piles (Based on Boring B-1)

PILE DIA, B (ft) = 2.50

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1
F.S. FOR UPLIFT = 1
PERIMETER OF THE PILE (ft) = 7.85 SQ. FT.
AREA OF THE PILE TIP 4.91 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	110	550	275.0	6	MEDIUM	0.55	750			0.00	0.00	0.00	0.00
5.0	10.0	2	CLAY	110	550	825.0	6	MEDIUM	0.55	750			8.10	8.10	5.67	5.67
10.0	15.0	2	CLAY	110	550	1375.0	5	MEDIUM	0.55	625			6.75	14.84	4.72	10.39
15.0	17.5	2	CLAY	110	275	1787.5	5	MEDIUM	0.55	625			3.37	18.21	2.36	12.75
17.5	22.5	2	CLAY	68	340	2095.0	7	MEDIUM	0.55	875			9.44	27.66	6.61	19.36
22.5	27.5	2	CLAY	68	340	2435.0	7	MEDIUM	0.55	875			9.44	37.10	6.61	25.97
27.5	32.5	2	CLAY	68	340	2775.0	7	MEDIUM	0.55	875			9.44	46.55	6.61	32.58
32.5	38.5	2	CLAY	68	408	3149.0	7	MEDIUM	0.55	875			11.33	57.88	7.93	40.52
38.5	43.5	1	SAND	73	365	3535.5	9	MED DENSE			33	0.38	26.46	84.34	13.23	53.75
43.5	48.5	1	SAND	73	365	3900.5	9	MED DENSE			33	0.35	26.84	111.18	13.42	67.17
48.5	55.0	1	SAND	73	474.5	4320.3	15	MED DENSE			34	0.53	58.29	169.47	29.14	96.31
55.0	60.0	2	CLAY	68	340	4727.5	18	V.STIFF	0.55	2250			24.29	193.76	17.00	113.31
60.0	68.0	2	CLAY	68	544	5169.5	18	V.STIFF	0.55	2250			38.86	232.61	27.20	140.51
68.0	70.0	2	CLAY	68	136	5509.5	18	V.STIFF	0.55	2250			9.71	242.33	6.80	147.31
70.0	75.0	2	CLAY	68	340	5747.5	15	STIFF	0.55	1875			20.24	262.57	14.17	161.48
75.0	81.0	2	CLAY	68	408	6121.5	33	STIFF	0.49	4125			47.96	310.53	33.58	195.05
81.0	86.0	2	CLAY	68	340	6495.5	33	STIFF	0.49	4125			39.97	350.50	27.98	223.03
86.0	92.0	2	CLAY	68	408	6869.5	30	V.STIFF	0.51	3750			45.26	395.76	31.68	254.72
92.0	97.0	1	SAND	68	340	7243.5	18	MED DENSE			34	0.25	35.54	431.30	17.77	272.48
97.0	102.0	1	SAND	68	340	7583.5	18	MED DENSE			33	0.25	37.21	468.51	18.60	291.09
102.0	107.0	1	SAND	68	340	7923.5	18	MED DENSE			33	0.25	38.87	507.38	19.44	310.53
107.0	112.0	1	SAND	73	365	8276.0	43	DENSE			36	0.25	40.60	547.99	20.30	330.83
112.0	117.0	1	SAND	73	365	8641.0	43	DENSE			36	0.25	42.39	590.38	21.20	352.02
117.0	119.5	1	SAND	73	182.5	8914.8	43	DENSE			36	0.25	21.87	612.25	10.93	362.96

Bent 4 Pile cut-off elevation is +17.2 feet. Finished grade elevation is +17.2 feet

Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71

Strength Limit (comp): 253 kips /(0.7*0.71) ≈ 510 kips (255 tons). Extreme Limit Comp): 240 kips/(1.0*0.71) ≈340 kips (170 tons). Extreme Limit (Tens): 60kips/(1.0*0.71) ≈ 90 kips (45.0 tons)

Pile Length @ Bent 4 = 75.0' (Strength Limit Comp). Pile Length = 55.0' (Extreme Lmit Comp). Pile Length = 43.5' (Extreme Limit Tension)

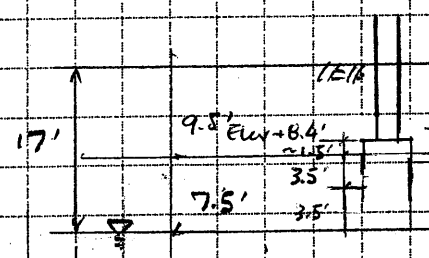
Pile tip at 75.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -57.5'(Strength Comp), -38.0' (Extreme Comp) -26.5'(Extreme Tens), pile cut-off @ Elev. +17.2') Ult. Pile Capacity=262.6 tons (Friction)> 510k/255 tons, OK

Sheet 8 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-1) Bent 5 (Int)

30" ϕ CIDH Concrete pile

Nominal Resistance = 350 kips



Elev 16.5' Assume

Silt (soft) SPT N = 6 c = 750 psf
γ' = 110 pcf

Clay (very soft to soft) Avg SPT N = 5 c = 625 psf
γ' = 110 psf

Clay (very soft to soft) Avg SPT N = 7
c = 875 psf
γsat = 125 pcf

21'

Silty sand (loose to medium dense) Avg SPT N = 9
ϕ' = 33° γsat = 135 pcf

10'

Sand (medium dense) SPT N = 15 ϕ' = 34°
γsat = 135 pcf

6.5'

Silty Clay (stiff to very stiff)
Avg SPT N = 18 c = 2000 psf
γsat = 130 pcf

15'

γsat = 125 pcf Clayey Silt (stiff) SPT N = 15 c = 1500 psf

6'

Silt (very stiff) Avg SPT N = 33
γsat = 130 pcf c = 3500 psf

10

γsat = 130 pcf Silt (very stiff) SPT N = 30 c = 3250 psf

6'

Silty Sand (medium dense) SPT N = 18
γsat = 130 pcf ϕ' = 35°

45'

Sand Avg SPT N = 43
γsat = 135 pcf ϕ' = 36°

125'

CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 5

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4/06/10

206127.BR

ALLOWABLE PILE CAPACTY

Bent 5 Interior Piles (Based on Boring B-1)

PILE DIA, B (ft) = 2.50

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 7.85 SQ. FT.
AREA OF THE PILE TIP 4.91 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	2	CLAY	51	154	77.0	6	MEDIUM	0.55	750			0.00	0.00	0.00	0.00
3.0	6.5	2	CLAY	110	385	346.5	5	MEDIUM	0.55	625			0.00	0.00	0.00	0.00
6.5	10.5	2	CLAY	110	440	759.0	5	MEDIUM	0.55	625			5.40	5.40	3.78	3.78
10.5	15.5	2	CLAY	68	340	1149.0	7	MEDIUM	0.55	875			9.44	14.84	6.61	10.39
15.5	20.5	2	CLAY	68	340	1489.0	7	MEDIUM	0.55	875			9.44	24.29	6.61	17.00
20.5	25.5	2	CLAY	68	340	1829.0	7	MEDIUM	0.55	875			9.44	33.73	6.61	23.61
25.5	31.5	2	CLAY	68	408	2203.0	7	MEDIUM	0.55	875			11.33	45.06	7.93	31.54 ←
31.5	36.5	1	SAND	73	365	2589.5	9	MED DENSE			40	0.43	21.73	66.80	10.87	42.41
36.5	41.5	1	SAND	73	365	2954.5	9	MED DENSE			40	0.39	22.85	89.65	11.43	53.84
41.5	48.0	1	SAND	73	474.5	3374.3	15	MED DENSE			40	0.60	51.39	141.04	25.69	79.53
48.0	54.0	2	CLAY	68	408	3815.5	18	V.STIFF	0.55	2250			29.14	170.18 ←	20.40	99.93
54.0	58.0	2	CLAY	68	272	4155.5	18	V.STIFF	0.55	2250			19.43	189.61	13.60	113.53
58.0	61.0	2	CLAY	68	204	4393.5	18	V.STIFF	0.55	2250			14.57	204.18	10.20	123.73
61.0	69.0	2	CLAY	68	544	4767.5	15	STIFF	0.55	1875			32.38	236.56	22.67	146.40
69.0	72.0	2	CLAY	68	204	5141.5	33	STIFF	0.49	4125			23.98	260.55 ←	16.79	163.19
72.0	79.0	2	CLAY	68	476	5481.5	33	STIFF	0.49	4125			55.96	316.50	39.17	202.36
79.0	85.0	2	CLAY	68	408	5923.5	30	V.STIFF	0.51	3750			45.26	361.76	31.68	234.04
85.0	90.0	1	SAND	68	340	6297.5	18	MED DENSE			34	0.25	30.90	392.66	15.45	249.49
90.0	95.0	1	SAND	68	340	6637.5	18	MED DENSE			34	0.25	32.57	425.23	16.28	265.77
95.0	100.0	1	SAND	68	340	6977.5	18	MED DENSE			34	0.25	34.23	459.46	17.12	282.89
100.0	105.0	1	SAND	68	340	7317.5	43	DENSE			37	0.25	35.90	495.36	17.95	300.84
105.0	110.0	1	SAND	68	340	7657.5	43	DENSE			37	0.25	37.57	532.93	18.78	319.62
110.0	112.5	1	SAND	68	170	7912.5	43	DENSE			37	0.25	19.41	552.34	9.71	329.33

Bent 5 Pile cut-off elevation is +8.4 feet. Finished grade elevation is +10.0 feet - 1.58 feet (scour depth) = 8.42 feet

Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71

Strength Limit (comp): 241 kips /(0.7*0.71) ≈ 490 kips (245 tons). Extreme Limit Comp): 240 kips/(1.0*0.71) ≈ 340 kips (170 tons). Extreme Limit (Tens): 30kips/(1.0*0.71) ≈ 50 kips (25.0 tons)

Pile Length @ Bent 5 = 72.0' -1.6' ≈ 70.4' (Strength Limit Comp). Pile Length = 54.0' - 1.6' ≈ 52.4' (Extreme Lmit Comp). Pile Length = 31.5' - 1.6' ≈ 29.9' (Extreme Limit Tension)

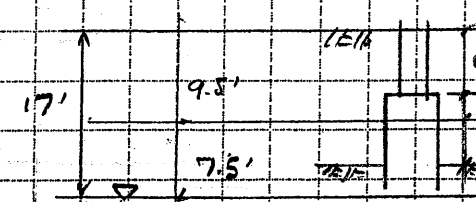
Pile tip at 70.4'(Strength Comp) below pile cut-off (pile tip @ Elev. -62.0'(Strength Comp),-44.0' (Extreme Comp),-21.5'(Extreme Tens), pile cut-off @ Elev. +8.4')Ult. Pile Capacity=260.6 tons (Friction)> 490k/245 tons, OK

Sheet 10 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-1) Bent 5a

60" ø CIDH concrete pile

Nominal Resistance 280 tons



Elev 16.5' Assume

Silt (soft) SPT N=6 c=750 psf

Elev +10' (MSL) γ'=110 pcf

Clay (very soft to soft) Avg SPT N=5 c=625 psf

Elev +8.0 γ'=110 pcf

Clay (very soft to soft) Avg SPT N=7

c=875 psf

21' γsat=130 pcf

Silty sand (loose to medium dense) Avg SPT N=9

10' φ'=33° γsat=135 pcf

Sand (medium dense) SPT N=15 φ'=34°

6.5' γsat=135 pcf

Silty Clay (stiff to very stiff)

15' Avg SPT N=18 c=2000 psf

γsat=130 pcf

6' γsat=125 pcf Clayey Silt (stiff) SPT N=15 c=1500 psf

Silt (very stiff) Avg SPT N=33

10 γsat=130 pcf c=3500 psf

6' γsat=130 pcf Silt (very stiff) SPT N=30 c=3250 psf

Silty Sand (medium dense) SPT N=18

15' γsat=130 pcf φ'=35°

Sand Avg SPT N=43

12.5' γsat=135 pcf φ'=36°

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 5a
206127.CRB

4/16/10
ALLOWABLE PILE CAPACTY
206127.BR

Bent 5a (Based on Boring B-1)

PILE DIA, B (ft) = 5.00

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 15.71 SQ. FT.
AREA OF THE PILE TIP 19.63 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	$\Delta\sigma'$ (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	ϕ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	2	CLAY	8	25.3	12.7	6	MEDIUM	0.55	750			0.00	0.00	0.00	0.00
3.0	7.0	2	CLAY	110	440	245.3	5	MEDIUM	0.55	625			0.00	0.00	0.00	0.00
7.0	10.5	2	CLAY	110	385	657.8	5	MEDIUM	0.55	625			9.45	9.45	6.62	6.62
10.5	15.5	2	CLAY	68	340	1020.3	7	MEDIUM	0.55	875			18.90	28.35	13.23	19.85
15.5	20.5	2	CLAY	68	340	1360.3	7	MEDIUM	0.55	875			18.90	47.25	13.23	33.08
20.5	25.5	2	CLAY	68	340	1700.3	7	MEDIUM	0.55	875			18.90	66.15	13.23	46.31
25.5	31.5	2	CLAY	68	408	2074.3	7	MEDIUM	0.55	875			22.68	88.84	15.88	62.18
31.5	36.5	1	SAND	68	340	2448.3	9	MED DENSE			33	0.43	41.13	129.96	20.56	82.75
36.5	41.5	1	SAND	68	340	2788.3	9	MED DENSE			33	0.39	43.16	173.12	21.58	104.33
41.5	48.0	1	SAND	68	442	3179.3	15	MED DENSE			35	0.60	96.89	270.02	48.45	152.78
48.0	52.5	2	CLAY	68	306	3553.3	18	V.STIFF	0.55	2250			43.74	313.76 ←	30.62	183.40
52.5	60.0	2	CLAY	68	510	3961.3	18	V.STIFF	0.55	2250			72.90	386.67	51.03	234.43
60.0	63.0	2	CLAY	68	204	4318.3	18	V.STIFF	0.55	2250			29.16	415.83	20.41	254.84
63.0	64.0	2	CLAY	68	68	4454.3	15	STIFF	0.55	1875			8.10	423.93	5.67	260.51
64.0	69.0	2	CLAY	68	340	4658.3	15	STIFF	0.55	1875			40.50	464.43	28.35	288.86
69.0	74.0	2	CLAY	68	340	4998.3	33	STIFF	0.49	4125			79.99	544.42	55.99	344.86
74.0	79.0	2	CLAY	68	340	5338.3	33	STIFF	0.49	4125			79.99	624.42	55.99	400.85
79.0	85.0	1	SAND	68	408	5712.3	30	DENSE			36	0.28	74.71	699.13	37.36	438.21
85.0	90.0	1	SAND	68	340	6086.3	18	MED DENSE			34	0.25	59.76	758.89	29.88	468.09
90.0	95.0	1	SAND	68	340	6426.3	18	MED DENSE			34	0.25	63.10	821.99	31.55	499.64
95.0	100.0	1	SAND	68	340	6766.3	18	MED DENSE			34	0.25	66.44	888.42	33.22	532.86
100.0	105.0	1	SAND	73	365	7118.8	43	DENSE			37	0.25	69.90	958.32	34.95	567.81
105.0	110.0	1	SAND	73	365	7483.8	43	DENSE			37	0.25	73.48	1031.80	36.74	604.55
110.0	112.5	1	SAND	73	182.5	7757.6	43	DENSE			37	0.25	38.08	1069.89	19.04	623.59

Bent 5a --> Pile cut-off elevation is +10.0 feet. Finished grade elevation is +10.0 feet -2.77 feet (scour depth) = 7.23 feet
560 kips/pile (Extreme Limit State). Required pile capacity = 560 kips /1.0 = 560 kips (280 tons)
Pile Length @ Bent 5a = 52.5'
Pile tip at 52.5' (comp) below pile cut-off (pile tip @ Elev. -42.5' (comp), pile cut-off @ Elev. +10.0'), Ult. Capacity = 313.8 tons (friction) > 280 tons (Extreme Limit State) OK

Sheet 12 of 27

Pile capacity Analysis. - Calaveras River Bridge (Widen) (Boring B-1) Bent 6
30" ϕ CIDH Concrete (Int)
Nominal Resistance = 360 kips

Elev 16.5' Assume

TEL

17'

9.5' — Silt (soft) SPT N=6 C=750 psf, γ'=110 pcf

7.5' — Clay (very soft to soft) Avg SPT N=5 γ'=110 pcf C=625 psf

4.5' Elev -5.2' (~0.6)

5' Clay (Very soft to soft) Avg SPT N=7 C=875 psf

Elev -10.0

5'

21' — γsat=130 pcf

6.5'

10' — Silty Sand (loose to medium dense) Avg SPT N=9 ϕ'=33° γsat=135 pcf

6.5' — Sand (medium dense) SPT N=15 ϕ'=34° γsat=135 pcf

Silty Clay (stiff to very stiff)

15' — Avg SPT N=18 C=2000 psf γsat=130 pcf

6' — γsat=125 pcf Clayey Silt (stiff) SPT N=15 C=1500 psf

Silt (very stiff) Avg SPT N=33

10 — γsat=130 pcf C=3500 psf

6' — γsat=130 pcf Silt (very stiff) SPT N=30 C=3250 psf

Silty Sand (medium dense) SPT N=18

15' — γsat=130 pcf ϕ'=35°

Sand Avg SPT N=43

125' — γsat=135 pcf ϕ'=36°

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 6
206127.CRB

4/06/10
206127.GDR

ALLOWABLE PILE CAPACTY

Bent 6 Interior Piles (Based on Boring B-1)

PILE DIA, B (ft) = 2.50

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 7.85 SQ. FT.
AREA OF THE PILE TIP 4.91 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	3	12.92	6.5	7	MEDIUM	0.55	875			0.00	0.00	0.00	0.00
5.0	10.0	2	CLAY	68	340	182.9	7	MEDIUM	0.55	875			0.00	0.00	0.00	0.00
10.0	16.5	2	CLAY	68	442	573.9	7	MEDIUM	0.55	875			12.28	12.28	8.59	8.59
16.5	21.5	1	SAND	73	365	977.4	9	MED DENSE			34	0.55	10.49	22.77	5.25	13.84
21.5	26.5	1	SAND	73	365	1342.4	9	MED DENSE			34	0.50	13.26	36.03	6.63	20.47
26.5	33.0	1	SAND	73	474.5	1762.2	15	DENSE			36	0.76	34.33	70.36	17.17	37.63
33.0	38.0	2	CLAY	68	340	2169.4	18	V.STIFF	0.55	2250			24.29	94.64	17.00	54.63
38.0	43.0	2	CLAY	68	340	2509.4	18	V.STIFF	0.55	2250			24.29	118.93	17.00	71.63
43.0	48.0	2	CLAY	68	340	2849.4	18	V.STIFF	0.55	2250			24.29	143.22	17.00	88.63
48.0	54.0	2	CLAY	68	408	3223.4	15	STIFF	0.55	1875			24.29	167.50 ←	17.00	105.63
54.0	62.0	2	CLAY	68	544	3699.4	33	STIFF	0.49	4125			63.95	231.45	44.77	150.40
62.0	65.0	2	CLAY	68	204	4073.4	33	STIFF	0.49	4125			23.98	255.44 ←	16.79	167.19
65.0	70.0	2	CLAY	68	340	4345.4	30	V.STIFF	0.51	3750			37.72	293.15	26.40	193.59
70.0	75.0	1	SAND	68	340	4685.4	18	MED DENSE			35	0.35	32.23	325.38	16.12	209.71
75.0	80.0	1	SAND	68	340	5025.4	18	MED DENSE			34	0.31	30.73	356.11	15.36	225.07
80.0	85.0	1	SAND	68	340	5365.4	18	MED DENSE			34	0.27	28.83	384.94	14.42	239.48
85.0	90.0	1	SAND	73	365	5717.9	43	DENSE			38	0.25	28.05	412.99	14.03	253.51
90.0	95.0	1	SAND	73	365	6082.9	43	DENSE			37	0.25	29.84	442.84	14.92	268.43
95.0	97.5	1	SAND	73	182.5	6356.7	43	DENSE			37	0.25	15.59	458.43	7.80	276.23

Bent 6 Pile cut-off elevation is -10.0 feet. Finished grade elevation is -5.2 feet - 4.81 feet (scour depth) = -10.01 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 247 kips /(0.7*0.71) ≈ 500 kips (250 tons). Extreme Limit Comp): 210 kips/(1.0*0.71) ≈ 300 kips (150 tons).
Pile Length @ Bent 6 = 65.0' -5.0' = 60.0' (Strength Limit Comp). Pile Length = 54.0' - 5.0' = 49.0' (Extreme Lmit Comp).
Pile tip at 60.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -70.0' (Strength Comp),-59.0' (Extreme Comp) pile cut-off @ Elev. -10.0')Ult. Pile Capacity=255.4 tons (Friction)> 500k/250 tons, OK

γ' of 3 pcf is the effective unit weight considering the scour [3=68*(5-4.81)/5]

Sheet 14 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-7) Bent 7 (Interior)

30" ϕ CIOH concrete pile

Nominal Resistance: 860 kips

Elev. 19.0' Assume

REF.

Silt & Clay (soft) SPT N = 6 c = 750 psf
γ' = 115 pcf

10'

15'

5' Dr = 60%. ϕ' = 33° Sand (loose) SPT N = 10 γ' = 120 pcf

Sand (very loose) SPT N = 3
γ' = 125 pcf

0' Dr = 25%. ϕ' = 30°

REF. 25' Elev. -8.5' (~0.6%)

5'

13.5' Elev. -13.5'

5'

Clayey Silt (stiff to very stiff) Avg SPT N = 21
γsat = 130 pcf c = 2500 psf

12'

Clayey Silt (stiff) Avg SPT N = 15 c = 1875 psf
γsat = 130 pcf

14'

Clayey Silt Avg SPT N = 23 c = 2750 psf
γsat = 130 pcf

5'

Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

16.5'

Clayey Silt Avg SPT N = 32 c = 3500 psf
γsat = 130 pcf

6.5'

Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 7
206127.CRB

4/06/10

Bent 7 Interior Piles (Based on Boring B-7)

PILE DIA, B (ft) = 2.5

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 7.85 SQ. FT.
AREA OF THE PILE TIP 4.91 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	0	0	0.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
5.0	11.0	2	CLAY	68	408	204.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
11.0	17.0	2	CLAY	68	408	612.0	15	STIFF	0.55	1875			24.29	24.29	17.00	17.00
17.0	23.0	2	CLAY	68	408	1020.0	15	STIFF	0.55	1875			24.29	48.57	17.00	34.00
23.0	30.0	2	CLAY	68	476	1462.0	23	V.STIFF	0.55	2875			43.44	92.02	30.41	64.41
30.0	37.0	2	CLAY	68	476	1938.0	23	V.STIFF	0.55	2875			43.44	135.46	30.41	94.82
37.0	42.0	2	CLAY	68	340	2346.0	15	STIFF	0.55	1875			20.24	155.70 ←	14.17	108.99
42.0	47.0	2	CLAY	68	340	2686.0	32	V.STIFF	0.50	4000			39.25	194.95	27.48	136.46
47.0	56.0	2	CLAY	68	612	3162.0	32	V.STIFF	0.50	4000			70.65	265.60 ←	49.46	185.92
56.0	58.5	2	CLAY	68	170	3553.0	32	V.STIFF	0.50	4000			19.63	285.22	13.74	199.66
58.5	65.0	2	CLAY	69	448.5	3862.3	15	STIFF	0.55	1875			26.31	311.53	18.42	218.07

Bent 7 Pile cut-off elevation is -13.6 feet. Finished grade elevation is -8.5 feet - 5.1 feet (scour depth) = -13.6 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 252 kips /(0.7*0.71) ≈ 510 kips (255 tons). Extreme Limit Comp): 210 kips/(1.0*0.71) ≈ 300 kips (150 tons).
Pile Length @ Bent 7 = 56.0' -5.0' = 51.0' (Strength Limit Comp). Pile Length = 42.0' - 5.0' = 37.0' (Extreme Lmit Comp).
Pile tip at 51.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -64.5' (Strength Comp),-50.5' (Extreme Comp) pile cut-off @ Elev. -13.6')Ult. Pile Capacity=265.6 tons (Friction)> 510k/255 tons, OK

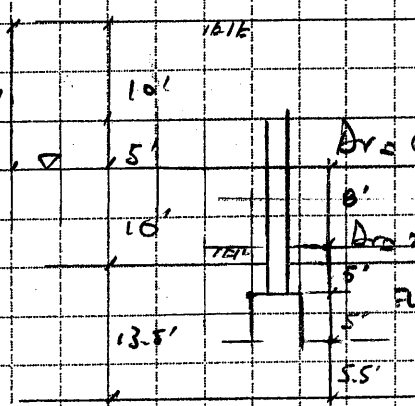
γ' of 0 pcf is the effective unit weight for the top five feet considering the scour from -8.5 feet to -13.6 feet.

Sheet 16 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widening) (Boring B-7) Bent B (Interior)

30" ϕ CIDH Concrete pile

Nominal Resistance = 360 kips



Elev. 19.0' Assume

Silt & Clay (soft) SPT N = 6 c = 750 psf
γ' = 115 pcf

φ' = 33° Sand (loose) SPT N = 10 γ' = 120 pcf

Sand (very loose) SPT N = 3
γ' = 125 pcf

Clayey Silt (stiff to very stiff) Avg SPT N = 21
γsat = 130 pcf c = 2500 psf

Clayey Silt (stiff) Avg SPT N = 15 c = 1875 psf
γsat = 130 pcf

Clayey Silt Avg SPT N = 23 c = 2750 psf
γsat = 130 pcf

Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

Clayey Silt Avg SPT N = 32 c = 3500 psf
γsat = 130 pcf

Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

PILE DIA, B (ft) = 2.50

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

I-5 Calaveras River Bridge (Widen) Bent 8
206127.CRB

Bent 8 Interior Piles (Based on Boring B-7)

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY 4/06/10
206127.BR

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 7.85 SQ. FT.
AREA OF THE PILE TIP 4.91 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	2	CLAY	0	0	0.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
2.0	5.0	2	CLAY	0	0	0.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
5.0	10.0	2	CLAY	68	340	170.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
10.0	15.5	2	CLAY	68	374	527.0	21	V.STIFF	0.55	2625			31.17	31.17	21.82	21.82
15.5	21.5	2	CLAY	68	408	918.0	15	STIFF	0.55	1875			24.29	55.45	17.00	38.82
21.5	27.5	2	CLAY	68	408	1326.0	15	STIFF	0.55	1875			24.29	79.74	17.00	55.82
27.5	34.5	2	CLAY	68	476	1768.0	23	V.STIFF	0.55	2875			43.44	123.18	30.41	86.23
34.5	41.5	2	CLAY	68	476	2244.0	23	V.STIFF	0.55	2875			43.44	166.63 ←	30.41	116.64
41.5	46.5	2	CLAY	68	340	2652.0	15	STIFF	0.55	1875			20.24	186.87	14.17	130.81
46.5	53.5	2	CLAY	68	476	3060.0	32	V.STIFF	0.50	4000			54.95	241.82	38.47	169.27
53.5	56.0	2	CLAY	68	170	3383.0	32	V.STIFF	0.50	4000			19.63	261.44 ←	13.74	183.01
56.0	63.0	2	CLAY	68	476	3706.0	32	V.STIFF	0.50	4000			54.95	316.39	38.47	221.47
63.0	69.5	2	CLAY	68	442	4165.0	15	STIFF	0.55	1875			26.31	342.70	18.42	239.89

Bent 8 Pile cut-off elevation is -9.0 feet. Finished grade elevation is -4.0 feet - 5.04 feet (scour depth) = -9.04 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 247 kips /(0.7*0.71) ≈ 500 kips (250 tons). Extreme Limit Comp): 210 kips/(1.0*0.71) ≈ 300 kips (150 tons).
Pile Length @ Bent 7 = 56.0' -5.0' = 51.0' (Strength Limit Comp). Pile Length = 41.5' - 5.0' = 36.5' (Extreme Lmit Comp).
Pile tip at 51.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -60.0' (Strength Comp),-45.5' (Extreme Comp) pile cut-off @ Elev. -9.0')Ult. Pile Capacity=261.4 tons (Friction)> 500k/250 tons, OK

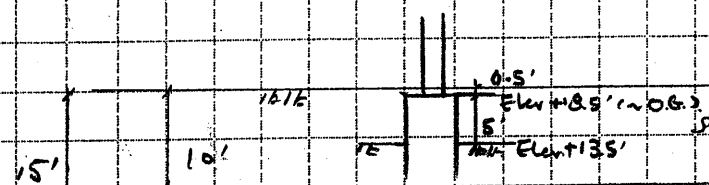
γ' of 0 pcf is the effective unit weight for the top five feet considering the scour from -4.0 feet to -9.0 feet.

Sheet 18 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-7) Bent 9 (Interior)

30" ⌀ CIDH concrete pile

Nominal Resistance 350 kips



Elev. 19.0' Assume

Bent 9 Clay (soft) SPT N = 6 c = 750 psf
γ' = 115 pcf

Dr = 60% φ' = 33° Sand (Loose) SPT N = 10 γ' = 120 pcf

10' Sand (very loose) SPT N = 3
Dr = 25% φ' = 30° γ' = 125 pcf

13.5' Clayey Silt (stiff to very stiff) Avg SPT N = 21
γsat = 130 pcf c = 2500 psf

12' Clayey Silt (stiff) Avg SPT N = 15 c = 1875 psf
γsat = 130 pcf

14' Clayey Silt Avg SPT N = 23 c = 2750 psf
γsat = 130 pcf

5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

16.5' Clayey Silt Avg SPT N = 32 c = 3500 psf
γsat = 130 pcf

6.5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

PILE DIA, B (ft) = 2.50

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

I-5 Calaveras River Bridge (Widen) Bent 9

206127.CRB

Bent 9 Interior Piles (Based on Boring B-7)

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY 4/06/10
206127.BR

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 7.85 SQ. FT.
AREA OF THE PILE TIP 4.91 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	115	575	287.5	6	MEDIUM	0.55	750			0.00	0.00	0.00	0.00
5.0	9.5	2	CLAY	115	517.5	833.8	6	MEDIUM	0.55	750			7.29	7.29	5.10	5.10
9.5	14.5	1	SAND	120	600	1392.5	10	MED DENSE			34	0.69	18.81	26.09	9.40	14.50
14.5	19.5	1	SAND	63	315	1850.0	3	LOOSE			31	0.25	9.08	35.17	4.54	19.04
19.5	24.5	1	SAND	63	315	2165.0	3	LOOSE			40	0.25	10.62	45.79	5.31	24.35
24.5	29.5	2	CLAY	68	340	2492.5	21	V.STIFF	0.55	2625			28.33	74.13	19.83	44.19 ←
29.5	34.5	2	CLAY	68	340	2832.5	21	V.STIFF	0.55	2625			28.33	102.46	19.83	64.02
34.5	38.0	2	CLAY	68	238	3121.5	21	V.STIFF	0.55	2625			19.83	122.29	13.88	77.90
38.0	44.0	2	CLAY	68	408	3444.5	15	STIFF	0.55	1875			24.29	146.58	17.00	94.90
44.0	50.0	2	CLAY	68	408	3852.5	15	STIFF	0.55	1875			24.29	170.86 ←	17.00	111.90
50.0	55.0	2	CLAY	68	340	4226.5	23	V.STIFF	0.55	2875			31.03	201.90	21.72	133.63
55.0	60.0	2	CLAY	68	340	4566.5	23	V.STIFF	0.55	2875			31.03	232.93	21.72	155.35
60.0	64.0	2	CLAY	68	272	4872.5	23	V.STIFF	0.55	2875			24.83	257.75 ←	17.38	172.73
64.0	69.0	2	CLAY	68	340	5178.5	15	STIFF	0.55	1875			20.24	277.99	14.17	186.89
69.0	74.0	2	CLAY	68	340	5518.5	32	V.STIFF	0.50	4000			39.25	317.24	27.48	214.37
74.0	80.0	2	CLAY	68	408	5892.5	32	V.STIFF	0.50	4000			47.10	364.34	32.97	247.34
80.0	85.5	2	CLAY	68	374	6283.5	32	V.STIFF	0.50	4000			43.18	407.52	30.22	277.56
85.5	92.0	2	CLAY	68	442	6691.5	15	STIFF	0.55	1875			26.31	433.83	18.42	295.98

Bent 9 Pile cut-off elevation is +18.5 feet. Finished grade elevation is +18.5 feet

Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71

Strength Limit (comp): 243 kips /(0.7*0.71) ≈ 490 kips (245 tons). Extreme Limit Comp): 240 kips/(1.0*0.71) ≈340 kips (170 tons). Extreme Limit (Tens): 50kips/(1.0*0.71) ≈ 75 kips (37.5 tons)

Pile Length @ Bent 9 = 64.0' (Strength Limit Comp). Pile Length = 50.0' (Extreme Lmit Comp). Pile Length = 29.5' (Extreme Limit Tension)

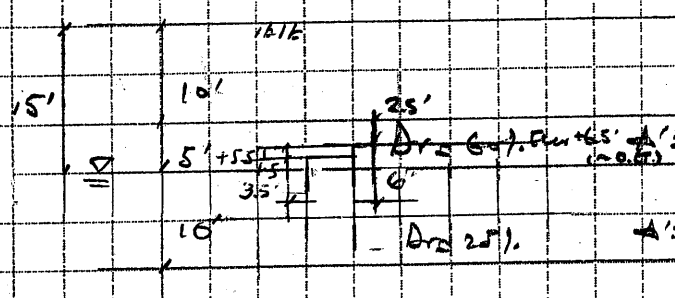
Pile tip at 64.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -45.5'(Strength Comp), -31.5' (Extreme Comp) -11.0'(Extreme Tens), pile cut-off @ Elev. +18.5') Ult. Pile Capacity=257.8 tons (Friction)> 490k/245 tons, OK

Sheet 20 of 27

Pile Capacity Analysis. - Calaveras River Bridge (Widen) (Boring B-7) Bent 10 (Interior)

30" ⌀ CIDH Concrete pile

Nominal Resistance 360 kips



Elev. 19.0' Assume

Silt & Clay (Soft) SPT N = 6 c = 750 psf
γ' = 115 pcf

Sand (Loose) SPT N = 10 γ' = 120 pcf

Sand (Very Loose) SPT N = 3
γ' = 125 pcf

13.5' Clayey Silt (Stiff to Very Stiff) Avg SPT N = 2
γsat = 130 pcf c = 2500 psf

12' Clayey Silt (Stiff) Avg SPT N = 15 c = 1875 psf
γsat = 130 pcf

14' Clayey Silt Avg SPT N = 23 c = 2750 psf
γsat = 130 pcf

5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

16.5' Clayey Silt Avg SPT N = 32 c = 3500 psf
γsat = 130 pcf

6.5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

PILE DIA, B (ft) = 2.50

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

I-5 Calaveras River Bridge (Widen) Bent 10

206127.CRB

Bent 10 Interior Piles (Based on Boring B-7)

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY 4/06/10
206127.BR

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 7.85 SQ. FT.
AREA OF THE PILE TIP 4.91 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.5	1	SAND	120	300	150.0	10	DENSE			36	0.90	0.00	0.00	0.00	0.00
2.5	5.0	1	SAND	63	157.5	378.8	3	LOOSE			32	0.25	0.00	0.00	0.00	0.00
5.0	7.5	1	SAND	63	157.5	536.3	3	LOOSE			32	0.25	1.32	1.32	0.66	0.66
7.5	12.5	1	SAND	63	315	772.5	3	LOOSE			31	0.25	3.79	5.11	1.90	2.55
12.5	17.5	2	CLAY	68	340	1100.0	21	V.STIFF	0.55	2625			28.33	33.44	19.83	22.39 ←
17.5	22.5	2	CLAY	68	340	1440.0	21	V.STIFF	0.55	2625			28.33	61.77	19.83	42.22
22.5	26.0	2	CLAY	68	238	1729.0	21	V.STIFF	0.55	2625			19.83	81.61	13.88	56.10
26.0	32.0	2	CLAY	68	408	2052.0	15	STIFF	0.55	1875			24.29	105.89	17.00	73.10
32.0	38.0	2	CLAY	68	408	2460.0	15	STIFF	0.55	1875			24.29	130.18	17.00	90.10
38.0	43.0	2	CLAY	68	340	2834.0	23	V.STIFF	0.55	2875			31.03	161.21 ←	21.72	111.83
43.0	48.0	2	CLAY	68	340	3174.0	23	V.STIFF	0.55	2875			31.03	192.24	21.72	133.55
48.0	52.0	2	CLAY	68	272	3480.0	23	V.STIFF	0.55	2875			24.83	217.07	17.38	150.93
52.0	57.0	2	CLAY	68	340	3786.0	15	STIFF	0.55	1875			20.24	237.31	14.17	165.09
57.0	61.0	2	CLAY	68	272	4092.0	32	V.STIFF	0.50	4000			31.40	268.71 ←	21.98	187.07
61.0	68.0	2	CLAY	68	476	4466.0	32	V.STIFF	0.50	4000			54.95	323.66	38.47	225.54
68.0	73.5	2	CLAY	68	374	4891.0	32	V.STIFF	0.50	4000			43.18	366.83	30.22	255.76
73.5	80.0	2	CLAY	68	442	5299.0	15	STIFF	0.55	1875			26.31	393.14	18.42	274.18

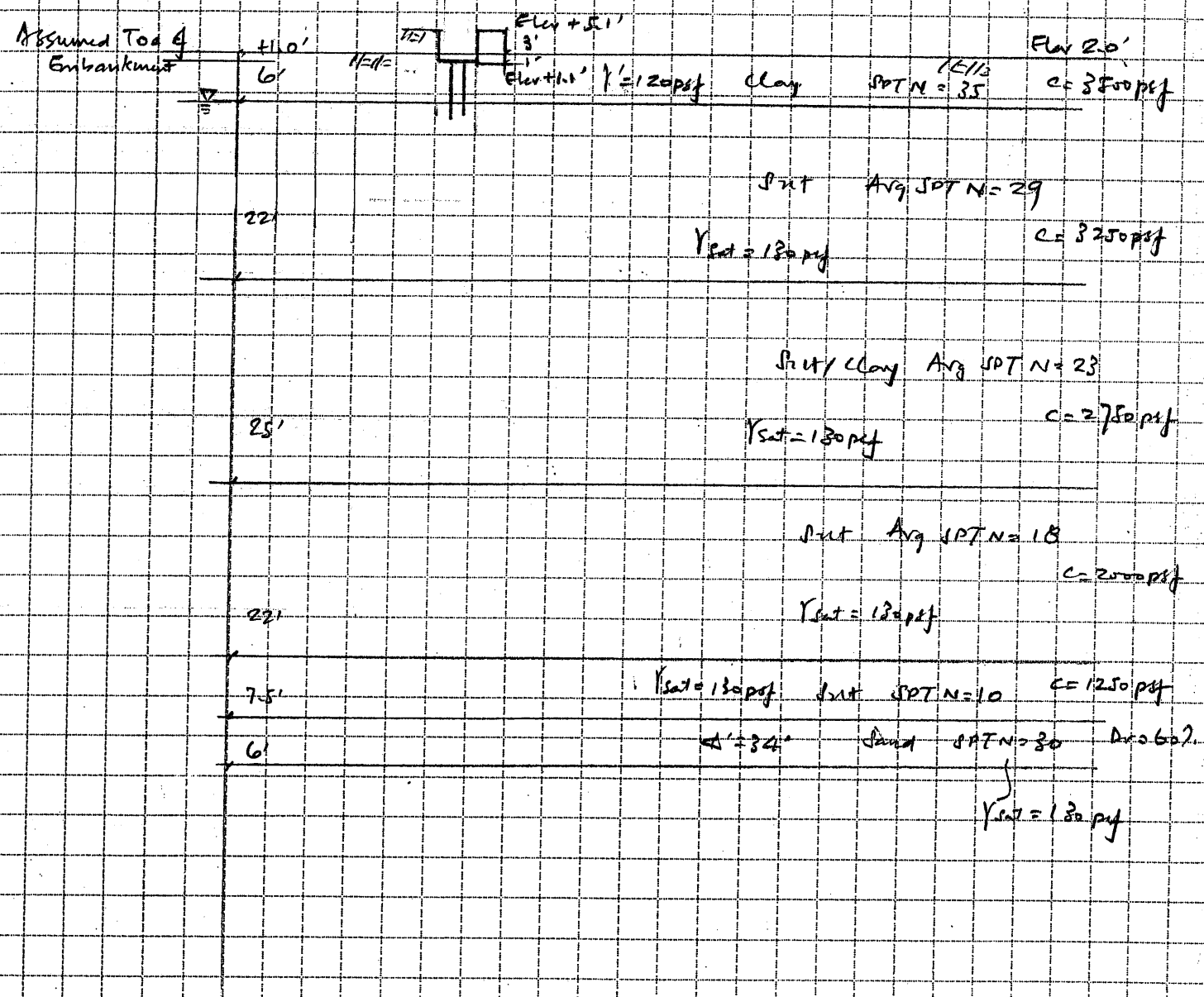
Bent 10 Pile cut-off elevation is +5.5 feet. Finished grade elevation is +6.5 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 250 kips /(0.7*0.71) ≈ 510 kips (255 tons). Extreme Limit Comp): 200 kips/(1.0*0.71) ≈ 290 kips (145 tons). Extreme Limit (Tens): 20kips/(1.0*0.71) ≈ 30 kips (15.0 tons)
Pile Length @ Bent 10 = 61.0' -1.0' = 60.0' (Strength Limit Comp). Pile Length = 43.0' - 1.0' = 42/0' (Extreme Lmit Comp). Pile Length = 17.5' - 1.0' = 16.5' (Extreme Limit Tension)
Pile tip at 60.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -54.5'(Strength Comp),-36.5' (Extreme Comp),-11.0'(Extreme Tens), pile cut-off @ Elev. +5.5')Ult. Pile Capacity=268.7 tons (Friction)> 510k/255 tons, OK

Sheet 22 of 27

Pile Capacity Analyses - Calaveras River Bridge (Widen) (Boring B-5, Bent 1)

Class 200 HX 14" □ precast concrete pile

Nominal Resistance: 380 kips



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Calaveras River Bridge (Widen)*** ***Bent 11-380 k(190 tons)Strength Limit Stat*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/16/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.17 — 1- GRAVELS, SANDS AND NONPLASTIC SILTS — (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 2 (1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 1 (1=CONCRETE ,2=STEEL) — SOIL TYPES — GRAVEL, SAND, NSILT Ref Boring : Boring B-5 — PERIMETER OF THE PILE (ft) : 4.50 SQ. FT.

DRIVEN/DRILLED = 1 (1=DRIVEN ,2=DRILLED) — CLAY, PSILT — AREA OF THE PILE TIP 1.35 SQ. FT.

Class 200 Alt. "X" - 14" square precast prestressed c

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	4.0	BT2 FTG	1	125	500	250	SAND		0		0.00	0.00	0.00	0.00	0.00
4.0	9.0		2	120	600	800	CLAY	V.STIFF	3500	0	0.25	9.84	9.84	9.84	9.84
9.0	14.0		2	68	340	1270	CLAY	V.STIFF	3250	0	0.25	9.14	18.98	9.14	18.98
14.0	19.0		2	68	340	1610	CLAY	V.STIFF	3250	0	0.30	10.97	29.95	10.97	29.95
19.0	25.0		2	68	408	1984	CLAY	V.STIFF	3250	0	0.35	15.36	45.31	15.36	45.31
25.0	31.0		2	68	408	2392	CLAY	V.STIFF	3250	0	0.35	15.36	60.67	15.36	60.67
31.0	36.0		2	68	340	2358	CLAY	V.STIFF	2750	0	0.40	12.38	73.04	12.38	73.04
36.0	41.0		2	68	340	3106	CLAY	V.STIFF	2750	0	0.40	12.38	85.42	12.38	85.42
41.0	46.0		2	68	340	3446	CLAY	V.STIFF	2750	0	0.40	12.38	97.79	12.38	97.79
46.0	51.0		2	68	340	3786	CLAY	V.STIFF	2750	0	0.40	12.38	110.17	12.38	110.17
51.0	56.0		2	68	340	3786	CLAY	V.STIFF	2750	0	0.40	12.38	122.54	12.38	122.54
56.0	61.0		2	68	340	4466	CLAY	STIFF	2000	0	0.85	19.13	141.67	19.13	141.67
61.0	66.0		2	68	340	4806	CLAY	STIFF	2000	0	0.85	19.13	160.79	19.13	160.79
66.0	76.0		2	68	680	5316	CLAY	STIFF	2000	0	0.85	38.25	199.04	38.25	199.04
76.0	78.0		2	68	136	5724	CLAY	STIFF	2000	0	0.85	7.65	206.69	7.65	206.69
78.0	82.0		2	68	272	5928	CLAY	STIFF	1250	0	1.00	11.25	217.94	11.25	217.94
82.0	85.5		2	68	238	4755	CLAY	STIFF	1250	0	1.00	9.84	227.78	9.84	227.78
85.5	91.5		1	68	408	5180	SAND	DENSE	0	41	1.00	0.00	227.78	0.00	227.78

Bent 11 Pile cut-off elevation is +1.10 feet. Assumed finished grade elevation is +5.10 feet

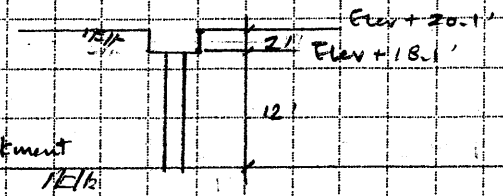
Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +1.0 feet

260 kips/pile (Strength Limit State). Required pile capacity (comp) = 260 kips /0.7 = 380 kips (190 tons).260 kips/pile (Extreme Limit State), Required pile capacity (tens) = 90 kips (45 tons)

Pile Length @ Bent 11 = 76.0' - 4.0' = 72.0 '

Pile tip at 72.0' (comp) & 25' (tens) below pile cut-off (pile tip @ Elev. -71.0'(comp)& -24.0'(tens), pile cut-off @ Elev. +1.1'), Ult. Pile Capacity =199.0 T(Friction)> required pile capacity (380 k/190 tons), OK

Pile Capacity Analysis - Calaveras River Bridge (CPT-09-001) Abutment 12
14" □ precast concrete pile - 80 tons service load



Assumed Top of Embankment Elev. +6.0'

16'	13.5'	Clay	c = 950 psf, γ' = 115 pcf
	6.5'	Clay	c = 850 psf, γ' = 105 pcf, γsat = 120 pcf
	13.5'	Clay	c = 650 psf, γsat = 125 pcf
	2.5'	Sand	φ' = 30°, γsat = 125 pcf
	3'	Clay	c = 2500 psf, γsat = 130 pcf
	4.5'	Clay	c = 3250 psf, γsat = 130 pcf
	5'	Clay	c = 2000 psf, γsat = 130 pcf
	5'	Clay	c = 4000 psf, γsat = 130 pcf
	6.5'	Clay	c = 2000 psf, γsat = 130 pcf
	2.5'	Clay	c = 1000 psf, γsat = 130 pcf
	9.5'	Clay	c = 3500 psf, γsat = 130 pcf
	6'	Sand	φ' = 33.5°, γsat = 130 pcf
	12'	Clay	c = 4000 psf, γsat = 130 pcf
	3'	Sand	φ' = 33.5°, γsat = 130 pcf
	7'	Clay	c = 4000 psf, γsat = 130 pcf

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Calaveras River Bridge (Widen)*** ***Abut 12 - 80 tons service load*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/16/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.17 — 1- GRAVELS, SANDS AND NONPLASTIC SILTS — (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 2 (1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 1 (1=CONCRETE ,2=STEEL) — SOIL TYPES — PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

DRIVEN/DRILLED = 1 (1=DRIVEN ,2=DRILLED) — GRAVEL, SAND, NSILT Ref Boring : CPT-09-001 (PARIKH 2009) — AREA OF THE PILE TIP 1.35 SQ. FT.

CLAY, PSILT

Class 200 Alt. "X" - 14" square precast prestressed

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	BT2 FTG	1	125	250	125	SAND		0		0.00	0.00	0.00	0.00	0.00
2.0	14.0		1	125	1500	1000	SAND	DENSE	0	38	0.00	38.91	38.91	13.04	13.04
14.0	18.5		2	115	517.5	2009	CLAY	MEDIUM	950	0	0.90	8.66	47.57	8.66	21.69
18.0	23.0		2	115	575	2555	CLAY	MEDIUM	950	0	0.90	9.62	57.19	9.62	31.31
23.0	27.5		2	115	517.5	3101	CLAY	MEDIUM	950	0	0.95	9.14	66.32	9.14	40.45
27.5	30.0		2	105	262.5	3491	CLAY	SOFT	350	0	1.00	1.97	68.29	1.97	42.42
30.0	34.0		2	58	232	3476	CLAY	SOFT	350	0	1.00	3.15	71.44	3.15	45.57
34.0	38.5		2	63	283.5	3996	CLAY	MEDIUM	650	0	1.00	6.58	78.02	6.58	52.15
38.5	43.0		2	63	283.5	4280	CLAY	MEDIUM	650	0	1.00	6.58	84.61	6.58	58.73
43.0	47.5		2	63	283.5	4563	CLAY	MEDIUM	650	0	1.00	6.58	91.19	0.00	58.73
47.5	50.0		1	63	157.5	4500	SAND	LOOSE	0	32	0.00	13.33	104.52	4.47	63.20
50.0	53.0		2	68	204	4965	CLAY	V.STIFF	2500	0	0.65	10.97	115.49	10.97	74.17
53.0	57.5		2	68	306	5220	CLAY	V.STIFF	3250	0	0.25	8.23	123.72 ←	15.19	89.35
57.5	62.5		2	68	340	5543	CLAY	STIFF	2000	0	0.65	14.63	138.34	9.84	99.20
62.5	67.5		2	68	340	5883	CLAY	V.STIFF	4000	0	0.30	13.50	151.84	0.00	99.20
67.5	74.0		2	68	442	6274	CLAY	STIFF	2000	0	0.75	21.94	173.78	23.40	122.60
74.0	76.5		2	68	170	5152	CLAY	MEDIUM	1000	0	0.75	8.44	182.22	8.44	131.03
76.5	81.5		2	68	340	5543	CLAY	V.STIFF	3500	0	0.95	10.69	192.90	9.84	140.88
81.5	86.0		2	68	306	6818	CLAY	V.STIFF	3500	0	0.25	8.86	201.76	8.86	149.74
86.0	92.0		1	68	408	7209	SAND	LOOSE	0	32	0.25	0.00	201.76	0.00	149.74
92.0	98.0		2	68	408	7515	CLAY	V.STIFF	4000	0	0.00	0.00	201.76	0.00	149.74
98.0	104.0		2	68	408	7923	CLAY	V.STIFF	4000	0	0.40	21.60	223.36	21.60	171.34
104.0	107.0		1	68	204	8229	SAND	MED DEN	0	33	0.40	0.00	223.36	0.00	171.34
107.0	114.0		2	68	476	8773	CLAY	V.STIFF	4000	0	0.00	0.00	223.36	0.00	171.34

Abut 12 Pile cut-off elevation is +20.10 feet. Assumed finished grade elevation is +18.10 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +6.0 feet

80 kips/pile (Service). Required pile capacity = 80 kips /0.5 = 160 kips (80 tons)

Pile Length @ Abutment 12 = 57.5' - 2.0' = 55.5'

Pile tip at approxiamtely 55.5' below pile cut-off (pile tip @ Elev. -37.5', pile cut-off @ Elev. +18.1'), available Ult. Pile Capacity =123.7 T(Friction)-38.9 (Predrilled)=84.8 tons > required capacity (160 k/80 tons), OK

I-5 Calaveras River Abut 12 Class 200 Alt X (4-16-10)

Sheet 26 of 27

PROJECT: Calaveras River Bridge (Widen)
BRIDGE NO: 29-0174 R/L

FOUNDATION DESIGN DATA

FOR EXTERIOR PILES ONLY

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size (ft)		Permissible Settlement (in.).	No. of Piles per Support Per Bridge
					B	L		
Abutment 1	WSD	Class 200 Alt 'X'	19.3	17.3	2.5	46.96	1	7
Bent 2	LRFD	Class 200 Alt 'X'	6.1	2.1	3	46.96	1	8
Bent 3	LRFD	3'-0" Dia CIDH	5.4	4.4	N/A	N/A	1	8
Bent 4	LRFD	3'-0" Dia CIDH	17.2	17.2	N/A	N/A	1	8
Bent 5	LRFD	3'-0" Dia CIDH	10.0	8.4	N/A	N/A	1	8
Bent 5a	LRFD	5'-0" Dia CIDH	10.0	10.0	N/A	N/A	1	2
Bent 6	LRFD	3'-0" Dia CIDH	-5.2	-10.0	N/A	N/A	1	8
Bent 7	LRFD	3'-0" Dia CIDH	-8.5	-13.6	N/A	N/A	1	8
Bent 8	LRFD	3'-0" Dia CIDH	-4.0	-9.0	N/A	N/A	1	8
Bent 9	LRFD	3'-0" Dia CIDH	18.5	18.5	N/A	N/A	1	8
Bent 10	LRFD	3'-0" Dia CIDH	6.5	5.5	N/A	N/A	1	8
Bent 11	LRFD	Class 200 Alt 'X'	5.1	1.1	3	48.02	1	8
Abutment 12	WSD	Class 200 Alt 'X'	20.1	18.1	2.5	48.02	1	7

Notes: Pile cut-off elev for Bents 4 and 9 are approximate FG elevations. Pile cut-off elev for Bents 3 and 10 are 1' below approximate elevations. Pile cut-off elev for Bents 5, 6, 7, and 8 are at the bottom of estimated local scour within the river. Pile cut-off at Abut 1 an reflect the bottom of abutment diaphragm elevation. Pile cut-off at Bent 2 and Bent 11 reflect the bottom of footing elevation.

FOUNDATION LOADS

FOR EXTERIOR PILES ONLY

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abutment 1	490	70	340	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	1000	N/A	710	1410	240	0	0	N/A	260	0	-90
Bent 3	930	N/A	660	1360	327	0	0	N/A	210	0	-50
Bent 4	1240	N/A	800	1600	378	0	0	N/A	240	0	-60
Bent 5	1030	N/A	740	1460	361	0	0	N/A	240	0	-30
Bent 5a	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1100	560	0	0
Bent 6	1020	N/A	760	1470	343	0	0	N/A	210	0	0
Bent 7	1040	N/A	770	1490	340	0	0	N/A	210	0	0
Bent 8	1050	N/A	760	1470	342	0	0	N/A	210	0	0
Bent 9	1230	N/A	800	1590	373	0	0	N/A	240	0	-50
Bent 10	980	N/A	710	1450	371	0	0	N/A	200	0	-20
Bent 11	1040	N/A	750	1460	260	0	0	N/A	260	0	-90
Abutment 12	510	80	360	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes: Total Load for Service-I Limit State @ Abutments 1 & 12 = DL + LL (HL93)

Total Load for Extreme Limit State @ Bent 5a = 1.0 DL only (collapse prevention only)

Strength Limit State = Governing LRFD Strength Limit State.

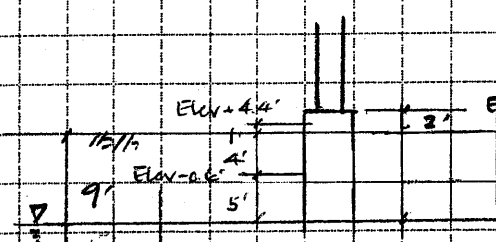
Extreme Limit State = DL + Seismic

LEGEND:

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-2) Bent 3 (Ext)

36" ø CIDH concrete pile

Nominal Resistance 470 kips



Elev. +5.4' (~ O.G.)

Elev. +3.5' Assume

9'		Clayey Silt	Avg SPT N = 7	c = 875 psf, γ' = 115 pcf
6'	γsat = 125 pcf	Clayey Silt	Avg SPT N = 10	c = 1250 psf
5'	γsat = 125 pcf	Clay	Avg SPT N = 13	c = 1500 psf
9'	γsat = 130 pcf	Clayey Silt	Avg SPT N = 15	c = 1750 psf
5'	Dr = 90%	φ' = 36° Gravel	SPT N = 43	γsat = 130 pcf
5'	γsat = 130 pcf	Silt	SPT N = 15	c = 1750 psf
6'	γsat = 130 pcf	Clayey Silt	SPT N = 16	c = 1750 psf
10'	γsat = 130 pcf	Clay	Avg SPT N = 29	c = 2500 psf
6.5'	γsat = 130 pcf	Clay	SPT N = 15	c = 1750 psf
7'	φ' = 33.5°	Clayey Sand	SPT N = 30	γsat = 130 pcf
19.5'	γsat = 130 pcf	Silt	Avg SPT N = 28	c = 3250 psf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 3
206127.CRB

Bent 3 Exterior Piles (Based on Boring B-2)

ALLOWABLE PILE CAPACTY

04/06/10
206127.BR

PILE DIA, B (ft) = 3.00

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

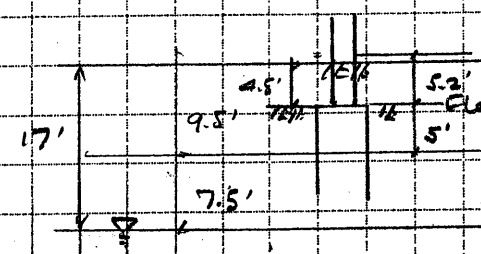
PERIMETER OF THE PILE (ft) = 9.42 SQ. FT.
AREA OF THE PILE TIP 7.07 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	6.0	2	CLAY	115	690	345.0	7	MEDIUM	0.55	875			0.00	0.00	0.00	0.00
6.0	11.0	2	CLAY	115	575	977.5	7	MEDIUM	0.55	875			11.33	11.33	7.93	7.93
11.0	17.0	2	CLAY	63	378	1454.0	10	STIFF	0.55	1250			19.43	30.76	13.60	21.53
17.0	23.0	2	CLAY	63	378	1832.0	13	STIFF	0.55	1625			25.26	56.02	17.68	39.21 ←
23.0	31.0	2	CLAY	68	544	2293.0	15	STIFF	0.55	1875			38.86	94.88	27.20	66.41
31.0	36.0	1	SAND	68	340	2735.0	43	V. DENSE			40	0.72	46.29	141.16	23.14	89.56
36.0	41.0	2	CLAY	68	340	3075.0	15	STIFF	0.55	1875			24.29	165.45 ←	17.00	106.56
41.0	47.0	2	CLAY	68	408	3449.0	16	STIFF	0.55	2000			31.09	196.54	21.76	128.32
47.0	51.0	2	CLAY	68	272	3789.0	29	V.STIFF	0.52	3625			35.43	231.96	24.80	153.12
51.0	55.0	2	CLAY	68	272	4061.0	29	V.STIFF	0.52	3625			35.43	267.39	24.80	177.92
55.0	57.0	2	CLAY	68	136	3993.0	29	V.STIFF	0.52	3625			17.71	285.11	24.80	202.72
57.0	63.5	2	CLAY	68	442	4146.0	15	STIFF	0.55	1875			31.57	316.68	24.80	227.52
63.5	66.5	1	SAND	68	204	4877.0	30	DENSE			36	0.41	28.36	345.04 ←	12.40	239.92
66.5	70.5	1	SAND	68	272	5115.0	30	DENSE			41	0.38	36.88	381.92	22.10	262.02
70.5	90.0	2	CLAY	68	1326	5914.0	28	V.STIFF	0.53	3500			168.77	550.68	14.18	276.20

Bent 3 Pile cut-off elevation is +4.4 feet. Finished grade elevation is +5.4 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 327 kips /(0.7*0.71) ≈ 660 kips (330 tons). Extreme Limit Comp): 210 kips/(1.0*0.71) ≈300 kips (150 tons). Extreme Limit (Tens): 50kips/(1.0*0.71) ≈ 75 kips (37.5 tons)
Pile Length @ Bent 3 = 66.5'- 1.0' = 65.5' (Strength Limit Comp). Pile Length = 41.0'-1.0' = 40.0' (Extreme Lmit Comp). Pile Length = 23.0' - 1.0' = 22.0' (Extreme Limit Tension)
Pile tip at 65.5'(Strength Comp) below pile cut-off (pile tip @ Elev. -61.0'(Strength Comp), -35.5' (Extreme Comp) -17.5'(Extreme Tens), pile cut-off @ Elev. +4.4') Ult. Pile Capacity=345.0 tons(Friction)> 660k/330 tons, OK

Sheet 6 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-1) Bent 4 (Ent
36" ϕ CIDH Concrete pile
Nominal Resistance 540 kips



Elev. +17.2' (MOG), Elev 16.5' Assume

Silt (soft) SPT N=6 c=750 psf
γ'=110 pcf

Clay (very soft to soft) Avg SPT N=5 c=625 psf
γ'=110 pcf

21'

Clay (very soft to soft) Avg SPT N=7
c=875 sf
γsat=130 pcf

10'

Silty Sand (loose to medium dense) Avg SPT N=9
ϕ'=33° γsat=135 pcf

6.5'

Sand (medium dense) SPT N=15 ϕ'=34°
γsat=135 pcf

15'

Silty Clay (stiff to very stiff)
Avg SPT N=18 c=2000 psf
γsat=130 pcf

6'

γsat=125 pcf Clayey Silt (stiff) SPT N=15 c=1500 psf

10

Silt (very stiff) Avg SPT N=33
γsat=130 pcf c=3500 psf

6'

γsat=130 pcf Silt (very stiff) SPT N=30 c=3250 psf

15'

Silty Sand (medium dense) SPT N=18
γsat=130 pcf ϕ'=35°

12.5'

Sand Avg SPT N=43
γsat=135 pcf ϕ'=36°

CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

PILE DIA, B (ft) = 3.00

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

I-5 Calaveras River Bridge (Widen) Bent 4

206127.CRB

Bent 4 Exterior Piles (Based on Boring B-1)

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

4/06/10

ALLOWABLE PILE CAPACTY 206127.BR

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 9.42 SQ. FT.
AREA OF THE PILE TIP 7.07 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	110	550	275.0	6	MEDIUM	0.55	750			0.00	0.00	0.00	0.00
5.0	10.0	2	CLAY	110	550	825.0	6	MEDIUM	0.55	750			9.71	9.71	6.80	6.80
10.0	15.0	2	CLAY	110	550	1375.0	5	MEDIUM	0.55	625			8.10	17.81	5.67	12.47
15.0	17.5	2	CLAY	110	275	1787.5	5	MEDIUM	0.55	625			4.05	21.86	2.83	15.30
17.5	22.5	2	CLAY	68	340	2095.0	7	MEDIUM	0.55	875			11.33	33.19	7.93	23.23
22.5	27.5	2	CLAY	68	340	2435.0	7	MEDIUM	0.55	875			11.33	44.52	7.93	31.17
27.5	32.5	2	CLAY	68	340	2775.0	7	MEDIUM	0.55	875			11.33	55.86	7.93	39.10
32.5	38.5	2	CLAY	68	408	3149.0	7	MEDIUM	0.55	875			13.60	69.46	9.52	48.62 ←
38.5	43.5	1	SAND	73	365	3535.5	9	MED DENSE			33	0.38	31.75	101.21	15.88	64.50
43.5	48.5	1	SAND	73	365	3900.5	9	MED DENSE			33	0.35	32.21	133.42	16.10	80.60
48.5	55.0	1	SAND	73	474.5	4320.3	15	MED DENSE			34	0.53	69.95	203.36 ←	34.97	115.57
55.0	60.0	2	CLAY	68	340	4727.5	18	V.STIFF	0.55	2250			29.14	232.51	20.40	135.97
60.0	68.0	2	CLAY	68	544	5169.5	18	V.STIFF	0.55	2250			46.63	279.14	32.64	168.61
68.0	70.0	2	CLAY	68	136	5509.5	18	V.STIFF	0.55	2250			11.66	290.79	8.16	176.77
70.0	75.0	2	CLAY	68	340	5747.5	15	STIFF	0.55	1875			24.29	315.08	17.00	193.77
75.0	83.0	2	CLAY	68	544	6189.5	33	STIFF	0.49	4125			76.74	391.82 ←	53.72	247.49
83.0	86.0	2	CLAY	68	204	6563.5	33	STIFF	0.49	4125			28.78	420.60	20.15	267.64
86.0	92.0	2	CLAY	68	408	6869.5	30	V.STIFF	0.51	3750			54.31	474.91	38.02	305.66
92.0	97.0	1	SAND	68	340	7243.5	18	MED DENSE			34	0.25	42.65	517.56	21.32	326.98
97.0	102.0	1	SAND	68	340	7583.5	18	MED DENSE			33	0.25	44.65	562.21	22.32	349.31
102.0	107.0	1	SAND	68	340	7923.5	18	MED DENSE			33	0.25	46.65	608.86	23.32	372.63
107.0	112.0	1	SAND	73	365	8276.0	43	DENSE			36	0.25	48.72	657.58	24.36	396.99
112.0	117.0	1	SAND	73	365	8641.0	43	DENSE			36	0.25	50.87	708.46	25.44	422.43
117.0	119.5	1	SAND	73	182.5	8914.8	43	DENSE			36	0.25	26.24	734.70	13.12	435.55

Bent 4 Pile cut-off elevation is +17.2 feet. Finished grade elevation is +17.2 feet

Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71

Strength Limit (comp): 378 kips /(0.7*0.71) ≈ 760 kips (380 tons). Extreme Limit Comp): 240 kips/(1.0*0.71) ≈340 kips (170 tons). Extreme Limit (Tens): 60kips/(1.0*0.71) ≈ 90 kips (45.0 tons)

Pile Length @ Bent 4 = 83.0' (Strength Limit Comp). Pile Length = 55.0' (Extreme Lmit Comp). Pile Length = 38.5' (Extreme Limit Tension)

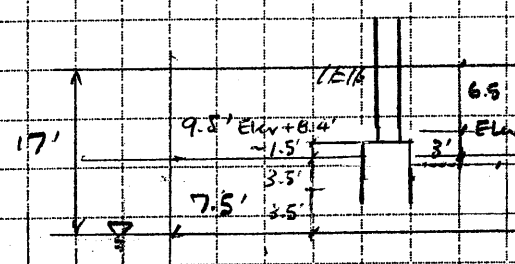
Pile tip at 83.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -66.0'(Strength Comp), -38.0' (Extreme Comp) -21.5'(Extreme Tens), pile cut-off @ Elev. +17.2') Ult. Pile Capacity=391.8 tons (Friction)> 760k/380 tons, OK

Sheet 8 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-1) Bent 5 (ext)

36" ø CIDH Concrete pile

Nominal Resistance = 530 Kips



Elev 16.5' Assume

Silt (soft) SPT N=6 C=750 psf
γ'=110 pcf

Clay (very soft to soft) Avg SPT N=5 c=625 psf
γ'=110 pcf

21' Clay (very soft to soft) Avg SPT N=7
c=875 psf
γsat=125 pcf

10' Silty Sand (loose to medium dense) Avg SPT N=9
φ'=33° γsat=135 pcf

6.5' Sand (medium dense) SPT N=15 φ'=34°
γsat=135 pcf

15' Silty Clay (stiff to very stiff)
Avg SPT N=18 c=2000 psf
γsat=130 pcf

6' γsat=125 pcf Clayey Silt (stiff) SPT N=15 c=1500 psf

10 Silt (very stiff) Avg SPT N=33
γsat=130 pcf c=3500 psf

6' γsat=130 pcf Silt (very stiff) SPT N=30 c=3250 psf

45' Silty Sand (medium dense) SPT N=18
γsat=130 pcf φ'=35°

125' Sand Avg SPT N=43
γsat=135 pcf φ'=36°

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 5
206127.CRB

4/06/10
206127.BR

Bent 5 Exterior Piles (Based on Boring B-1)

PILE DIA, B (ft) = 3.00

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 9.42 SQ. FT.
AREA OF THE PILE TIP 7.07 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	2	CLAY	51	154	77.0	6	MEDIUM	0.55	750			0.00	0.00	0.00	0.00
3.0	6.5	2	CLAY	110	385	346.5	5	MEDIUM	0.55	625			0.00	0.00	0.00	0.00
6.5	10.5	2	CLAY	110	440	759.0	5	MEDIUM	0.55	625			6.48	6.48	4.53	4.53
10.5	15.5	2	CLAY	68	340	1149.0	7	MEDIUM	0.55	875			11.33	17.81	7.93	12.47
15.5	20.5	2	CLAY	68	340	1489.0	7	MEDIUM	0.55	875			11.33	29.14	7.93	20.40
20.5	25.5	2	CLAY	68	340	1829.0	7	MEDIUM	0.55	875			11.33	40.48	7.93	28.33 ←
25.5	31.5	2	CLAY	68	408	2203.0	7	MEDIUM	0.55	875			13.60	54.08	9.52	37.85
31.5	36.5	1	SAND	73	365	2589.5	9	MED DENSE			40	0.43	26.08	80.16	13.04	50.89
36.5	41.5	1	SAND	73	365	2954.5	9	MED DENSE			40	0.39	27.42	107.58	13.71	64.61
41.5	48.0	1	SAND	73	474.5	3374.3	15	MED DENSE			40	0.60	61.66	169.25 ←	30.83	95.44
48.0	54.0	2	CLAY	68	408	3815.5	18	V.STIFF	0.55	2250			34.97	204.22	24.48	119.92
54.0	58.0	2	CLAY	68	272	4155.5	18	V.STIFF	0.55	2250			23.31	227.53	16.32	136.24
58.0	61.0	2	CLAY	68	204	4393.5	18	V.STIFF	0.55	2250			17.49	245.02	12.24	148.48
61.0	69.0	2	CLAY	68	544	4767.5	15	STIFF	0.55	1875			38.86	283.88	27.20	175.68
69.0	72.0	2	CLAY	68	204	5141.5	33	STIFF	0.49	4125			28.78	312.65	20.15	195.82
72.0	79.0	2	CLAY	68	476	5481.5	33	STIFF	0.49	4125			67.15	379.80 ←	47.01	242.83
79.0	85.0	2	CLAY	68	408	5923.5	30	V.STIFF	0.51	3750			54.31	434.12	38.02	280.85
85.0	90.0	1	SAND	68	340	6297.5	18	MED DENSE			34	0.25	37.08	471.19	18.54	299.39
90.0	95.0	1	SAND	68	340	6637.5	18	MED DENSE			34	0.25	39.08	510.27	19.54	318.93
95.0	100.0	1	SAND	68	340	6977.5	18	MED DENSE			34	0.25	41.08	551.35	20.54	339.47
100.0	105.0	1	SAND	68	340	7317.5	43	DENSE			37	0.25	43.08	594.43	21.54	361.01
105.0	110.0	1	SAND	68	340	7657.5	43	DENSE			37	0.25	45.08	639.52	22.54	383.55
110.0	112.5	1	SAND	68	170	7912.5	43	DENSE			37	0.25	23.29	662.81	11.65	395.19

Bent 5 Pile cut-off elevation is +8.4 feet. Finished grade elevation is +10.0 feet - 1.58 feet (scour depth) = 8.42 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 361 kips /(0.7*0.71) ≈ 730 kips (365 tons). Extreme Limit Comp): 240 kips/(1.0*0.71) ≈ 340 kips (170 tons). Extreme Limit (Tens): 30kips/(1.0*0.71) ≈ 50 kips (25.0 tons)
Pile Length @ Bent 5 = 79.0' -1.6' ≈ 77.4' (Strength Limit Comp). Pile Length = 48.0' - 1.6' ≈ 46.4' (Extreme Lmit Comp). Pile Length = 25.5' - 1.6' ≈ 23.9' (Extreme Limit Tension)
Pile tip at 77.4'(Strength Comp) below pile cut-off (pile tip @ Elev. -69.0'(Strength Comp),-38.0' (Extreme Comp),-15.5'(Extreme Tens), pile cut-off @ Elev. +8.4')Ult. Pile Capacity=379.8 tons (Friction)> 730k/365 tons, OK

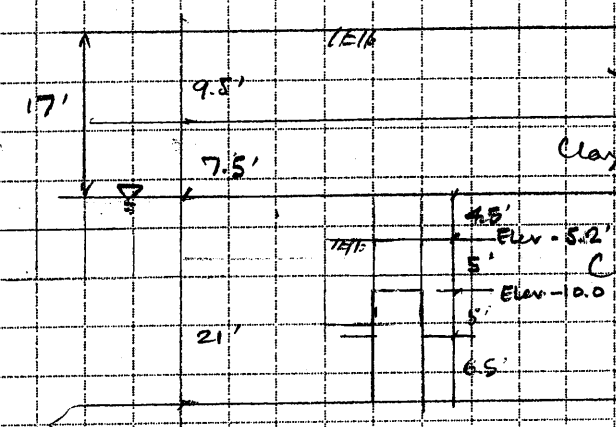
Sheet 10 of 27

Pile capacity Analysis - Calaveras River Bridge (Widen) (Boring B-1) Bent 6 (ext)

36" ϕ CIDH Concrete

Nominal Resistance = 490 kips

Elev 16.5'



Assume

Silt (soft) SPT N = 6 c = 750 psf
γ' = 110 pcf

Clay (very soft to soft) Avg SPT N = 5 c = 625 psf
γ' = 110 pcf

Elev -5.2' (~0.6)

Clay (Very soft to soft) Avg SPT N = 7
c = 875 psf
γsat = 130 pcf

Elev -10.0

Silty sand (loose to medium dense) Avg SPT N = 9
ϕ' = 33° γsat = 135 pcf

Sand (medium dense) SPT N = 15 ϕ' = 34°
γsat = 135 pcf

Silty Clay (stiff to very stiff)
Avg SPT N = 18 c = 2000 psf
γsat = 130 pcf

γsat = 125 pcf Clayey Silt (stiff) SPT N = 15 c = 1500 psf

Silt (very stiff) Avg SPT N = 33
γsat = 130 pcf c = 3500 psf

γsat = 130 pcf Silt (very stiff) SPT N = 30 c = 3250 psf

Silty Sand (medium dense) SPT N = 18
γsat = 130 pcf ϕ' = 35°

Sand Avg SPT N = 43
γsat = 135 pcf ϕ' = 36°

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 6
206127.CRB

4/06/10
206127.GDR

Bent 6 Exterior Piles (Based on Boring B-1)

PILE DIA, B (ft) = 3.00

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 9.42 SQ. FT.
AREA OF THE PILE TIP 7.07 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	3	12.92	6.5	7	MEDIUM	0.55	875			0.00	0.00	0.00	0.00
5.0	10.0	2	CLAY	68	340	182.9	7	MEDIUM	0.55	875			0.00	0.00	0.00	0.00
10.0	16.5	2	CLAY	68	442	573.9	7	MEDIUM	0.55	875			14.73	14.73	10.31	10.31
16.5	21.5	1	SAND	73	365	977.4	9	MED DENSE			34	0.55	12.59	27.32	6.29	16.61
21.5	26.5	1	SAND	73	365	1342.4	9	MED DENSE			34	0.50	15.91	43.23	7.95	24.56
26.5	33.0	1	SAND	73	474.5	1762.2	15	DENSE			36	0.76	41.20	84.43	20.60	45.16
33.0	38.0	2	CLAY	68	340	2169.4	18	V.STIFF	0.55	2250			29.14	113.57	20.40	65.56
38.0	45.0	2	CLAY	68	476	2577.4	18	V.STIFF	0.55	2250			40.80	154.37 ←	28.56	94.12
45.0	48.0	2	CLAY	68	204	2917.4	18	V.STIFF	0.55	2250			17.49	171.86	12.24	106.36
48.0	54.0	2	CLAY	68	408	3223.4	15	STIFF	0.55	1875			29.14	201.00	20.40	126.76
54.0	62.0	2	CLAY	68	544	3699.4	33	STIFF	0.49	4125			76.74	277.75	53.72	180.48
62.0	65.0	2	CLAY	68	204	4073.4	33	STIFF	0.49	4125			28.78	306.52	20.15	200.63
65.0	70.0	2	CLAY	68	340	4345.4	30	V.STIFF	0.51	3750			45.26	351.78	31.68	232.31
70.0	71.0	1	SAND	68	68	4549.4	18	MED DENSE			35	0.37	7.85	359.64 ←	3.93	236.24
71.0	80.0	1	SAND	68	612	4889.4	18	MED DENSE			34	0.33	67.77	427.41	33.88	270.12
80.0	85.0	1	SAND	68	340	5365.4	18	MED DENSE			34	0.27	34.60	462.00	17.30	287.42
85.0	90.0	1	SAND	73	365	5717.9	43	DENSE			38	0.25	33.66	495.67	16.83	304.25
90.0	95.0	1	SAND	73	365	6082.9	43	DENSE			37	0.25	35.81	531.48	17.91	322.16
95.0	97.5	1	SAND	73	182.5	6356.7	43	DENSE			37	0.25	18.71	550.19	9.36	331.51

Bent 6 Pile cut-off elevation is -10.0 feet. Finished grade elevation is -5.2 feet - 4.81 feet (scour depth) = -10.01 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 343 kips /(0.7*0.71) ≈ 690 kips (345 tons). Extreme Limit Comp): 210 kips/(1.0*0.71) ≈ 300 kips (150 tons).
Pile Length @ Bent 6 = 71.0' -5.0' = 66.0' (Strength Limit Comp). Pile Length = 45.0' - 5.0' = 40.0' (Extreme Lmit Comp).
Pile tip at 66.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -76.0' (Strength Comp),-50.0' (Extreme Comp) pile cut-off @ Elev. -10.0')Ult. Pile Capacity=359.6 tons (Friction)> 690k/345 tons, OK

γ' of 3 pcf is the effective unit weight considering the scour [3=68*(5-4.81)/5]

Sheet 14 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widening) (Boring B-7) Bent 7 (Exterior)

36" ⌀ CIDH concrete pile

Nominal Resistance = 490 kips

Elev. 19.0' Assume

GWT

Silt & Clay (Soft) SPT N = 6 c = 750 psf
γ' = 115 pcf

15' 10' 5'

Dr = 60%. φ' = 33° Sand (Loose) SPT N = 10 γ' = 125 pcf

Sand (Very Loose) SPT N = 3
γ' = 125 pcf

10' Dr = 25%. φ' = 30°

GWT 2.5' Elev. -8.5' (~ 0.6%)
5'
13.5' Elev. -13.5'
5
1'

Clayey Silt (Stiff to Very Stiff) Avg SPT N = 21
γsat = 130 pcf c = 2500 psf

12' Clayey Silt (Stiff) Avg SPT N = 15 c = 1875 psf
γsat = 130 pcf

14' Clayey Silt Avg SPT N = 23 c = 2750 psf
γsat = 130 pcf

5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

16.5' Clayey Silt Avg SPT N = 32 c = 3500 psf
γsat = 130 pcf

6.5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

PILE DIA, B (ft) = 3.00

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

I-5 Calaveras River Bridge (Widen) Bent 7

206127.CRB

Bent 7 Exterior Piles (Based on Boring B-7)

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

4/06/10

ALLOWABLE PILE CAPACTY

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 9.42 SQ. FT.
AREA OF THE PILE TIP 7.07 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	0	0	0.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
5.0	11.0	2	CLAY	68	408	204.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
11.0	17.0	2	CLAY	68	408	612.0	15	STIFF	0.55	1875			29.14	29.14	20.40	20.40
17.0	23.0	2	CLAY	68	408	1020.0	15	STIFF	0.55	1875			29.14	58.29	20.40	40.80
23.0	30.0	2	CLAY	68	476	1462.0	23	V.STIFF	0.55	2875			52.13	110.42	36.49	77.29
30.0	37.0	2	CLAY	68	476	1938.0	23	V.STIFF	0.55	2875			52.13	162.55 ←	36.49	113.79
37.0	42.0	2	CLAY	68	340	2346.0	15	STIFF	0.55	1875			24.29	186.84	17.00	130.79
42.0	47.0	2	CLAY	68	340	2686.0	32	V.STIFF	0.50	4000			47.10	233.94	32.97	163.76
47.0	56.0	2	CLAY	68	612	3162.0	32	V.STIFF	0.50	4000			84.78	318.72	59.35	223.10
56.0	58.5	2	CLAY	68	170	3553.0	32	V.STIFF	0.50	4000			23.55	342.27	16.49	239.59
58.5	62.0	2	CLAY	69	241.5	3758.8	15	STIFF	0.55	1875			17.00	359.27 ←	11.90	251.49
62.0	65.0	2	CLAY	69	207	3983.0	15	STIFF	0.55	1875			14.57	373.84	10.20	261.69

Bent 7 Pile cut-off elevation is -13.6 feet. Finished grade elevation is -8.5 feet - 5.1 feet (scour depth) = -13.6 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 340 kips /(0.7*0.71) ≈ 690 kips (345 tons). Extreme Limit Comp): 210 kips/(1.0*0.71) ≈ 300 kips (150 tons).
Pile Length @ Bent 7 = 62.0' -5.0' = 57.0' (Strength Limit Comp). Pile Length = 37.0' - 5.0' = 32.0' (Extreme Lmit Comp).
Pile tip at 57.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -70.5' (Strength Comp),-45.5' (Extreme Comp) pile cut-off @ Elev. -13.6')Ult. Pile Capacity=359.3 tons (Friction)> 690 k/345 tons, OK

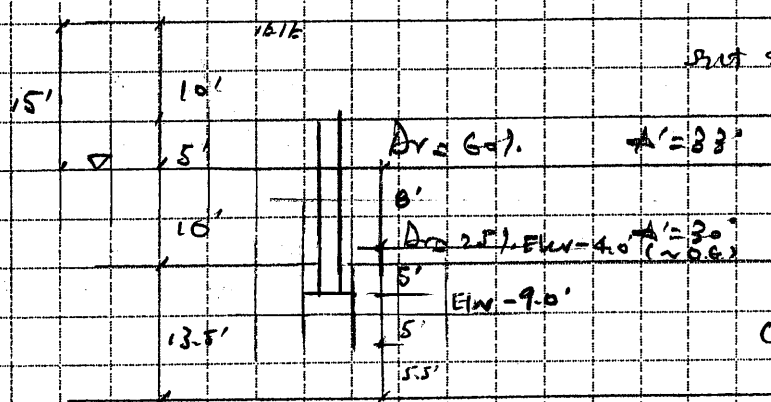
γ' of 0 pcf is the effective unit weight for the top five feet considering the scour from -8.5 feet to -13.6 feet.

Sheet 16 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-7) Bent B (Exterior)

36" ϕ CIDH Concrete pile

Nominal Resistance: 490 kips



Elev. 19.0' Assume

Silt & Clay (soft) SPT N = 6 c = 750 psf
γ' = 115 pcf

φ' = 33° Sand (Loose) SPT N = 10 γ' = 120 pcf

Sand (very loose) SPT N = 3
γ' = 125 pcf

φ' = 30° (~0.6)

Clayey Silt (stiff to very stiff) Avg SPT N = 21
γsat = 130 pcf c = 2500 psf

Clayey Silt (stiff) Avg SPT N = 15 c = 1875 psf
γsat = 130 pcf

Clayey Silt Avg SPT N = 23 c = 2750 psf
γsat = 130 pcf

Clayey Silt SPT N = 15 c = 1875 psf
γsat = 120 pcf

Clayey Silt Avg SPT N = 32 c = 3500 psf
γsat = 130 pcf

Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

I-5 Calaveras River Bridge (Widen) Bent 8

206127.CRB

Bent 8 Exterior Piles (Based on Boring B-7)

ALLOWABLE PILE CAPACTY

4/06/10

206127.BR

PILE DIA, B (ft) = 3.00

SIDE MOBILIZATION

FACTOR (CLAY) = 1

FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

SOIL GROUPS

1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES

GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 9.42 SQ. FT.

AREA OF THE PILE TIP 7.07 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	2	CLAY	0	0	0.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
2.0	5.0	2	CLAY	0	0	0.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
5.0	10.0	2	CLAY	68	340	170.0	21	V.STIFF	0.55	2625			0.00	0.00	0.00	0.00
10.0	15.5	2	CLAY	68	374	527.0	21	V.STIFF	0.55	2625			37.40	37.40	26.18	26.18
15.5	21.5	2	CLAY	68	408	918.0	15	STIFF	0.55	1875			29.14	66.54	20.40	46.58
21.5	27.5	2	CLAY	68	408	1326.0	15	STIFF	0.55	1875			29.14	95.69	20.40	66.98
27.5	35.0	2	CLAY	68	510	1785.0	23	V.STIFF	0.55	2875			55.86	151.54 ←	39.10	106.08
35.0	41.5	2	CLAY	68	442	2261.0	23	V.STIFF	0.55	2875			48.41	199.95	33.89	139.97
41.5	46.5	2	CLAY	68	340	2652.0	15	STIFF	0.55	1875			24.29	224.24	17.00	156.97
46.5	53.5	2	CLAY	68	476	3060.0	32	V.STIFF	0.50	4000			65.94	290.18	46.16	203.13
53.5	61.0	2	CLAY	68	510	3553.0	32	V.STIFF	0.50	4000			70.65	360.83 ←	49.46	252.58
61.0	63.0	2	CLAY	68	136	3876.0	32	V.STIFF	0.50	4000			18.84	379.67	13.19	265.77
63.0	69.5	2	CLAY	68	442	4165.0	15	STIFF	0.55	1875			31.57	411.24	22.10	287.87

Bent 8 Pile cut-off elevation is -9.0 feet. Finished grade elevation is -4.0 feet - 5.04 feet (scour depth) = -9.04 feet

Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71

Strength Limit (comp): 342 kips /(0.7*0.71) ≈ 690 kips (345 tons). Extreme Limit Comp): 210 kips/(1.0*0.71) ≈ 300 kips (150 tons).

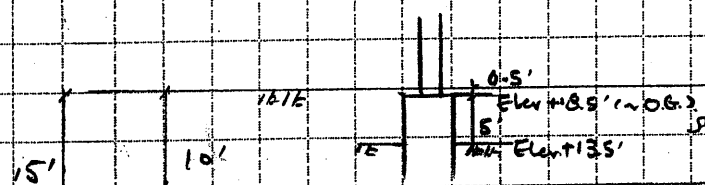
Pile Length @ Bent 7 = 61.0' -5.0' = 56.0' (Strength Limit Comp). Pile Length = 35.0' - 5.0' = 30.0' (Extreme Lmit Comp).

Pile tip at 56.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -65.0' (Strength Comp),-39.0' (Extreme Comp) pile cut-off @ Elev. -9.0')Ult. Pile Capacity=360.8 tons (Friction)> 690k/345 tons, OK

γ' of 0 pcf is the effective unit weight for the top five feet considering the scour from -4.0 feet to -9.0 feet.

Sheet 18 of 27

Pile Capacity Analysis - Calaveras River Bridge (Widen) (Boring B-7) Bent 9 (Exterior)
36" ⌀ CIDH concrete pile
Nominal Resistance = 540 kips



Elev. 19.0' Assume

Bent 9 Clay (soft) SPT N = 6 c = 750 psf
γ' = 115 pcf

Dr = 60%. φ' = 33° Sand (loose) SPT N = 10 γ' = 120 pcf

10' Dr = 25%. φ' = 30° Sand (very loose) SPT N = 3
γ' = 125 pcf

13.5' Clayey Silt (stiff to very stiff) Avg SPT N = 21
γsat = 130 pcf c = 2500 psf

12' Clayey Silt (stiff) Avg SPT N = 15 c = 1875 psf
γsat = 130 pcf

14' Clayey Silt Avg SPT N = 23 c = 2750 psf
γsat = 130 pcf

5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

16.5' Clayey Silt Avg SPT N = 32 c = 3500 psf
γsat = 130 pcf

6.5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

PILE DIA, B (ft) = 3.00

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

I-5 Calaveras River Bridge (Widen) Bent 9
206127.CRB

Bent 9 Exterior Piles (Based on Boring B-7)

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY 4/06/10
206127.BR

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 9.42 SQ. FT.
AREA OF THE PILE TIP 7.07 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	115	575	287.5	6	MEDIUM	0.55	750			0.00	0.00	0.00	0.00
5.0	9.5	2	CLAY	115	517.5	833.8	6	MEDIUM	0.55	750			8.74	8.74	6.12	6.12
9.5	14.5	1	SAND	120	600	1392.5	10	MED DENSE			34	0.69	22.57	31.31	11.28	17.40
14.5	19.5	1	SAND	63	315	1850.0	3	LOOSE			31	0.25	10.89	42.20	5.45	22.85
19.5	24.5	1	SAND	63	315	2165.0	3	LOOSE			40	0.25	12.75	54.95	6.37	29.22
24.5	28.0	2	CLAY	68	238	2441.5	21	V.STIFF	0.55	2625			23.80	78.75	16.66	45.88
28.0	34.5	2	CLAY	68	442	2781.5	21	V.STIFF	0.55	2625			44.20	122.95	30.94	76.82
34.5	38.0	2	CLAY	68	238	3121.5	21	V.STIFF	0.55	2625			23.80	146.75	16.66	93.48
38.0	44.0	2	CLAY	68	408	3444.5	15	STIFF	0.55	1875			29.14	175.89	20.40	113.88
44.0	50.0	2	CLAY	68	408	3852.5	15	STIFF	0.55	1875			29.14	205.04	20.40	134.28
50.0	55.0	2	CLAY	68	340	4226.5	23	V.STIFF	0.55	2875			37.24	242.28	26.07	160.35
55.0	60.0	2	CLAY	68	340	4566.5	23	V.STIFF	0.55	2875			37.24	279.51	26.07	186.42
60.0	64.0	2	CLAY	68	272	4872.5	23	V.STIFF	0.55	2875			29.79	309.31	20.85	207.27
64.0	69.0	2	CLAY	68	340	5178.5	15	STIFF	0.55	1875			24.29	333.59	17.00	224.27
69.0	75.0	2	CLAY	68	408	5552.5	32	V.STIFF	0.50	4000			56.52	390.11	39.56	263.84
75.0	80.0	2	CLAY	68	340	5926.5	32	V.STIFF	0.50	4000			47.10	437.21	32.97	296.81
80.0	85.5	2	CLAY	68	374	6283.5	32	V.STIFF	0.50	4000			51.81	489.02	36.27	333.07
85.5	92.0	2	CLAY	68	442	6691.5	15	STIFF	0.55	1875			31.57	520.59	22.10	355.17

Bent 9 Pile cut-off elevation is +18.5 feet. Finished grade elevation is +18.5 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 373 kips /(0.7*0.71) ≈ 750 kips (375 tons). Extreme Limit Comp): 240 kips/(1.0*0.71) ≈340 kips (170 tons). Extreme Limit (Tens): 50kips/(1.0*0.71) ≈ 75 kips (37.5 tons)
Pile Length @ Bent 9 = 75.0' (Strength Limit Comp). Pile Length = 44.0' (Extreme Lmit Comp). Pile Length = 28.0' (Extreme Limit Tension)
Pile tip at 75.0'(Strength Comp)below pile cut-off(pile tip @ Elev. -56.5'(Strength Comp), -25.5' (Extreme Comp) -9.5'(Extreme Tens), pile cut-off@Elev. +18.5') Ult. Pile Capacity=390.1 tons (Friction)> 750k/375 tons, OK

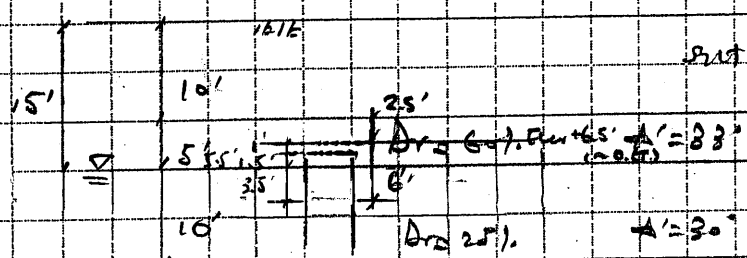
Sheet 20 of 27

Pile Capacity Analysis. - Calaveras River Bridge (Widening) (Boring B-7) Bent 10 (Exterior)

36" ⌀ CIDH Concrete pile

Nominal Resistance: 530 kips

Elev. 19.0' Assume



MTF

Silt & Clay (soft) SPT N = 6 c = 750 psf
γ' = 115 pcf

Sand (Loose) SPT N = 10 γ' = 120 pcf

Sand (very loose) SPT N = 3
γ' = 125 pcf

13.5' Clayey Silt (stiff to very stiff) Avg SPT N = 21
γsat = 130 pcf c = 2500 psf

12' Clayey Silt (stiff) Avg SPT N = 15 c = 1875 psf
γsat = 130 pcf

14' Clayey Silt Avg SPT N = 23 c = 2750 psf
γsat = 130 pcf

5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

16.5' Clayey Silt Avg SPT N = 32 c = 3500 psf
γsat = 130 pcf

6.5' Clayey Silt SPT N = 15 c = 1875 psf
γsat = 130 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

PILE DIA, B (ft) = 3.00

SIDE MOBILIZATION
FACTOR (CLAY) = 1
FACTOR (SAND) = 1

INCREAMENTS (FT)= 5

I-5 Calaveras River Bridge (Widen) Bent 10

206127.CRB

Bent 10 Exterior Piles (Based on Boring B-7)

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

ALLOWABLE PILE CAPACTY 4/06/10
206127.BR

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 9.42 SQ. FT.
AREA OF THE PILE TIP 7.07 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.5	1	SAND	120	300	150.0	10	DENSE			36	0.90	0.00	0.00	0.00	0.00
2.5	6.0	1	SAND	63	220.5	410.3	3	LOOSE			32	0.25	0.00	0.00	0.00	0.00
6.0	9.0	1	SAND	63	189	615.0	3	LOOSE			32	0.25	2.17	2.17	1.09	1.09
9.0	12.5	1	SAND	63	220.5	819.8	3	LOOSE			31	0.25	3.38	5.55	1.69	2.78
12.5	16.5	2	CLAY	68	272	1066.0	21	V.STIFF	0.55	2625			27.20	32.75	19.04	21.82 ←
16.5	22.5	2	CLAY	68	408	1406.0	21	V.STIFF	0.55	2625			40.80	73.55	28.56	50.38
22.5	26.0	2	CLAY	68	238	1729.0	21	V.STIFF	0.55	2625			23.80	97.35	16.66	67.04
26.0	32.0	2	CLAY	68	408	2052.0	15	STIFF	0.55	1875			29.14	126.49	20.40	87.44
32.0	38.0	2	CLAY	68	408	2460.0	15	STIFF	0.55	1875			29.14	155.64 ←	20.40	107.84
38.0	43.0	2	CLAY	68	340	2834.0	23	V.STIFF	0.55	2875			37.24	192.88	26.07	133.90
43.0	48.0	2	CLAY	68	340	3174.0	23	V.STIFF	0.55	2875			37.24	230.11	26.07	159.97
48.0	52.0	2	CLAY	68	272	3480.0	23	V.STIFF	0.55	2875			29.79	259.91	20.85	180.82
52.0	57.0	2	CLAY	68	340	3786.0	15	STIFF	0.55	1875			24.29	284.19	17.00	197.82
57.0	61.0	2	CLAY	68	272	4092.0	32	V.STIFF	0.50	4000			37.68	321.87	26.38	224.20
61.0	68.0	2	CLAY	68	476	4466.0	32	V.STIFF	0.50	4000			65.94	387.81 ←	46.16	270.36
68.0	73.5	2	CLAY	68	374	4891.0	32	V.STIFF	0.50	4000			51.81	439.62	36.27	306.62
73.5	80.0	2	CLAY	68	442	5299.0	15	STIFF	0.55	1875			31.57	471.19	22.10	328.73

Bent 10 Pile cut-off elevation is +5.5 feet. Finished grade elevation is +6.5 feet
Pile spacing at 5.5 feet (or 1.83 pile diameter) center-to-center. Vertical pile group efficiency is 0.71
Strength Limit (comp): 371 kips /(0.7*0.71) ≈ 750 kips (375 tons). Extreme Limit Comp): 200 kips/(1.0*0.71) ≈ 290 kips (145 tons). Extreme Limit (Tens): 20kips/(1.0*0.71) ≈ 30 kips (15.0 tons)
Pile Length @ Bent 10 = 68.0' -1.0' = 67.0' (Strength Limit Comp). Pile Length = 38.0' - 1.0' = 37.0' (Extreme Lmit Comp). Pile Length = 16.5' - 1.0' = 15.5' (Extreme Limit Tension)
Pile tip at 67.0'(Strength Comp) below pile cut-off (pile tip @ Elev. -61.5'(Strength Comp),-31.5' (Extreme Comp),-10.0'(Extreme Tens), pile cut-off @ Elev. +5.5')Ult. Pile Capacity=387.8 tons (Friction)> 750k/375 tons, OK

Sheet 22 of 27

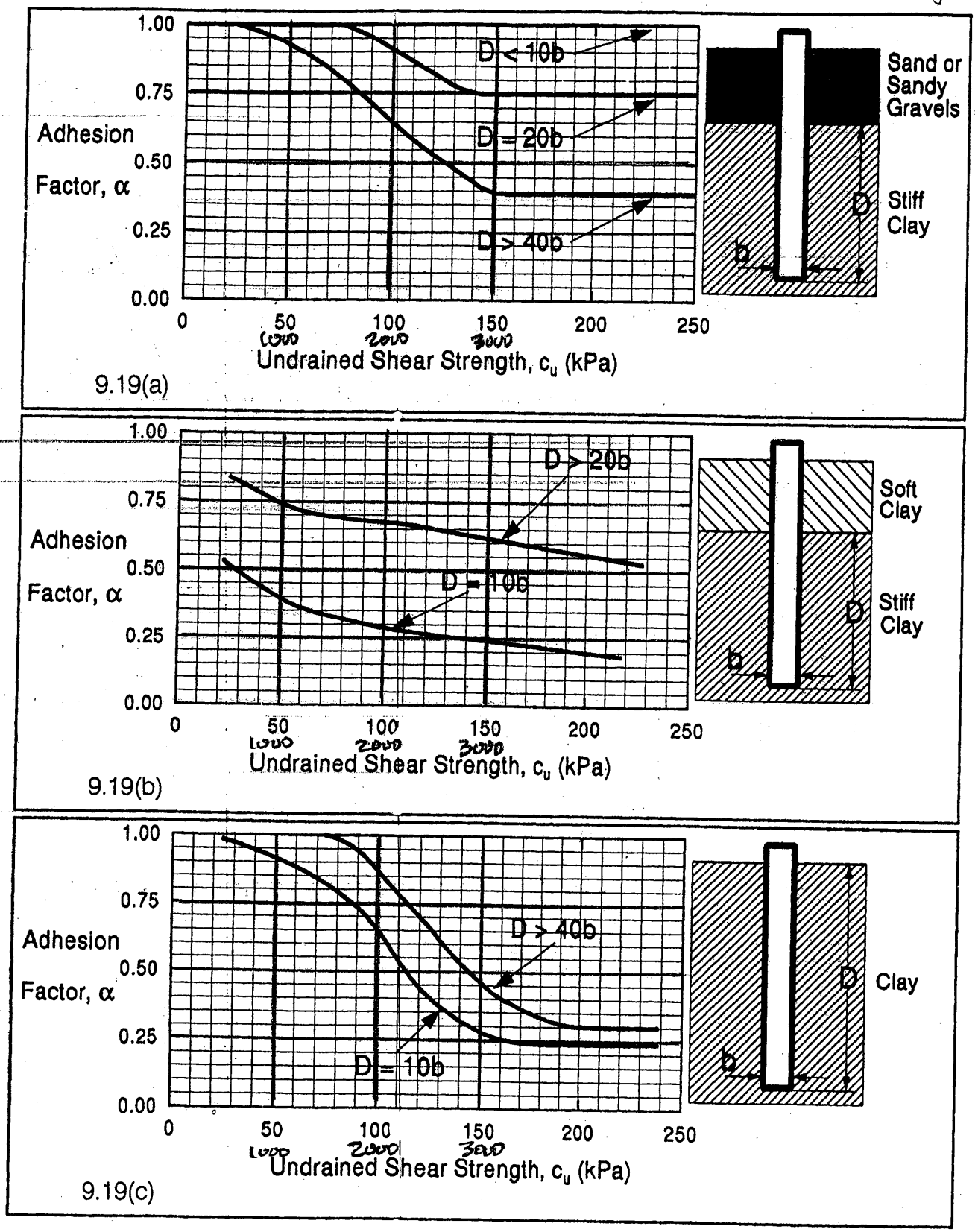


Figure 9.19 Adhesion Factors for Driven Piles in Clay (after Tomlinson, 1980)

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

Boring R-09-CR-001 (Abutment 1) (14-square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
5.0* to -3.5	0 to 8.5	Clay (stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.067	-
-3.5 to -7.0	8.5 to 12	Clay (very stiff)	$\phi = 0°$; c=3500 psf	3, Clay	500	0.007	0.069	-
-7.0 to -13.5	12 to 18.5	Clay (very stiff)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 12 feet deep below existing ground surface for design.
-13.5 to -28.0	18.5 to 33	Clay (stiff to very stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.039	-
-28.0 to -33.5	33 to 38.5	Clay (firm)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.036	-
-33.5 to -43.5	38.5 to 48.5	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-43.5 to -48.0	48.5 to 53	Clay (firm)	$\phi = 0°$; c=1000 psf	3, Clay	500	0.007	0.036	-
-48.0 to -53.0	53 to 58	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-53.0 to -61.0	58 to 66	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.036	-
-61.0 to -71.0	66 to 76	Clay (very stiff)	$\phi = 0°$; c=3000 psf	3, Clay	1000	0.005	0.039	-
-71.0 to -81.0	76 to 86	Clay (firm to stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.036	-
-81.0 to -88.5	86 to 93.5	Clay (soft)	$\phi = 0°$; c=875 psf	3, Clay	100	0.01	0.036	-
-88.5 to -100.0	93.5 to 105	Sand (medium dense)	$\phi = 30°$; c= 0 psf	4, Sand	60	-	0.039	-
-100.0 to -106.5	105 to 111.5	Sand (loose)	$\phi = 28°$; c= 0 psf	4, Sand	20	-	0.039	-

*Finish ground elevation of +19.30 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +5.00 feet to +19.30 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-2 (8-1963)(Bent 2) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
3.5* to -5.5	0 to 9	Clay (soft to firm)	$\phi = 0°$; c=875 psf	3, Clay	100	0.01	0.067	-
-5.5 to -11.5	9 to 15	Clay (firm to stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.036	Assumed GWL at 9 feet deep below existing ground surface for design.
-11.5 to -16.5	15 to 20	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.036	-
-16.5 to -25.5	20 to 29	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-25.5 to -30.5	29 to 34	Sand (Dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-
-30.5 to -35.5	34 to 39	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-35.5 to -41.5	39 to 45	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-41.5 to -51.5	45 to 55	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-51.5 to -58.0	55 to 61.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-58.0 to -65.0	61.5 to 68.5	Sand (medium dense)	$\phi = 33{,}5°$; c=0 psf	4, Sand	60	-	0.039	-
-65.0 to -84.5	68.5 to 88	Clay (very stiff)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +6.10 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +3.50 feet to +6.10 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-2 (8-1963)(Bent 3) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
3.5* to -5.5	0 to 9	Clay (soft to firm)	$\phi = 0°$; c=875 psf	3, Clay	100	0.01	0.067	-
-5.5 to -11.5	9 to 15	Clay (firm to stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.036	Assumed GWL at 9 feet deep below existing ground surface for design.
-11.5 to -16.5	15 to 20	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.036	-
-16.5 to -25.5	20 to 29	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-25.5 to -30.5	29 to 34	Sand (Dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-
-30.5 to -35.5	34 to 39	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-35.5 to -41.5	39 to 45	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-41.5 to -51.5	45 to 55	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-51.5 to -58.0	55 to 61.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-58.0 to -65.0	61.5 to 68.5	Sand (medium dense)	$\phi = 33{,}5°$; c=0 psf	4, Sand	60	-	0.039	-
-65.0 to -84.5	68.5 to 88	Clay (very stiff)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +5.40 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +3.50 feet to +5.40 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-1 (8-1963)(Bent 4) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
16.5* to 7.0	0 to 9.5	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.064	-
7.0 to -0.5	9.5 to 17	Clay (soft)	$\phi = 0°$; c=500 psf	3, Clay	100	0.01	0.064	-
-0.5 to -21.5	17 to 38	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.036	Assumed GWL at 17 feet deep below existing ground surface for design.
-21.5 to -31.5	38 to 48	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.036	-
-31.5 to -38.0	48 to 54.5	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-38.0 to -53.0	54.5 to 69.5	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.039	-
-53.0 to -59.0	69.5 to 75.5	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.039	-
-59.0 to -69.0	75.5 to 85.5	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-69.0 to -75.0	85.5 to 91.5	Clay (very stiff to hard)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-75.0 to -90.0	91.5 to 106.5	Sand (medium dense)	$\phi = 33°$; c=0 psf	4, Sand	60	-	0.039	-
-90.0 to -102.5	106.5 to 119	Sand (dense)	$\phi = 36°$; c=0 psf	4, Sand	125	-	0.042	-

* Finish ground elevation of +17.20 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +16.50 feet to +17.20 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-1 (8-1963)(Bent 5) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
16.5* to 7.0	0 to 9.5	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.064	-
7.0 to -0.5	9.5 to 17	Clay (soft)	$\phi = 0°$; c=500 psf	3, Clay	100	0.01	0.064	-
-0.5 to -21.5	17 to 38	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.036	Assumed GWL at 17 feet deep below existing ground surface for design.
-21.5 to -31.5	38 to 48	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.036	-
-31.5 to -38.0	48 to 54.5	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-38.0 to -53.0	54.5 to 69.5	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.039	-
-53.0 to -59.0	69.5 to 75.5	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.039	-
-59.0 to -69.0	75.5 to 85.5	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-69.0 to -75.0	85.5 to 91.5	Clay (very stiff to hard)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-75.0 to -90.0	91.5 to 106.5	Sand (medium dense)	$\phi = 33°$; c=0 psf	4, Sand	60	-	0.039	-
-90.0 to -102.5	106.5 to 119	Sand (dense)	$\phi = 36°$; c=0 psf	4, Sand	125	-	0.042	-

* Finish ground elevation of +10.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-1 (8-1963)(Bent 5a) [60-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
16.5* to 7.0	0 to 9.5	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.064	-
7.0 to -0.5	9.5 to 17	Clay (soft)	$\phi = 0°$; c=500 psf	3, Clay	100	0.01	0.064	-
-0.5 to -21.5	17 to 38	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.036	Assumed GWL at 17 feet deep below existing ground surface for design.
-21.5 to -31.5	38 to 48	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.036	-
-31.5 to -38.0	48 to 54.5	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-38.0 to -53.0	54.5 to 69.5	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.039	-
-53.0 to -59.0	69.5 to 75.5	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.039	-
-59.0 to -69.0	75.5 to 85.5	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-69.0 to -75.0	85.5 to 91.5	Clay (very stiff to hard)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-75.0 to -90.0	91.5 to 106.5	Sand (medium dense)	$\phi = 33°$; c=0 psf	4, Sand	60	-	0.039	-
-90.0 to -102.5	106.5 to 119	Sand (dense)	$\phi = 36°$; c=0 psf	4, Sand	125	-	0.042	-

* Finish ground elevation of +10.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-1 (8-1963)(Bent 6) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
16.5* to 7.0	0 to 9.5	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.064	-
7.0 to -0.5	9.5 to 17	Clay (soft)	$\phi = 0°$; c=500 psf	3, Clay	100	0.01	0.064	-
-0.5 to -21.5	17 to 38	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.036	Assumed GWL at 17 feet deep below existing ground surface for design.
-21.5 to -31.5	38 to 48	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.036	-
-31.5 to -38.0	48 to 54.5	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-38.0 to -53.0	54.5 to 69.5	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.039	-
-53.0 to -59.0	69.5 to 75.5	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.039	-
-59.0 to -69.0	75.5 to 85.5	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-69.0 to -75.0	85.5 to 91.5	Clay (very stiff to hard)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-75.0 to -90.0	91.5 to 106.5	Sand (medium dense)	$\phi = 33°$; c=0 psf	4, Sand	60	-	0.039	-
-90.0 to -102.5	106.5 to 119	Sand (dense)	$\phi = 36°$; c=0 psf	4, Sand	125	-	0.042	-

* Finish ground elevation of -5.20 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-7 (1963)(Bent 7) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
19.0* to 9.0	0 to 10	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.067	-
9.0 to 4.0	10 to 15	Sand (loose)	$\phi = 33°$; c= 0 psf	4, Sand	25	-	0.069	-
4.0 to -6.0	15 to 25	Sand (loose)	$\phi = 30°$; c= 0 psf	4, Sand	20	-	0.036	Assumed GWL at 15 feet deep below existing ground surface for design.
-6.0 to -19.5	25 to 38.5	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-19.5 to -31.5	38.5 to 50.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-
-31.5 to -45.5	50.5 to 64.5	Clay (very stiff)	$\phi = 0°$; c=2750 psf	3, Clay	1000	0.005	0.039	-
-45.5 to -50.5	64.5 to 69.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-
-50.5 to -67.0	69.5 to 86	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-67.0 to -73.5	86 to 92.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-

* Finish ground elevation of -8.50 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-7 (1963)(Bent 8) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
19.0* to 9.0	0 to 10	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.067	-
9.0 to 4.0	10 to 15	Sand (loose)	$\phi = 33°$; c= 0 psf	4, Sand	25	-	0.069	-
4.0 to -6.0	15 to 25	Sand (loose)	$\phi = 30°$; c= 0 psf	4, Sand	20	-	0.036	Assumed GWL at 15 feet deep below existing ground surface for design.
-6.0 to -19.5	25 to 38.5	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-19.5 to -31.5	38.5 to 50.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-
-31.5 to -45.5	50.5 to 64.5	Clay (very stiff)	$\phi = 0°$; c=2750 psf	3, Clay	1000	0.005	0.039	-
-45.5 to -50.5	64.5 to 69.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-
-50.5 to -67.0	69.5 to 86	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-67.0 to -73.5	86 to 92.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-

* Finish ground elevation of -4.0 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-7 (1963)(Bent 9) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
19.0* to 9.0	0 to 10	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.067	-
9.0 to 4.0	10 to 15	Sand (loose)	$\phi = 33°$; c= 0 psf	4, Sand	25	-	0.069	-
4.0 to -6.0	15 to 25	Sand (loose)	$\phi = 30°$; c= 0 psf	4, Sand	20	-	0.036	Assumed GWL at 15 feet deep below existing ground surface for design.
-6.0 to -19.5	25 to 38.5	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-19.5 to -31.5	38.5 to 50.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-
-31.5 to -45.5	50.5 to 64.5	Clay (very stiff)	$\phi = 0°$; c=2750 psf	3, Clay	1000	0.005	0.039	-
-45.5 to -50.5	64.5 to 69.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-
-50.5 to -67.0	69.5 to 86	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-67.0 to -73.5	86 to 92.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-

*Finish ground elevation of +18.50 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-7 (1963)(Bent 10) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **6-7-10**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
19.0* to 9.0	0 to 10	Clay (soft)	$\phi = 0°$; c=750 psf	3, Clay	100	0.01	0.067	-
9.0 to 4.0	10 to 15	Sand (loose)	$\phi = 33°$; c= 0 psf	4, Sand	25	-	0.069	-
4.0 to -6.0	15 to 25	Sand (loose)	$\phi = 30°$; c= 0 psf	4, Sand	20	-	0.036	Assumed GWL at 15 feet deep below existing ground surface for design.
-6.0 to -19.5	25 to 38.5	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-19.5 to -31.5	38.5 to 50.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-
-31.5 to -45.5	50.5 to 64.5	Clay (very stiff)	$\phi = 0°$; c=2750 psf	3, Clay	1000	0.005	0.039	-
-45.5 to -50.5	64.5 to 69.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-
-50.5 to -67.0	69.5 to 86	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-67.0 to -73.5	86 to 92.5	Clay (stiff to very stiff)	$\phi = 0°$; c=1875 psf	3, Clay	500	0.007	0.039	-

*Finish ground elevation of +6.50 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.66 in the longitudinal direction and 0.44 in the transverse direction should be adopted in the lateral pile analysis for pile spacing of 1.83 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

As-built Boring B-5 (1963)(Bent 11) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
2.0* to -4.0	0 to 6	Clay (very stiff to hard)	$\phi = 0°$; c= 3500 psf	3, Clay	1000	0.005	0.069	-
-4.0 to -26.0	6 to 28	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 6 feet deep below existing ground surface for design.
-26.0 to -51.0	28 to 53	Clay (very stiff)	$\phi = 0°$; c= 2750 psf	3, Clay	1000	0.005	0.039	-
-51.0 to -73.0	53 to 75	Clay (very stiff)	$\phi = 0°$; c= 2000 psf	3, Clay	500	0.007	0.039	-
-73.0 to -80.5	75 to 82.5	Clay (firm to stiff)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.039	-
-80.5 to -86.5	82.5 to 88.5	Sand (medium dense to dense)	$\phi = 34°$; c= 0 psf	4, Sand	125	-	0.039	-

*Finish ground elevation of +5.10 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +2.00 feet to +5.10 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Calaveras River Bridge (Widen)

CPT-09-CR-001 (Abutment 12) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
6.0* to -7.5	0 to 13.5	Clay (stiff)	$\phi = 0°$; c= 950 psf	3, Clay	100	0.01	0.067	-
-7.5 to -10.0	13.5 to 16	Clay (soft)	$\phi = 0°$; c= 350 psf	3, Clay	100	0.01	0.061	-
-10.0 to -14.0	16 to 20	Clay (soft)	$\phi = 0°$; c= 350 psf	3, Clay	100	0.01	0.033	Assumed GWL at 16 feet deep below existing ground surface for design.
-14.0 to -27.5	20 to 33.5	Clay (soft to firm)	$\phi = 0°$; c= 650 psf	3, Clay	100	0.01	0.036	-
-27.5 to -30.0	33.5 to 36	Sand (loose)	$\phi = 30°$; c= 0 psf	4, Sand	20	-	0.036	-
-30.0 to -33.0	36 to 39	Clay (very stiff)	$\phi = 0°$; c= 2500 psf	3, Clay	1000	0.005	0.039	-
-33.0 to -37.5	39 to 43.5	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	-
-37.5 to -42.5	43.5 to 48.5	Clay (very stiff)	$\phi = 0°$; c= 2000 psf	3, Clay	500	0.007	0.039	-
-42.5 to -47.5	48.5 to 53.5	Clay (very stiff to hard)	$\phi = 0°$; c= 4000 psf	3, Clay	1000	0.005	0.039	-
-47.5 to -54.0	53.5 to 60	Clay (very stiff)	$\phi = 0°$; c= 2000 psf	3, Clay	500	0.007	0.039	-
-54.0 to -56.5	60 to 62.5	Clay (firm)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.039	-
-56.5 to -66.0	62.5 to 72	Clay (very stiff to hard)	$\phi = 0°$; c= 3500 psf	3, Clay	1000	0.005	0.039	-
-66.0 to -72.0	72 to 78	Sand (medium dense)	$\phi = 33.5°$; c= 0 psf	4, Sand	60	-	0.039	-
-72.0 to -84.0	78 to 90	Clay (hard)	$\phi = 0°$; c= 4000 psf	3, Clay	1000	0.006	0.039	-
-84.0 to -87.0	90 to 93	Sand (medium dense)	$\phi = 33.5°$; c= 0 psf	4, Sand	60	-	0.039	-
-87.0 to -94.0	93 to 100	Clay (hard)	$\phi = 0°$; c= 4000 psf	3, Clay	1000	0.005	0.039	-

*Current ground elevation of +20.10 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +6.00 feet to +20.10 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	25.0/32.7		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

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

REGISTERED PROFESSIONAL ENGINEER RONALD L OEAN No. 46945 Exp. 9/30/11 CIVIL STATE OF CALIFORNIA

BIGGS CARDOSA ASSOCIATES INC.
865 THE ALAMEDA
SAN JOSE, CALIFORNIA 95126

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 E. WEBER AVENUE
STOCKTON, CA 95202

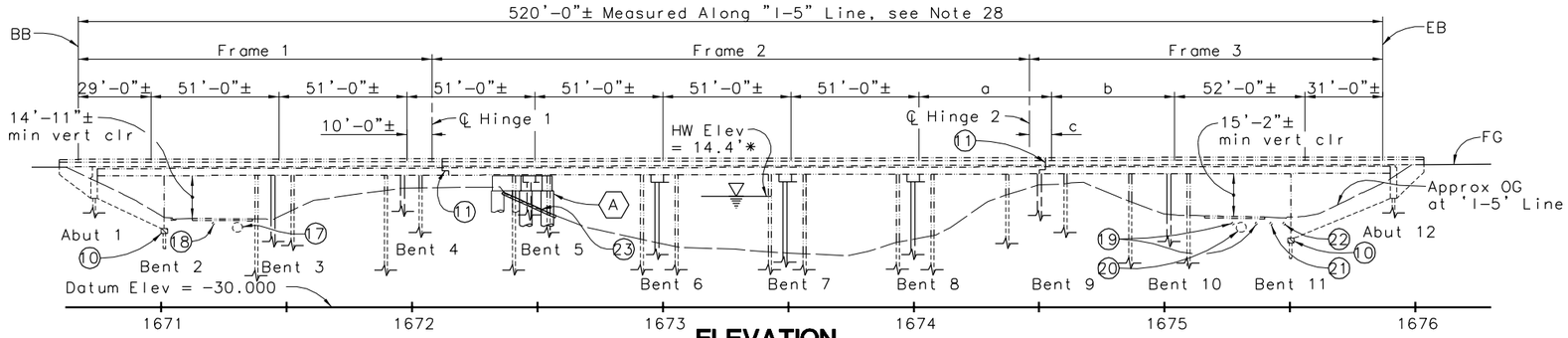
'I-5' Line 1670+37.00±
Elev 25.49

See 'DECK GRADES' sheets

'I-5' Line 1676+21.38±
Elev 26.48

PROFILE GRADE

NO SCALE



ELEVATION

1" = 30'

* See 'HYDROLOGIC SUMMARY' on 'GENERAL NOTES No. 2' sheet

Retrofit Legend:

(A) Construct Bent Retrofit (Piling, Cap)

(B) Install pipe seat extenders & construct diaphragm bolsters

	a	b	c
Left bridge	53'-0"±	49'-0"±	9'-0"±
Widen	49'-6"±	52'-6"±	Varies 9'-0"± min 10'-6"± max
Right bridge	46'-0"±	56'-0"±	10'-6"±

Legend:

Bridge Removal (Portion)

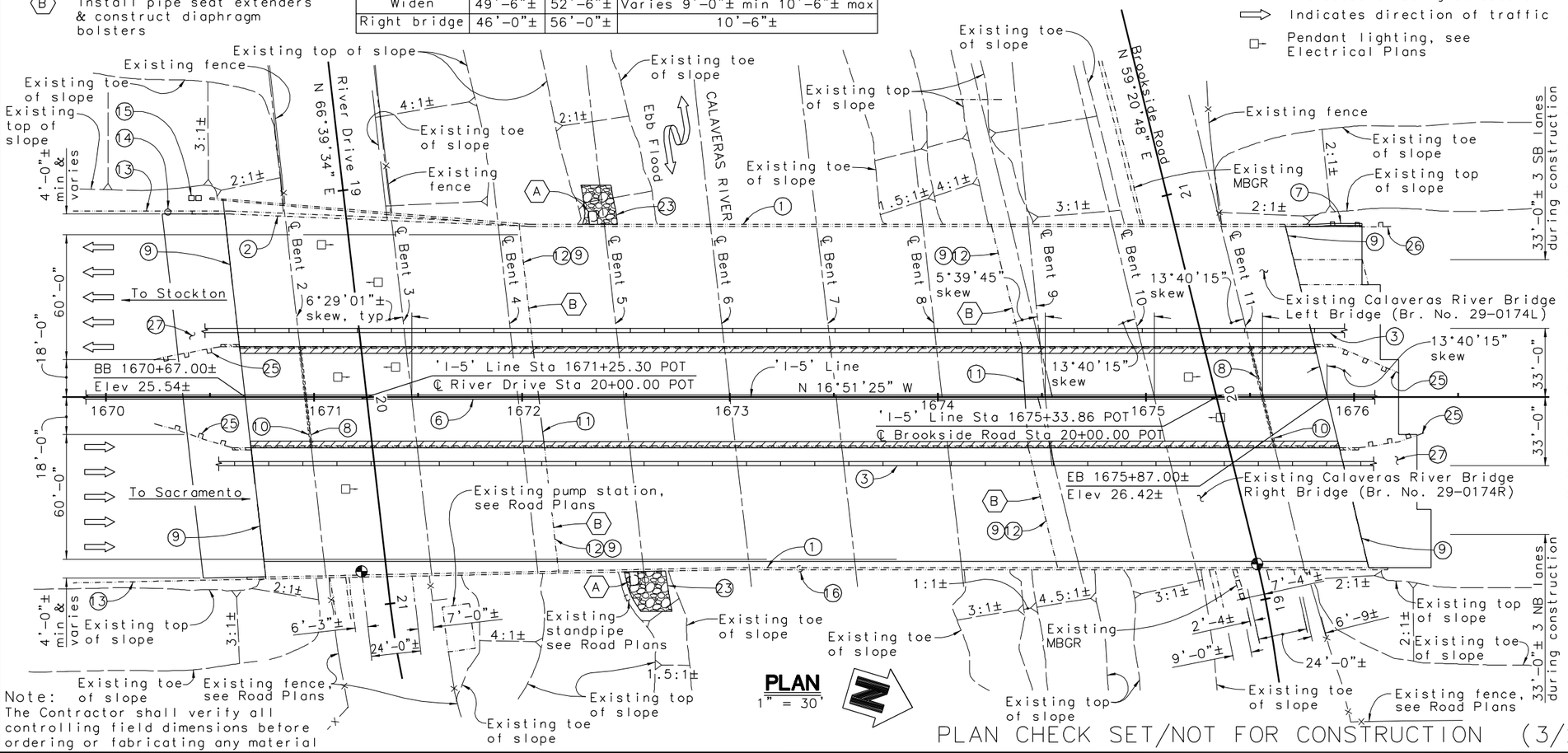
Closure Pour

Indicates point of minimum vertical clearance

Indicates existing structure

Indicates direction of traffic

Pendant lighting, see Electrical Plans



PLAN

1" = 30'

Note: The Contractor shall verify all controlling field dimensions before ordering or fabricating any material

Notes:

1. Existing Type 9 Barrier Railing
2. Existing Concrete Barrier Type 27R
3. Temporary Railing Type K for Stage 3 Construction, see Road Plans
4. Salvage existing Type 9 Barrier Railing
5. Remove existing overhang
6. Concrete Barrier Type 60A (Mod)
7. Concrete Barrier Type 736B (Mod)
8. Remove and reconstruct existing sidewalk, see Road Plans
9. Remove and replace existing joint seal, full width
10. Remove portion of existing curtain wall and footing, and construct new Bent 2 and Bent 11
11. Cable Restrainers at new hinge
12. Existing Cable Restrainers at hinge to remain
13. Existing Soundwall
14. Existing Overhead Sign
15. Existing Electrical Box
16. Existing Electrolier
17. Existing 18" dia SD, see Road Plans
18. Existing 12" dia Water line (California Water Co.), see Road Plans
19. Existing electrical line (PG&E), see Road Plans
20. Existing 48" dia SD, see Road Plans
21. Existing 12" dia water line, see Road Plans
22. Existing 10" dia SS line, see Road Plans
23. Rock slope protection, see Civil Plans
24. 4" dia FO (Future), see Electrical Plans
25. Salvage existing TBB, see Road Plans
26. Remove existing MBGR, see Road Plans
27. Structure Approach Type N(30D) and Type R(30D), see 'LIMITS OF STRUCTURE APPROACH' sheet
28. Indicates limits of Clean Bridge Deck, and Furnish and Place Polyester Concrete Overlay
29. For Typical Sections, see 'GENERAL PLAN No. 2' sheet
30. For Pile Data Table, see 'FOUNDATION PLAN' sheet
31. For General Notes, Quantities, Hydrologic Summary and Index to Bridge Plans, see 'GENERAL NOTES' sheets.

PLAN CHECK SET/NOT FOR CONSTRUCTION (3/25/10)

DESIGN OVERSIGHT	DESIGN	BY J. ALCIATI	CHECKED J. HO	LOAD & RESISTANCE FACTOR DESIGN	LIVE LOADING: HL93 W/"LOW-BOY"; PERMIT DESIGN VEHICLE
	DETAILS	BY R. YAMANE	CHECKED A. NOTARO	LAYOUT BY R. YAMANE	CHECKED J. HO
SIGN OFF DATE	QUANTITIES	BY C. TOGAN	CHECKED D. ROSELLINI	SPECIFICATIONS BY F. ARON	PLANS AND SPECS COMPAREDA. NOTARO

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

JOHN A. ALCIATI
PROJECT ENGINEER

BRIDGE NO. 29-0174 R/L
POST MILES 29.56

CALAVERAS RIVER BRIDGE (WIDEN)

GENERAL PLAN No. 1

DESIGN GENERAL PLAN SHEET (ENGLISH) (REV. 6-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

FILE => $REQUEST

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 6/2/09 9/4/09 10/16/09 12/4/09 1/15/10

SHEET 1 OF 58

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

20060048 (20060485TB)

APPENDIX E

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__) __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection __ 65% PS&E Unchecked Details _X PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 7/8/2010 **Structure Name*:** Calaveras River Bridge **Br No*:** 29-0174 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	_____	_____	_____	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 4th review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated June 11, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___) __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** Calaveras River Bridge **Br No*:** 29-0174 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	6/07/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	11	The specified tip elevation for Bent 3 in Table 5A is –58.0, in Table 6A it is –59.0, please clarify the design tip elevation for the interior piles at Bent 3.	Comment is incorporated. The design tip elevation and specified tip elevation in Table 6A is amended to -58.0 feet	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Calaveras River Bridge **Br No*:** 29-0174 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection X 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 12/16/2009 **Structure Name*:** Calaveras River Bridge **Br No*:** 29-0174 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	p.6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
3	FR	p.7, S.8.2	The report states that the liquefaction potential is low although the analysis states that the FS for layers 12 and 13 are below 1.1. Please clarify.	Comment is will be clarified. It is our opinion that these loose to medium dense sand layers (Nos. 12 and 13) are in form of pocket/lens and localized. Hence the overall liquefaction potential at the project site is considered relatively low.	
4	FR	p.8	Tables 2 and 3, Although the pile tips appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Submittal Data (Reviewer to complete)

Dist-EA: 10-0G4701 Reviewer: BHB Str Name*: Calaveras River Bridge

Date of Review: 12/16/09 Functional Unit: 59-323 Br No*. 29-0174 *=if applicable

5	FR	p.10	Table 4 needs to be updated to our standard LRFD Pile Data Table showing the resistance factor used.	Comment is incorporated. Caltrans standard LRFD Pile Data Table showing the resistance factor used is shown in Table 4.	
6	FR	p.14, S.9.7	A construction consideration may be to abide by Army Corps of Engineers and/or Department of Water Resources construction and backfill requirements, please respond.	This statement is correct. This will be included in the construction consideration. The details provided in this section were discussed with the design team and the agency. Some of these will be incorporated in the specifications.	
7	FR	Plate No. 2	Please delineate Bent 5a on plan.	Comment is incorporated. Bent 5a is delineated on Plate No. 2.	
8	FR	LOTB	As is stated in the Soil Legend, the sample number or unconfined compressive strength is generally not located to the left of the graphic on the LOTB.	Comment is incorporated.	
9	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #4 above.	
10	FR	App. C	Layer 5 of the liquefaction analysis was analyzed as a clay/silt, please correct to show layer is a sand.	Comment is incorporated. The soil type is changed to "sand" for layer 5.	
11	FR	NA	C3 – Not Approved (Resubmittal to GS Required)	NA	

FOUNDATION REPORT
EBMUD AQUEDUCT UC (WIDEN)
(BRIDGE NO. 29-0226)
STOCKTON, CALIFORNIA
10-SJ-05, PM 29.83 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

PARIKH CONSULTANTS, INC.
2360 Qume Drive, Suite A, San Jose, CA 95131
(408) 452-9000

April 20, 2010 Job No. 206127.BR

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As-built Log of Test Borings (Br. No. 29-0226 R/L)
CPT Correlation Data (CPT-09-EM-001)

APPENDIX B

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APPENDIX C

Evaluation of Liquefaction Potential
Vertical Pile Capacity Calculations
Geotechnical Parameters for LPILE (v. 5) Analyses

APPENDIX D

Office of Special Funded Projects Comment & Response Form (with PARIKH Consultants, Inc. Responses)

FOUNDATION REPORT
EBMUD AQUEDUCT UC (WIDEN)
(BRIDGE NO. 29-0226)
STOCKTON, CALIFORNIA
10-SJ-05 PM 29.83 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "EBMUD Aqueduct Undercrossing (Widen) (Bridge No. 29-0226)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the EBMUD Aqueduct Undercrossing (UC) (Widen) (Bridge No. 29-0226).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter

unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are four-span structures. Based on the as-built general plan and foundation plan of the EBMUD Aqueduct UC, the structures were constructed in 1970. The existing left and right bridge (Bridge No. 29-0226 L/R) are approximately 248 feet long. The left bridge (29-0226 L) is approximately 67 feet to 81 feet wide. The right bridge (29-0226 R) is approximately 50 feet wide. The existing ramp connector bridge (29-0226 S) is approximately 256 feet and 26 feet wide. The left and right bridges have not been widened since their construction in 1970. The ramp connector bridge was widened in 1997.

The abutments of the existing left and right bridge are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class II 45 tons) based on the as-built data.

Proposed Construction

The widening of the EBMUD Aqueduct UC is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. Exterior widening of Left Bridge is proposed and no widening for the ramp connector bridge.

According to Rajappan & Meyer Consulting Engineers Inc., the proposed EBMUD Aqueduct UC (Widen) will match the existing span with supports at the abutments and bents. The widen structure will be a cast-in-place reinforced concrete box girder with approximate length of 248

feet measured along the "I-5" Line. The width of the inside widening will be 53 feet 8 inches. The abutments will incorporate diaphragm type abutments.

Based on our discussions with the designer, the proposed bridge abutments are to be supported on pile cap with 24-inch diameter Cast-In-Drilled-Hole (CIDH) concrete piles. The bents will be supported on 48-inch diameter CIDH concrete piles.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The EBMUD Aqueduct UC is located at the intersection of Interstate 5 (I-5) and EBMUD Aqueduct. It is approximately 0.1 miles south of March Lane UC. The Project Location Map, Plate 1, shows the location of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the EBMUD Aqueduct UC (July 1965) have been used as a reference (Ref. 1). Based on the as-built LOTB, Borings B-2 and B-4 were drilled and Cone Penetration Test (CPT) at B-1, B-3, B-5 and B-6 were performed by Caltrans in 1965. The soil borings were drilled to the approximate depths between 66 feet and 76 feet and to the bottom elevation between approximately -63 feet and -75.5 feet. The CPTs were pushed to the approximate depths between 59 feet and 70 feet and to the bottom elevation between approximately -59 feet and -67 feet.

Field Exploration in 2009

Additional field exploration was conducted in March and April 2009. Boring A-09-EM-001 was drilled on the south side and CPT-09-EM-001 was performed on the north side of EBMUD Aqueduct. The soil boring was drilled to the depth of 111.0 feet and the CPT was pushed to the depth of 109.5 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit weight, Plasticity Index and grain size classification of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built borings and CPT, the subsurface soils generally consist of interbedded layers of stiff to hard silts and medium dense to dense silty sands. A relatively thin (approximately 8 feet) layer of very soft clayey silt was encountered at the top of Boring B-4.

Based on Boring A-09-EM-001 and CPT-09-EM-001, the subsurface soils generally consist of dense silty sand (embankment fill), underlain by firm to stiff lean clays, underlain by medium dense silty sands, underlain by stiff to hard silt/lean clays.

Groundwater was recorded between approximate Elevation -4 feet and -4.5 feet in the CPTs in July 1965. Groundwater was measured at Elevation -8 feet (the depth of 33 feet below existing ground surface) during drilling for A-09-EM-001 in March 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of dense silty sand, underlain by firm to stiff lean clays, underlain by medium dense silty sands, underlain by stiff to hard silt/lean clays. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring A-09-EM-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-EM-001 included in this report, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure, different foundation systems have been evaluated with respect to their design capacity and construction considerations. No driven

pile is allowed in the vicinity of the aqueduct according to East Bay Municipal Utility District. 24-inch diameter CIDH concrete piles and 48-inch diameter CIDH concrete piles are recommended to be the foundation system for the abutments and bents of the proposed bridge structure, respectively. Pertinent foundation design information provided by Biggs Cardosa Associates (BCA), including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2: FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	24" Dia. CIDH	19.0	17.0	2.5	53.67	1.00	6
Bent 2	LRFD	48" Dia. CIDH	5.8	3.8	N/A	N/A	1.00	3
Bent 3	LRFD	48" Dia. CIDH	4.5	2.5	N/A	N/A	1.00	3
Bent 4	LRFD	48" Dia. CIDH	5.5	3.5	N/A	N/A	1.00	3
Abut 5	WSD	24" Dia. CIDH	20.5	18.5	2.5	53.67	1.00	6

TABLE 3: FOUNDATION DESIGN LOADS

Support No.	**Service-I Limit State (kips)**			**Strength Limit State (Controlling Group, kips)**				**Extreme Limit State (Controlling Group, kips)**			
	Total Load		**Permanent Loads**	**Compression**		**Tension**		**Compression**		**Tension**	
	Per Support	**Per Pile**	**Per Support**	**Per Support**	**Max. Per Pile.**	**Per Support**	**Max. Per Pile.**	**Per Support**	**Max. Per Pile.**	**Per Support**	**Max. Per Pile.**
Abut 1	470	160	310	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	1580	670	1190	2160	1070	0	0	1190	690	0	0
Bent 3	1930	790	1530	2640	1290	0	0	1530	690	0	0
Bent 4	1580	670	1190	2160	1070	0	0	1190	690	0	0
Abut 5	470	160	231	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

* Entire support remains in overall compression

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations and Load and Resistance Factor Design (LRFD) is used for the bent foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads". The Bent foundations are evaluated for the foundation design data and loading conditions using AASHTO LRFD Bridge Design Specifications–4th Edition, with interims through December 2008 and current Caltrans Amendments.

The evaluation of Load Demands on the piles, based upon WSD and LRFD are presented in Table 3 above. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Table 4 and Table 5 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	24" Dia. CIDH	17.0 ±	470	310	160	320	-30.0 (a)	-30.0
Abut 5	24" Dia. CIDH	18.5 ±	470	231	160	320	-27.5 (a)	-27.5

Notes: (i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively.
(i) The specified tip elevation shall not be raised above the design tip elevations for lateral.

TABLE 5: FOUDNATION RECOMMENDATION FOR BENTS

Location	Pile Type	Cut-off Elev. (feet)	Service-I Limit State Load (kips) per Support	Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev.[(1)] (ft)	Specified Tip Elev. (ft)[(ii)]
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1.0)	Tension (φ=1.0)		
Bent 2	48" Dia. CIDH	3.8	1190	1	1070	0	690	0	-62.5 (a), -26.5(c)	-62.5
Bent 3	48" Dia. CIDH	2.5	1530	1	1290	0	690	0	-70.5 (a), -27.0 (c)	-70.5
Bent 4	48" Dia. CIDH	3.5	1190	1	1070	0	690	0	-63.0 (a), -24.5 (c)	-63.0

Notes

(i) Design tip elevations are controlled by (a) Compression (Strength Limit), (b) Tension (Strength Limit), (c) Compression (Extreme Event), (d) Tension (Extreme Event), (e) Lateral Load, (f) Settlement respectively.
(ii) The design tip elevation due to the lateral loads are estimated by BCA and not included in the table above.
(iii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iv) The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the penetrated soil layers, if any, which do not contribute to the design resistance.

TABLE 6: PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)
		Compression	Tension		
Abut 1	24" Dia. CIDH	320	0	-30.0 (a)	-30.0
Bent 2	48" Dia. CIDH	1530	0	-62.5 (a)	-62.5
Bent 3	48" Dia. CIDH	1850	0	-70.5 (a)	-70.5
Bent 4	48" Dia. CIDH	1530	0	-63.0 (a)	-63.0
Abut 5	24" Dia. CIDH	320	0	-27.5 (a)	-27.5

Notes

(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression (b) Lateral Load.
(ii) Design tip elevations for the ***Bents*** are controlled by (a) Compression, (b) Lateral Load.
(iii) The design tip elevation due to the lateral loads are estimated by BCA and not included in the pile data table above.
(iv) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

The estimation of the pile capacity in clay and sand are based on the adhesion factors per FHWA manual (O'Neil and Reese 1999). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 6, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles should not be spaced closer than 3 times the pile diameter measured center-to-center. For piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter, there is no group effect for vertical pile capacity.

Caltrans standard specification for "Cast-in-Place Concrete Piling" should be used for the construction of CIDH piles. Groundwater is expected during pile construction. Due to

presence of sand/gravel layers, and groundwater, raveling or caving may be expected, which may require additional drilling and cleaning effort and may increase the concrete volume for the piles. The CIDH holes are not expected to remain open without implementation of appropriate measures. Temporary steel casing and using slurry displacement method of construction should be used at all times to maintain the integrity of the piles. Caltrans Standard Specifications and SSPs should be used for construction and quality assurance procedures. All pile excavations should be observed by the Geotechnical Engineer prior to the placement of reinforcement and concrete so that if conditions differ from those anticipated, appropriate recommendations can be made.

It is prudent to make the contractor aware of these conditions so that appropriate steps can be taken to comply with the standard and maintain the integrity of the pile foundations.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. It is our understanding that the structural designer will perform lateral load analyses for the piles using LPILE Program (v. 5). "Generalized Vertical Soil Profile and Strength Parameters" based on A-09-EM-001 and CPT-09-EM-001 are included in Appendix C. To account for group effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D, respectively.

Generally, abutment piles are designed for service loads only, whereas bent piles are governed by seismic loading conditions. A maximum pile head deflection of 1/4 inch is recommended for service condition for abutment piles. For seismic loading condition of the bent piles, a maximum pile head deflection up to about one inch is generally tolerable and accepted for pile design.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

Applied Lateral Earth Pressures

Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for the widening of the existing structures. Settlement of the existing embankment is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Boring A-09-EM-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
A-09-EM-001	29.5	1100	6.9	53.2	96.6

Based on the corrosion test results, the native subsurface soils in Boring A-09-EM-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.8 Construction Observation

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

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,

PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

AL/GP: S:\Ongoing Projects\2006\206127\I-5 EBMUD Aqueduct UC (Widen) Foundation Report (4-2010).rpt

REFERENCES

1. EBMUD Aqueduct UC As-built Log of Test Borings (July 1965).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

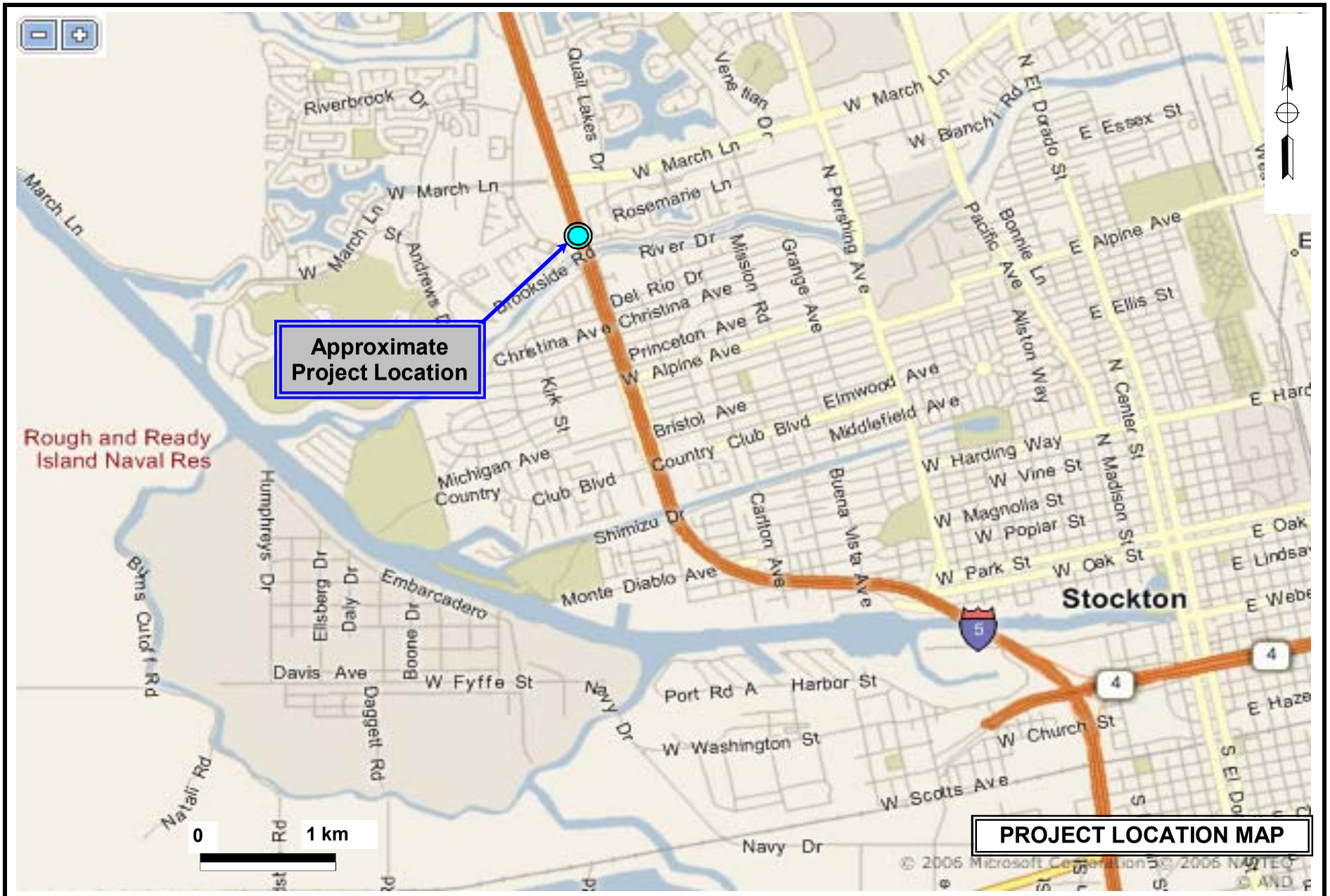
3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

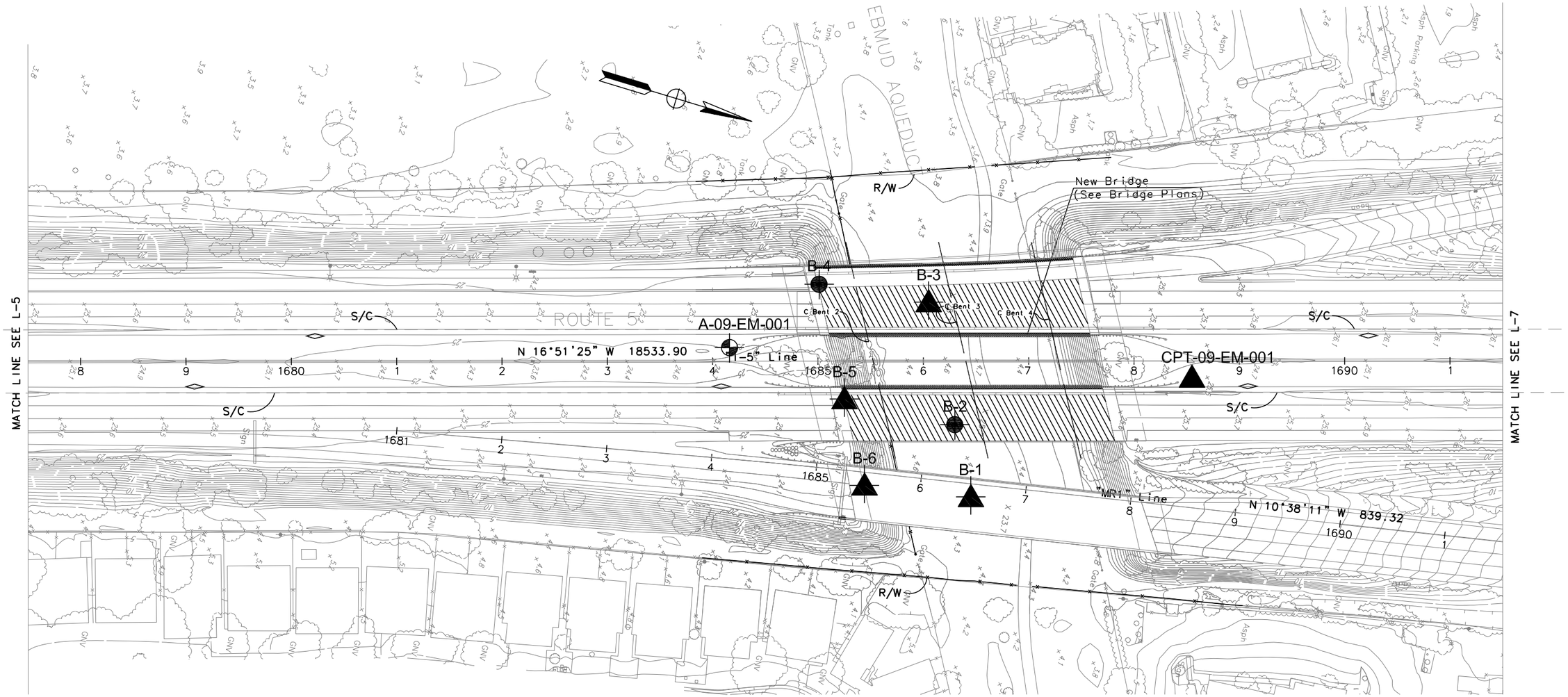
7. State of California Department of Transportation Standard Specifications May 2006.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

EBMUD AQUEDUCT UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0226)	PLATE NO.: 1



LEGEND

A-09-EM-001
Approximate Boring Location

B-1
Approximate CPT location performed in 1965

B-2
Approximate Boring location drilled in 1965

CPT-09-EM-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

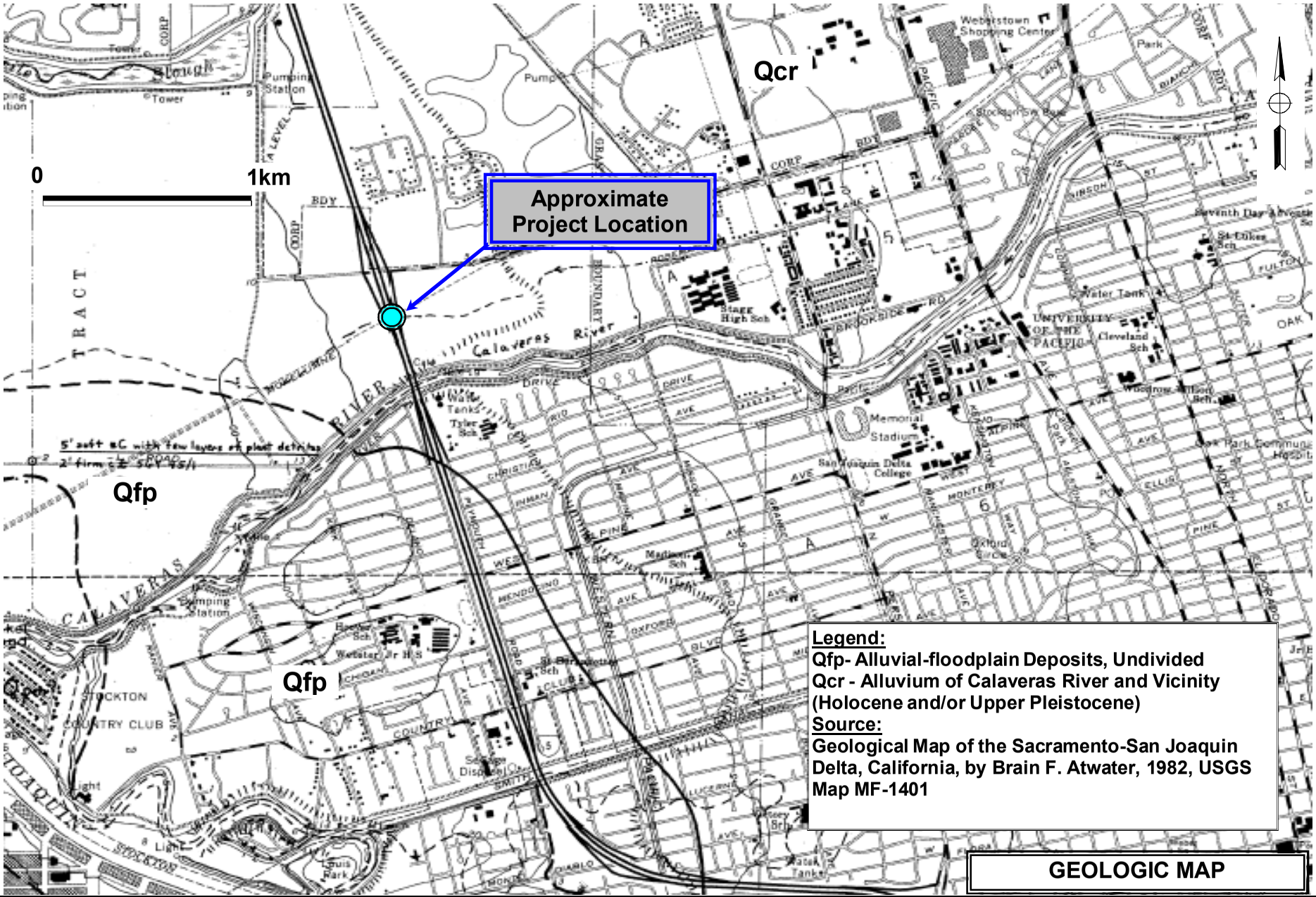
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

EBMUD AQUEDUCT UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

PLATE NO: 2

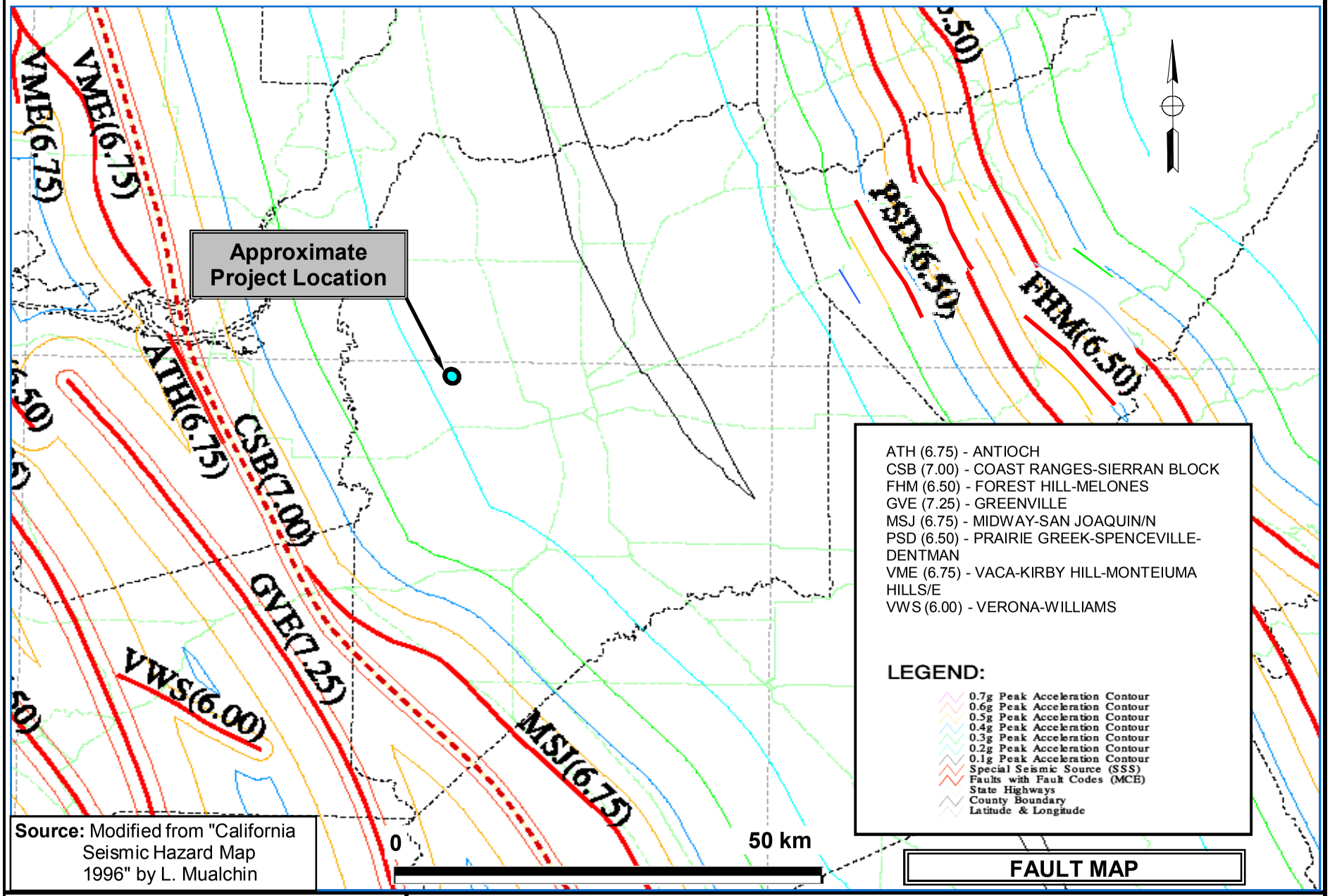


GEOLOGIC MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

EBMUD AQUEDUCT UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0226)	PLATE NO.: 3



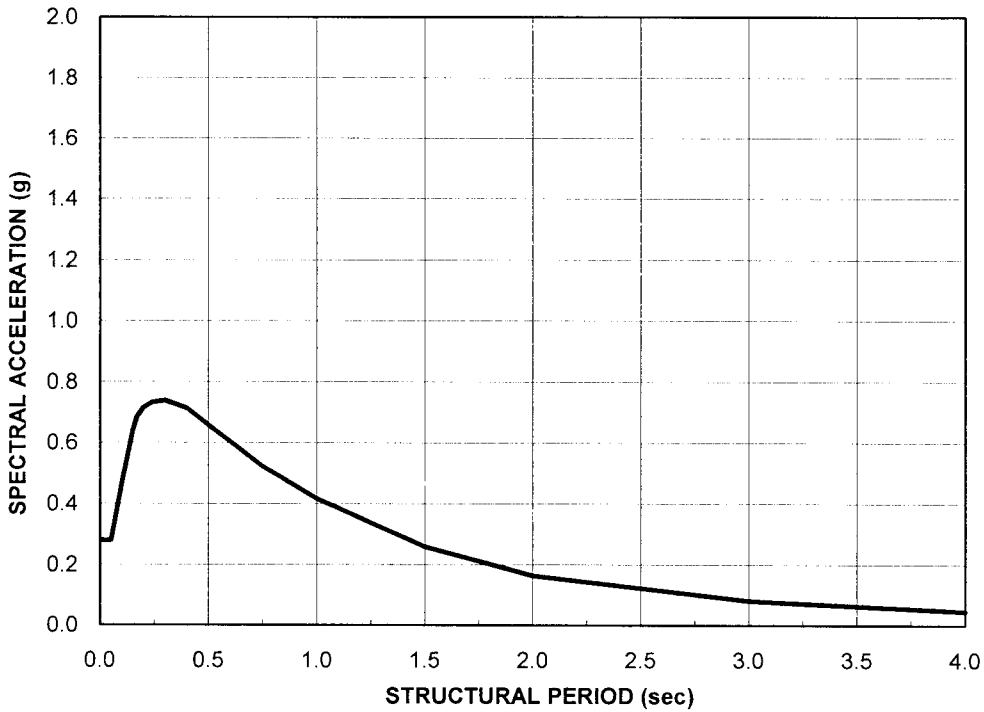
Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

EBMUD AQUEDUCT UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0226) | **PLATE NO.: 4**

ARS DESIGN CURVE
EBMUD AQUEDUCT UC (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

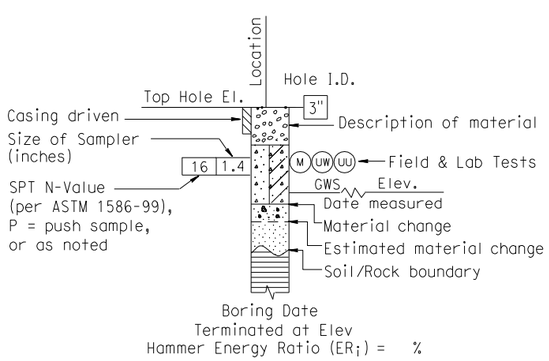
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

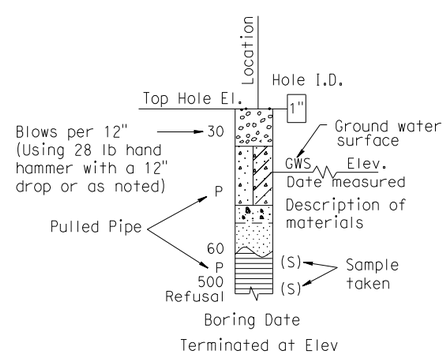
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

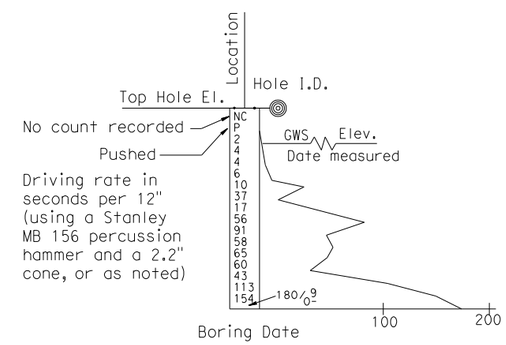
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



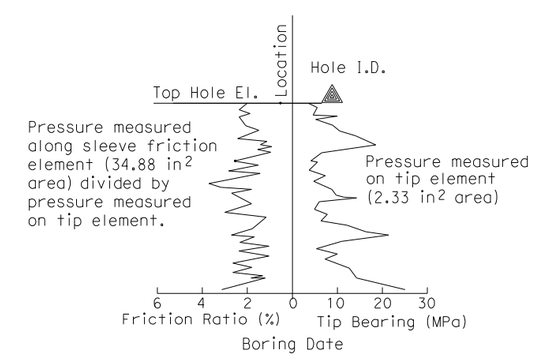
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER
SIGN OFF DATE	CHECKED BY	D. WANG	DATE: MARCH 2009 - APRIL 2009		

BRIDGE NO. 29-0226 — POST MILES 29.83

EBMUD AQUEDUCT UC (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES — REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 30 OF 33

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

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

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)
- (VS) Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0226
POST MILES 29.83

EBMUD AQUEDUCT UC (WIDEN)
SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 31 OF 33

...\EBMUD\29-0226-z-ltb02.dgn 6/8/2010 9:23:58 AM

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

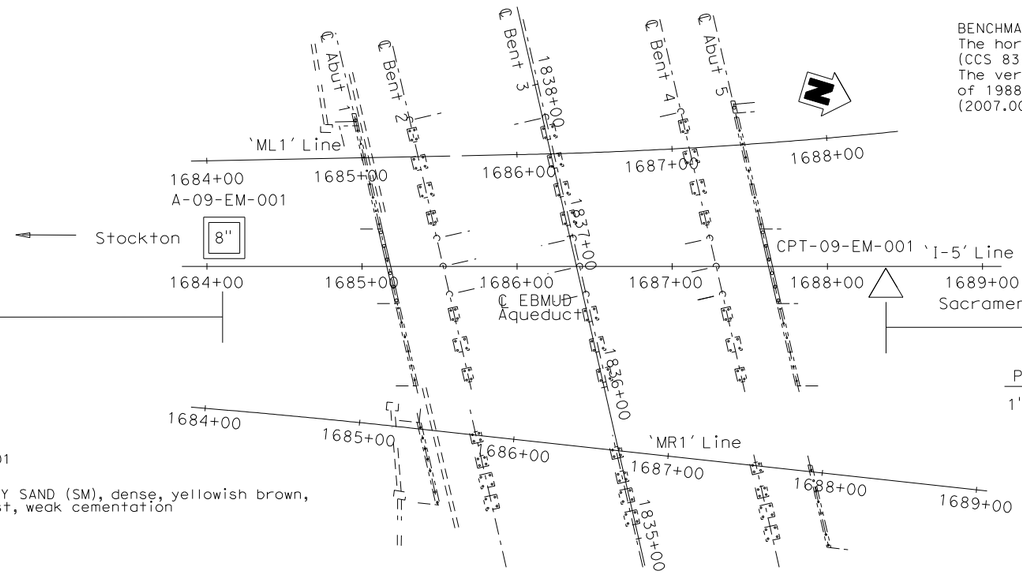
REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

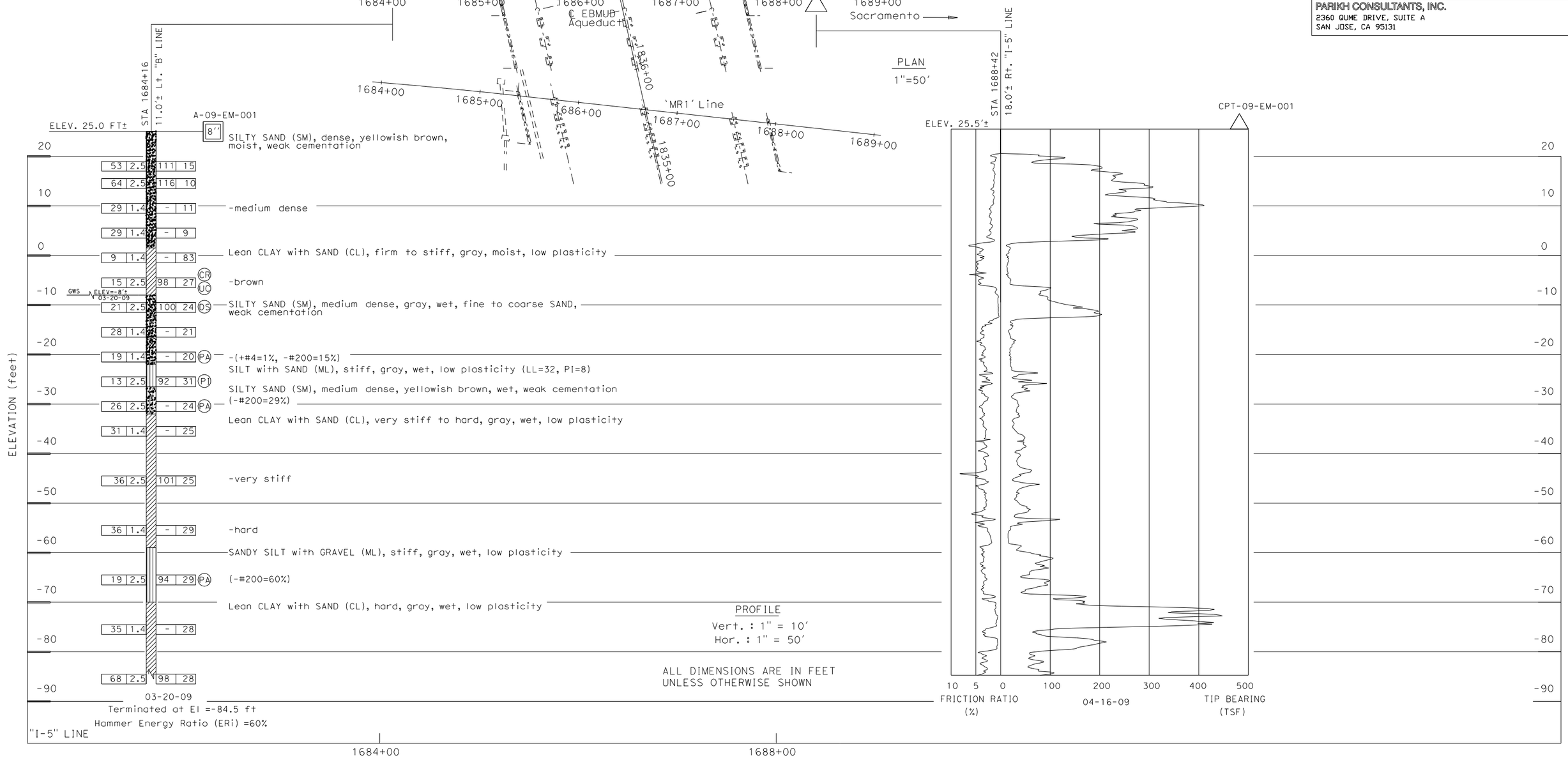
SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.
The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.



PLAN
1"=50'



PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	A. LAM	BRIDGE NO. 29-0226
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PROJECT ENGINEER	POST MILES 29.83

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

EBMUD AQUEDUCT UC (WIDEN)
LOG OF TEST BORINGS 1 OF 2

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES → 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 32 OF 33

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

...\EBMUD\29-0226-z-ltb03.dgn 6/8/2010 9:24:53 AM

DIST.	COUNTY	ROUTE	KILOMETER POST-TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5			

Dist.	County	Route	PM	Project No.	Total Sheets
10	SJ	5	25.3/31.5	343	372

District Geologist — Civil Engineer—Licensed Land Surveyor

DATE APPROVED September 5, 1967

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

REGISTERED CIVIL ENGINEER _____ DATE _____

EBMUD AQUEDUCT UNDERCROSSING (WIDEN)

LOG OF TEST BORINGS 2 OF 2

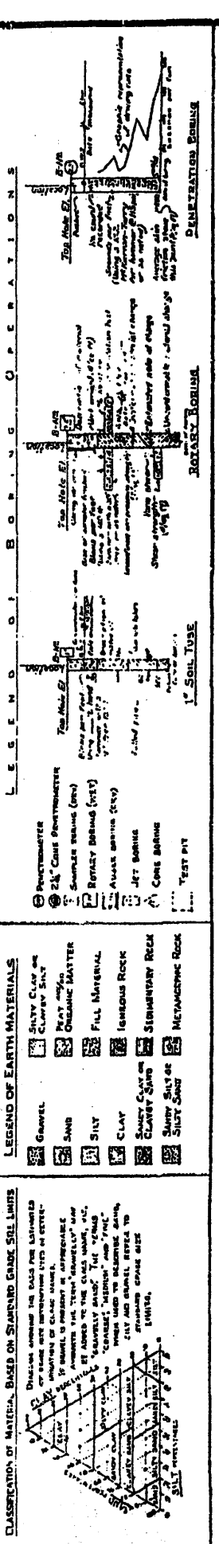
NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0C4701

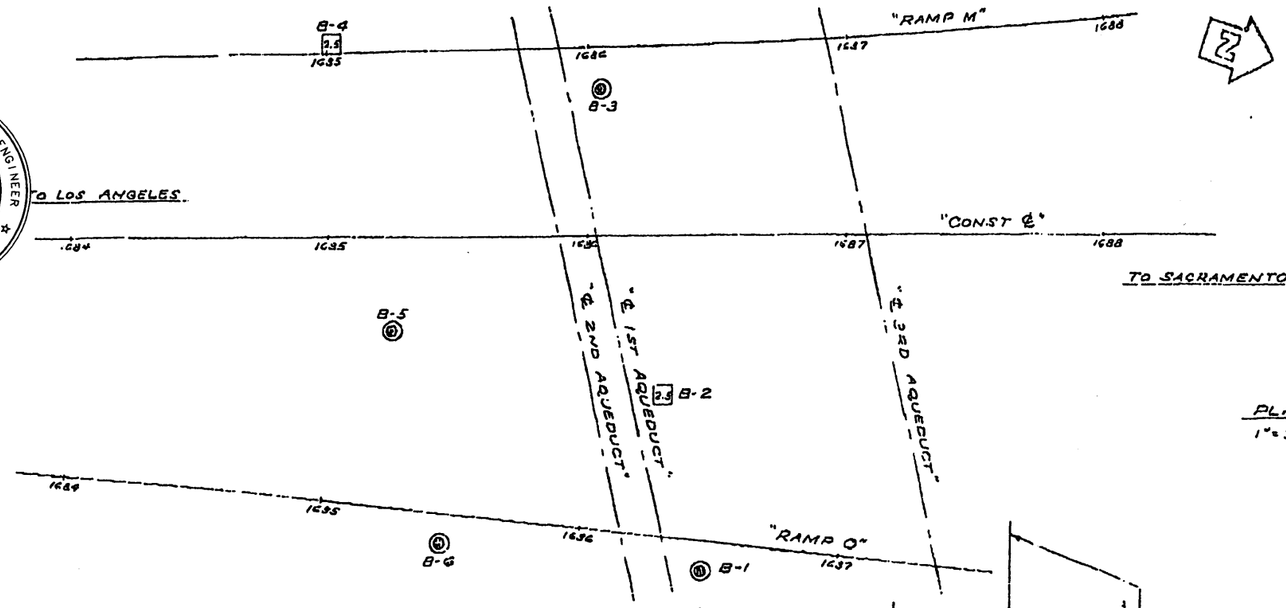
BRIDGE No. 29-0226

Sheet 32 of 32

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

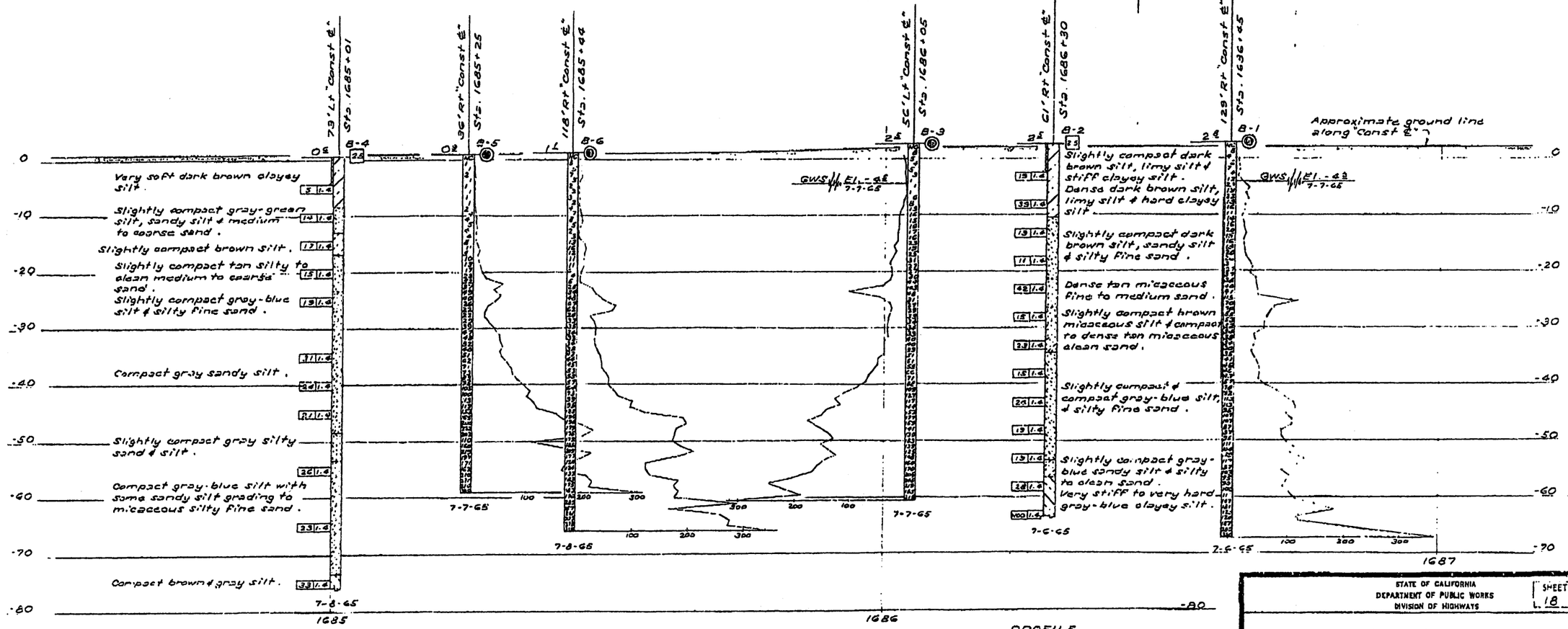


NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.



BM "NO 5"
Bronze disc in PCC 100'±
E "Const ℄" 1693+60
Elev. 193

PLAN
1" = 30'



PROFILE
1" = 10'

AS BUILT PLANS
Contract No. 10-080924
Date Completed _____

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

SHEET 18 OF 18

EBMUD AQUEDUCT UC

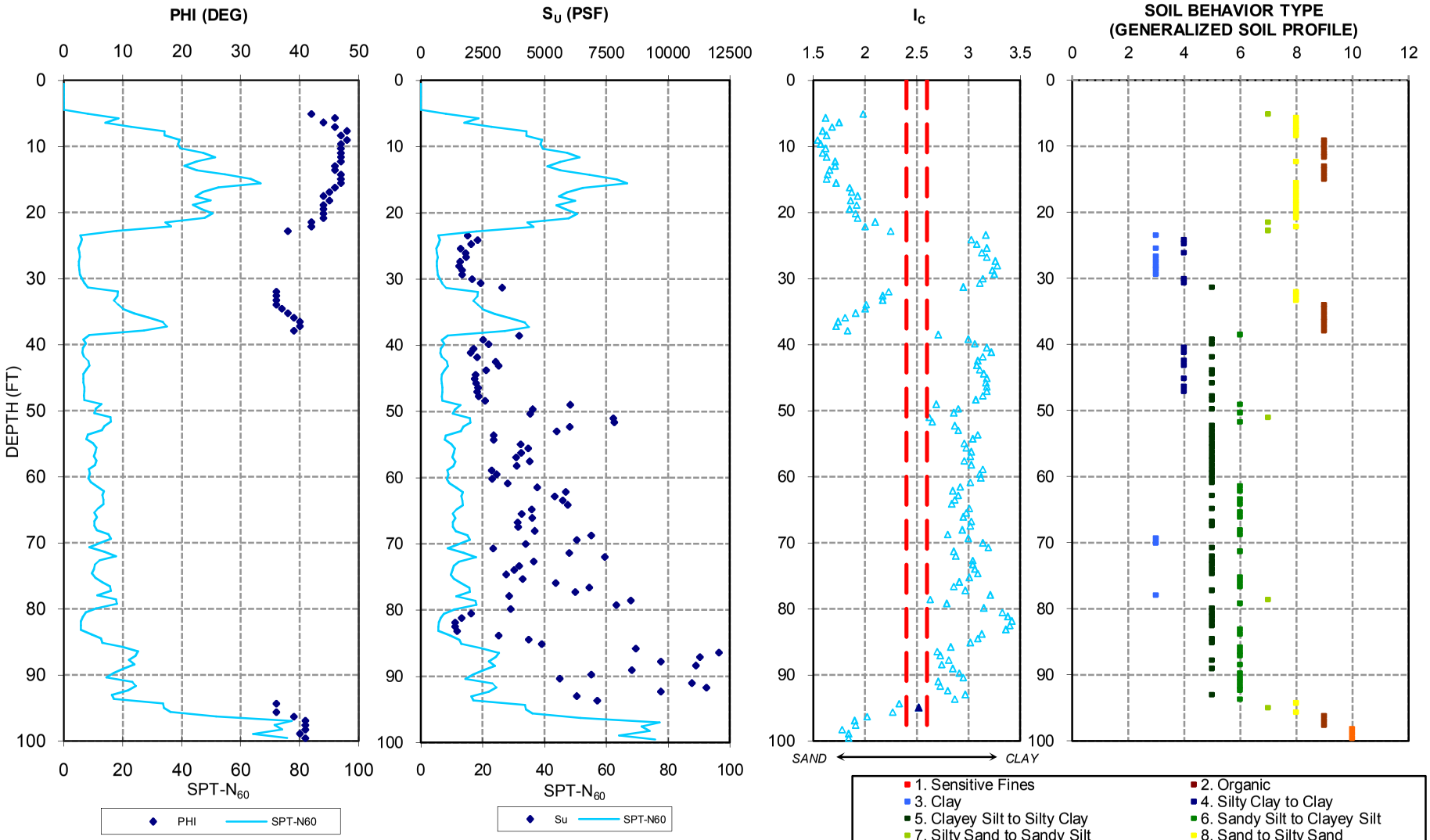
LOG OF TEST BORINGS

SCALE As Noted | R/L & QR BRIDGE 29-226 | FILE | DRAWING 29226-16

10202

CPT DATA CORRELATION

CPT-09-EM-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

EBMUD AQUEDUCT UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0226)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

	I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-EBMUD		
PROJECT NAME	***AQUEDUCT UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0226)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-EM-006***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
5.085	7	Silty Sand to Sandy Silt	41.597	0.72	0.299		0.299	1.98	1.46765	0.99	0.129	1.980	1.47	61.05	1.275	77.822	0.12	1.00	1	
5.741	8	Sand to Silty Sand	108.706	1.293	0.339		0.339	1.62	1.4295	0.99	0.129	1.620	1.43	155.40	1.000	155.395	0.43	1.00	1	
6.398	8	Sand to Silty Sand	78.498	0.97	0.379		0.379	1.75	1.39329	0.99	0.128	1.750	1.39	109.37	1.072	117.201	0.23	1.00	1	
7.054	8	Sand to Silty Sand	130.234	1.838	0.418		0.418	1.68	1.3597	0.99	0.128	1.680	1.36	177.08	1.024	181.376		1.00	1	
7.71	8	Sand to Silty Sand	200.777	2.736	0.458		0.458	1.59	1.3269	0.98	0.128	1.590	1.33	266.41	1.000	266.411		1.00	1	
8.366	8	Sand to Silty Sand	197.966	2.836	0.498		0.498	1.63	1.29564	0.98	0.128	1.630	1.30	256.49	1.000	256.493		1.00	1	
9.022	9	Sand	234.801	2.761	0.539		0.539	1.54	1.26509	0.98	0.128	1.540	1.27	297.05	1.000	297.046		1.00	1	
9.678	9	Sand	229.347	2.771	0.579		0.579	1.57	1.23665	0.98	0.127	1.570	1.24	283.62	1.000	283.622		1.00	1	
10.335	9	Sand	230.095	3.071	0.62		0.62	1.62	1.20879	0.98	0.127	1.620	1.21	278.14	1.000	278.137		1.00	1	
10.991	9	Sand	277.479	3.703	0.661		0.661	1.59	1.18216	0.98	0.127	1.590	1.18	328.02	1.000	328.025		1.00	1	
11.647	9	Sand	297.527	4.497	0.701		0.701	1.63	1.15729	0.98	0.127	1.630	1.16	344.32	1.000	344.324		1.00	1	
12.303	8	Sand to Silty Sand	255.493	4.168	0.741		0.741	1.71	1.13344	0.97	0.127	1.710	1.13	289.59	1.045	302.500		1.00	1	
12.959	9	Sand	231.291	3.342	0.782		0.782	1.71	1.10999	0.97	0.126	1.710	1.11	256.73	1.045	268.181		1.00	1	
13.615	9	Sand	259.103	3.485	0.823		0.823	1.66	1.08749	0.97	0.126	1.660	1.09	281.77	1.010	284.705		1.00	1	
14.272	9	Sand	316.002	4.429	0.863		0.863	1.64	1.06641	0.97	0.126	1.640	1.07	336.99	1.000	336.987		1.00	1	
14.928	9	Sand	367.82	5.392	0.904		0.904	1.63	1.04563	0.97	0.126	1.630	1.05	384.60	1.000	384.603		1.00	1	
15.584	8	Sand to Silty Sand	375.938	6.933	0.944		0.944	1.72	1.02612	0.97	0.126	1.720	1.03	385.76	1.051	405.557		1.00	1	
16.24	8	Sand to Silty Sand	282.841	5.94	0.983		0.983	1.85	1.00779	0.97	0.126	1.850	1.01	285.04	1.145	326.398		1.00	1	
16.896	8	Sand to Silty Sand	252.021	5.057	1.023		1.023	1.87	0.98965	0.96	0.125	1.870	0.99	249.41	1.162	289.778		0.99	1	
17.552	8	Sand to Silty Sand	233.246	5.108	1.063		1.063	1.93	0.97216	0.96	0.125	1.930	0.97	226.75	1.218	276.290		0.98	1	
18.209	8	Sand to Silty Sand	267.27	5.115	1.102		1.102	1.86	0.95569	0.96	0.125	1.860	0.96	255.43	1.153	294.596		0.96	1	
18.865	8	Sand to Silty Sand	229.282	4.635	1.142		1.142	1.92	0.93937	0.96	0.125	1.920	0.94	215.38	1.208	260.238		0.95	1	
19.521	8	Sand to Silty Sand	251.945	4.333	1.182		1.182	1.85	0.92359	0.96	0.125	1.850	0.92	232.69	1.145	266.454		0.94	1	
20.177	8	Sand to Silty Sand	265.583	5.439	1.222		1.222	1.91	0.90834	0.96	0.124	1.910	0.91	241.24	1.198	289.103		0.92	1	
20.833	8	Sand to Silty Sand	249.018	5.061	1.261		1.261	1.93	0.89395	0.95	0.124	1.930	0.89	222.61	1.218	271.240		0.91	1	
21.49	7	Silty Sand to Sandy Silt	168.107	4.001	1.3		1.3	2.1	0.88	0.95	0.124	2.100	0.88	147.93	1.455	215.208		0.92	1	
22.146	8	Sand to Silty Sand	184.95	3.424	1.34		1.34	2	0.86614	0.95	0.124	2.000	0.87	160.19	1.300	208.251		0.91	1	
22.802	7	Silty Sand to Sandy Silt	82.298	1.44	1.378		1.378	2.25	0.85337	0.95	0.123	2.250	0.85	70.23	1.798	126.293	0.27	0.93	1	
23.458	3	Clay	15.527	0.832	1.415		1.415	3.17	0.8413	0.95	0.123							0.93	1	
24.114	4	Silty Clay to Clay	18.677	0.749	1.452		1.452	3.03	0.82956	0.94	0.123							0.93	1	
24.77	4	Silty Clay to Clay	16.734	0.665	1.49		1.49	3.08	0.81784	0.94	0.123							0.92	1	
25.427	3	Clay	13.551	0.541	1.526		1.526	3.18	0.80704	0.94	0.122							0.92	1	
26.083	4	Silty Clay to Clay	15.03	0.549	1.564		1.564	3.13	0.79595	0.94	0.122							0.91	1	
26.739	3	Clay	15.216	0.689	1.601		1.601	3.18	0.78543	0.94	0.122							0.91	1	
27.395	3	Clay	13.48	0.636	1.637		1.637	3.26	0.77547	0.93	0.121							0.91	1	
28.051	3	Clay	13.196	0.627	1.674		1.674	3.28	0.76548	0.93	0.121							0.90	1	
28.707	3	Clay	14.102	0.61	1.71		1.71	3.23	0.75601	0.93	0.120							0.90	1	
29.364	3	Clay	14.201	0.63	1.747		1.747	3.25	0.74652	0.92	0.120							0.89	1	
30.02	4	Silty Clay to Clay	17.294	0.685	1.784		1.784	3.14	0.73727	0.92	0.120							0.89	1	
30.676	4	Silty Clay to Clay	19.911	0.867	1.822	0.021	1.801	3.11	0.73309	0.92	0.121							0.89	1	
31.332	5	Clayey Silt to Silty Clay	26.446	0.943	1.86	0.041	1.818	2.95	0.72896	0.91	0.122							0.89	1	
31.988	8	Sand to Silty Sand	85.563	0.994	1.899	0.062	1.837	2.23	0.7244	0.91	0.122	2.230	0.72	61.98	1.743	108.063	0.20	0.89	1	1.93
32.644	8	Sand to Silty Sand	86.093	0.803	1.939	0.082	1.857	2.17	0.71966	0.91	0.123	2.170	0.72	61.96	1.596	98.908	0.17	0.88	1	1.65
33.301	8	Sand to Silty Sand	80.491	0.653	1.979	0.103	1.876	2.17	0.71521	0.90	0.124	2.170	0.72	57.57	1.596	91.901	0.15	0.88	1	1.47
33.957	9	Sand	93.086	0.45	2.019	0.123	1.896	2.01	0.71059	0.90	0.124	2.010	0.71	66.15	1.313	86.869	0.14	0.88	1	1.35
34.613	9	Sand	101.456	0.554	2.06	0.144	1.916	2	0.70603	0.89	0.125	2.000	0.71	71.63	1.300	93.121	0.16	0.87	1	1.48
35.269	9	Sand	126.281	0.651	2.101	0.164	1.937	1.91	0.70131	0.89	0.125	1.910	0.70	88.56	1.198	106.133	0.19	0.85	1	1.82
35.925	9	Sand	156.554	0.739	2.142	0.185	1.957	1.81	0.69686	0.88	0.126	1.810	0.70	109.10	1.114	121.535	0.25	0.83	1	2.34
36.581	9	Sand	187.543	0.876	2.182	0.205	1.977	1.74	0.69248	0.88	0.126	1.740	0.69	129.87	1.065	138.286	0.33	0.82	1	3.08
37.238	9	Sand	196.552	0.872	2.223	0.226	1.997	1.72	0.68815	0.87	0.126	1.720	0.69	135.26	1.051	142.199	0.35	0.81	1	3.28
37.894	9	Sand	147.131	0.631	2.264	0.246	2.018	1.83	0.68365	0.87	0.127	1.830	0.68	100.59	1.129	113.581	0.22	0.83	1	2.04
38.55	6	Sandy Silt to Clayey Silt	32.141	0.486	2.301	0.267	2.035	2.71	0.68006	0.86	0.127							0.87	1	
39.206	5	Clayey Silt to Silty Clay	20.883	0.506	2.339	0.287	2.052	3	0.67651	0.86	0.127							0.87	1	
39.862	5	Clayey Silt to Silty Clay	22.439	0.794	2.377	0.307	2.069	3.06	0.67299	0.85	0.127							0.86	1	
40.518	4	Silty Clay to Clay	18.132	0.695	2.414	0.328	2.086	3.18	0.66951	0.85	0.127							0.86	1	
41.175	4	Silty Clay to Clay	17.132	0.663	2.452	0.348	2.103	3.22	0.66606	0.84	0.127							0.86	1	
41.831	5	Clayey Silt to Silty Clay	19.327	0.663	2.489	0.369	2.12	3.14	0.66265	0.83	0.127							0.86	1	
42.487	4	Silty Clay to Clay	24.798	1.035	2.527	0.389	2.138	3.09	0.65908	0.83	0.127							0.86	1	
43.143	4	Silty Clay to Clay	25.846	1.091	2.564	0.41	2.155	3.08	0.65574	0.82	0.127							0.86	1	
43.799	5	Clayey Silt to Silty Clay	22.171	0.803	2.602	0.43	2.172	3.11	0.65243	0.82	0.127							0.86	1	
44.455	5	Clayey Silt to Silty Clay	19.021	0.622	2.64	0.451	2.189	3.15	0.64916	0.81	0.127							0.85	1	
45.112	4	Silty Clay to Clay	18.366	0.673	2.677	0.471	2.206	3.18	0.64592	0.80	0.127							0.85	1	
45.768	5	Clayey Silt to Silty Clay	18.705	0.662	2.715	0.492	2.223	3.17	0.64271	0.80	0.126							0.85	1	
46.424	4	Silty Clay to Clay	19.409	0.757	2.752	0.512	2.24	3.18	0.63953	0.79	0.126							0.85	1	
47.08	4	Silty Clay to Clay	19.19	0.719	2.79	0.533	2.257	3.18	0.63639	0.78	0.126							0.85	1	
47.736	5	Clayey Silt to Silty Clay	19.398	0.654	2.828	0.553	2.275	3.14	0.63309	0.78	0.125							0.85	1	
48.392	5	Clayey Silt to Silty Clay	20.938	0.629	2.865	0.574	2.292	3.07	0.63001	0.77	0.125							0.85	1	
49.049	6	Sandy Silt to Clayey Silt	47.307	1.038	2.903	0.594	2.309	2.69	0.62696	0.76	0.125							0.85	1	
49.705	5	Clayey Silt to Silty Clay	36.443	1.15	2.94	0.615	2.326	2.9	0.62394	0.76	0.124							0.84	1	
50.361	6	Sandy Silt to Clayey Silt	34.98	0.933	2.978	0.635	2.343	2.86	0.62094	0.75	0.124							0.84	1	
51.017	7	Silty Sand to Sandy Silt	61.044	1.383	3.017	0.655	2.361	2.62	0.6178	0.74	0.123							0.84	1	
51.673	6	Sandy Silt to Clayey Silt	61.191	1.549	3.054	0.676	2.378	2.65	0.61487	0.74	0.123							0.84	1	
52.329	5	Clayey Silt to Silty Clay	46.652	1.906	3.092	0.696	2.395	2.87	0.61196	0.73	0.122							0.84	1	
52.986	5	Clayey Silt to Silty Clay	43.093	1.728	3.129	0.717	2.413	2.9	0.60891	0.72	0.122							0.84	1	
53.642	5	Clayey Silt to Silty Clay	23.897	0.834	3.167	0.737	2.43	3.09	0.60606	0.72	0.121							0.84	1	
54.298	5	Clayey Silt to Silty Clay	23.651	0.662	3.205	0.758	2.447	3.04	0.60324	0.71	0.121							0.84	1	
54.954	5	Clayey Silt to Silty Clay	31.955	1.032	3.242	0.778	2.464	2.96	0.60044	0.70	0.120							0.83	1	
55.61	5	Clayey Silt to Silty Clay	34.718	1.343	3.28	0.799	2.481	2.98	0.59766	0.70	0.120							0.83	1	
56.266	5	Clayey Silt to Silty Clay	32.96	1.346	3.317	0.819	2.498	3.03	0.59492	0.69	0.119							0.83	1	
56.923	5	Clayey Silt to Silty Clay	31.491	1.164	3.355	0.84	2.515	3.02	0.59219	0.69	0.119							0.83	1	
57.579	5	Clayey Silt to Silty Clay	35.618	1.22	3.393	0.86	2.532	2.96	0.5895	0.68	0.118							0.83	1	
58.235	5	Clayey Silt to Silty Clay	32.032	1.211	3.43	0.881	2.549	3.03	0.58682	0.67	0.118							0.83	1	
58.891	5	Clayey Silt to Silty Clay	24.376	0.892	3.468	0.901	2.567	3.14	0.58402	0.67	0.117							0.83	1	
59.547	5	Clayey Silt to Silty Clay	25.524	0.93	3.505	0.922	2.584	3.11	0.5814	0.66	0.117							0.83	1	
60.203	5	Clayey Silt to Silty Clay	23.984	0.816	3.543	0.942	2.601	3.12	0.5788	0.66	0.116							0.83	1	
60.86	5	Clayey Silt to Silty Clay	28.319	0.883	3.581	0.963	2.618	3.02	0.57622	0.65	0.116							0.82	1	
61.516	6	Sandy Silt to Clayey Silt	37.3	1.185	3.618	0.983	2.635	2.92	0.57366	0.65	0.116							0.82	1	
62.172	6	Sandy Silt to Clayey Silt	46.347	1.499	3.656	1.004	2.652	2.85	0.57113	0.64	0.115							0.82	1	
62.828	5	Clayey Silt to Silty Clay	43.235	1.488	3.693	1.024	2.669	2.9	0.56862	0.64	0.115							0.82	1	
63.484	6	Sandy Silt to Clayey Silt	45.309	1.512	3.731	1.044	2.686	2.87	0.56613	0.63	0.114							0.82	1	
64.14	6	Sandy Silt to Clayey Silt	46.62	1.436	3.769	1.065	2.704	2.84	0.56352	0.63	0.114							0.82	1	
64.797	5	Clayey Silt to Silty Clay	36.317	1.437	3.806	1.085	2.721	3.01	0.56108	0.62	0.113							0.82	1	
65.453	6	Sandy Silt to Clayey Silt	32.861	0.991	3.844	1.106	2.738	2.98	0.55866	0.62	0.113							0.82	1	
66.109	6	Sandy Silt to Clayey Silt	35.979	1.125	3.881	1.126	2.755	2.95	0.55626	0.61	0.113							0.82	1	
66.765	5	Clayey Silt to Silty Clay	31.769	1.073	3.919	1.147	2.772	3.03	0.55388	0.61	0.112							0.82	1	
67.421	5	Clayey Silt to Silty Clay	31.928	1.056	3.956	1.167	2.789	3.02	0.55152	0.61	0.112							0.81	1	
68.077	6	Sandy Silt to Clayey Silt	36.432	1.125	3.994	1.188	2.806	2.94	0.54918	0.60	0.112							0.81	1	
68.734	6	Sandy Silt to Clayey Silt	53.018	1.67	4.032	1.208	2.823	2.8	0.54686	0.60	0.111							0.81	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-EBMUD AQUEDUCT UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0226)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-EM-006***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
69.39	3	Clay	49.862	2.717	4.068	1.229	2.84	3	0.54455	0.60	0.111							0.81	1	
70.046	3	Clay	34.384	1.923	4.105	1.249	2.856	3.14	0.54241	0.59	0.111							0.81	1	
70.702	5	Clayey Silt to Silty Clay	24.093	0.966	4.142	1.27	2.873	3.19	0.54014	0.59	0.110							0.81	1	
71.358	6	Sandy Silt to Clayey Silt	46.783	1.455	4.18	1.29	2.89	2.86	0.5379	0.58	0.110							0.81	1	
72.014	5	Clayey Silt to Silty Clay	59.204	2.553	4.218	1.311	2.907	2.88	0.53567	0.58	0.110							0.81	1	
72.671	5	Clayey Silt to Silty Clay	37.207	1.513	4.255	1.331	2.924	3.04	0.53346	0.58	0.109							0.81	1	
73.327	5	Clayey Silt to Silty Clay	32.337	1.075	4.293	1.352	2.941	3.04	0.53127	0.58	0.109							0.81	1	
73.983	5	Clayey Silt to Silty Clay	30.716	1.019	4.33	1.372	2.958	3.06	0.5291	0.57	0.109							0.81	1	
74.639	5	Clayey Silt to Silty Clay	28.308	0.898	4.368	1.393	2.975	3.09	0.52695	0.57	0.109							0.80	1	
75.295	6	Sandy Silt to Clayey Silt	33.14	1.026	4.406	1.413	2.993	3.01	0.52468	0.57	0.108							0.80	1	
75.951	6	Sandy Silt to Clayey Silt	43.06	1.331	4.443	1.433	3.01	2.91	0.52257	0.56	0.108							0.80	1	
76.608	6	Sandy Silt to Clayey Silt	53.553	1.831	4.481	1.454	3.027	2.86	0.52046	0.56	0.108							0.80	1	
77.264	5	Clayey Silt to Silty Clay	50.491	2.26	4.518	1.474	3.044	2.97	0.51838	0.56	0.108							0.80	1	
77.92	3	Clay	30.989	1.511	4.555	1.495	3.06	3.21	0.51643	0.56	0.108							0.80	1	
78.576	7	Silty Sand to Sandy Silt	67.579	1.251	4.594	1.515	3.078	2.63	0.51426	0.55	0.107							0.80	1	
79.232	6	Sandy Silt to Clayey Silt	63.714	1.942	4.631	1.536	3.095	2.79	0.51222	0.55	0.107							0.80	1	
79.888	5	Clayey Silt to Silty Clay	31.377	1.234	4.669	1.556	3.112	3.15	0.5102	0.55	0.107							0.80	1	
80.545	5	Clayey Silt to Silty Clay	18.552	0.604	4.706	1.577	3.13	3.33	0.50808	0.55	0.107							0.80	1	
81.201	5	Clayey Silt to Silty Clay	15.38	0.43	4.744	1.597	3.147	3.38	0.5061	0.54	0.107							0.80	1	
81.857	5	Clayey Silt to Silty Clay	13.349	0.334	4.781	1.618	3.164	3.42	0.50412	0.54	0.106							0.79	1	
82.513	5	Clayey Silt to Silty Clay	13.349	0.307	4.819	1.638	3.181	3.4	0.50217	0.54	0.106							0.79	1	
83.169	6	Sandy Silt to Clayey Silt	13.916	0.291	4.857	1.659	3.198	3.36	0.50023	0.54	0.106							0.79	1	
83.825	6	Sandy Silt to Clayey Silt	26.681	0.752	4.894	1.679	3.215	3.13	0.4983	0.54	0.106							0.79	1	
84.482	5	Clayey Silt to Silty Clay	36.891	1.432	4.932	1.7	3.232	3.09	0.49639	0.53	0.106							0.79	1	
85.138	5	Clayey Silt to Silty Clay	40.215	1.447	4.969	1.72	3.249	3.02	0.49449	0.53	0.106							0.79	1	
85.794	6	Sandy Silt to Clayey Silt	68.578	2.561	5.007	1.741	3.267	2.83	0.4925	0.53	0.106							0.79	1	
86.45	6	Sandy Silt to Clayey Silt	93.72	3.284	5.045	1.761	3.284	2.7	0.49063	0.53	0.106							0.79	1	
87.106	6	Sandy Silt to Clayey Silt	88.986	3.122	5.082	1.781	3.301	2.73	0.48878	0.53	0.105							0.79	1	
87.762	5	Clayey Silt to Silty Clay	76.975	2.982	5.12	1.802	3.318	2.81	0.48694	0.52	0.105							0.79	1	
88.419	6	Sandy Silt to Clayey Silt	87.015	3.075	5.157	1.822	3.335	2.74	0.48512	0.52	0.105							0.79	1	
89.075	5	Clayey Silt to Silty Clay	68.616	2.59	5.195	1.843	3.352	2.85	0.4833	0.52	0.105							0.79	1	
89.731	6	Sandy Silt to Clayey Silt	56.098	1.963	5.233	1.863	3.369	2.91	0.48151	0.52	0.105							0.78	1	
90.387	6	Sandy Silt to Clayey Silt	46.33	1.455	5.27	1.884	3.386	2.95	0.47972	0.52	0.105							0.78	1	
91.043	6	Sandy Silt to Clayey Silt	85.552	2.654	5.308	1.904	3.403	2.71	0.47795	0.52	0.105							0.78	1	
91.699	6	Sandy Silt to Clayey Silt	89.947	3.125	5.345	1.925	3.421	2.73	0.47609	0.52	0.105							0.78	1	
92.356	6	Sandy Silt to Clayey Silt	76.795	2.734	5.383	1.945	3.438	2.8	0.47434	0.51	0.105							0.78	1	
93.012	5	Clayey Silt to Silty Clay	51.981	1.93	5.421	1.966	3.455	2.97	0.47261	0.51	0.105							0.78	1	
93.668	6	Sandy Silt to Clayey Silt	57.555	1.791	5.458	1.986	3.472	2.87	0.47089	0.51	0.104							0.78	1	
94.324	8	Sand to Silty Sand	149.053	2.223	5.498	2.007	3.491	2.33	0.46898	0.51	0.104	2.330	0.47	69.90	2.049	143.214	0.35	0.77	1	4.04
94.98	7	Silty Sand to Sandy Silt	138.056	3.756	5.536	2.027	3.509	2.52	0.46719	0.51	0.104	2.520	0.47	64.50	2.872	185.215		0.78	1	
95.636	8	Sand to Silty Sand	164.624	2.184	5.576	2.048	3.529	2.27	0.46521	0.51	0.104	2.270	0.47	76.59	1.856	142.149	0.35	0.76	1	3.98
96.293	9	Sand	260.707	2.509	5.617	2.068	3.549	2.02	0.46326	0.51	0.104	2.020	0.46	120.77	1.327	160.269		0.70	1	
96.949	9	Sand	408.51	4.425	5.658	2.089	3.569	1.9	0.46131	0.50	0.104	1.900	0.46	188.45	1.189	224.038		0.63	1	
97.605	9	Sand	376.314	3.83	5.698	2.109	3.589	1.91	0.45939	0.50	0.104	1.910	0.46	172.87	1.198	207.172		0.64	1	
98.261	10	Gravelly Sand to Sand	408.313	2.838	5.74	2.13	3.611	1.78	0.45729	0.50	0.104	1.780	0.46	186.72	1.092	203.958		0.62	1	
98.917	10	Gravelly Sand to Sand	346.303	2.436	5.782	2.15	3.632	1.84	0.4553	0.50	0.104	1.840	0.46	157.67	1.137	179.277		0.64	1	
99.573	10	Gravelly Sand to Sand	408.98	3.533	5.824	2.17	3.653	1.84	0.45333	0.50	0.103	1.840	0.45	185.40	1.137	210.808		0.62	1	
100.23	9	Sand	336.962	3.347	5.864	2.191	3.673	1.95	0.45147	0.50	0.103	1.950	0.45	152.13	1.240	188.617		0.77	2	
100.886	7	Silty Sand to Sandy Silt	127.247	3.141	5.903	2.211	3.692	2.54	0.44971	0.50	0.103	2.540	0.45	57.22	2.979	170.471		0.77	3	
101.542	6	Sandy Silt to Clayey Silt	68.938	2.348	5.941	2.232	3.709	2.85	0.44816	0.50	0.103							0.77	4	
102.198	6	Sandy Silt to Clayey Silt	88.746	2.687	5.978	2.252	3.726	2.73	0.44661	0.49	0.103							0.77	5	
102.854	9	Sand	164.307	1.643	6.019	2.273	3.746	2.22	0.4448	0.49	0.103	2.220	0.44	73.08	1.717	125.499	0.26	0.77	6	3.05
103.511	9	Sand	201.678	1.255	6.06	2.293	3.766	2.03	0.44301	0.49	0.103	2.030	0.44	89.35	1.341	119.831	0.24	0.77	7	2.78
104.167	9	Sand	179.72	1.191	6.1	2.314	3.787	2.09	0.44115	0.49	0.103	2.090	0.44	79.28	1.437	113.923	0.22	0.77	8	2.52
104.823	9	Sand	144.849	1.323	6.141	2.334	3.807	2.25	0.43938	0.49	0.103	2.250	0.44	63.64	1.798	114.449	0.22	0.77	9	2.55
105.479	6	Sandy Silt to Clayey Silt	77.444	2.399	6.179	2.355	3.824	2.8	0.4379	0.49	0.103							0.76	10	
106.135	6	Sandy Silt to Clayey Silt	60.782	2.122	6.216	2.375	3.841	2.91	0.43642	0.49	0.103							0.76	11	
106.791	6	Sandy Silt to Clayey Silt	58.986	2.066	6.254	2.396	3.858	2.93	0.43495	0.49	0.103							0.76	12	
107.448	6	Sandy Silt to Clayey Silt	76.789	2.496	6.291	2.416	3.875	2.81	0.4335	0.49	0.103							0.76	13	
108.104	5	Clayey Silt to Silty Clay	63.473	2.509	6.329	2.437	3.892	2.94	0.43205	0.49	0.103							0.76	14	
108.76	6	Sandy Silt to Clayey Silt	60.28	1.814	6.367	2.457	3.91	2.88	0.43053	0.48	0.103							0.76	15	
109.416	6	Sandy Silt to Clayey Silt	85.083	2.757	6.404	2.478	3.927	2.77	0.4291	0.48	0.102							0.76	16	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. The result was used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The result of the test is presented on Plate B-2, "Plasticity Chart".

Grain Size Classification

Grain size classification test (ASTM Test Method D 420) was performed on selected samples of granular soil to aid in the classification. The result is presented on Plate B-3, "Grain Size Distribution Curves".

Unconfined Compression Tests

Strength test was performed on selected undisturbed sample using unconfined compression machine. Unconfined compression test was performed in general accordance with ASTM Test Method D 2166-00. The result is presented on Plate B-6.

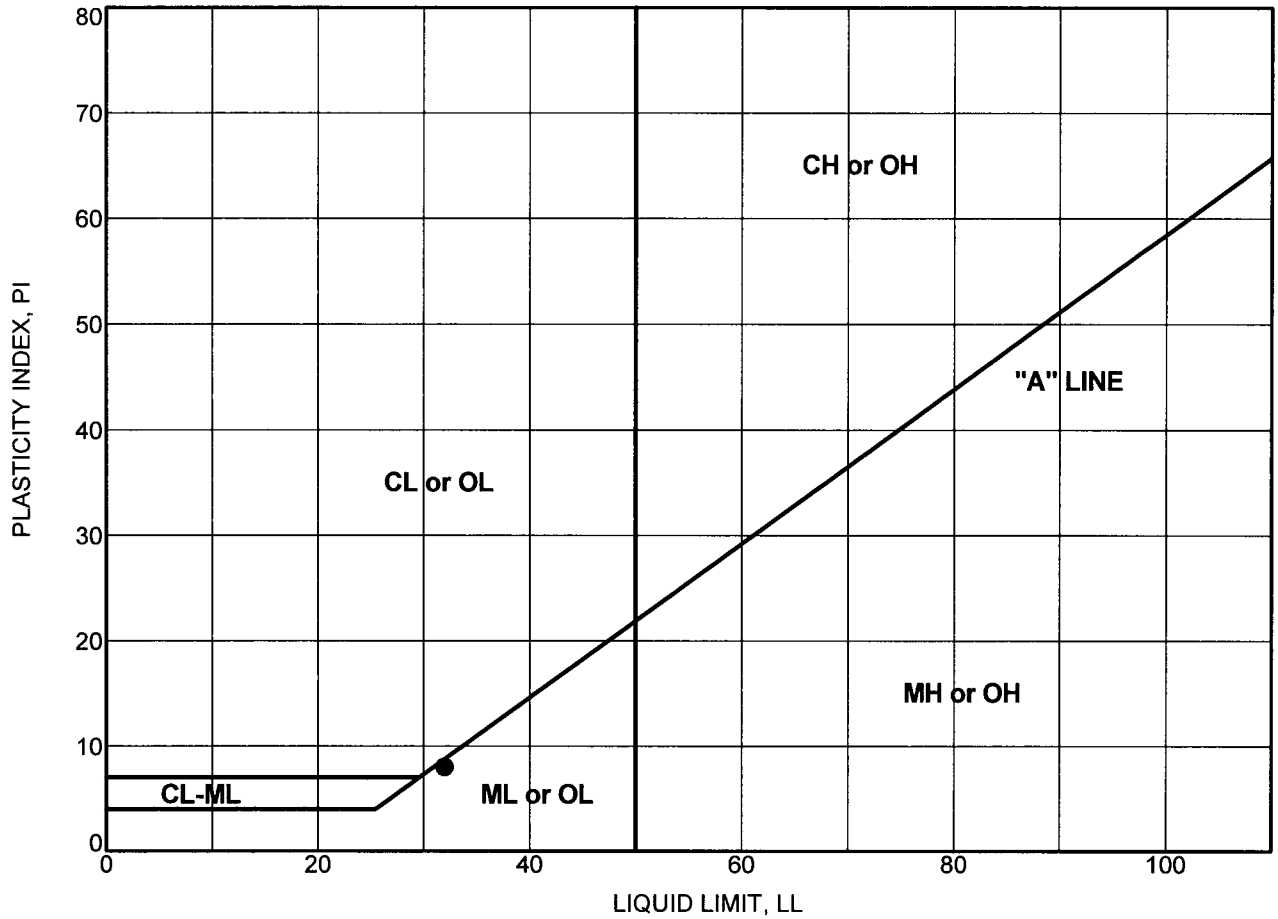
Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-4.

LABORATORY TESTS
(Continued)

Direct Shear Tests

Direct Shear test was performed on selected relatively undisturbed sample to determine the shear strength of a soil material in direct shear. The test was performed according to ASTM Test Method D 3080 by Cooper . The test result is presented on Plate B-5.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A-09-001	MC-10	49.5	●	31	32	24	8	COHESIONLESS SILT (ML)

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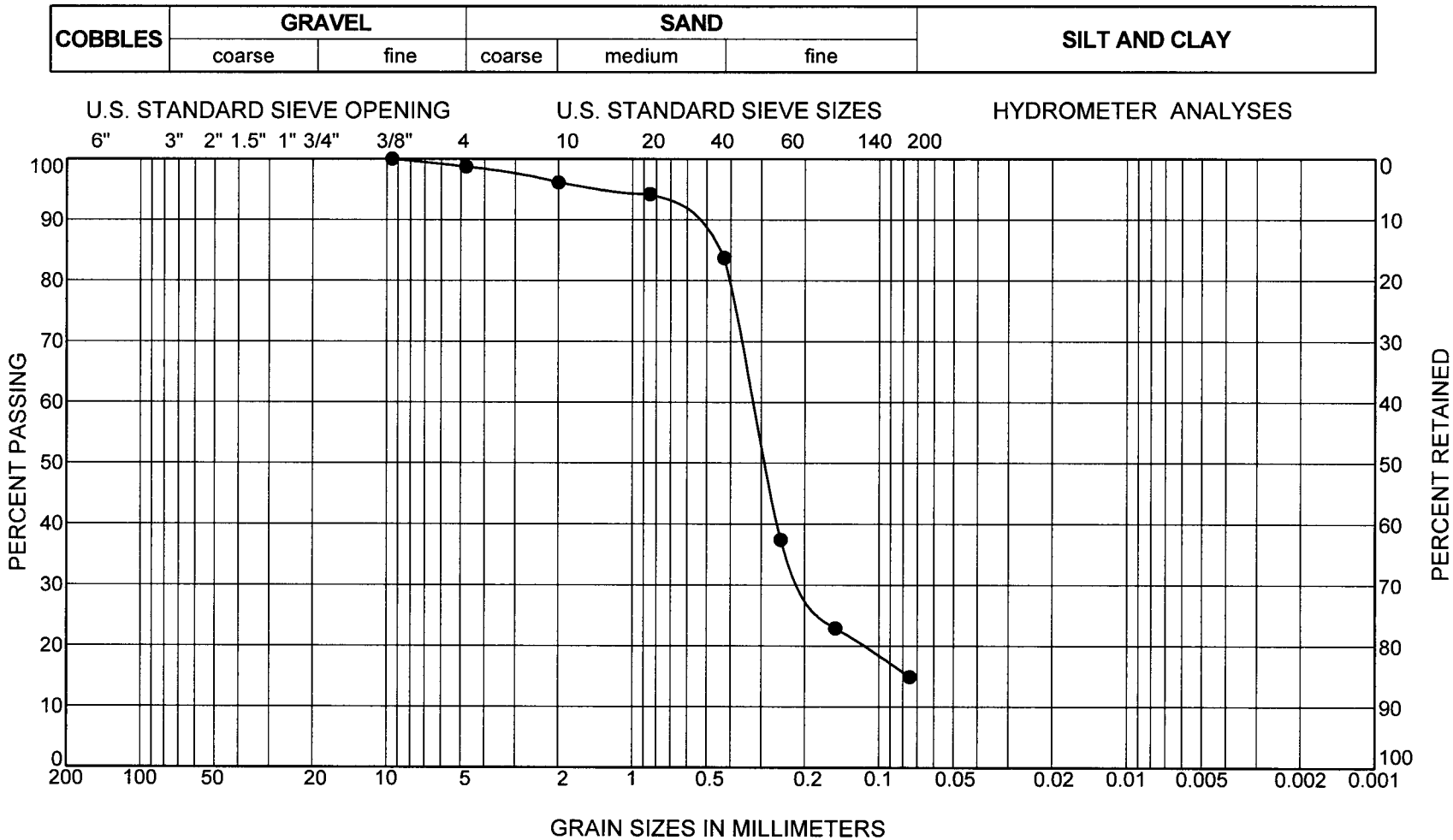
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

EBMUD Aqueduct UC (Widen)
Stockton, California

JOB NO: 206127.BR (MAQ)

PLATE NO: B-2

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A-09-001	SPT-9	44.5	●			SILTY SAND (SM)

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MATERIALS ENGINEERING

JOB NO: 206127.BR (MAQ)

EBMUD Aqueduct UC (Widen)
Stockton, California

PLATE NO: B-3

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/29/2009
Date Submitted 04/24/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR Site ID : A-09-EM-001#6@29.5'
Thank you for your business.

* For future reference to this analysis please use SUN # 55554-111857.

EVALUATION FOR SOIL CORROSION

Soil pH	6.87	
Minimum Resistivity	1.10 ohm-cm (x1000)	
Chloride	53.2 ppm	00.00532 %
Sulfate	96.6 ppm	00.00966 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

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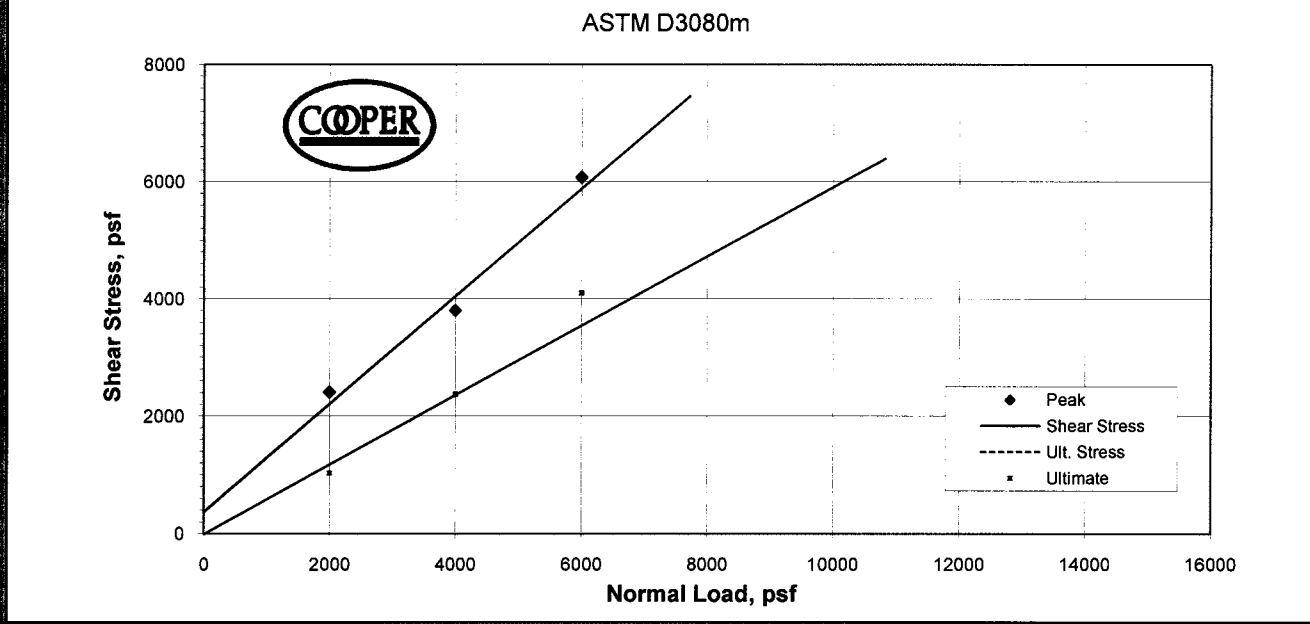
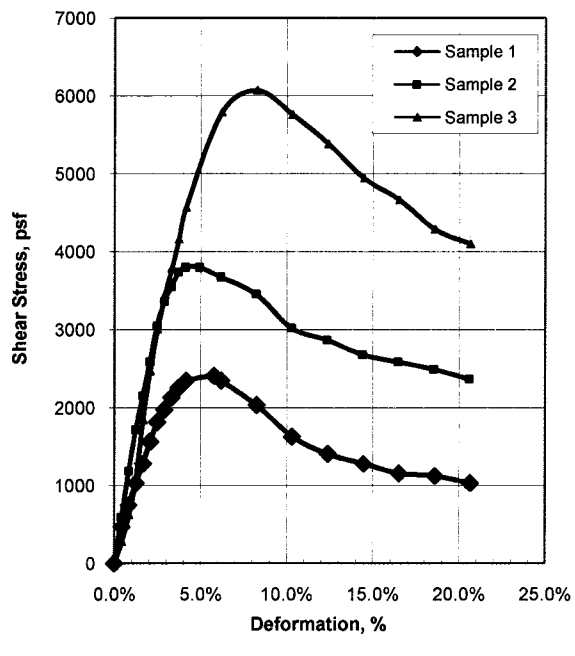
By

Direct Shear



P. Phi (degrees)	40.0	Ult. Phi (degrees)	28.5
P. Cohesion(psf)	400	Ult. Cohesion (psf)	0



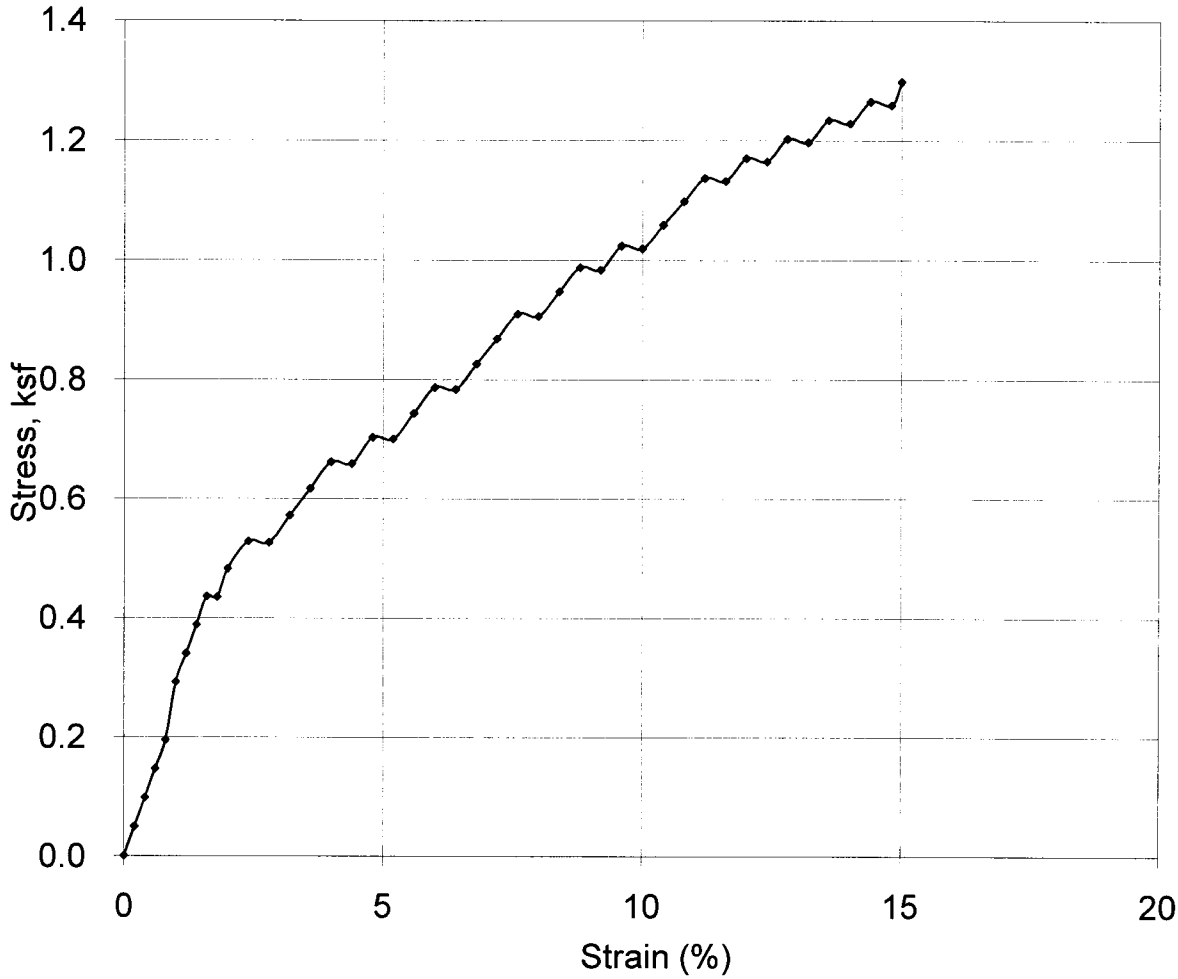
	Sample Data: Initial			
	1	2	3	4
Moisture %	25.8	22.6	23.4	
Dry Dens., pcf	97.5	102.0	99.3	
Void Ratio	0.729	0.652	0.697	
Saturation %	95.7	93.7	90.7	
Diameter	2.42	2.43	2.42	
Height	1.00	1.00	1.00	
	Sample Data: At Test			
Moisture %	25.2	22.6	22.8	
Dry Dens., pcf	99.1	104.6	102.4	
Void Ratio	0.701	0.611	0.646	
Saturation %	97.2	100.0	95.2	
Diameter	2.42	2.43	2.42	
Height	0.98	0.98	0.97	
Normal Stress, psf	2000	4000	6000	
Shear Stress, psf	2411	3804	6074	
Strengths picked at	5.8%	4.9%	8.3%	
Ult. Stress, psf	1033	2370	4101	
Strain Rate, in/min	0.020	0.020	0.020	

CTL #	157-272
Client:	Parikh Consultants, Inc.
Project	206127.GDR
Tested By:	MD
Reduced By:	RU
Date:	4/20/2009

Specimen	Boring	Sample	Depth, ft	Visual Soil Classification
1	A-09-EM-001	#7	34.5	Greenish Gray Silty SAND
2	A-09-EM-001	#7	34.5	Greenish Gray Silty SAND
3	A-09-EM-001	#7	34.5	Greenish Gray Silty SAND
4				

Remarks: *DS-CU* A fully undrained condition may not be attained in this test.

UNCONFINED COMPRESSION TEST



Boring No.: A-09-EM-001

Sample No. : MC-6

Depth (feet): 29.5

Material Description:
Sandy Lean Clay

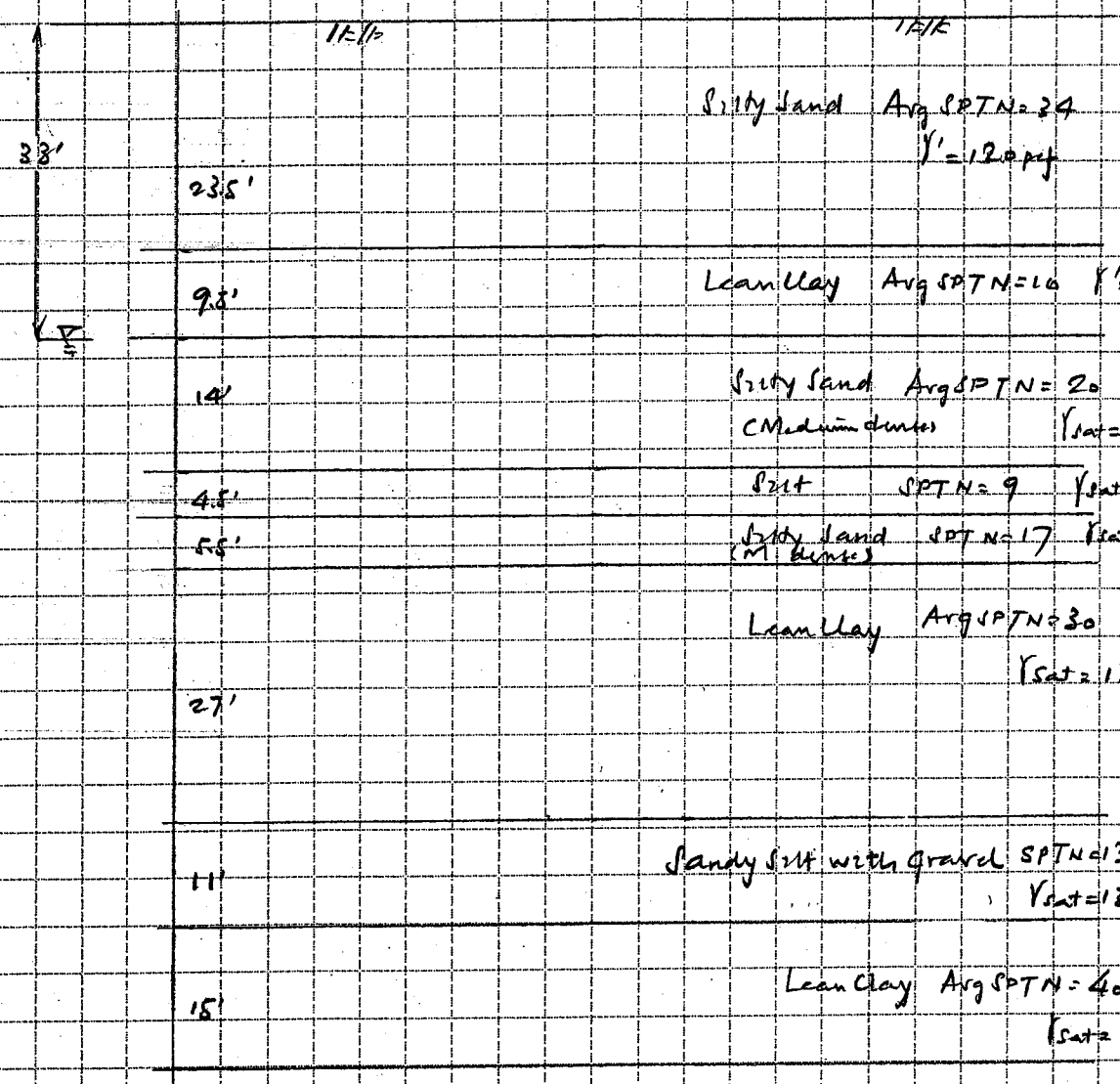
Unconfined Compressive Strength: 1.3 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**EDMUD AQUEDUCT. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-6

APPENDIX C

Evaluation of Liquefaction Potential

Evaluation of Liquefaction Potential - EBMUD Aqueduct use (Boring A-09-EM-001)



LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-EM-001 (EBMUD Aqueduct UC)(Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 33

SOIL GROUPS

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR$_{7.0}$/CSR)*MSF*Kσ*Kα*

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	23.5	1	34	SPT	2820.0	2820.0	0.89	0.12	39	0.84	1	0.95	1.2	1.15	43.31	15%	47.89		0.90	1	
2	33.0	2	10	SPT	3912.5	3912.5	0.82		12	0.71	1	0.80	1.2	1.15	9.11	55%			0.82	1	
3	47.0	1	20	SPT	5732.5	4857.5	0.68	0.10	23	0.64	1	0.80	1.2	1.15	16.35	15%	19.63	0.21	0.77	1	1.84
4	51.5	2	9	SPT	6295.0	5138.8	0.64		10	0.62	1	0.80	1.2	1.15	7.15	55%			0.75	1	
5	57.0	1	17	SPT	7010.0	5510.0	0.60	0.10	20	0.60	1	0.80	1.2	1.15	13.05	29%	19.59	0.21	0.74	1	1.87
6	84.0	2	30	SPT	10520.0	7332.5	0.49		35	0.52	1	0.80	1.2	1.15	19.96	55%			0.68	1	
7	95.0	2	13	SPT	11950.0	8075.0	0.47		15	0.50	1	0.80	1.2	1.15	8.24	55%			0.66	1	
8	110.0	2	40	SPT	13900.0	9087.5	0.46		46	0.47	1	0.80	1.2	1.15	23.90	55%			0.64	1	

Vertical Pile Capacity Calculations

PROJECT: EBMUD UC (Interior Widening)
BRIDGE NO: 29-0226 R/L

FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size (ft)		Permissible Settlement (in.).	No. of Piles per Support Per Bridge
					B	L		
Abutment 1	WSD	24" Dia CIDH	19.0	17.0	2.5	53.67	1"	6
Bent 2	LRFD	48" Dia CIDH	5.8	3.8	N/A	N/A	1"	3
Bent 3	LRFD	48" Dia CIDH	4.5	2.5	N/A	N/A	1"	3
Bent 4	LRFD	48" Dia CIDH	5.5	3.5	N/A	N/A	1"	3
Abutment 5	WSD	24" Dia CIDH	20.5	18.5	2.5	53.67	1"	6

FOUNDATION LOADS

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	470	160	310	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	1580	670	1190	2160	1070	0	0	1190	690	0	0
Bent 3	1930	790	1530	2640	1290	0	0	1530	690	0	0
Bent 4	1580	670	1190	2160	1070	0	0	1190	690	0	0
Abut 5	470	160	310	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

(1) Total Load for Service-I Limit State @ Abutments 1 & 5 = DL + LL (HL93) + Earth

(2) Strength Limit State = Assumes LRFD at Bents 2, 3 & 4 using 1.25DC + 1.50DW + 1.75LL (HL-93) or 1.35LL (P-15), WHICHEVER CONTROLS

(3) Extreme Limit State = DL + Seismic

(4) 'Per Support' Columns are the total reactions at each Abut or Bent cap obtained from VBridge.

(5) 'Per Pile' Columns are the maximum reactions for any column within a bent cap obtained from VBent.

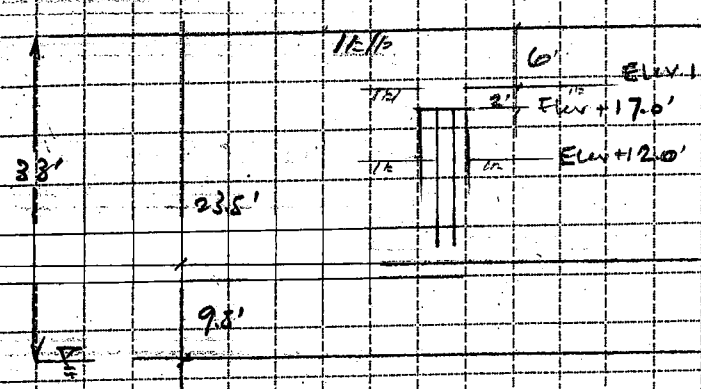
(6) For HL-93 loading, 4 lanes of traffic were used with a Multiple Presence Factor of 0.65

(7) For CA P-15 loading, a maximum of 2 traffic lanes were used.

Pile Capacity Analysis : EBMUD Aqueduct Use (Boring A-09-006) Abutment 1

2' ø CIDH Concrete pile - 71.4 tons

Strength Limit State



Silty Sand Avg SPT N = 24
$\gamma' = 120$ pcf
$\phi = 36°$

9.5' Lean Clay Avg SPT N = 10 $\gamma' = 115$ pcf
c = 1250 psf

14' Silty Sand Avg SPT N = 20
$\phi' = 32°$ (Medium dense) $\gamma_{sat} = 130$ pcf

c = 1125 psf Silt SPT N = 9 $\gamma_{sat} = 125$ pcf

45' $\phi' = 31°$ Silty Sand (M Dense) SPT N = 17 $\gamma_{sat} = 130$ pcf

55' Lean Clay Avg SPT N = 30
$\gamma_{sat} = 130$ pcf

27' c = 3500 psf

Sandy Silt with Gravel SPT N = 13
c = 1500 psf, $\gamma_{sat} = 130$ pcf

11' Lean Clay Avg SPT N = 40

15' c = 4000 psf $\gamma_{sat} = 130$ pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

I-5 EBMUD Aqueduct UC (Widen) Abutment 1

206127.EBD

ALLOWABLE PILE CAPACTY 01/11/10

Abut 1 (Based on Boring A-09-EM-001)

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	1	SAND	120	240	120.0	14	DENSE			37	1.20	0.00	0.00	0.00	0.00
2.0	7.0	1	SAND	120	600	540.0	14	DENSE			37	1.13	0.00	0.00	0.00	0.00
7.0	12.0	1	SAND	120	600	1140.0	14	DENSE			36	1.01	17.20	17.20	8.60	8.60
12.0	17.5	1	SAND	120	660	1770.0	15	DENSE			36	0.98	28.50	45.70	14.25	22.85
17.5	22.5	2	CLAY	115	575	2387.5	10	STIFF	0.55	1250			10.25	55.96	7.18	30.03
22.5	27.0	2	CLAY	115	517.5	2933.8	10	STIFF	0.55	1250			9.23	65.19	6.46	36.49
27.0	32.0	1	SAND	68	340	3362.5	10	MED DENSE			33	0.51	25.64	90.82	12.82	49.31
32.0	37.0	1	SAND	68	340	3702.5	10	MED DENSE			33	0.47	26.03	116.85	13.02	62.32
37.0	41.0	1	SAND	68	272	4008.5	10	MED DENSE			33	0.44	20.95	137.80	10.47	72.80
41.0	45.5	2	CLAY	63	283.5	4286.3	9	STIFF	0.55	1125			8.31	146.11	5.81	78.61
45.5	49.0	1	SAND	68	238	4547.0	17	MED DENSE			34	0.57	27.16	173.26	13.58	92.19
49.0	56.0	1	SAND	68	476	4904.0	17	MED DENSE			34	0.52	53.44	226.70	26.72	118.91
56.0	61.0	2	CLAY	68	340	5312.0	30	V.STIFF	0.51	3750			28.66	255.36	20.07	138.97
61.0	66.0	2	CLAY	68	340	5652.0	30	V.STIFF	0.51	3750			28.66	284.03	20.07	159.04
66.0	72.0	2	CLAY	68	408	6026.0	30	V.STIFF	0.51	3750			34.40	318.42	24.08	183.12
72.0	78.0	2	CLAY	68	408	6434.0	30	V.STIFF	0.51	3750			34.40	352.82	24.08	207.19
78.0	83.5	2	CLAY	68	374	6825.0	13	STIFF	0.55	1625			14.66	367.49	10.26	217.46
83.5	89.0	2	CLAY	68	374	7199.0	13	STIFF	0.55	1625			14.66	382.15	10.26	227.72
89.0	94.0	2	CLAY	68	340	7556.0	40	STIFF	0.45	5000			33.56	415.71	23.49	251.21
94.0	99.0	2	CLAY	68	340	7896.0	40	STIFF	0.45	5000			33.56	449.27	23.49	274.70
99.0	104.0	2	CLAY	68	340	8236.0	40	STIFF	0.45	5000			33.56	482.83	23.49	298.20

Abutment 1 --: Pile cut-off elevation is +17.0 feet. Finished grade elevation is +19.0 feet
Pile Length @ Abutment 1 = 49.0' - 2.0' = 47.0 '
160 kips/pile (Service Load). Required pile capacity (comp) = 160 kips /(0.5) = 320 kips (160 tons)
Pile tip at 47.0' below pile cut-off (pile tip @ Elev. -30.0', pile cut-off @ Elev. +17.0'), Ult. Compression Capacity = 173.3 tons > 160 tons (Service-I Limit State) OK

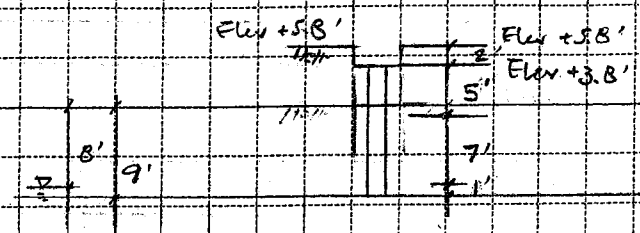
Sheet 2 of 10

PARIKH
Practicing in the Geosciences

JOB I-5 North Stockton Interchange Improvement - EBMUD Aqueduct Ue (Widen)
SHEET NO 3 OF 10
CALCULATED BY A Lam DATE 10/09
CHECKED BY DATE
SCALE

Pile Capacity Analysis - EBMUD Aqueduct Ue (Widen) & As-built Boring B-4) Bent 2

4' ⌀ CIDH Concrete pile
753.6 tons Strength Limit State



Elev. +0.0'

Clay (very soft) SPT N = 5 $\gamma = 100$ pcf
c = 625 psf $\gamma_{sat} = 110$ pcf

Sand Avg SPT N = 16
$\phi' = 34°$ $\gamma_{sat} = 130$ pcf
(Medium dense)

22'
Dr = 70%

Sand SPT N = 31 $\gamma_{sat} = 130$ pcf
$\phi' = 36°$ (dense)

8'

Sand Avg SPT N = 23.5 (medium dense)
$\gamma_{sat} = 130$ pcf
$\phi' = 33°$

37'

Clay SPT N = 33 $\gamma_{sat} = 130$ pcf
c = 3750 psf

3'

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 EBMUD Aqueduct UC (Widen) Bent 2
206127.EBD

ALLOWABLE PILE CAPACTY 01/10/10

Bent 2 (Based on As-built Boring B-4)

PILE DIA, B (ft) = 4.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 12.56 SQ. FT.
AREA OF THE PILE TIP 12.56 SQ. FT.

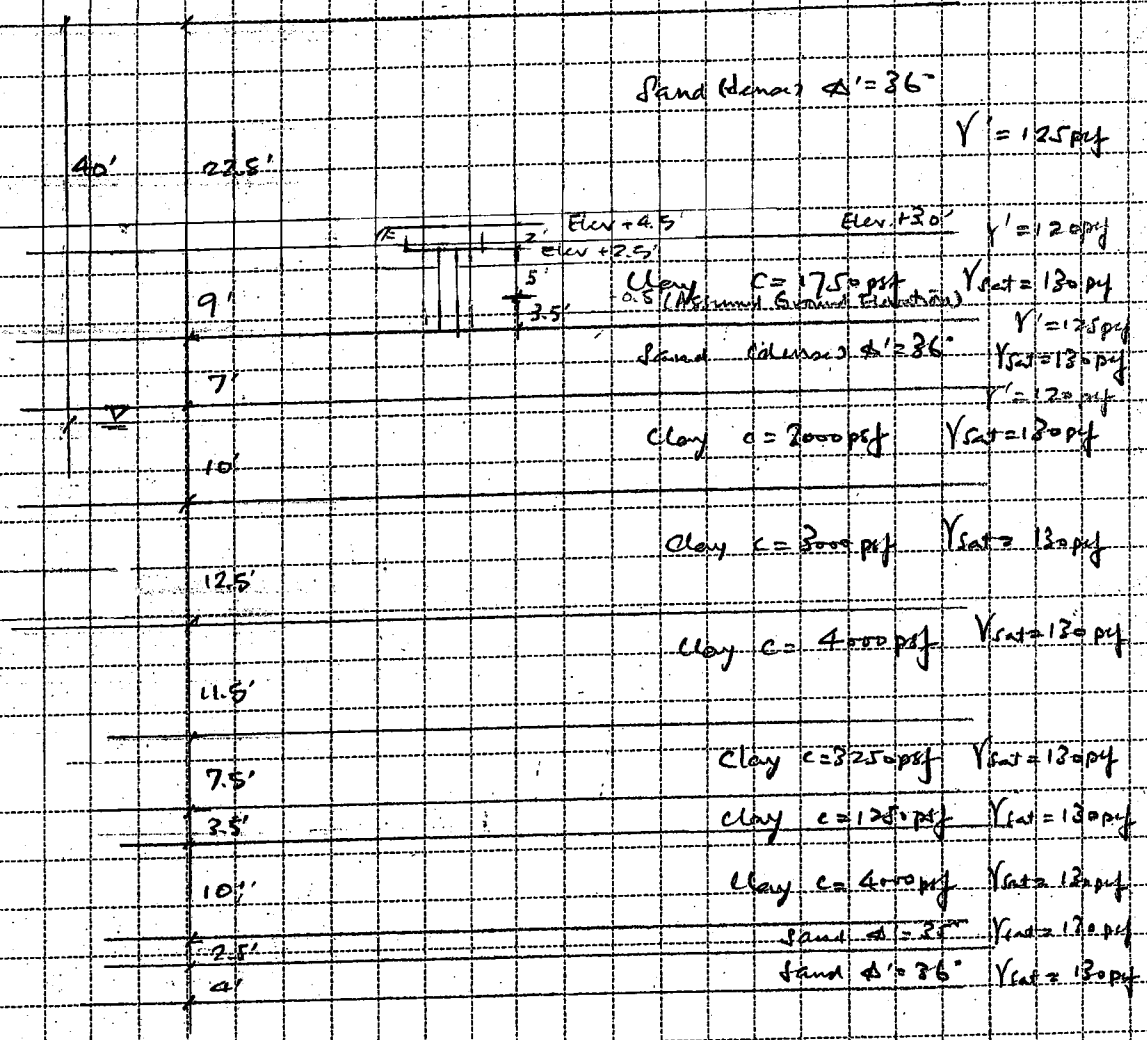
DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	2	CLAY	100	200	100.0	5	MEDIUM	0.55	625			0.00	0.00	0.00	0.00
2.0	7.0	2	CLAY	100	500	450.0	5	MEDIUM	0.55	625			0.00	0.00	0.00	0.00
7.0	14.0	2	CLAY	100	700	1050.0	5	MEDIUM	0.55	625			14.36	14.36	10.05	10.05
14.0	15.0	2	CLAY	48	48	1424.0	5	MEDIUM	0.55	625			2.05	16.41	1.44	11.48
15.0	20.0	1	SAND	68	340	1618.0	16	DENSE			36	0.94	45.14	61.55	22.57	34.05
20.0	26.0	1	SAND	68	408	1992.0	16	DENSE			36	0.85	60.79	122.34	30.40	64.45
26.0	32.0	1	SAND	68	408	2400.0	16	MED DENSE			35	0.77	66.41	188.75 ←	33.20	97.66
32.0	37.0	1	SAND	68	340	2774.0	16	MED DENSE			35	0.71	58.51	247.26	29.25	126.91
37.0	41.0	1	SAND	68	272	3080.0	31	DENSE			38	0.66	48.28	295.54	24.14	151.05
41.0	45.0	1	SAND	68	272	3352.0	31	DENSE			38	0.61	49.17	344.72	24.59	175.64
45.0	50.0	1	SAND	68	340	3658.0	24	DENSE			36	0.57	62.15	406.87	31.08	206.71
50.0	55.0	1	SAND	68	340	3998.0	24	DENSE			36	0.52	62.23	469.10	31.12	237.83
55.0	60.0	1	SAND	68	340	4338.0	24	DENSE			36	0.48	61.64	530.74	30.82	268.65
60.0	65.0	1	SAND	68	340	4678.0	24	DENSE			36	0.43	60.39	591.12	30.19	298.84
65.0	68.0	1	SAND	68	204	4950.0	24	MED DENSE			35	0.40	35.36	626.48 ←	17.68	316.52
68.0	76.0	1	SAND	68	544	5324.0	24	MED DENSE			35	0.35	90.08	716.56	45.04	361.56
76.0	82.0	1	SAND	68	408	5800.0	24	MED DENSE			35	0.30	62.30	778.86	31.15	392.71
82.0	85.0	2	CLAY	68	204	6106.0	33	STIFF	0.49	4125			36.45	815.31	25.52	418.23

Bent 2 Pile cut-off elevation is +3.8 feet. Finished grade elevation is +5.8 feet
1070 kips/pile (Strength Limit State). Required pile capacity (comp)= 1070 kips /(0.7) = 1530 kips (765 tons).
Pile Length @ Bent 2 = 68.0'-2.0' = 66.0' (Strength Limit Comp), = 32.0'-2.0'= 30.0' (Extreme Limit State).
Pile tip at 66.0'(comp) below pile cut-off(pile tip@Elev.-62.5', pile cut-off @ Elev. +3.8.'),Ult. Pile Capacity =626.5 T (friction)+177.1 T (end bearing)(0.6 Nspt x area)= 803.6 T > required pile capacity (1530k/765 tons), OK
Pile tip at 30.0'(comp) below pile cut-off(pile tip@Elev.-26.5', pile cut-off @ Elev. +3.8.'),Ult. Pile Capacity =188.8 T (friction)+177.1 T (end bearing)(0.6 Nspt x area)= 365.9 T > required pile capacity (690k/345 tons), OK

Sheet 4 of 10

I-5 EBMUD Aqueduct UC Bent 2 4' Dia CIDH

Pile Capacity Analysis - EBMUD Aqueduct 44 (Widen) (CPT-09-001) Bent 3 4' ø CIDM concrete pile - 925 tons strength Limit State



CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 EBMUD Aqueduct UC (Widen) Bent 3
206127.MAQ

Bent 3 (Based on CPT-09-EB-001)

ALLOWABLE PILE CAPACTY 01/11/10

PILE DIA, B (ft) = 4.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

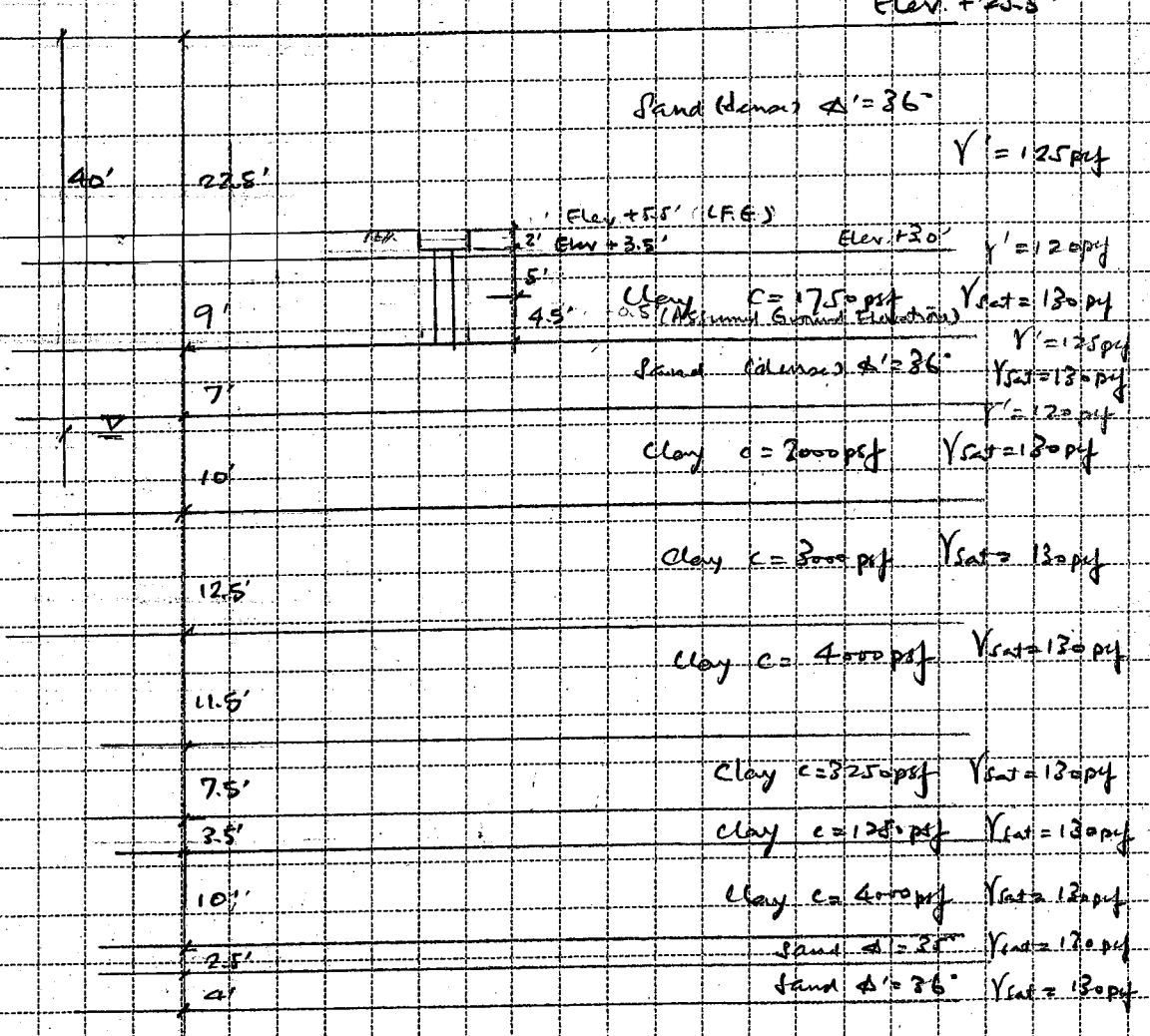
PERIMETER OF THE PILE (ft) = 12.56 SQ. FT.
AREA OF THE PILE TIP 12.56 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	1.5	1	SAND	125	187.5	93.8	12	DENSE			37	1.11	0.00	0.00	0.00	0.00
1.5	7.0	2	CLAY	120	660	517.5	14	STIFF	0.55	1750			0.00	0.00	0.00	0.00
7.0	10.5	2	CLAY	120	420	1057.5	14	STIFF	0.55	1750			20.10	20.10	14.07	14.07
10.5	14.0	1	SAND	125	437.5	1486.3	16	DENSE			36	1.03	31.89	51.99	15.94	30.01
14.0	17.5	1	SAND	125	437.5	1923.8	16	DENSE			36	0.96	38.73	90.72	19.37	49.38
17.5	19.0	2	CLAY	120	180	2232.5	16	STIFF	0.55	2000			9.84	100.56	6.89	56.27
19.0	23.5	2	CLAY	68	306	2475.5	16	STIFF	0.55	2000			29.53	130.09	20.67	76.94
23.5	27.5	2	CLAY	68	272	2764.5	16	STIFF	0.55	2000			26.25	156.34	18.38	95.32
27.5	31.5	2	CLAY	68	272	3036.5	24	V.STIFF	0.55	3000			39.38	195.72 ←	27.56	122.88
31.5	35.5	2	CLAY	68	272	3308.5	24	V.STIFF	0.55	3000			39.38	235.10	27.56	150.44
35.5	40.0	2	CLAY	68	306	3597.5	24	V.STIFF	0.55	3000			44.30	279.39	31.01	181.45
40.0	46.0	2	CLAY	68	408	3954.5	32	V.STIFF	0.50	4000			71.59	350.99	50.11	231.57
46.0	51.5	2	CLAY	68	374	4345.5	32	V.STIFF	0.50	4000			65.63	416.61	45.94	277.50
51.5	55.5	2	CLAY	68	272	4668.5	26	V.STIFF	0.54	3250			41.69	458.30	29.18	306.69
55.5	59.0	2	CLAY	68	238	4923.5	26	V.STIFF	0.54	3250			36.48	494.78	25.53	332.22
59.0	62.5	2	CLAY	68	238	5161.5	10	STIFF	0.55	1250			14.36	509.13	10.05	342.27
62.5	67.5	2	CLAY	68	340	5450.5	32	V.STIFF	0.50	4000			59.66	568.79	41.76	384.03
67.5	72.5	2	CLAY	68	340	5790.5	32	V.STIFF	0.50	4000			59.66	628.45	41.76	425.79
72.5	75.0	1	SAND	68	170	6045.5	26	MED DENSE			35	0.34	30.72	659.17 ←	15.36	441.15
75.0	79.0	1	SAND	68	272	6266.5	36	DENSE			37	0.32	47.16	706.33	23.58	464.73

Bent 3 Pile cut-off elevation is +2.5 feet. Finished grade elevation is +4.5 feet
1290 kips/pile (Strength Limit State). Required pile capacity (comp)= 1290 kips /(0.7) = 1850 kips (925 tons).
Pile Length @ Bent 3 = 75.0' - 2.0' = 73.0 ' (Strength Limit Comp), = 31.5' - 2.0' = 29.5' (Extreme Limit Comp)
Pile tip at 73.0'(comp)below pile cut-off(pile tip@Elev.-70.5',pile cut-off @ Elev. +2.5'),Ult. Pile Capacity=659.2 tons (friction)+282.6 tons (end bearing)(9c x area)=941.8 tons >(1850k/925 tons)(Strength Limit State)OK
Assume c= 5000 psf in the calculation of end bearing capacity
Pile tip at 29.5'(comp)below pile cut-off (pile tip@Elev.-27.0', pile cut-off@Elev. +2.5'),Ult. Pile Capacity=195.7 tons (friction)+170.0 tons (end bearing)(9c x area) =365.7 tons >(690k/345 tons)(Extreme Limit State) OK
Assume c= 3000 psf in the calculation of end bearing capacity

Sheet 6 of 10

Pile Capacity Analysis - EBMUD Aqueduct UC (Widen) (CPT-09-001) Bent 4 4' ϕ x 70 ft
Concrete pile - 753.6 tons
Strength Limit State



CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

I-5 EBMUD Aqueduct UC (Widen) Bent 4

206127.MAQ

Bent 4 (Based on CPT-09-EM-001)

ALLOWABLE PILE CAPACTY 1/11/10

PILE DIA, B (ft) = 4.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 12.56 SQ. FT.
AREA OF THE PILE TIP 12.56 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	1	SAND	125	250	125.0	12	DENSE			36	1.09	0.00	0.00	0.00	0.00
2.0	7.0	2	CLAY	120	600	550.0	14	STIFF	0.55	1750			0.00	0.00	0.00	0.00
7.0	11.5	2	CLAY	120	540	1120.0	14	STIFF	0.55	1750			25.84	25.84	18.09	18.09
11.5	15.0	1	SAND	125	437.5	1608.8	16	DENSE			36	1.01	33.88	59.72	16.94	35.03
15.0	18.5	1	SAND	125	437.5	2046.3	16	DENSE			36	0.95	40.48	100.21	20.24	55.27
18.5	20.0	2	CLAY	120	180	2355.0	16	STIFF	0.55	2000			9.84	110.05	6.89	62.16
20.0	24.5	2	CLAY	68	306	2598.0	16	STIFF	0.55	2000			29.53	139.58	20.67	82.83
24.5	28.5	2	CLAY	68	272	2887.0	16	STIFF	0.55	2000			26.25	165.83	18.38	101.21
28.5	30.0	2	CLAY	68	102	3074.0	24	V.STIFF	0.55	3000			14.77	180.60 ←	10.34	111.54
30.0	36.5	2	CLAY	68	442	3346.0	24	V.STIFF	0.55	3000			63.99	244.58	44.79	156.33
36.5	41.0	2	CLAY	68	306	3720.0	24	V.STIFF	0.55	3000			44.30	288.88	31.01	187.34
41.0	47.0	2	CLAY	68	408	4077.0	32	V.STIFF	0.50	4000			71.59	360.47	50.11	237.46
47.0	52.5	2	CLAY	68	374	4468.0	32	V.STIFF	0.50	4000			65.63	426.10	45.94	283.40
52.5	56.5	2	CLAY	68	272	4791.0	26	V.STIFF	0.54	3250			41.69	467.79	29.18	312.58
56.5	60.0	2	CLAY	68	238	5046.0	26	V.STIFF	0.54	3250			36.48	504.26	25.53	338.11
60.0	63.5	2	CLAY	68	238	5284.0	10	STIFF	0.55	1250			14.36	518.62	10.05	348.16
63.5	68.0	2	CLAY	68	306	5556.0	32	V.STIFF	0.50	4000			53.69	572.31 ←	37.59	385.75
68.0	73.5	2	CLAY	68	374	5896.0	32	V.STIFF	0.50	4000			65.63	637.94	45.94	431.68
73.5	76.0	1	SAND	68	170	6168.0	26	MED DENSE			35	0.33	30.62	668.56	15.31	446.99
76.0	80.0	1	SAND	68	272	6389.0	36	DENSE			36	0.31	46.92	715.47	23.46	470.45

Bent 4 --> Pile cut-off elevation is +3.5 feet. Finished grade elevation is +5.5 feet

1070 kips/pile (Strength Limit State). Required pile capacity (comp)= 1070 kips /(0.7) = 1530 kips (765 tons).

Pile Length @ Bent 4 = 68.0' - 2.0' = 66.0' (Strength Limit Comp), =30.0'-2.0' = 28.0'(Extreme Limit State)

Pile tip at 66.5' (comp) below pile cut-off (pile tip @ Elev. -63.0', pile cut-off @ Elev. +3.5'), Ult. Pile Capacity =572.3 tons (friction)+226.1 tons (end bearing)(9c x area) =798.4T > 765T(Strength Limit State) OK

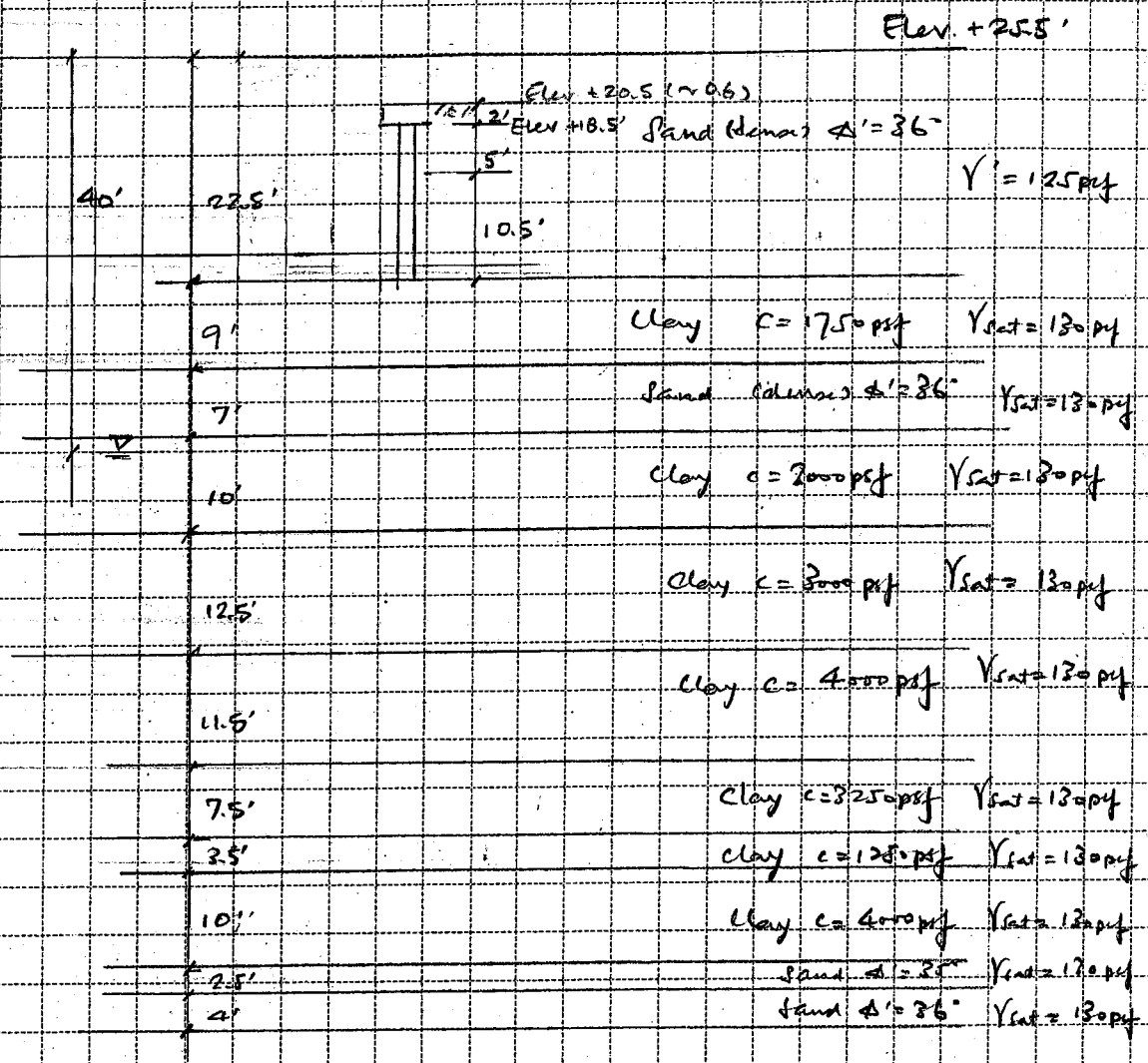
Pile tip at 28.0'(comp) below pile cut-off(pile tip@Elev.-24.5', pile cut-off @ Elev. +3.5.'),Ult. Pile Capacity =180.6 T (friction)+177.1 T (end bearing)(0.6 Nspt x area)= 357.7 T > 345& (Extreme Limit State), OK

Sheet 8 of 10

PARIKH
Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement - EBMUD Aqueduct 66 (Widen)
SHEET NO 9 OF 10
CALCULATED BY A Lam DAT 10/09
CHECKED BY DATE
SCALE

Pile Capacity Analysis - EBMUD Aqueduct 66 (Widen) (CPT-09-001) Abutment 5
2' ϕ CIDH Concrete pile - 71.4 tons
Strength Limit State



CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 EBMUD Aqueduct UC (Widen) Abut 5
206127.MAQ

Abut 5 (Based on CPT-09-EM-001)

ALLOWABLE PILE CAPACTY 1/11/10

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	$\Delta\sigma'$ (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	ϕ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	2.0	1	SAND	125	250	125.0	12	DENSE			36	1.09	0.00	0.00	0.00	0.00
2.0	7.0	1	SAND	125	625	562.5	12	DENSE			36	0.97	0.00	0.00	0.00	0.00
7.0	12.0	1	SAND	125	625	1187.5	14	DENSE			36	1.01	17.92	17.92	8.96	8.96
12.0	17.5	1	SAND	125	687.5	1843.8	16	DENSE			36	0.98	29.69	47.61	14.85	23.80
17.5	22.0	2	CLAY	120	540	2457.5	14	STIFF	0.55	1750			12.92	60.53	9.04	32.85
22.0	26.5	2	CLAY	120	540	2997.5	14	STIFF	0.55	1750			12.92	73.45	9.04	41.89
26.5	32.0	1	SAND	125	687.5	3611.3	20	DENSE			36	0.77	45.61	119.06	22.81	64.70
32.0	33.5	1	SAND	125	187.5	4048.8	22	DENSE			36	0.73	13.18	132.24	6.59	71.29
33.5	35.0	2	CLAY	120	180	4232.5	16	STIFF	0.55	2000			4.92	137.16	3.45	74.73
35.0	39.0	2	CLAY	68	272	4458.5	16	STIFF	0.55	2000			13.13	150.29	9.19	83.92
39.0	45.5	2	CLAY	68	442	4815.5	16	STIFF	0.55	2000			21.33	171.62 ←	14.93	98.85
45.5	48.0	2	CLAY	68	170	5121.5	24	V.STIFF	0.55	3000			12.30	183.92	8.61	107.46
48.0	51.5	2	CLAY	68	238	5325.5	24	V.STIFF	0.55	3000			17.23	201.15	12.06	119.52
51.5	56.0	2	CLAY	68	306	5597.5	24	V.STIFF	0.55	3000			22.15	223.30	15.50	135.03
56.0	62.0	2	CLAY	68	408	5954.5	32	V.STIFF	0.50	4000			35.80	259.09	25.06	160.08
62.0	67.5	2	CLAY	68	374	6345.5	32	V.STIFF	0.50	4000			32.81	291.91	22.97	183.05
67.5	71.5	2	CLAY	68	272	6668.5	26	V.STIFF	0.54	3250			20.84	312.75	14.59	197.64
71.5	75.0	2	CLAY	68	238	6923.5	26	V.STIFF	0.54	3250			18.24	330.99	12.77	210.41
75.0	78.5	2	CLAY	68	238	7161.5	10	STIFF	0.55	1250			7.18	338.17	5.02	215.44
78.5	83.5	2	CLAY	68	340	7450.5	32	V.STIFF	0.50	4000			29.83	368.00	20.88	236.32
83.5	88.5	2	CLAY	68	340	7790.5	32	V.STIFF	0.50	4000			29.83	397.83	20.88	257.20
88.5	91.0	1	SAND	68	170	8045.5	32	MED DENSE			35	0.25	15.00	412.83	7.50	264.70
91.0	95.0	1	SAND	68	272	8266.5	36	DENSE			36	0.25	24.66	437.48	12.33	277.03

Abut 5 --> Pile cut-off elevation is +18.5 feet. Finished grade elevation is +20.5 feet
160 kips/pile (Service Load). Required pile capacity (comp) = 160 kips /(0.5) = 320 kips (160 tons)
Pile Length @ Abutment 5 = 45.5' - 2.0' = 43.5 '
Pile tip at approxiamtely 43.5' below pile cut-off (pile tip @ Elev. -25.0', pile cut-off @ Elev. +18.5'), Ult. Compression Capacity = 171.6 tons > 160 tons (Service-I Limit State) OK

Sheet 10 of 10

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
EBMUD Aqueduct UC (Widen)

Boring A-09-EM-001 (Abutment 1) [24-inch diameter Cast-In-Drilled Hole (CIDH) Concrete pile] **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
25.0* to 1.5	0 to 23.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.072	-
1.5 to -8.0	23.5 to 33	Clay (firm)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.067	-
-8.0 to -22.0	33 to 47	Sand (medium dense)	$\phi = 32°$; c= 0 psf	4, Sand	60	-	0.039	Assumed GWL at 33 feet deep below existing ground surface for design.
-22.0 to -26.5	47 to 51.5	Clay (firm)	$\phi = 0°$; c=1125 psf	3, Clay	500	0.007	0.036	
-26.5 to -32.0	51.5 to 57	Sand (medium dense)	$\phi = 31°$; c= 0 psf	4, Sand	60	-	0.039	-
-32.0 to -59.0	57 to 84	Clay (very stiff)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-59.0 to -70.0	84 to 95	Clay (stiff)	$\phi = 0°$; c= 1500 psf	3, Clay	500	0.007	0.039	-
-70.0 to -85.0	95 to 110	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +19.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
EBMUD Aqueduct UC (Widen)

As-Built Boring B-4 (Bent 2) (48-inch diameter CIDH Concrete pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
0.0* to -8.0	0 to 8	Clay (soft)	$\phi = 0°$; c=625 psf	3, Clay	100	0.01	0.058	-
-8.0 to -9.0	8 to 9	Clay (soft)	$\phi = 0°$; c=625 psf	3, Clay	100	0.01	0.028	Assumed GWL at 8 feet deep below existing ground surface for design.
-9.0 to -31.0	9 to 31	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-31.0 to -39.0	31 to 39	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-
-39.0 to -76.0	39 to 76	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.039	-
-76.0 to -79.0	76 to 79	Clay (very stiff to hard)	$\phi = 0°$; c=3750 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +5.80 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +0.00 feet to +5.80 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
EBMUD Aqueduct UC (Widen)

As-Built Boring B-4 (Bent 3) (48-inch diameter CIDH Concrete pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
2.0* to -6.0	0 to 8	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.067	-
-6.0 to -8.0	8 to 10	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.069	
-8.0 to -11.0	10 to 13	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 10 feet deep below existing ground surface for design.
-11.0 to -21.0	13 to 23	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.036	-
-21.0 to -26.0	23 to 28	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.042	-
-26.0 to -58.0	28 to 60	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.039	-
-58.0 to -61.0	60 to 63	Clay (very stiff to hard)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-61.0 to -65.0	64 to 67	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

*Finish ground elevation of +4.50 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +2.00 feet to +4.50 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
EBMUD Aqueduct UC (Widen)

CPT-09-EM-001 (Bent 4) (48-inch diameter CIDH Concrete pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
25.5* to 3.0	0 to 22.5	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	225	-	0.072	-
3.0 to -6.0	22.5 to 31.5	Clay (stiff)	ϕ = 0°; c=1750 psf	3, Clay	500	0.007	0.069	-
-6.0 to -13.0	31.5 to 38.5	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	225	-	0.072	
-13.0 to -14.5	38.5 to 40	Clay (very stiff)	ϕ = 0°; c=2000 psf	3, Clay	500	0.007	0.069	
-14.5 to -23.0	40 to 48.5	Clay (very stiff)	ϕ = 0°; c=2000 psf	3, Clay	500	0.007	0.039	Assumed GWL at 40 feet deep below existing ground surface for design.
-23.0 to -35.5	48.5 to 61	Clay (very stiff)	ϕ = 0°; c=3000 psf	3, Clay	1000	0.005	0.039	-
-35.5 to -47.0	61 to 72.5	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-47.0 to -54.5	72.5 to 80	Clay (very stiff to hard)	ϕ = 0°; c=3250 psf	3, Clay	1000	0.005	0.039	-
-54.5 to -58.0	80 to 83.5	Clay (stiff)	ϕ = 0°; c=1250 psf	3, Clay	500	0.007	0.039	-
-58.0 to -68.0	83.5 to 93.5	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-68.0 to -70.5	93.5 to 96	Sand (medium dense)	ϕ = 35°; c= 0 psf	4, Sand	60	-	0.039	-
-70.5 to -74.5	96 to 100	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	125	-	0.039	-

*Finish ground elevation of +5.50 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
EBMUD Aqueduct UC (Widen)

CPT-09-EM-001 (Abutment 5) (24-inch diameter CIDH Concrete pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
25.5* to 3.0	0 to 22.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.072	-
3.0 to -6.0	22.5 to 31.5	Clay (stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.069	-
-6.0 to -13.0	31.5 to 38.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.072	
-13.0 to -14.5	38.5 to 40	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.069	
-14.5 to -23.0	40 to 48.5	Clay (very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.039	Assumed GWL at 40 feet deep below existing ground surface for design.
-23.0 to -35.5	48.5 to 61	Clay (very stiff)	$\phi = 0°$; c=3000 psf	3, Clay	1000	0.005	0.039	-
-35.5 to -47.0	61 to 72.5	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-47.0 to -54.5	72.5 to 80	Clay (very stiff to hard)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-54.5 to -58.0	80 to 83.5	Clay (stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.039	-
-58.0 to -68.0	83.5 to 93.5	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-68.0 to -70.5	93.5 to 96	Sand (medium dense)	$\phi = 35°$; c= 0 psf	4, Sand	60	-	0.039	-
-70.5 to -74.5	96 to 100	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-

* Finish ground elevation of +20.5 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** EBMUD Aqueduct UC **Br No*:** 29-0226 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection _X 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** EBMUD Aqueduct UC **Br No*:** 29-0226 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 EA: 0G4701 Project Name: I-5 North Stockton Widening OSFP Liaison: John Fujimoto Phone: 916-227-8757 e-mail: john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection X 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	Reviewer Name: Brandon Badeker Functional Unit: Geotechnical Services Cost Center: 59-323 Phone Number: 916-227-1046 e-mail: brandon_badeker@dot.ca.gov Date of Review: 12/16/2009 Structure Name*: EBMUD Aqueduct UC Br No*: 29-0226 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/12/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
3	FR	8	Tables 2 and 3, Although the pile tip appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	
4	FR	9	Table 4: Please verify pile tip elevation for Bent 3.	The design tip elevation and specified tip elevation at Bent 3 have been revised.	
5	FR	9	Table 4 needs to be updated to our standard LRFD Pile Data Table showing the resistance factor used.	Comment is incorporated. Caltrans standard LRFD Pile Data Table showing the resistance factor used is shown in Table 4.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

6	FR	LOTB	As is stated in the Soil Legend, the sample number or unconfined compressive strength is generally not located to the left of the graphic on the LOTB.	Comment is incorporated.	
7	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #3 above.	
8	FR	NA	C3 – Not Approved (Resubmittal to GS Required)	N/A	

FOUNDATION REPORT
MARCH LANE UC (WIDEN)
(BRIDGE NO. 29-0193)
STOCKTON, CALIFORNIA
10-SJ-05, PM 29.99 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

PARIKH CONSULTANTS, INC.

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June 11, 2010

Job No. 206127.BR

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Office of Special Funded Projects Comment & Response Form (with PARIKH Consultants, Inc. Responses)

FOUNDATION REPORT
MARCH LANE UC (WIDEN)
(BRIDGE NO. 29-0193)
STOCKTON, CALIFORNIA
10-SJ-05 PM 29.99 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "March Lane Undercrossing (Widen) (Bridge No. 29-0193)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the March Lane Undercrossing (UC) (Widen) (Bridge No. 29-0193).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter

unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are single-span structures. Based on the as-built general plan and foundation plan of the March Lane UC, the structures were constructed in 1970. The existing left and right bridge (Bridge No. 29-0193 L/R) are approximately 134 feet long and 53 feet wide. The left and right bridges have not been widened since their construction in 1970.

The abutments of the existing left and right bridge are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class I 50 tons) based on the as-built data.

Proposed Construction

The widening of the March Lane UC is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. There will be no outside widening for the March Lane UC.

According to Rajappan & Meyer Consulting Engineers Inc., the proposed March Lane UC (Widen) will match the existing span with supports at the abutments. The widen structure will be a cast-in-place prestressed concrete box girder with approximate length of 134 feet measured along the "I-5" Line. The width of the inside widening will be 48 feet. The abutments will incorporate diaphragm type abutments.

Based on our discussions with the designer, the proposed bridge abutments are to be supported on Caltrans Standard Class 200 Alternate "W" (16-inch diameter) piles.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The March Lane UC is located at the intersection of Interstate 5 (I-5) and West March Lane. It is approximately 0.4 miles northwest of Calaveras River. The Project Location Map, Plate 1, shows the location of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the March Lane UC (February and July 1964) have been used as a reference (Ref. 1). Based on the as-built LOTB, Borings B-2 and B-3 were drilled and Cone Penetration Test (CPT) at B-1 and B-4 were performed by Caltrans in 1964. The soil borings were drilled to the approximate depths between 46.5 feet and 70.5 feet and to the bottom elevation between approximately -46.5 feet and -70 feet. The CPTs were pushed to the approximate depths between 50 feet and 60 feet and to the bottom elevation between approximately -50 feet and -59 feet.

Field Exploration in 2009

Additional field exploration was conducted in March and April 2009. Boring A-09-ML-001 was drilled on the south side and CPT-09-ML-001 was performed on the north side of March Lane. The soil boring was drilled to the depth of 110.0 feet and the CPT was pushed to the depth of 102.0 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are

presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit weight and Plasticity Index of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built borings and CPT, the subsurface soils generally consist of very stiff to hard clays/silt with pockets/lenses/layers of medium dense to dense clayey/silty sands. A relatively thin layer of soft clayey silt was encountered at the top of Boring B-2.

Based on Boring A-09-ML-001 and CPT-09-ML-001, the subsurface soils generally consist of medium dense to dense silty sand (embankment fill), underlain by interbedded layers of stiff to hard lean clays and medium dense to dense silty sand.

Groundwater was recorded at Elevation -5.5 feet in CPT at B-1 in February 1964. Groundwater was measured at Elevation -6 feet (the depth of 32 feet below existing ground surface) during drilling for A-09-ML-001 in March 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 21 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of medium dense to dense silty sand, underlain by interbedded layers of stiff to hard lean clays and medium dense to dense silty sand. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which

usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring A-09-ML-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-ML-001 included in this report, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure, different foundation systems have been evaluated with respect to their design capacity and construction considerations. Caltrans standard Class 200 Alternative "W" Piles are recommended to be the foundation system for the abutments of the proposed bridge structure. These piles may be designed for service load of 85 tons at the abutments to the indicated pile tip elevations. Pertinent

foundation design information provided by Rajappan & Meyer Consulting Engineers, Inc. (Structural designer), including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2: FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 200 Alt "W"	14.80	11.80	2.5	52.0	1.00	12
Abut 2	WSD	Class 200 Alt "W"	14.00	10.87	2.5	52.0	1.00	12

TABLE 3: FOUNDATION DESIGN LOADS

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	2,001	167	1,547	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	2,001	167	1,547	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads", based upon WSD is presented in Table 3 above.

According to the designer, there are existing tieback walls at the abutments of March Lane UC. The tieback anchors are at an approximate horizontal spacing of 7'3" center-to-center. The proposed Alt "W" piles will be spaced at 3 feet center-to-center to allow adequate clearance between the proposed pile and the existing tieback anchors. The pile group efficiency is reduced to 81.25% for pile spacing of 2.25 diameter center-to-center. The evaluation of Load Demands on the piles, based upon WSD is presented in Table 4 below. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Table 4 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	Class 200 Alt "W"	11.80	2,001	1,547	170	340	-57.5 (a), -7.0 (b)	-68.5
Abut 2	Class 200 Alt "W"	10.87	2,001	1,547	170	340	-69.5 (a), -7.5 (b)	-69.5

Notes: (i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

TABLE 5: PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 200 Alt "W"	260	0	-57.5 (a), -7.0 (c)	-68.5	260
Abut 2	Class 200 Alt "W"	260	0	-69.5 (a), -7.5 (c)	-69.5	260

Notes
(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression, (b) Tension, (c) Lateral Load.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iii) Based on Caltrans review comment, it was recommended to keep the same pile length (specified tip elevation) for "ease of pile fabrication".

The estimation of the driven pile capacity in clay is based on the adhesion factor per FHWA manual (Tomlinson 1980). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 5, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003.

It is anticipated that the pile capacity will develop after driving as a result of soil "freeze"

and dissipation of excess pore water pressures. The final pile capacity can be verified and the gain of pile capacity after initial driving may be evaluated based on "re-driving" after 24-hour (min.) set up. Caltrans Standard Specifications (Section 49-1.08) may be used as the pile driving acceptance criteria. As an option, Pile Driving Analyzer (PDA) may be used to evaluate the pile capacity of the driven piles when unanticipated conditions arise. Typical applications of the PDA include capacity evaluation (for both during driving and re-driving) and integrity testing for piles that have experienced hard driving.

Based on the available boring/CPT information, dense sands were encountered in the embankment fill. It is recommended to follow Caltrans standard specifications (Section 49-1.06) (Ref. 7) for oversize predrill through the existing embankment.

Due to the hard driving condition that may be encountered during pile driving, pile driving should be allowed to terminate short of the specified tip elevation provided the following conditions are satisfied:

- Other requirements including tension and lateral demands are met;
- Pile attaining its capacity and refusal within 5 feet above the specified tip elevation.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. Lateral load analyses for the piles were performed using LPILE Program (v. 5). "Generalized Vertical Soil Profile and Strength Parameters" based on A-09-ML-001 and CPT-09-ML-001 are included in Appendix C. To account for group effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D, respectively.

Generally, abutment piles are designed for service loads only, and batter piles are used for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following

applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for new structures. Settlement is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Boring A-09-ML-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
A-09-ML-001	39.5	880	7.2	54.3	71.9

Based on the corrosion test results, the native subsurface soils in Boring A-09-ML-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.8 Construction Observation

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

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

AL/GP: S:\Ongoing Projects\2006\206127\I-5 March Lane UC (Widen) Foundation Report (6-2010).rpt

REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

REFERENCES

1. March Lane Drive UC As-built Log of Test Borings (February 1964).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

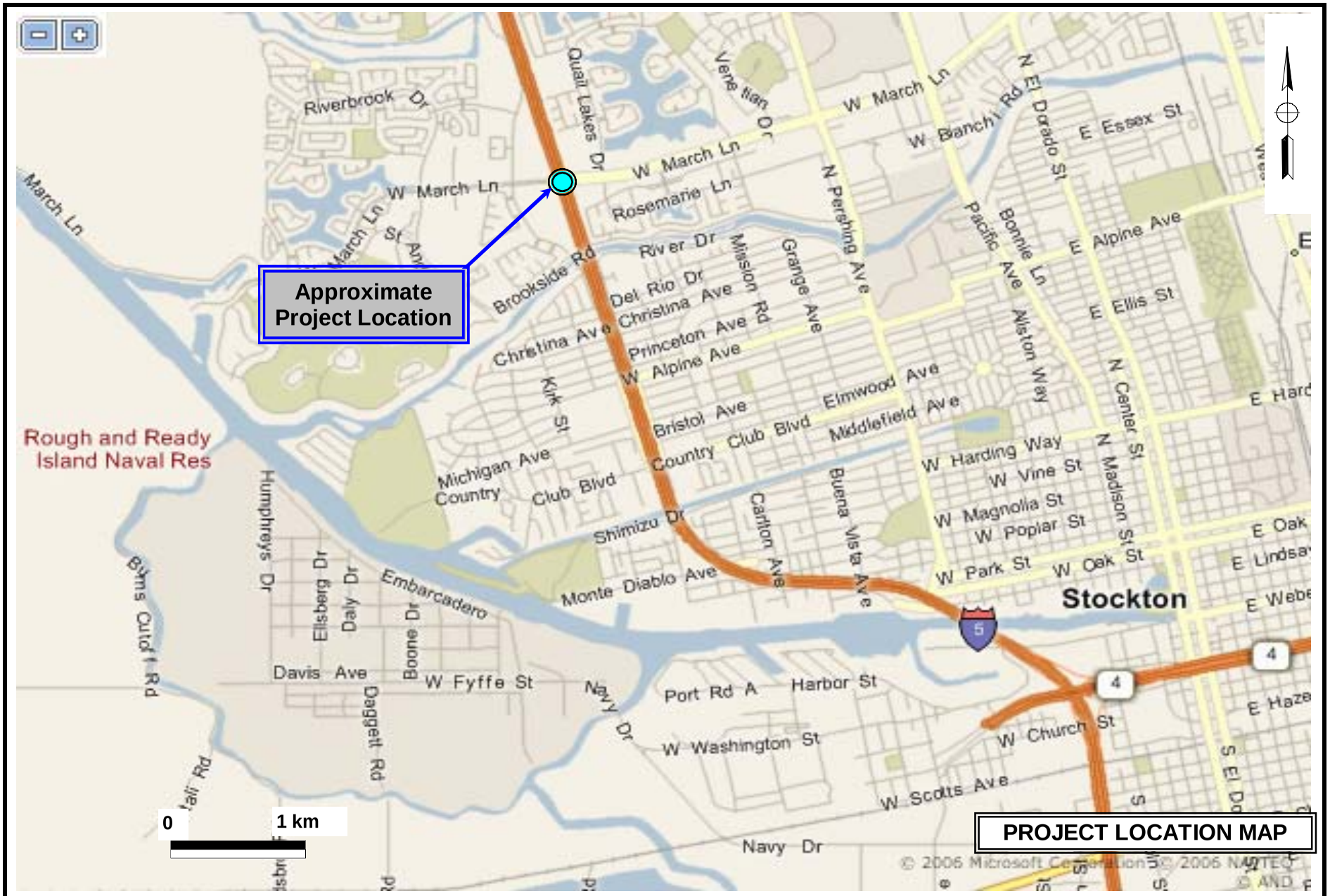
3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

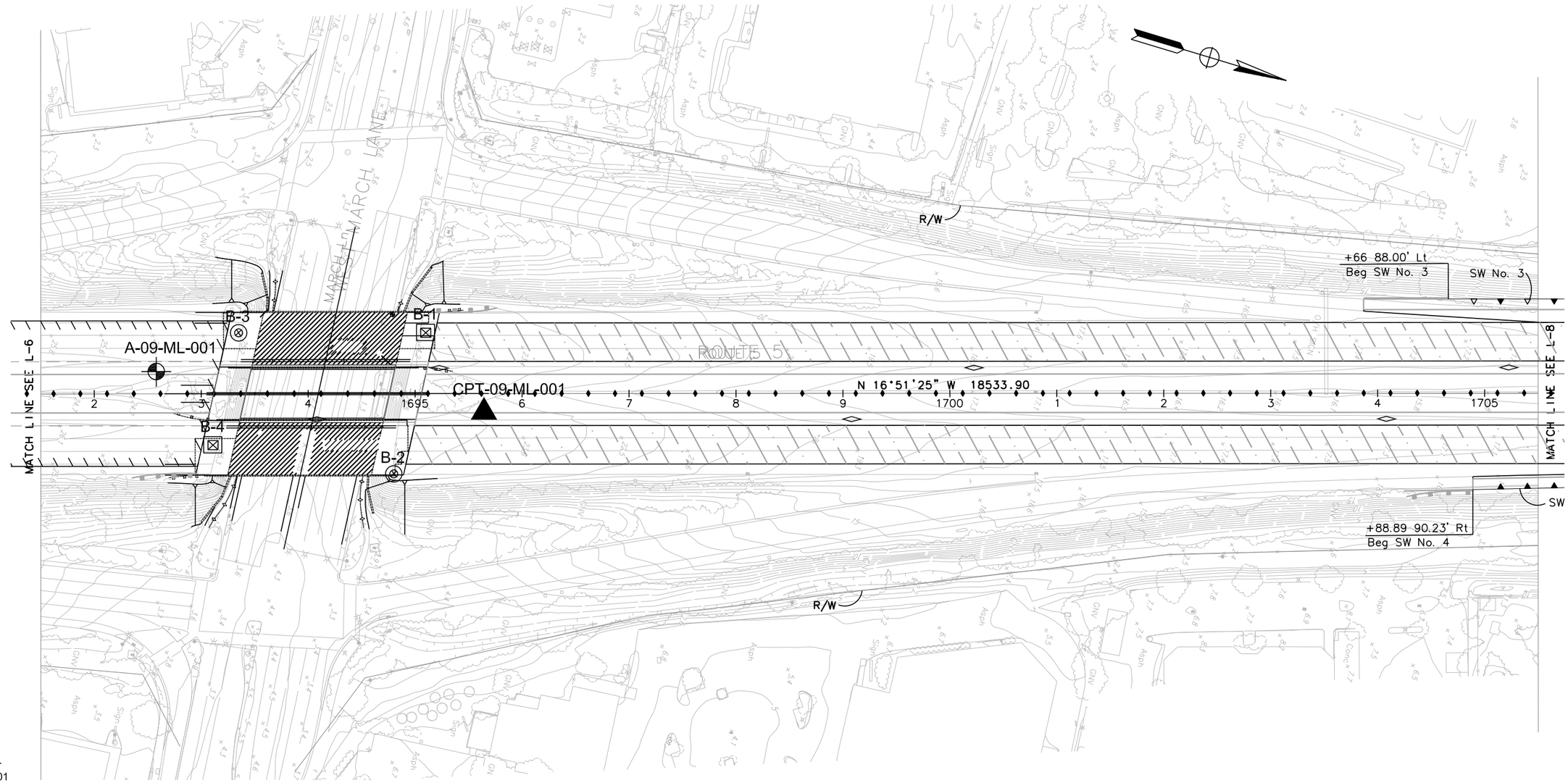
7. State of California Department of Transportation Standard Specifications May 2006.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MARCH LANE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0193)	PLATE NO.: 1



LEGEND

A-09-ML-001
Approximate Boring Location

B-4
Approximate CPT location performed in 1964

B-3
Approximate Boring location drilled in 1964

CPT-09-ML-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

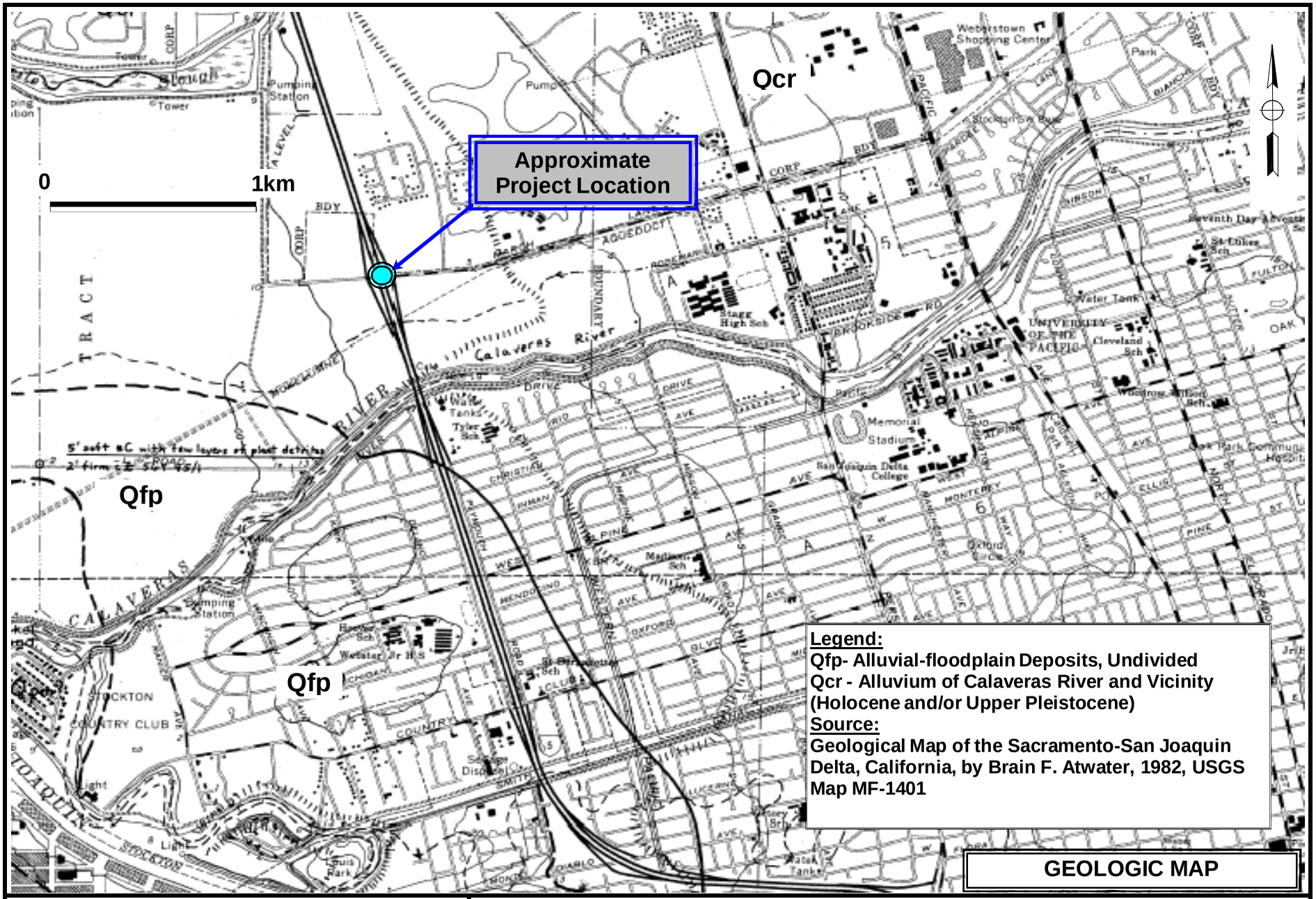
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

MARCH LANE UC (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

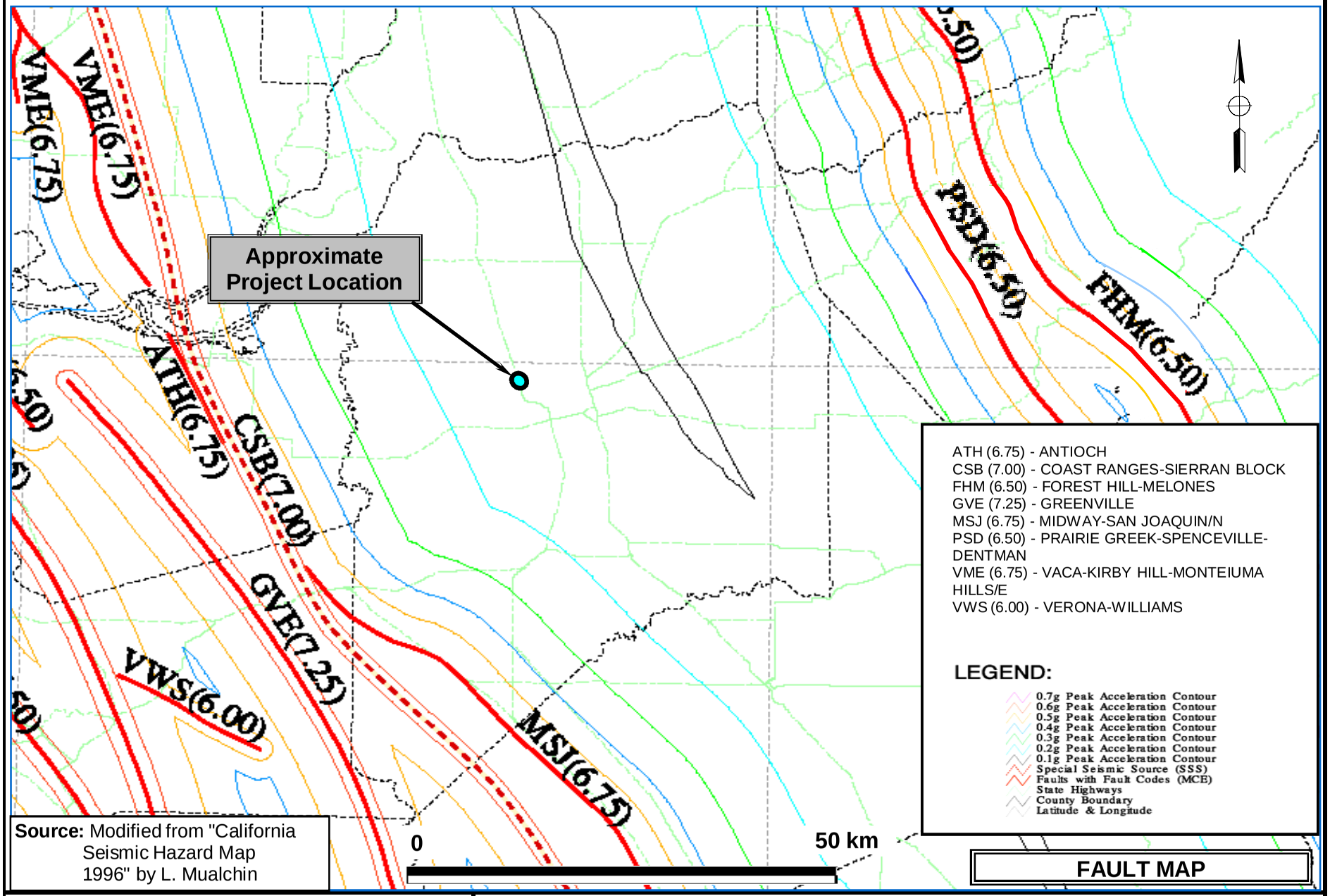
PLATE NO: 2



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MARCH LANE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0193)	PLATE NO.: 3



Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

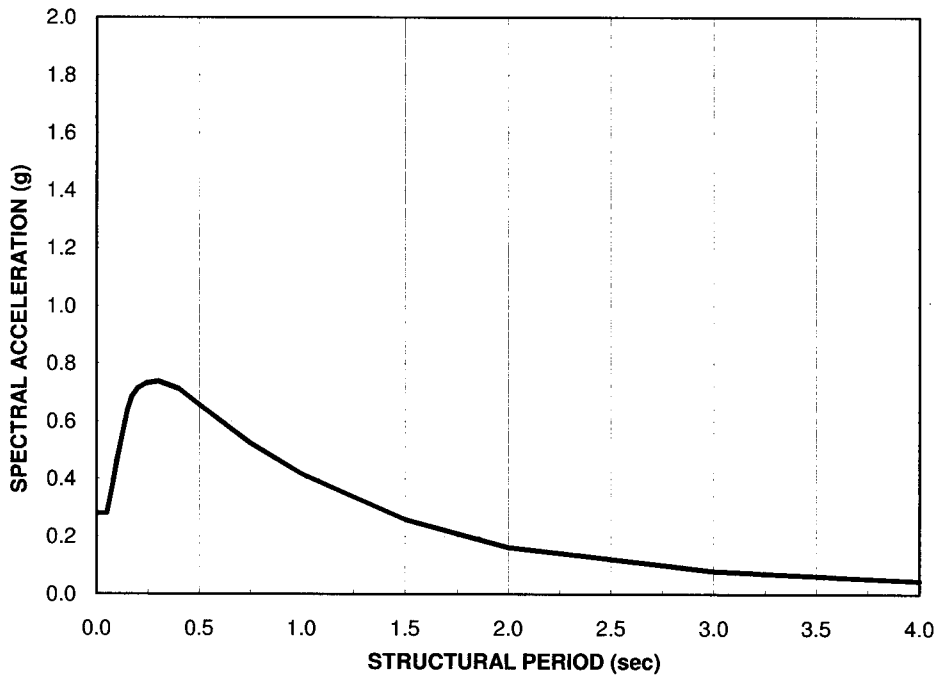
FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MARCH LANE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0193) | **PLATE NO.: 4**

ARS DESIGN CURVE
MARCH LANE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

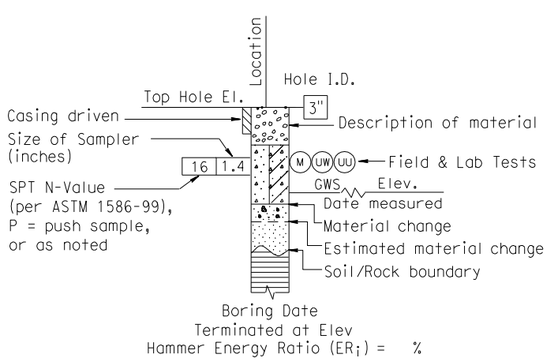
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

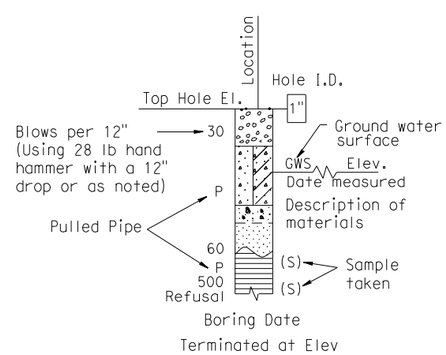
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

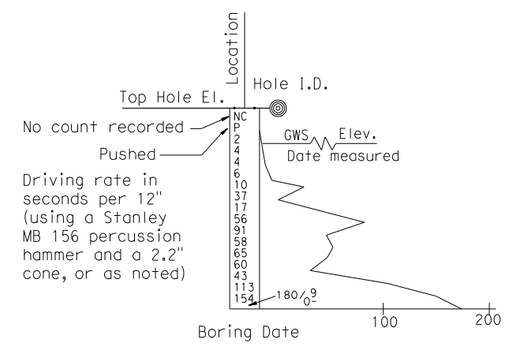
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



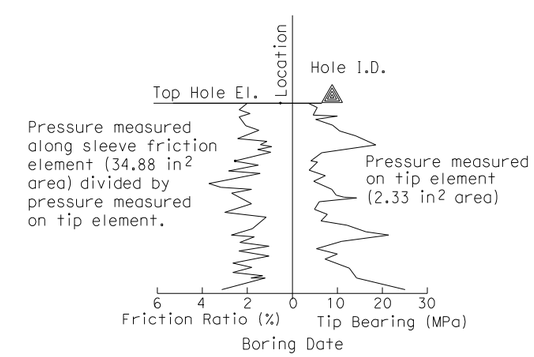
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY: O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER	BRIDGE NO. 29-0193 POST MILES 29.99
SIGN OFF DATE	CHECKED BY: D. WANG				

MARCH LANE UC (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 16 OF 20

...\29-0193-z-ltb01.dgn 6/10/2010 8:56:25 AM

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

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

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)
- (VS) Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0193
POST MILES 29.99

MARCH LANE UC (WIDEN)
SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 17 OF 20

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

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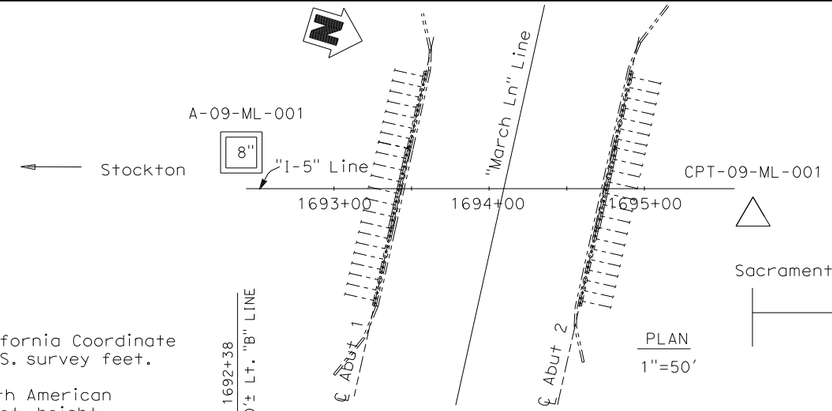
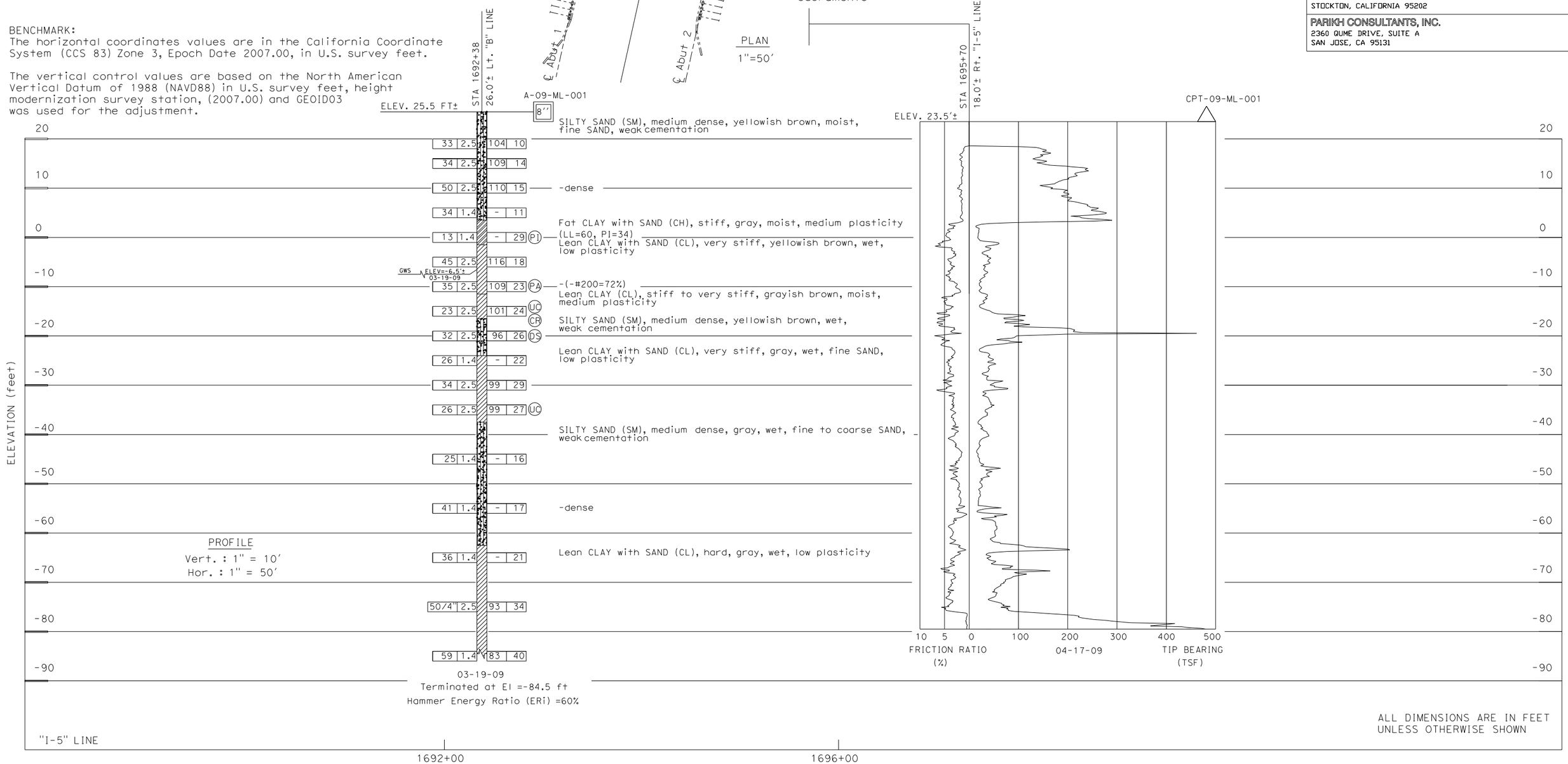
SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131



BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.

The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.



ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER

BRIDGE NO. 29-0193
POST MILES 29.99

MARCH LANE UC (WIDEN)
LOG OF TEST BORINGS 1 OF 3

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES → 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 18 OF 20

...\29-0193-z-ltb03.dgn 6/10/2010 8:58:13 AM

DIST.	COUNTY	ROUTE	SECTION	SHEET No.	TOTAL SHEETS
10	SJ	5	R.S.	357	392

DATE APPROVED September 5, 1967

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5			

REGISTERED CIVIL ENGINEER DATE

MARCH LANE UC (WIDEN)

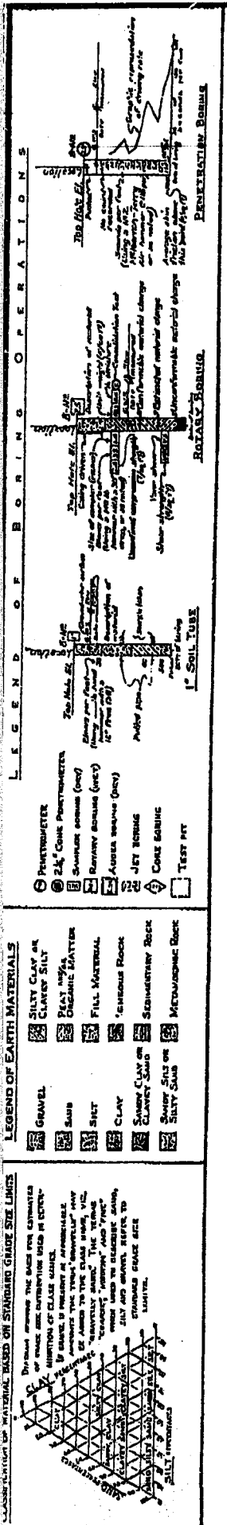
LOG OF TEST BORINGS 2 OF 3

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

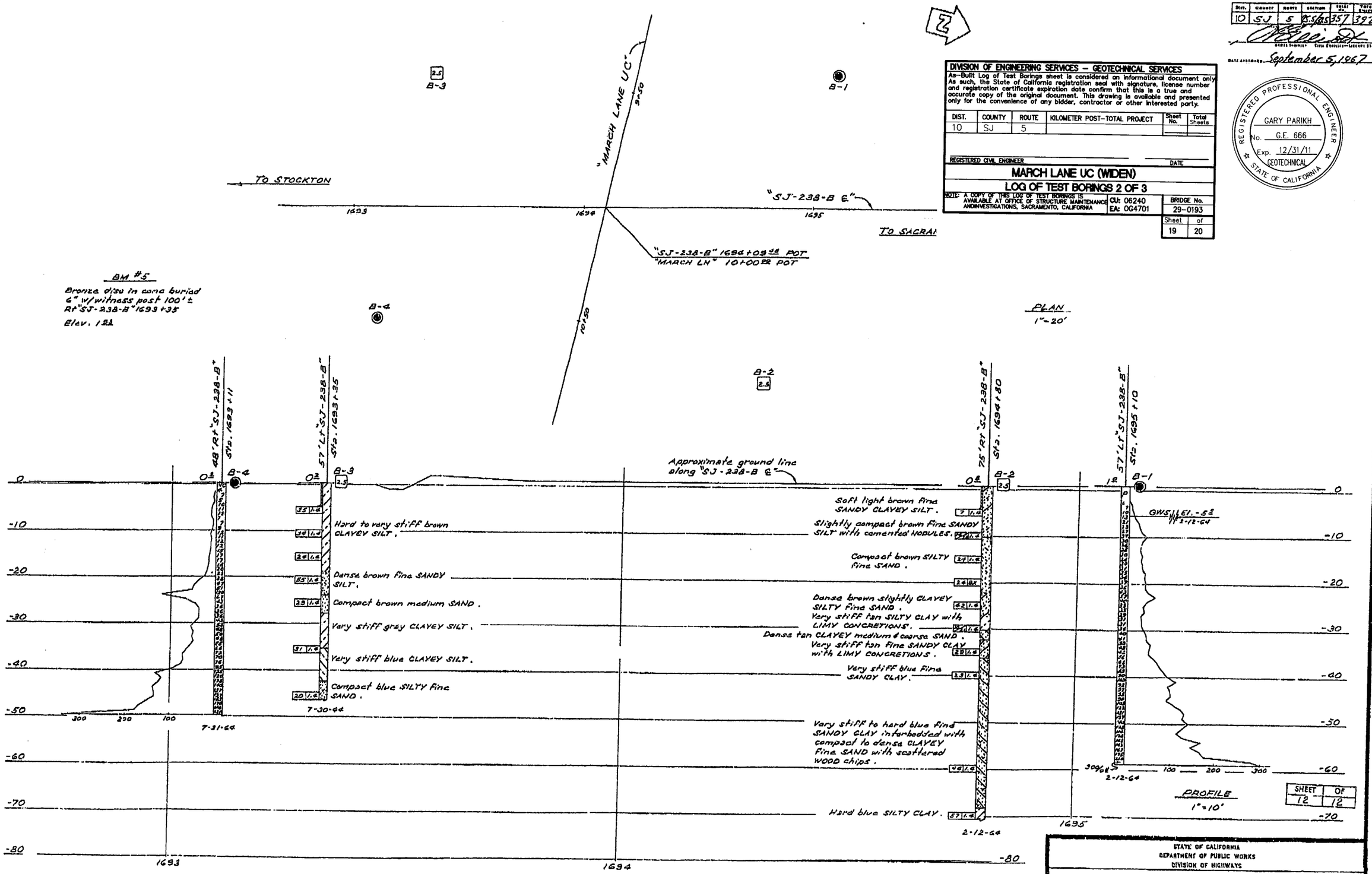
CU: 06240
EA: 0G4701

BRIDGE No. 29-0193

Sheet 19 of 20



NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.



AS BUILT PLANS
Contract No. 10-080924
Date Completed
Document No.

CONFIRMATION OF ACTUAL CONDITIONS ON FILE IN BRIDGE GEOLOGY SECTION

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

MARCH LANE UNDERCROSSING

LOG OF TEST BORINGS

SCALE As Noted | BRIDGE 29-193 | FILE | DRAWING 29193-8

DIST.	COUNTY	ROUTE	KILOMETER POST-TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5	—		

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES

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REGISTERED CIVIL ENGINEER _____ DATE _____

MARCH LANE UC (WIDEN)

LOG OF TEST BORINGS 3 OF 3

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0G4701

BRIDGE No. 29-0193

Sheet 20 of 20

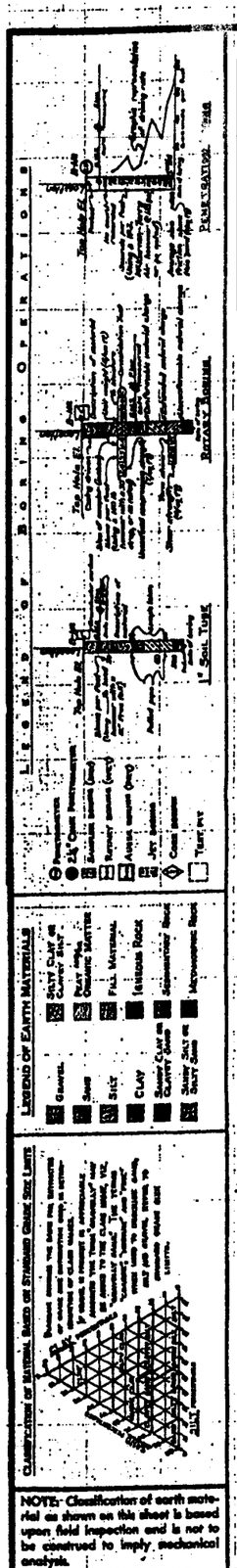
REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

Dist.	County	Route	Bridge No.	Total Sheets
10	SJ	5	29-0193	57 / 57

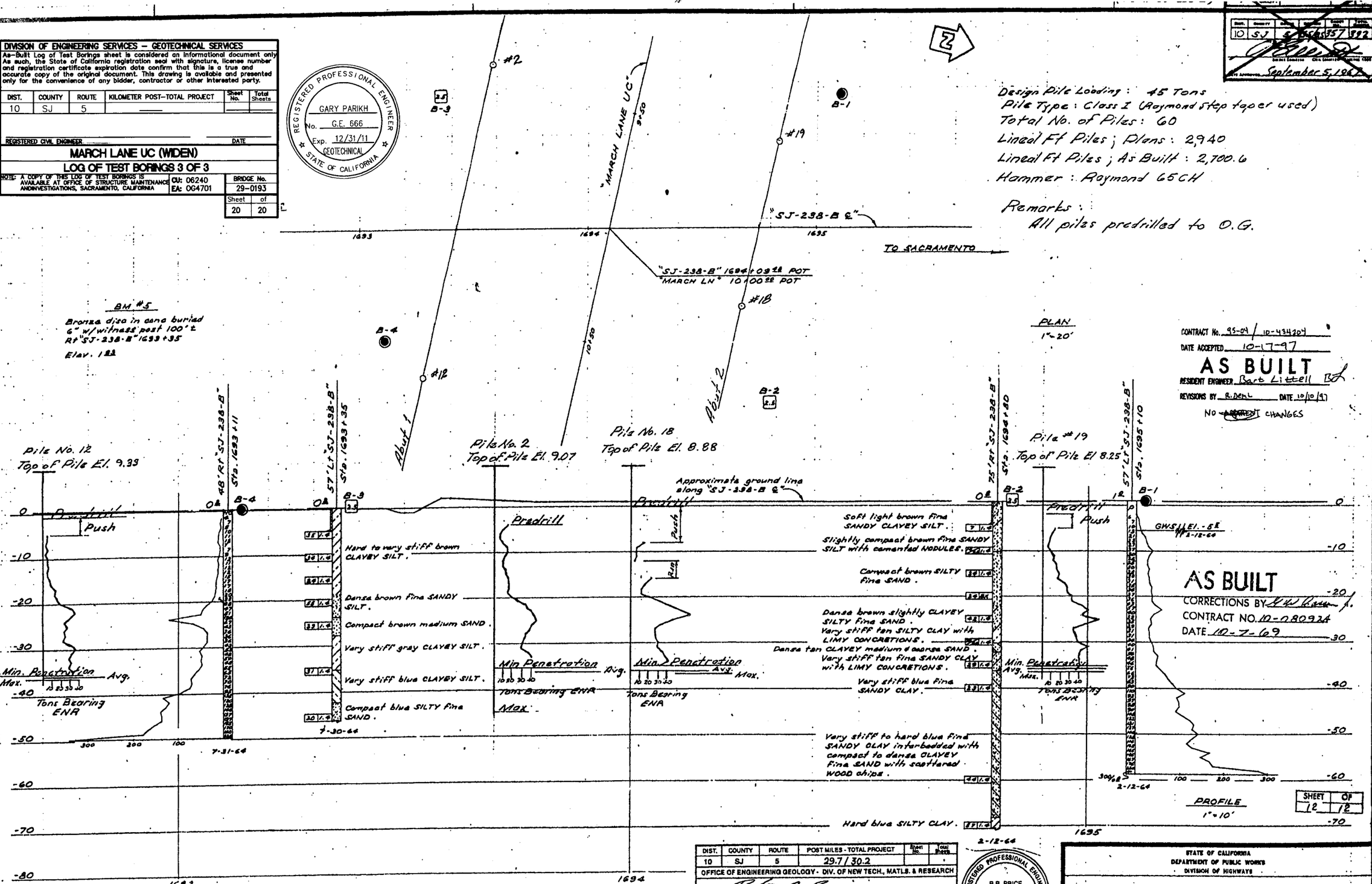
Date Engineer Completed: September 5, 1967

Design Pile Loading : 45 Tons
Pile Type : Class I (Raymond step taper used)
Total No. of Piles : 60
Lineal Ft Piles ; Plans : 2,940
Lineal Ft Piles ; As Built : 2,700.6
Hammer : Raymond 65CH

Remarks :
All piles predrilled to O.G.



NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.



CONTRACT No. 95-04 / 10-434204
DATE ACCEPTED 10-17-97

AS BUILT
RESIDENT ENGINEER Bart Littell
REVISIONS BY R. DENL DATE 10/10/97
NO ~~SUBSTANTIAL~~ CHANGES

AS BUILT
CORRECTIONS BY
CONTRACT NO. 10-080924
DATE 10-7-69

SHEET	OF
12	12

DIST.	COUNTY	ROUTE	POST MILES - TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5	29.7 / 30.2		

OFFICE OF ENGINEERING GEOLOGY - DIV. OF NEW TECH., MATLS. & RESEARCH

REGISTERED ENGINEER - CIVIL

MARCH LANE UC RETAINING WALLS

LOG OF TEST BORINGS

NOTE: THIS LOG OF TEST BORINGS IS AVAILABLE ON MICROFILM AT OFFICE OF STRUCTURES DESIGN SACRAMENTO, CALIFORNIA

CU: 10
EA: 434201

BRIDGE No. 29-0193R/L

2-12-64

REGISTERED PROFESSIONAL ENGINEER
R.R. PRICE
No. 49593
Exp. 9-30-98
CIVIL
STATE OF CALIFORNIA

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

MARCH LANE UNDERCROSSING

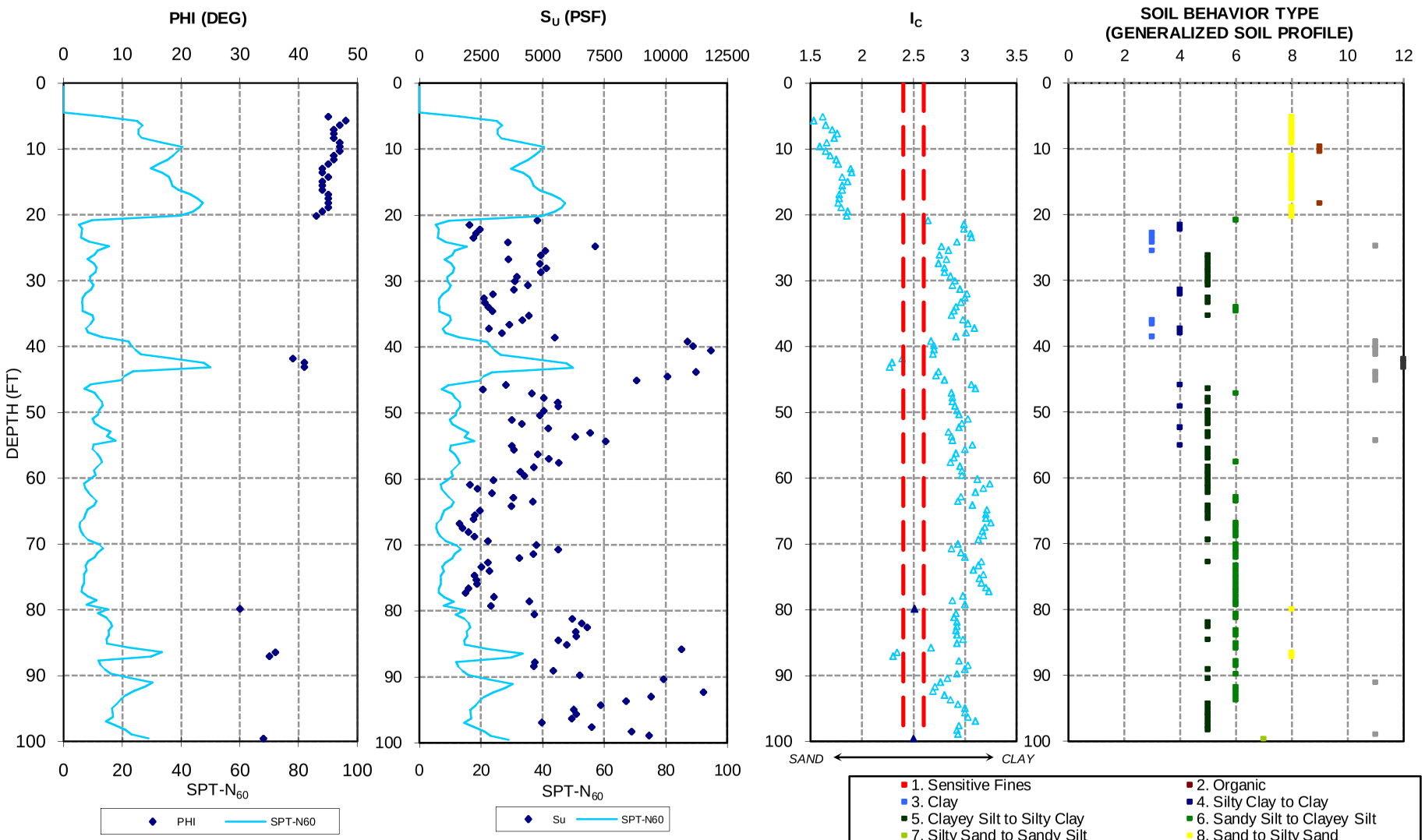
LOG OF TEST BORINGS

SCALE As Noted | BRIDGE 29-193 R/L | FILE | DRAWING 29193-8

3838.121 C

CPT DATA CORRELATION

CPT-09-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

MARCH LANE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR/29 0193

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-MARCH LANE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***206127.MLB***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)	Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
5.085	8 Sand to Silty Sand	76.244	0.74	0.308		0.308	1.62	1.45889	0.99	0.129	1.620	1.46	111.23	1.000	111.231	0.21	1.00	1	
5.741	8 Sand to Silty Sand	150.325	1.719	0.347		0.347	1.53	1.42211	0.99	0.129	1.530	1.42	213.78	1.000	213.778		1.00	1	
6.398	8 Sand to Silty Sand	154.818	2.368	0.387		0.387	1.65	1.38626	0.99	0.128	1.650	1.39	214.62	1.003	215.335		1.00	1	
7.054	8 Sand to Silty Sand	143.539	2.319	0.427		0.427	1.71	1.35218	0.99	0.128	1.710	1.35	194.09	1.045	202.747		1.00	1	
7.71	8 Sand to Silty Sand	141.153	2.397	0.466		0.466	1.76	1.32053	0.98	0.128	1.760	1.32	186.40	1.078	201.017		1.00	1	
8.366	8 Sand to Silty Sand	149.692	2.334	0.506		0.506	1.73	1.28957	0.98	0.128	1.730	1.29	193.04	1.058	204.245		1.00	1	
9.022	8 Sand to Silty Sand	190.907	2.735	0.546		0.546	1.66	1.26002	0.98	0.128	1.660	1.26	240.55	1.010	243.050		1.00	1	
9.678	9 Sand	237.302	3.117	0.586		0.586	1.59	1.2318	0.98	0.127	1.590	1.23	292.31	1.000	292.309		1.00	1	
10.335	9 Sand	225.029	3.176	0.627		0.627	1.65	1.20416	0.98	0.127	1.650	1.20	270.97	1.003	271.875		1.00	1	
10.991	8 Sand to Silty Sand	213.062	3.096	0.667		0.667	1.69	1.17836	0.98	0.127	1.690	1.18	251.06	1.031	258.868		1.00	1	
11.647	8 Sand to Silty Sand	196.704	3.08	0.706		0.706	1.75	1.15425	0.98	0.127	1.750	1.15	227.05	1.072	243.301		1.00	1	
12.303	8 Sand to Silty Sand	179.163	2.693	0.746		0.746	1.77	1.13052	0.97	0.127	1.770	1.13	202.55	1.085	219.835		1.00	1	
12.959	8 Sand to Silty Sand	158.083	2.886	0.786		0.786	1.89	1.10775	0.97	0.126	1.890	1.11	175.12	1.180	206.563		1.00	1	
13.615	8 Sand to Silty Sand	177.459	3.52	0.825		0.825	1.9	1.08642	0.97	0.126	1.900	1.09	192.79	1.189	229.203		1.00	1	
14.272	8 Sand to Silty Sand	195.689	3.095	0.865		0.865	1.81	1.06538	0.97	0.126	1.810	1.07	208.48	1.114	232.251		1.00	1	
14.928	8 Sand to Silty Sand	195.765	3.436	0.905		0.905	1.86	1.04513	0.97	0.126	1.860	1.05	204.60	1.153	235.974		1.00	1	
15.584	8 Sand to Silty Sand	201.891	3.079	0.945		0.945	1.81	1.02564	0.97	0.126	1.810	1.03	207.07	1.114	230.676		1.00	1	
16.24	8 Sand to Silty Sand	212.554	3.302	0.984		0.984	1.81	1.00733	0.97	0.126	1.810	1.01	214.11	1.114	238.522		1.00	1	
16.896	8 Sand to Silty Sand	237.499	3.517	1.024		1.024	1.78	0.98921	0.96	0.125	1.780	0.99	234.94	1.092	256.631		0.99	1	
17.552	8 Sand to Silty Sand	252.016	3.805	1.064		1.064	1.78	0.97173	0.96	0.125	1.780	0.97	244.89	1.092	267.506		0.98	1	
18.209	9 Sand	262.078	3.84	1.104		1.104	1.77	0.95486	0.96	0.125	1.770	0.95	250.25	1.085	271.606		0.96	1	
18.865	8 Sand to Silty Sand	252.649	3.854	1.144		1.144	1.8	0.93857	0.96	0.125	1.800	0.94	237.13	1.107	262.420		0.95	1	
19.521	8 Sand to Silty Sand	235.249	3.869	1.184		1.184	1.86	0.92282	0.96	0.125	1.860	0.92	217.09	1.153	250.382		0.94	1	
20.177	8 Sand to Silty Sand	212.75	3.018	1.223		1.223	1.85	0.90797	0.96	0.124	1.850	0.91	193.17	1.145	221.195		0.93	1	
20.833	6 Sandy Silt to Clayey Silt	37.153	1.09	1.261		1.261	2.64	0.89395	0.95	0.124							0.95	1	
21.49	4 Silty Clay to Clay	16.587	0.559	1.299		1.299	2.99	0.88035	0.95	0.124							0.95	1	
22.146	4 Silty Clay to Clay	19.694	0.821	1.336		1.336	2.99	0.86751	0.95	0.124							0.94	1	
22.802	3 Clay	18.53	0.846	1.373		1.373	3.05	0.85503	0.95	0.123							0.94	1	
23.458	3 Clay	17.886	0.784	1.409		1.409	3.06	0.84323	0.95	0.123							0.93	1	
24.114	3 Clay	28.27	1.347	1.446		1.446	2.92	0.83144	0.94	0.123							0.93	1	
24.77	11 Sitff Fine-Grained	55.044	3.208	1.489		1.489	2.77	0.81815	0.94	0.123							0.92	1	
25.427	3 Clay	39.708	2.025	1.525		1.525	2.84	0.80734	0.94	0.122							0.92	1	
26.083	5 Clayey Silt to Silty Clay	38.359	1.311	1.563		1.563	2.75	0.79624	0.94	0.122							0.91	1	
26.739	5 Clayey Silt to Silty Clay	28.652	0.844	1.6		1.6	2.82	0.78571	0.94	0.122							0.91	1	
27.395	5 Clayey Silt to Silty Clay	37.671	1.201	1.638		1.638	2.74	0.77519	0.93	0.121							0.91	1	
28.051	5 Clayey Silt to Silty Clay	39.626	1.629	1.676		1.676	2.8	0.76495	0.93	0.121							0.90	1	
28.707	5 Clayey Silt to Silty Clay	37.966	1.463	1.713		1.713	2.8	0.75524	0.93	0.120							0.90	1	
29.364	5 Clayey Silt to Silty Clay	30.634	1.074	1.751		1.751	2.86	0.74551	0.92	0.120							0.89	1	
30.02	5 Clayey Silt to Silty Clay	29.99	1.166	1.788		1.788	2.9	0.73628	0.92	0.120							0.89	1	
30.676	5 Clayey Silt to Silty Clay	34.281	1.409	1.826	0.021	1.805	2.88	0.73211	0.92	0.121							0.89	1	
31.332	4 Silty Clay to Clay	29.99	1.322	1.864	0.041	1.822	2.95	0.72799	0.91	0.121							0.89	1	
31.988	4 Silty Clay to Clay	23.171	0.964	1.901	0.062	1.839	3.02	0.72392	0.91	0.122							0.89	1	
32.644	5 Clayey Silt to Silty Clay	20.086	0.651	1.939	0.082	1.856	3	0.7199	0.91	0.123							0.88	1	
33.301	5 Clayey Silt to Silty Clay	20.283	0.571	1.976	0.103	1.874	2.96	0.71568	0.90	0.124							0.88	1	
33.957	6 Sandy Silt to Clayey Silt	21.298	0.519	2.014	0.123	1.891	2.91	0.71174	0.90	0.124							0.88	1	
34.613	6 Sandy Silt to Clayey Silt	21.757	0.53	2.051	0.144	1.908	2.89	0.70785	0.89	0.125							0.88	1	
35.269	5 Clayey Silt to Silty Clay	33.473	1.277	2.089	0.164	1.925	2.87	0.704	0.89	0.125							0.88	1	
35.925	3 Clay	32.206	1.639	2.126	0.185	1.941	2.98	0.70041	0.88	0.126							0.88	1	
36.581	3 Clay	28.325	1.44	2.162	0.205	1.957	3.03	0.69686	0.88	0.126							0.87	1	
37.238	4 Silty Clay to Clay	22.308	1.002	2.2	0.226	1.974	3.09	0.69313	0.87	0.127							0.87	1	
37.894	4 Silty Clay to Clay	26.037	1.072	2.237	0.246	1.991	3.01	0.68944	0.87	0.127							0.87	1	
38.55	3 Clay	42.678	2.235	2.274	0.267	2.007	2.91	0.686	0.86	0.127							0.87	1	
39.206	11 Sitff Fine-Grained	83.259	3.97	2.317	0.287	2.03	2.67	0.68111	0.86	0.127							0.87	1	
39.862	11 Sitff Fine-Grained	85.476	4.556	2.36	0.307	2.052	2.7	0.67651	0.85	0.127							0.87	1	
40.518	11 Sitff Fine-Grained	90.613	5.077	2.402	0.328	2.074	2.7	0.67196	0.85	0.127							0.86	1	
41.175	11 Sitff Fine-Grained	98.399	5.816	2.445	0.348	2.097	2.69	0.66727	0.84	0.127							0.86	1	
41.831	12 Sand to Clayey Sand	158.695	5.604	2.485	0.369	2.116	2.39	0.66345	0.83	0.127	2.390	0.66	105.29	2.272	239.212		0.83	1	
42.487	12 Sand to Clayey Sand	215.709	7.248	2.525	0.389	2.135	2.29	0.65967	0.83	0.127	2.290	0.66	142.30	1.917	272.790		0.80	1	
43.143	12 Sand to Clayey Sand	227.377	7.538	2.564	0.41	2.154	2.27	0.65593	0.82	0.127	2.270	0.66	149.14	1.856	276.824		0.79	1	
43.799	11 Sitff Fine-Grained	85.847	4.982	2.607	0.43	2.177	2.74	0.65147	0.82	0.127							0.86	1	
44.455	11 Sitff Fine-Grained	76.577	3.773	2.65	0.451	2.199	2.72	0.64725	0.81	0.127							0.85	1	
45.112	11 Sitff Fine-Grained	68.387	3.643	2.693	0.471	2.221	2.8	0.64309	0.80	0.126							0.85	1	
45.768	4 Silty Clay to Clay	28.194	1.27	2.73	0.492	2.239	3.06	0.63972	0.80	0.126							0.85	1	
46.424	5 Clayey Silt to Silty Clay	21.216	0.691	2.768	0.512	2.256	3.1	0.63657	0.79	0.126							0.85	1	
47.08	6 Sandy Silt to Clayey Silt	35.646	1.121	2.805	0.533	2.273	2.87	0.63346	0.78	0.126							0.85	1	
47.736	5 Clayey Silt to Silty Clay	39.337	1.4	2.843	0.553	2.29	2.88	0.63037	0.78	0.125							0.85	1	
48.392	5 Clayey Silt to Silty Clay	44.048	1.815	2.881	0.574	2.307	2.88	0.62732	0.77	0.125							0.85	1	
49.049	4 Silty Clay to Clay	44.141	1.949	2.918	0.594	2.324	2.9	0.62429	0.76	0.124							0.84	1	
49.705	5 Clayey Silt to Silty Clay	39.654	1.617	2.956	0.615	2.341	2.92	0.62129	0.76	0.124							0.84	1	
50.361	5 Clayey Silt to Silty Clay	38.496	1.555	2.993	0.635	2.358	2.94	0.61832	0.75	0.124							0.84	1	
51.017	5 Clayey Silt to Silty Clay	30.519	1.206	3.031	0.655	2.376	3.03	0.61521	0.74	0.123							0.84	1	
51.673	5 Clayey Silt to Silty Clay	33.713	1.212	3.069	0.676	2.393	2.97	0.6123	0.74	0.123							0.84	1	
52.329	4 Silty Clay to Clay	41.755	1.812	3.106	0.696	2.41	2.94	0.60942	0.73	0.122							0.84	1	
52.986	5 Clayey Silt to Silty Clay	54.531	2.346	3.144	0.717	2.427	2.84	0.60656	0.72	0.122							0.84	1	
53.642	5 Clayey Silt to Silty Clay	49.732	2.115	3.181	0.737	2.444	2.87	0.60373	0.72	0.121							0.84	1	
54.298	11 Sitff Fine-Grained	59.559	3.1	3.224	0.758	2.466	2.88	0.60011	0.71	0.121							0.83	1	
54.954	4 Silty Clay to Clay	30.492	1.323	3.262	0.778	2.483	3.07	0.59734	0.70	0.120							0.83	1	
55.61	5 Clayey Silt to Silty Clay	30.747	1.083	3.299	0.799	2.501	3	0.59443	0.70	0.120							0.83	1	
56.266	5 Clayey Silt to Silty Clay	37.791	1.265	3.337	0.819	2.518	2.91	0.59172	0.69	0.119							0.83	1	
56.923	5 Clayey Silt to Silty Clay	41.499	1.403	3.375	0.84	2.535	2.89	0.58902	0.69	0.119							0.83	1	
57.579	6 Sandy Silt to Clayey Silt	44.414	1.488	3.412	0.86	2.552	2.86	0.58635	0.68	0.118							0.83	1	
58.235	5 Clayey Silt to Silty Clay	37.349	1.319	3.45	0.881	2.569	2.95	0.58371	0.67	0.118							0.83	1	
58.891	5 Clayey Silt to Silty Clay	32.971	1.054	3.487	0.901	2.586	2.97	0.58109	0.67	0.117							0.83	1	
59.547	5 Clayey Silt to Silty Clay	34.265	1.172	3.525	0.922	2.603	2.97	0.57849	0.66	0.117							0.83	1	
60.203	5 Clayey Silt to Silty Clay	25.404	0.884	3.563	0.942	2.62	3.12	0.57592	0.66	0.116							0.82	1	
60.86	5 Clayey Silt to Silty Clay	18.421	0.584	3.6	0.963	2.638	3.24	0.57322	0.65	0.116							0.82	1	
61.516	5 Clayey Silt to Silty Clay	20.25	0.623	3.638	0.983	2.655	3.18	0.57069	0.65	0.115							0.82	1	
62.172	5 Clayey Silt to Silty Clay	24.787	0.773	3.675	1.004	2.672	3.1	0.56818	0.64	0.115							0.82	1	
62.828	6 Sandy Silt to Clayey Silt	30.913	0.821	3.713	1.024	2.689	2.96	0.5657	0.64	0.114							0.82	1	
63.484	6 Sandy Silt to Clayey Silt	36.689	1.114	3.751	1.044	2.706	2.93	0.56324	0.63	0.114							0.82	1	
64.14	5 Clayey Silt to Silty Clay	31.164	1.191	3.788	1.065	2.723	3.07	0.5608	0.63	0.114							0.82	1	
64.797	5 Clayey Silt to Silty Clay	21.866	0.753	3.826	1.085	2.74	3.21	0.55838	0.62	0.113							0.82	1	
65.453	5 Clayey Silt to Silty Clay	20.223	0.571	3.863	1.106	2.757	3.2	0.55598	0.62	0.113							0.82	1	
66.109	5 Clayey Silt to Silty Clay	19.098	0.512	3.901	1.126	2.775	3.2	0.55346	0.61	0.112							0.82	1	
66.765	6 Sandy Silt to Clayey Silt	14.632	0.29	3.938	1.147	2.792	3.25	0.5511	0.61	0.112							0.81	1	
67.421	6 Sandy Silt to Clayey Silt	15.189	0.263	3.976	1.167	2.809	3.19	0.54877	0.61	0.112							0.81	1	
68.077	6 Sandy Silt to Clayey Silt	16.848	0.337	4.014	1.188	2.826	3.17	0.54645	0.60	0.111							0.81	1	
68.734	6 Sandy Silt to Clayey Silt	19.032	0.475	4.051	1.208	2.843	3.18	0.54415	0.60	0.111							0.81	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-MARCH LANE

PROJECT NAME	***UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***206127.MLB***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*Kσ*Kα*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	Kσ	Kα	F.S.
69.39	5	Clayey Silt to Silty Clay	23.853	0.669	4.089	1.229	2.86	3.13	0.54187	0.60	0.111							0.81	1	
70.046	6	Sandy Silt to Clayey Silt	39.134	1.094	4.126	1.249	2.877	2.93	0.53961	0.59	0.110							0.81	1	
70.702	6	Sandy Silt to Clayey Silt	45.369	1.277	4.164	1.27	2.894	2.87	0.53737	0.59	0.110							0.81	1	
71.358	6	Sandy Silt to Clayey Silt	36.825	1.124	4.202	1.29	2.912	2.96	0.53502	0.58	0.110							0.81	1	
72.014	6	Sandy Silt to Clayey Silt	33.206	0.978	4.239	1.311	2.929	3	0.53282	0.58	0.109							0.81	1	
72.671	5	Clayey Silt to Silty Clay	23.498	0.749	4.277	1.331	2.946	3.16	0.53063	0.58	0.109							0.81	1	
73.327	6	Sandy Silt to Clayey Silt	21.314	0.493	4.314	1.352	2.963	3.13	0.52847	0.58	0.109							0.80	1	
73.983	6	Sandy Silt to Clayey Silt	23.64	0.537	4.352	1.372	2.98	3.08	0.52632	0.57	0.109							0.80	1	
74.639	6	Sandy Silt to Clayey Silt	19.458	0.459	4.39	1.393	2.997	3.18	0.52418	0.57	0.108							0.80	1	
75.295	6	Sandy Silt to Clayey Silt	19.633	0.403	4.427	1.413	3.014	3.14	0.52207	0.57	0.108							0.80	1	
75.951	6	Sandy Silt to Clayey Silt	19.879	0.44	4.465	1.433	3.031	3.16	0.51997	0.56	0.108							0.80	1	
76.608	6	Sandy Silt to Clayey Silt	17.302	0.337	4.502	1.454	3.048	3.2	0.51789	0.56	0.108							0.80	1	
77.264	6	Sandy Silt to Clayey Silt	16.608	0.325	4.54	1.474	3.066	3.23	0.51571	0.56	0.107							0.80	1	
77.92	6	Sandy Silt to Clayey Silt	25.509	0.388	4.578	1.495	3.083	2.98	0.51366	0.56	0.107							0.80	1	
78.576	6	Sandy Silt to Clayey Silt	37.515	0.696	4.615	1.515	3.1	2.88	0.51163	0.55	0.107							0.80	1	
79.232	6	Sandy Silt to Clayey Silt	24.663	0.374	4.653	1.536	3.117	3	0.50961	0.55	0.107							0.80	1	
79.888	8	Sand to Silty Sand	60.787	0.529	4.692	1.556	3.136	2.51	0.50738	0.55	0.107	2.510	0.51	30.84	2.820	86.960	0.14	0.80	1	1.58
80.545	6	Sandy Silt to Clayey Silt	38.933	0.876	4.73	1.577	3.153	2.91	0.5054	0.55	0.107							0.79	1	
81.201	6	Sandy Silt to Clayey Silt	49.049	1.541	4.768	1.597	3.17	2.89	0.50343	0.54	0.106							0.79	1	
81.857	5	Clayey Silt to Silty Clay	52.467	2.014	4.805	1.618	3.188	2.92	0.50137	0.54	0.106							0.79	1	
82.513	5	Clayey Silt to Silty Clay	54.094	2.156	4.843	1.638	3.205	2.92	0.49943	0.54	0.106							0.79	1	
83.169	6	Sandy Silt to Clayey Silt	50.349	1.757	4.88	1.659	3.222	2.91	0.49751	0.54	0.106							0.79	1	
83.825	6	Sandy Silt to Clayey Silt	50.84	1.782	4.918	1.679	3.239	2.92	0.49561	0.54	0.106							0.79	1	
84.482	5	Clayey Silt to Silty Clay	46.232	1.729	4.956	1.7	3.256	2.98	0.49372	0.53	0.106							0.79	1	
85.138	6	Sandy Silt to Clayey Silt	48.198	1.562	4.993	1.72	3.273	2.92	0.49184	0.53	0.106							0.79	1	
85.794	6	Sandy Silt to Clayey Silt	84.28	2.24	5.031	1.741	3.29	2.67	0.48998	0.53	0.105							0.79	1	
86.45	8	Sand to Silty Sand	147.623	2.463	5.07	1.761	3.309	2.34	0.48791	0.53	0.105	2.340	0.49	72.03	2.084	150.088	0.39	0.78	1	4.47
87.106	8	Sand to Silty Sand	134.137	1.606	5.11	1.781	3.329	2.3	0.48576	0.53	0.105	2.300	0.49	65.16	1.949	126.976	0.27	0.79	1	3.07
87.762	6	Sandy Silt to Clayey Silt	38.791	0.882	5.148	1.802	3.346	2.94	0.48394	0.52	0.105							0.79	1	
88.419	6	Sandy Silt to Clayey Silt	38.73	1.253	5.185	1.822	3.363	3.03	0.48214	0.52	0.105							0.78	1	
89.075	5	Clayey Silt to Silty Clay	44.261	1.589	5.223	1.843	3.38	3	0.48035	0.52	0.105							0.78	1	
89.731	6	Sandy Silt to Clayey Silt	52.254	1.81	5.261	1.863	3.397	2.92	0.47857	0.52	0.105							0.78	1	
90.387	5	Clayey Silt to Silty Clay	78.482	3.267	5.298	1.884	3.414	2.83	0.47681	0.52	0.105							0.78	1	
91.043	11	Sitff Fine-Grained	109.132	5.17	5.341	1.904	3.437	2.76	0.47444	0.52	0.104							0.78	1	
91.699	6	Sandy Silt to Clayey Silt	102.379	4.079	5.379	1.925	3.454	2.71	0.47271	0.52	0.104							0.78	1	
92.356	6	Sandy Silt to Clayey Silt	89.172	2.745	5.416	1.945	3.471	2.69	0.47099	0.51	0.104							0.78	1	
93.012	6	Sandy Silt to Clayey Silt	74.158	2.564	5.454	1.966	3.488	2.8	0.46928	0.51	0.104							0.78	1	
93.668	6	Sandy Silt to Clayey Silt	66.662	2.426	5.491	1.986	3.505	2.86	0.46759	0.51	0.104							0.78	1	
94.324	5	Clayey Silt to Silty Clay	59.363	2.316	5.529	2.007	3.522	2.93	0.4659	0.51	0.104							0.78	1	
94.98	5	Clayey Silt to Silty Clay	51.774	2.069	5.567	2.027	3.539	3	0.46423	0.51	0.104							0.78	1	
95.636	5	Clayey Silt to Silty Clay	52.227	2.127	5.604	2.048	3.556	3	0.46257	0.51	0.104							0.78	1	
96.293	5	Clayey Silt to Silty Clay	51.495	2.24	5.642	2.068	3.574	3.03	0.46083	0.51	0.104							0.78	1	
96.949	5	Clayey Silt to Silty Clay	42.487	1.741	5.679	2.089	3.591	3.1	0.45919	0.50	0.104							0.77	1	
97.605	5	Clayey Silt to Silty Clay	57.097	2.082	5.717	2.109	3.608	2.94	0.45757	0.50	0.104							0.77	1	
98.261	5	Clayey Silt to Silty Clay	69.179	3.098	5.754	2.13	3.625	2.92	0.45596	0.50	0.104							0.77	1	
98.917	11	Sitff Fine-Grained	75.496	3.721	5.797	2.15	3.647	2.93	0.45389	0.50	0.103							0.77	1	
99.573	7	Silty Sand to Sandy Silt	118.583	2.295	5.836	2.17	3.665	2.5	0.45221	0.50	0.103	2.500	0.45	53.62	2.768	148.456	0.38	0.77	1	4.43
100.23	9	Sand	202.661	0.923	5.877	2.191	3.686	1.96	0.45027	0.50	0.103	1.960	0.45	91.25	1.251	114.165	0.22	0.77	2	2.52
100.886	9	Sand	243.815	1.417	5.917	2.211	3.706	1.94	0.44843	0.50	0.103	1.940	0.45	109.33	1.229	134.370	0.31	0.77	3	3.53
101.542	10	Gravelly Sand to Sand	323.384	2.009	5.959	2.232	3.727	1.85	0.44652	0.50	0.103	1.850	0.45	144.40	1.145	165.346		0.77	4	
102.198	10	Gravelly Sand to Sand	399.89	1.908	6.001	2.252	3.749	1.71	0.44453	0.49	0.103	1.710	0.44	177.76	1.045	185.693		0.77	5	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. The result was used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The result of the test is presented on Plate B-2, "Plasticity Chart".

Unconfined Compression Tests

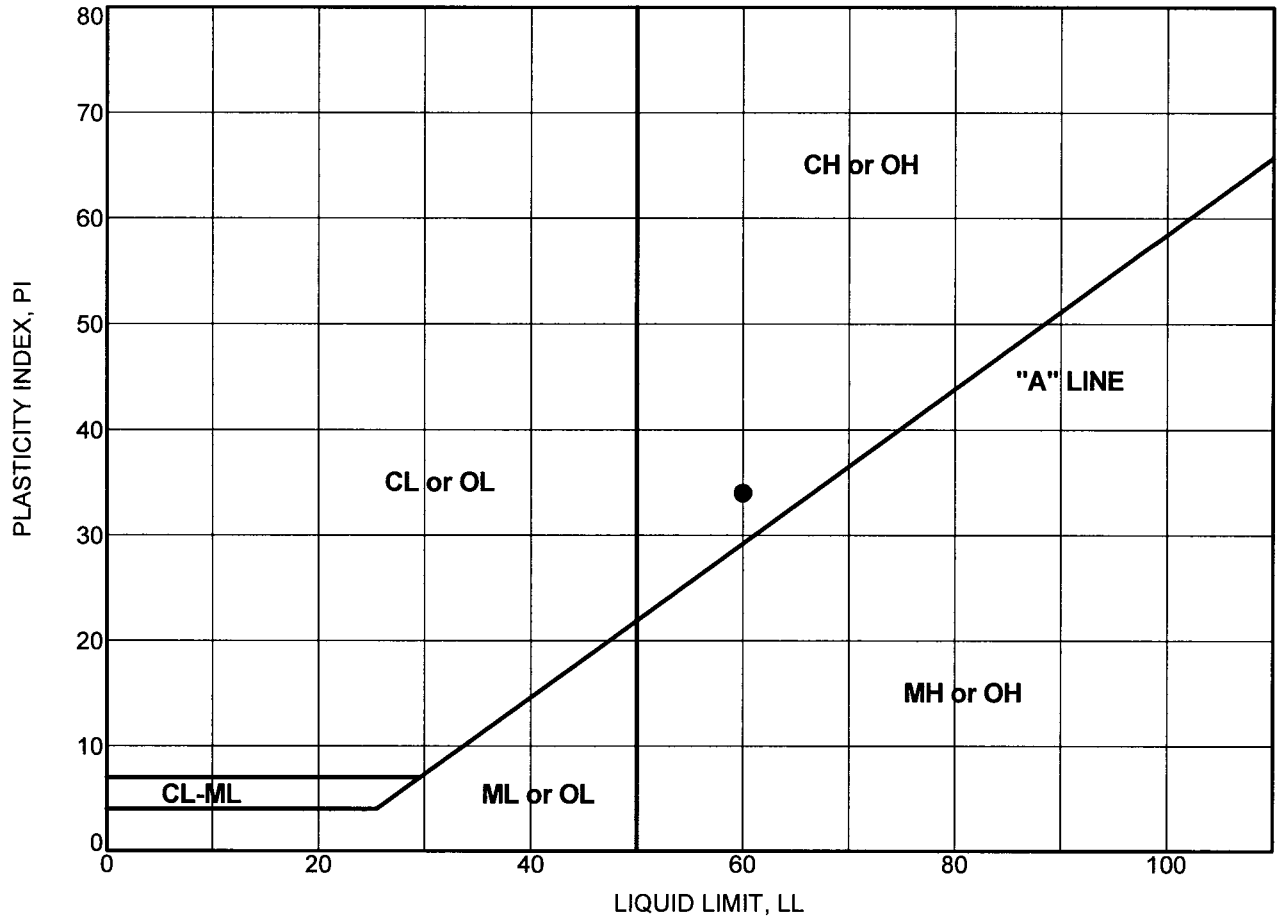
Strength tests were performed on selected undisturbed samples using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-5A through B-5B.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-3.

Direct Shear Tests

Direct Shear test was performed on selected relatively undisturbed sample to determine the shear strength of a soil material in direct shear. The test was performed according to ASTM Test Method D 3080 by Cooper . The test result is presented on Plate B-4.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A-09-001	SPT-5	24.5	●	29	60	26	34	FAT CLAY (CH)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

March Lane UC (Widen)
Stockton, California

JOB NO: 206127.BR (MLB)

PLATE NO: B-2

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/29/2009
Date Submitted 04/24/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR Site ID : R-09-ML-001#8@39.5'
Thank you for your business.

* For future reference to this analysis please use SUN # 55554-111852.

EVALUATION FOR SOIL CORROSION

Soil pH	7.19	
Minimum Resistivity	0.88 ohm-cm (x1000)	
Chloride	54.3 ppm	00.00543 %
Sulfate	71.9 ppm	00.00719 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

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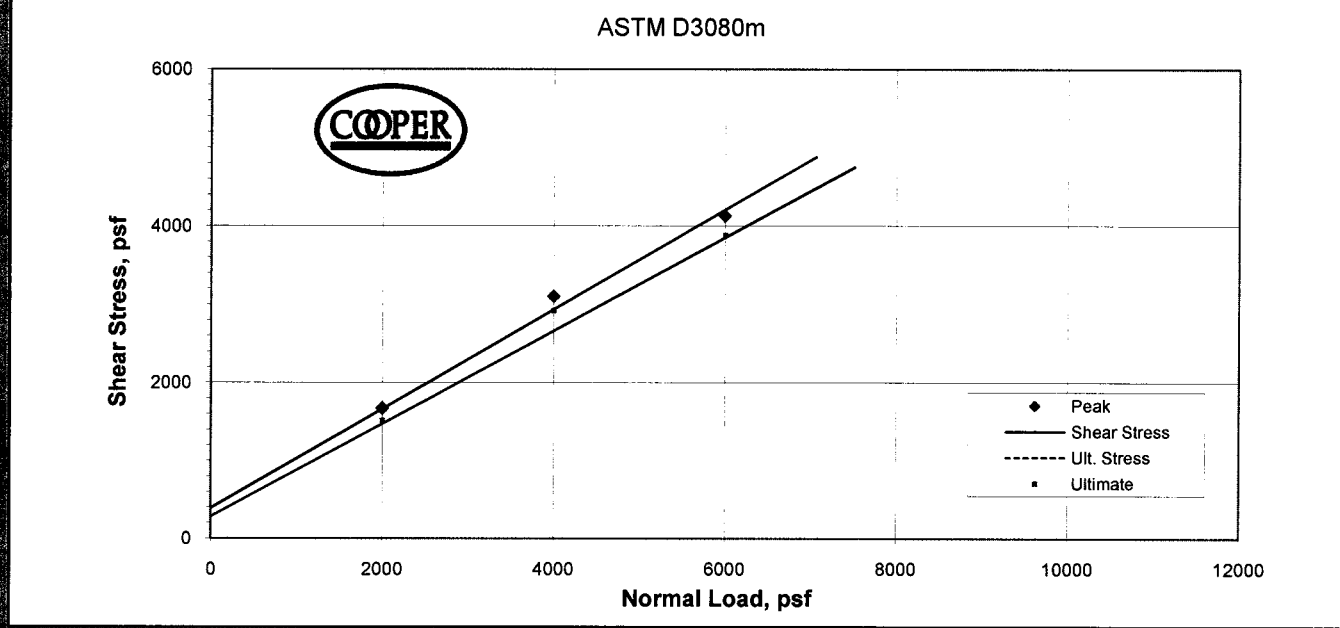
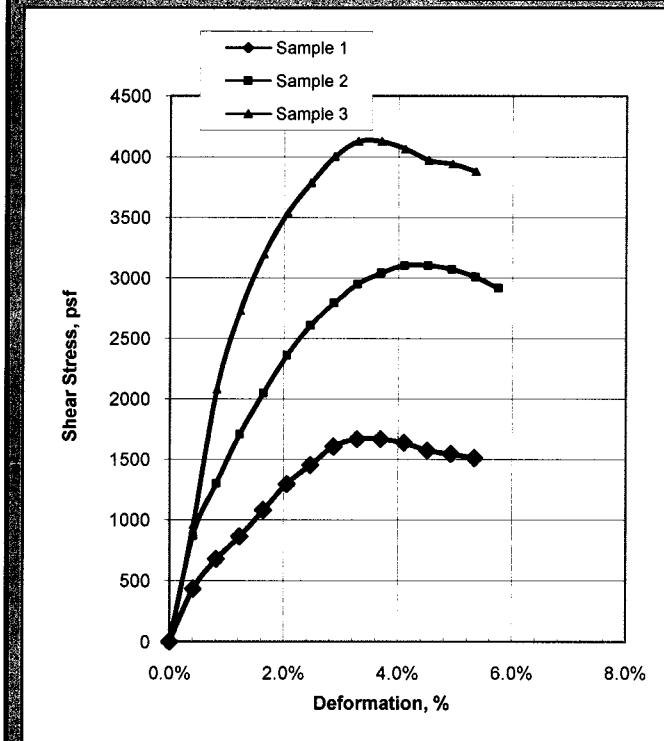
MAY 1 2009
By

Direct Shear



P. Phi (degrees)	30.0	Ult. Phi (degrees)	27.5
P. Cohesion(psf)	400	Ult. Cohesion (psf)	300



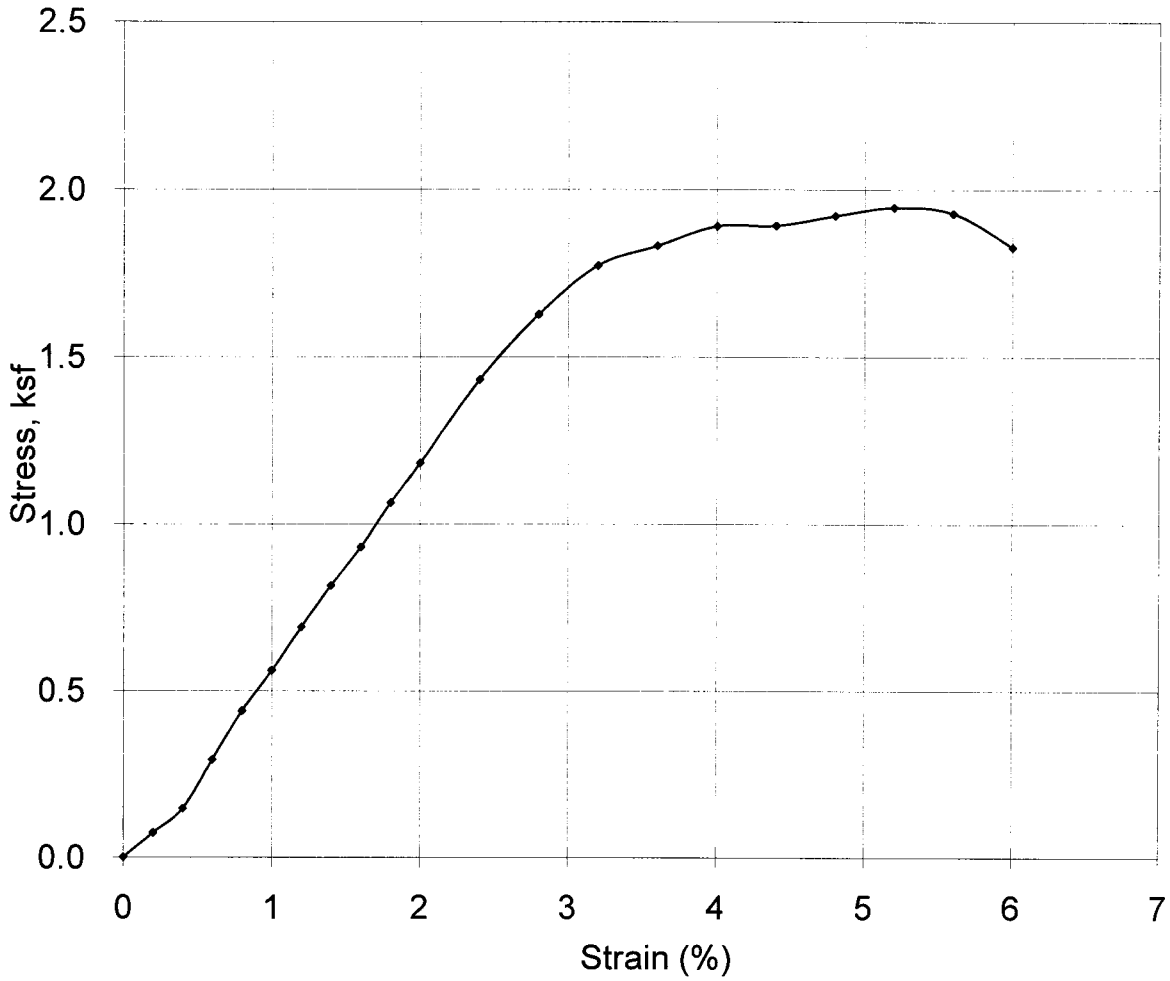
	Sample Data: Initial			
	1	2	3	4
Moisture %	26.5	26.7	26.0	
Dry Dens., pcf	96.7	95.6	96.1	
Void Ratio	0.743	0.763	0.754	
Saturation %	96.5	94.5	93.0	
Diameter	2.44	2.43	2.43	
Height	1.01	1.02	1.02	
	Sample Data: At Test			
Moisture %	25.9	25.6	24.6	
Dry Dens., pcf	99.0	99.3	100.3	
Void Ratio	0.703	0.697	0.680	
Saturation %	99.4	99.2	97.8	
Diameter	2.44	2.43	2.43	
Height	0.99	0.98	0.98	
Normal Stress, psf	2000	4000	6000	
Shear Stress, psf	1670	3105	4130	
Strengths picked at	3.7%	4.5%	3.7%	
Ult. Stress, psf	1515	2919	3881	
Strain Rate, in/min	0.020	0.020	0.020	

CTL #	157-272
Client:	Parikh Consultants, Inc.
Project	206127.GDR
Tested By:	MD
Reduced By:	RU
Date:	4/20/2009

Specimen #	Boring:	Sample:	Depth, ft:	Visual Soil Classification
1	A-09-ML-001	#9	44.5	Olive Brown Clayey SAND (Silty)
2	A-09-ML-001	#9	44.5	Olive Brown Clayey SAND (Silty)
3	A-09-ML-001	#9	44.5	Olive Brown Clayey SAND (Silty)
4				

Remarks: *DS-CU* A fully undrained condition may not be attained in this test.

UNCONFINED COMPRESSION TEST



Boring No.: A-09-ML-001

Sample No. : MC-8

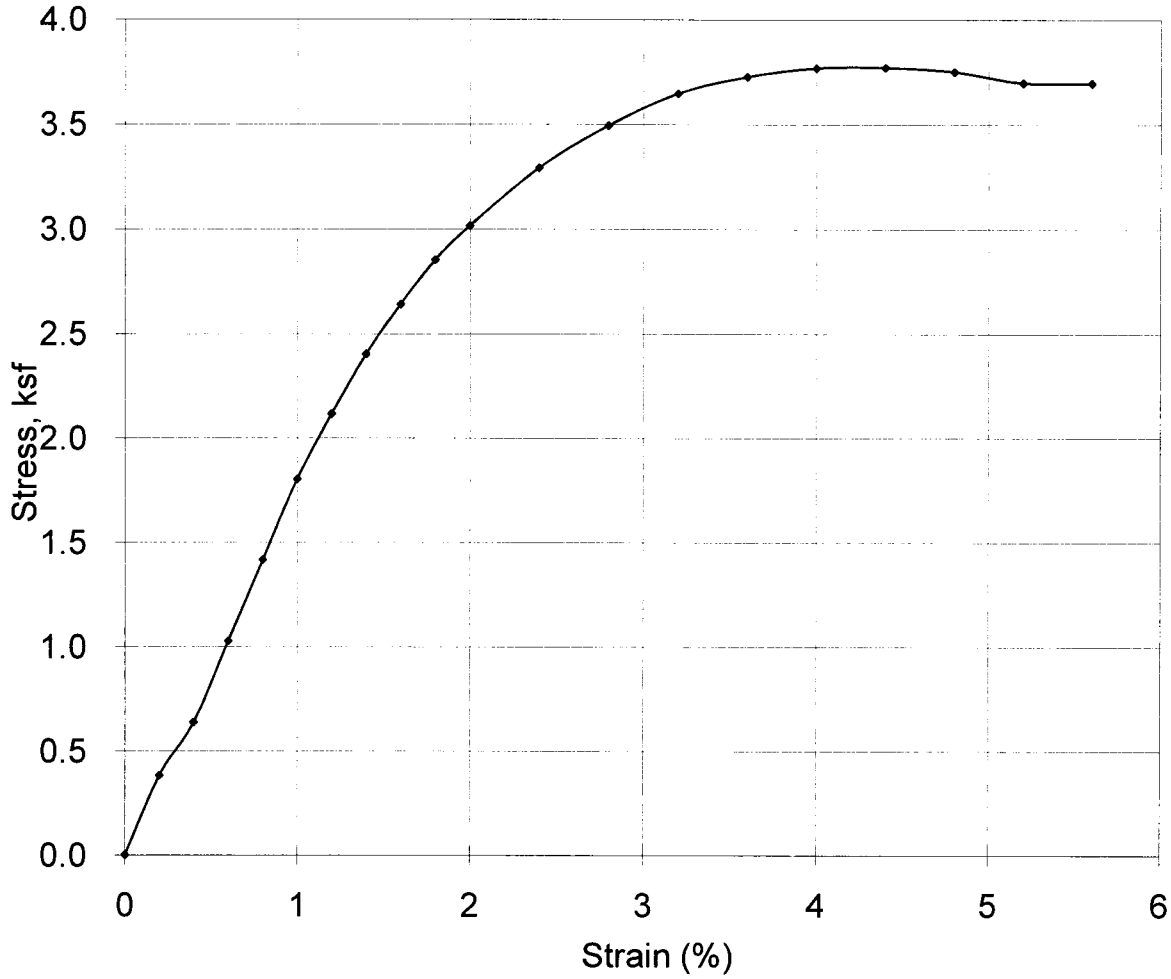
Depth (feet): 39.5

Material Description:
Lean Clay

Unconfined Compressive Strength: 2.0 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**MARCH LANE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5A

UNCONFINED COMPRESSION TEST



Boring No.: A-09-ML-001

Sample No. : MC-12

Depth (feet): 59.5

Material Description:
Fat Clay

Unconfined Compressive Strength: 3.8 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**MARCH LANE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5B

APPENDIX C

Evaluation of Liquefaction Potential

Evaluation of Liquefaction Potential + Marsh Lane (Boring A-09-ML-001)

TEF

TEF

Depth	Soil	Avg SPT N	γ'
12'	Silty Sand (Medium dense)	23	120 pcf
32'	Silty Sand (Dense)	34	125 pcf
16'			
5'	Lean Clay	13	115 pcf
10'	Sandy Lean Clay	27	120 pcf, γ_{sat} = 130 pcf
5'	Lean Clay	15	γ_{sat} = 130 pcf
7.5'	Silty Sand (Medium dense)	21	γ_{sat} = 130 pcf
13.5'	Lean Clay	22	γ_{sat} = 130 pcf
25'	Silty Sand (Dense)	33	γ_{sat} = 135 pcf
22'	Lean Clay	48	γ_{sat} = 130 pcf

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-ML-001 (March Lane UC)(Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 32

SOIL GROUPS

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR$_{7.0}$/CSR)*MSF*Kσ*Kα*

Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	*CYCLIC STRESS RATIO (CSR)* σ_v (psf)	σ_v' (psf)	γ_d	CSR	*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)* SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	12.0	1	23	SPT	1440.0	1440.0	0.95	0.12	27	1.18	1	0.80	1.2	1.15	34.53	15%	38.69		1.00	1	
2	22.0	1	34	SPT	2690.0	2690.0	0.90	0.12	39	0.86	1	0.80	1.2	1.15	37.35	15%	41.64		0.91	1	
3	27.0	2	13	SPT	3265.0	3265.0	0.86		15	0.78	1	0.80	1.2	1.15	12.96	55%			0.86	1	
4	37.0	2	27	SPT	4515.0	4202.5	0.78		31	0.69	1	0.80	1.2	1.15	23.73	55%			0.80	1	
5	42.0	2	15	SPT	5165.0	4540.0	0.73		17	0.66	1	0.80	1.2	1.15	12.68	55%			0.78	1	
6	49.5	1	21	SPT	6140.0	5046.3	0.66	0.10	24	0.63	1	0.80	1.2	1.15	16.84	15%	20.15	0.22	0.76	1	1.88
7	63.0	2	22	SPT	7895.0	5957.5	0.56		25	0.58	1	0.80	1.2	1.15	16.24	55%			0.72	1	
8	88.0	1	33	SPT	11270.0	7770.0	0.48	0.09	38	0.51	1	0.80	1.2	1.15	21.33	15%	24.85	0.29	0.67	1	2.50
9	110.0	2	48	SPT	14130.0	9255.0	0.46		55	0.46	1	0.80	1.2	1.15	28.42	55%			0.63	1	

Vertical Pile Capacity Calculations/Lateral Pile Capacity Analyses

Rajappan & Meyer
CONSULTING ENGINEERS INC

Project Name: ***I-5 North Stockton Corridor Improvement***
Bridge Name: ***March Lane UC (Widen)***
Br No: ***29-0193 R/L***
Date: ***April 7, 2010***

FOUNDATION DESIGN INFORMATION FROM BRIDGE DESIGNER TO GEOTECHNICAL ENGINEER

MARCH LANE UC (WIDEN)

Table 1. Foundation Design Data Sheet

Location	Design Method	Pile Type	Finish Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		No. of Piles Per Support
					B	L	
Abut 1	WSD	Class 200 Alt "W"	14.80	11.80	2.50	52.00	12
Abut 2	WSD	Class 200 Alt "W"	14.00	10.87	2.50	52.00	12

Note: Finish Grade is at the front face of abutment diaphragm; Pile Cut-off Elevations are the Bottom of Footing Elevations

Table 2. Foundation Design Loads

Location	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent	Compression		Tension		Compression		Tension	
	Per Support	Max per Pile	Per Support	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile
Abut 1	2001	167	1547	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	2001	167	1547	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

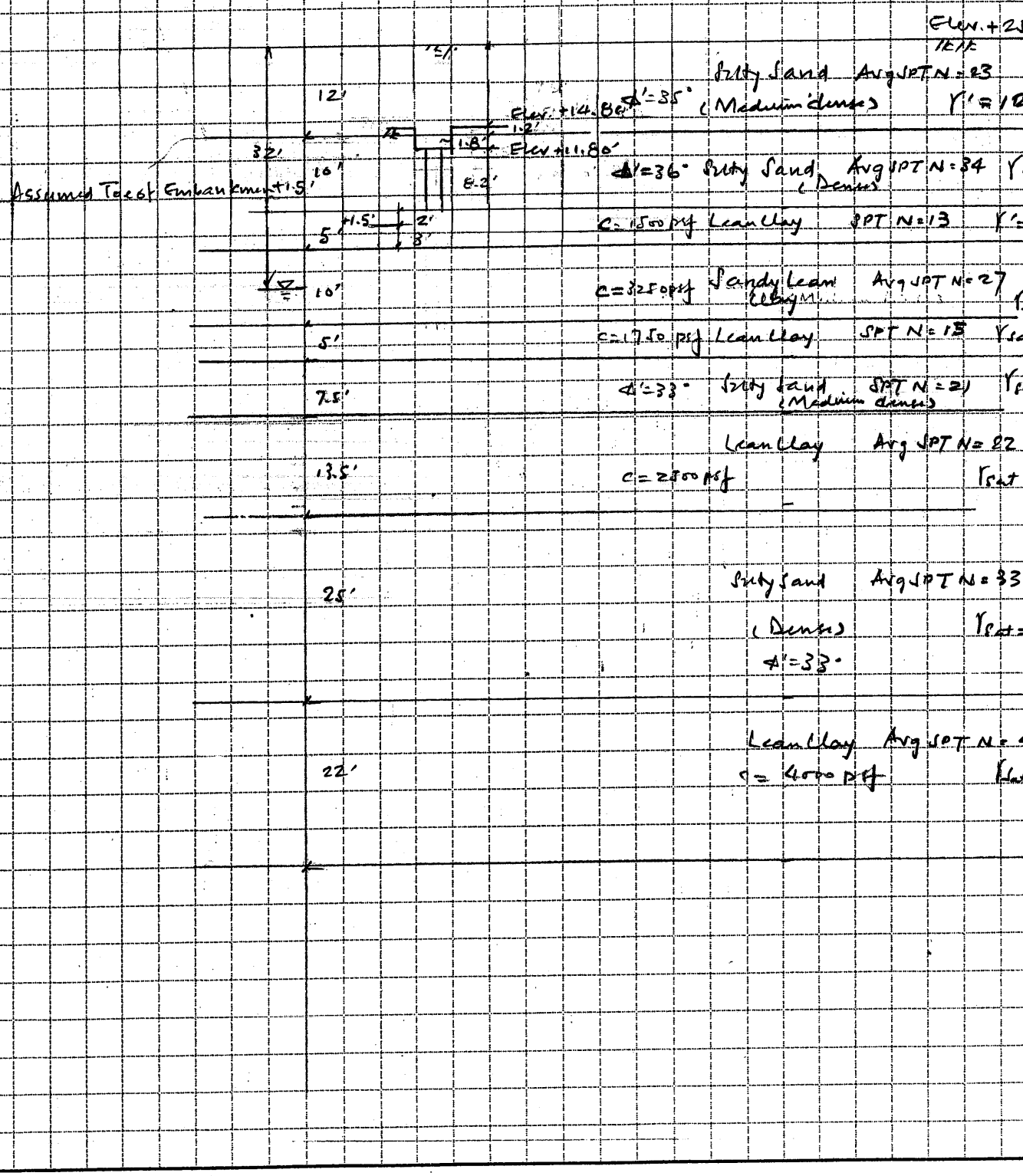
Table 3. Lateral Load on Piles

Per Support	Service Limit (kips/pile)	Max per Pile
Abut 1	Negligible	36
Abut 2	Negligible	36

PARIKH
Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement 26627.80
SHEET NO 1 OF 5
CALCULATED BY A. Lam DATE 4/2010
CHECKED BY DATE
SCALE

Pile Capacity Analysis : March Lane (Boring A-09-001) - Abutment 1
16" ⌀ Alt "W" pile - 94 tons Service Load



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 March Lane UC (Widen)*** ***Abutment 1 - 85 tons service load (170 ton*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/07/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.33 — 1- GRAVELS, SANDS AND NONPLASTIC SILTS — (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 1 (1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 2 (1=CONCRETE ,2=STEEL) — SOIL TYPES — GRAVEL, SAND, NSILT Ref Boring : A-09-ML-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) 4.19 SQ. FT.

DRIVEN/DRILLED = 1 (1=DRIVEN ,2=DRILLED) — CLAY, PSILT — AREA OF THE PILE TIP 1.40 SQ. FT.

Class 200 Alt. "W" - 16" diameter steel pipe pile

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	125	375	188	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	11.0		1	125	1000	875	SAND	V. DENSE	0	41	0.00	17.24	17.24	5.77	5.77
11.0	13.0		2	115	230	1490	CLAY	STIFF	1500	0	0.80	5.03	22.26	5.03	10.80
13.0	16.0		2	115	345	1778	CLAY	STIFF	1500	0	0.80	7.54	29.81	7.54	18.34
16.0	21.0		2	120	600	2250	CLAY	V.STIFF	3250	0	0.25	8.51	38.32	8.51	26.85
21.0	26.0		2	68	340	2720	CLAY	V.STIFF	3250	0	0.25	8.51	46.83	8.51	35.37
26.0	31.0		2	68	340	3060	CLAY	STIFF	1750	0	0.75	13.75	60.58	13.75	49.11
31.0	35.0		1	68	272	3366	SAND	DENSE	0	36	0.00	19.12	79.70	6.41	55.52
35.0	38.5		1	68	238	3621	SAND	DENSE	0	36	0.00	16.65	96.35	5.58	61.10
38.5	43.5		2	68	340	3910	CLAY	V.STIFF	2500	0	0.80	20.95	117.30	20.95	82.05
43.5	48.5		2	68	340	4250	CLAY	V.STIFF	2500	0	0.80	20.95	138.25	20.95	103.00
48.5	52.0		2	68	238	4539	CLAY	V.STIFF	2500	0	0.80	14.67	152.91	14.67	117.66
52.0	57.0		1	68	340	4828	SAND	DENSE	0	37	0.00	24.71	177.62	8.28	125.94
57.0	62.0		1	68	340	5168	SAND	DENSE	0	37	0.00	24.56	202.18	8.23	134.17
62.0	65.5		1	68	238	5457	SAND	DENSE	0	37	0.00	17.11	219.30	5.73	139.90
65.5	72.0		1	68	442	5797	SAND	DENSE	0	36	0.00	31.61	250.91	10.59	150.49
72.0	77.0		1	68	340	6188	SAND	DENSE	0	36	0.00	24.17	275.08	8.10	158.59
77.0	82.0		2	68	340	6528	CLAY	V.STIFF	4000	0	0.50	20.95	296.03	20.95	179.54
82.0	86.0		2	68	272	6834	CLAY	V.STIFF	4000	0	0.50	16.76	312.79	16.76	196.30
86.0	93.0		2	68	476	7208	CLAY	V.STIFF	4000	0	0.40	23.46	336.25	23.46	219.76
93.0	99.0		2	68	408	7650	CLAY	V.STIFF	4000	0	0.40	20.11	356.37	20.11	239.88

Abutment 1 --> Pile cut-off elevation is +11.8 feet. Finished grade elevation is +14.8 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +1.5 feet

Pile Length @ Abutment 1 = 72.0' - 3.0' = 69.0'. Pile spacing at 3 feet (or 2.25 pile diameter) center-to-center. Vertical pile group efficiency is 0.8125

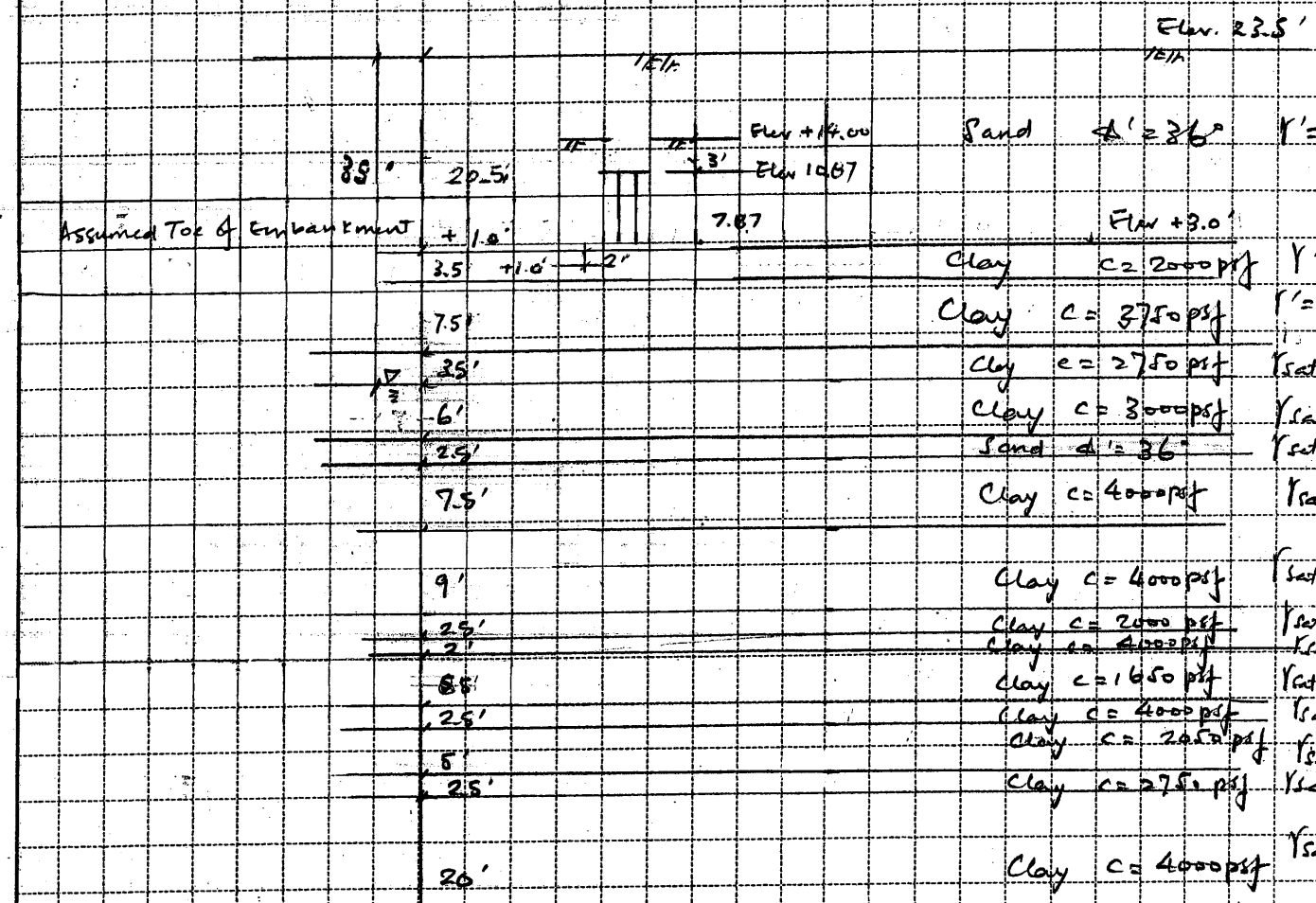
167 kips/pile (Service Load). Required pile capacity = 167 kips /(0.5*0.8125) = 420 kips (210 tons)

Pile tip at 69.0' below pile cut-off (pile tip @ Elev. -57.5', pile cut-off @ Elev. +11.8'), Ult. Capacity = 250.9 tons (friciton) - 22.3 tons (predrilled) = 228.6 tons > required capacity of 420 k/210 tons (Service Limit State), OK

I-5 March Lane UC Abut 1 Class 200 Alt W (4-07-10)

Sheet 2 of 5

Pile capacity Analyses - March Lane IC (Widen) - CPT-09-001 Abutment 2 - 16" ϕ
Alt "W" pile - 94 tons service loads.



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 March Lane UC (Widen)*** ***Abutment 2 - 85 tons service load (170 ton*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/07/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.33 — 1- GRAVELS, SANDS AND NONPLASTIC SILTS — (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 1 (1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 2 (1=CONCRETE ,2=STEEL) — SOIL TYPES: GRAVEL, SAND, NSILT — Ref Boring : CPT-09-ML-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) : 4.19 SQ. FT.

DRIVEN/DRILLED = 1 (1=DRIVEN ,2=DRILLED) — CLAY, PSILT — AREA OF THE PILE TIP 1.40 SQ. FT.

Alt. "W" - 16" diameter steel pipe pile

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	125	375	188	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	11.0		1	125	1000	875	SAND	V. DENSE	0	41	0.00	17.24	17.24	5.77	5.77
11.0	13.0		2	115	230	1490	CLAY	STIFF	2000	0	0.80	6.70	23.94	6.70	12.48
13.0	14.5		2	115	172.5	1691	CLAY	STIFF	2000	0	0.80	5.03	28.97	5.03	17.51
14.5	22.0		2	120	900	2228	CLAY	V.STIFF	3750	0	0.25	14.73	43.70	14.73	32.24
22.0	25.5		2	115	402.5	2879	CLAY	V.STIFF	2750	0	0.75	15.12	58.82	15.12	47.36
25.5	28.5		2	68	204	3182	CLAY	V.STIFF	3000	0	0.40	7.54	66.36	7.54	54.90
28.5	31.5		2	68	204	3386	CLAY	V.STIFF	3000	0	0.40	7.54	73.91	7.54	62.44
31.5	34.0		1	68	170	3573	SAND	DENSE	0	36	0.00	11.99	85.90	4.02	66.46
34.0	39.0		2	68	340	3828	CLAY	V.STIFF	4000	0	0.40	16.76	102.66	16.76	83.22
39.0	44.0		2	68	340	4168	CLAY	V.STIFF	4000	0	0.40	16.76	119.42	16.76	99.98
44.0	50.5		2	68	442	4559	CLAY	V.STIFF	4000	0	0.40	21.79	141.21	21.79	121.77
50.5	53.0		2	68	170	4865	CLAY	STIFF	2000	0	0.85	8.90	150.11	8.90	130.67
53.0	55.0		2	68	136	5018	CLAY	V.STIFF	4000	0	0.30	5.03	155.14	5.03	135.70
55.0	60.5		2	68	374	5273	CLAY	STIFF	2000	0	0.90	20.74	175.88	20.74	156.44
60.5	63.0		2	68	170	5545	CLAY	V.STIFF	4000	0	0.30	6.29	182.16	6.29	162.73
63.0	68.0		2	68	340	5800	CLAY	V.STIFF	2500	0	0.85	22.26	204.42	22.26	184.99
68.0	70.5		2	68	170	6055	CLAY	V.STIFF	3000	0	0.55	8.64	213.06	8.64	193.63
70.5	78.0		2	68	510	6395	CLAY	V.STIFF	4000	0	0.30	18.86	231.92	18.86	212.48
78.0	83.0		2	68	340	6820	CLAY	V.STIFF	4000	0	0.30	12.57	244.49 ←	12.57	225.05
83.0	86.0		2	68	204	7092	CLAY	V.STIFF	4000	0	0.30	7.54	252.03	7.54	232.59
86.0	90.5		2	68	306	7347	CLAY	V.STIFF	4000	0	0.30	11.31	263.34	11.31	243.91

Abutment 2 --> Pile cut-off elevation is +10.87 feet. Finished grade elevation is +14.0 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +1.0 feet

Pile Length @ Abutment 2 = 83.0' - 3.0' = 80.0'. Pile spacing at 3 feet (or 2.25 pile diameter) center-to-center. Vertical pile group efficiency is 0.8125

167 kips/pile (Service Load). Required pile capacity = 167 kips /(0.5*0.8125) = 420 kips (210 tons)

Pile tip at 80.0' below pile cut-off(pile tip @ Elev. -69.5', pile cut-off @ Elev. +10.87'), Ult. Capacity = 244.5 tons (friciton) - 23.9 tons (predrill) = 220.6 tons > required capacity of 420 k/210 tons (Service Limit State), OK

I-5 March Lane UC Abut 2 Class 200 Alt W (4-07-10)

Sheet 4 of 5

MARCH LANE UC (WIDEN)
Sheet 5 of 5

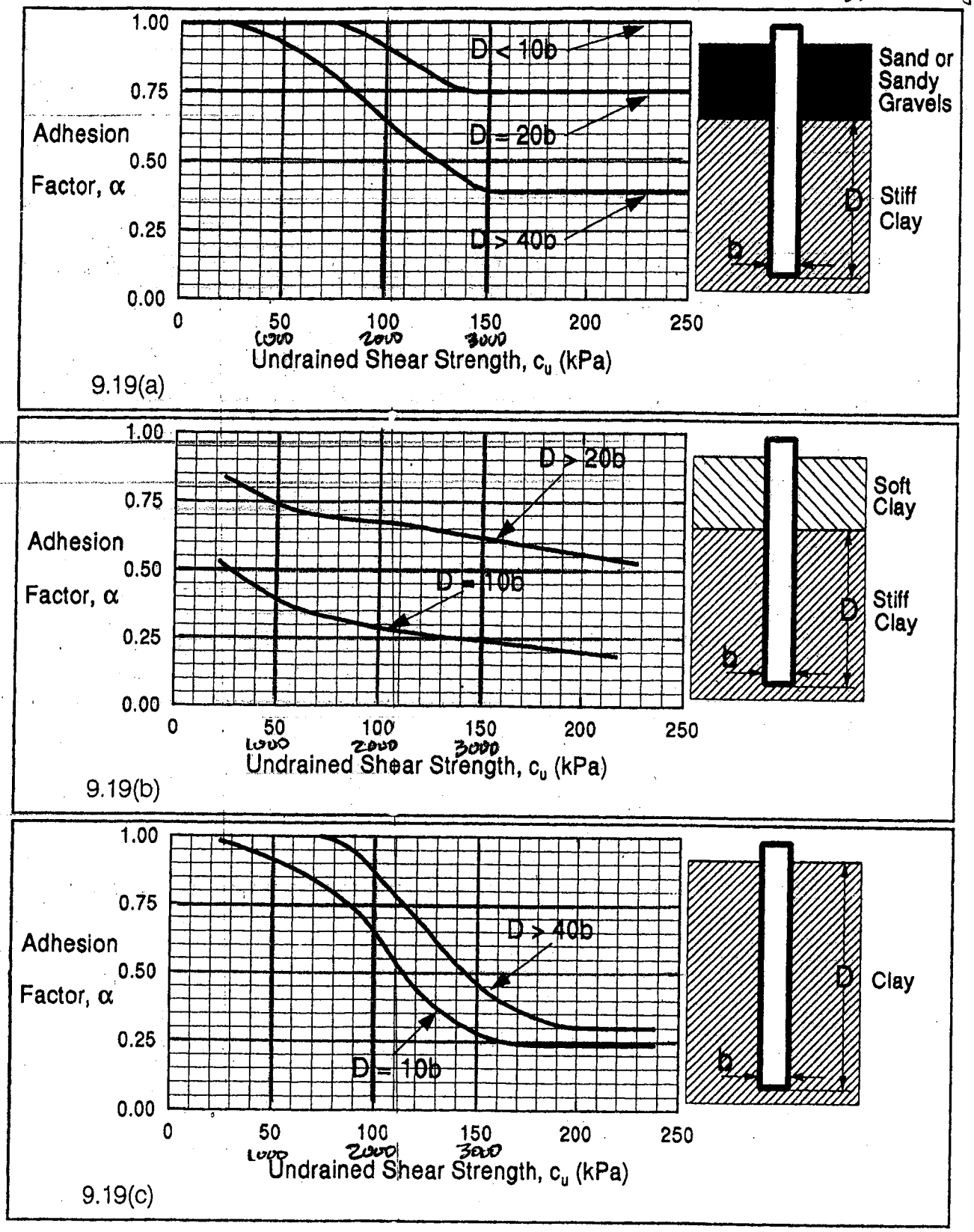
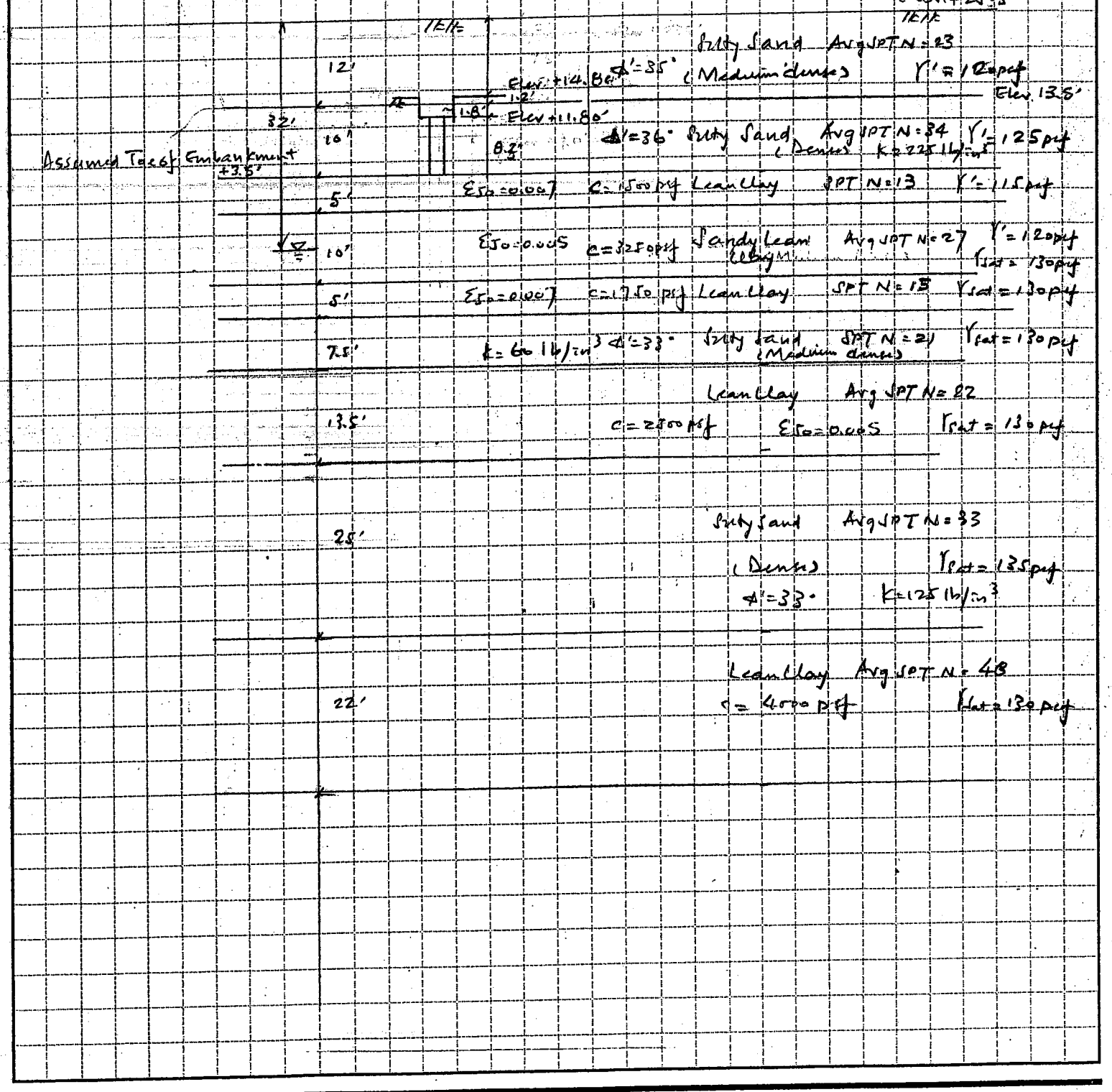


Figure 9.19 Adhesion Factors for Driven Piles in Clay (after Tomlinson, 1980)

Lateral Pile Capacity Analysis : March Lane (Boring A-09-001) - Abutment 1
16" ⌀ Alt "W" pile - 94 tons Service Loads



```
LPILEP5
I-5 Stockton-March-Abut-1
1       1       0       0       0       0
100     2       -36     480     0
0       16      731.94  24.74   30000000
480     16      731.94  24.74   30000000
8       16      16      0       2
4       -36     102     225     225
3       102     162     0       0
3       162     222     0       0
3       222     282     0       0
3       282     342     0       0
4       342     432     60      60
3       432     594     0       0
4       594     894     125     125
-36     0.072
102     0.072
102     0.067
162     0.067
162     0.069
222     0.069
222     0.039
282     0.039
282     0.039
342     0.039
342     0.039
432     0.039
432     0.039
594     0.039
594     0.042
894     0.042
-36     0       36      0       0
102     0       36      0       0
102     10.4    0       0.007   0
162     10.4    0       0.007   0
162     22.56   0       0.005   0
222     22.56   0       0.005   0
222     22.56   0       0.005   0
282     22.56   0       0.005   0
282     12.15   0       0.007   0
342     12.15   0       0.007   0
342     0       33      0       0
432     0       33      0       0
432     17.36   0       0.005   0
594     17.36   0       0.005   0
594     0       33      0       0
894     0       33      0       0
0       0.6     1
1248    0.6     1
0       1       1
1
1       36000   0       167000
0
1       1       0
100     1E-5    100
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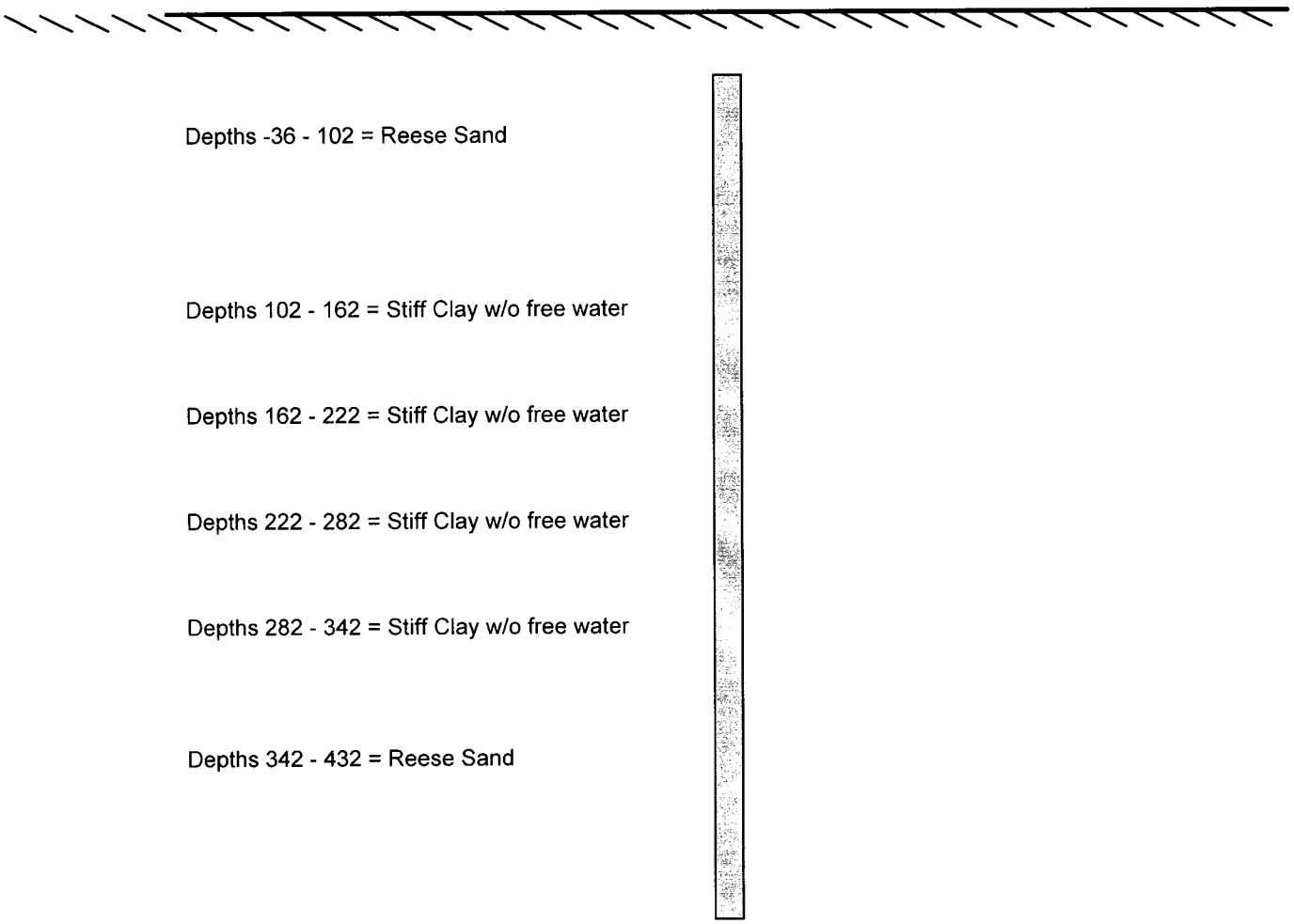
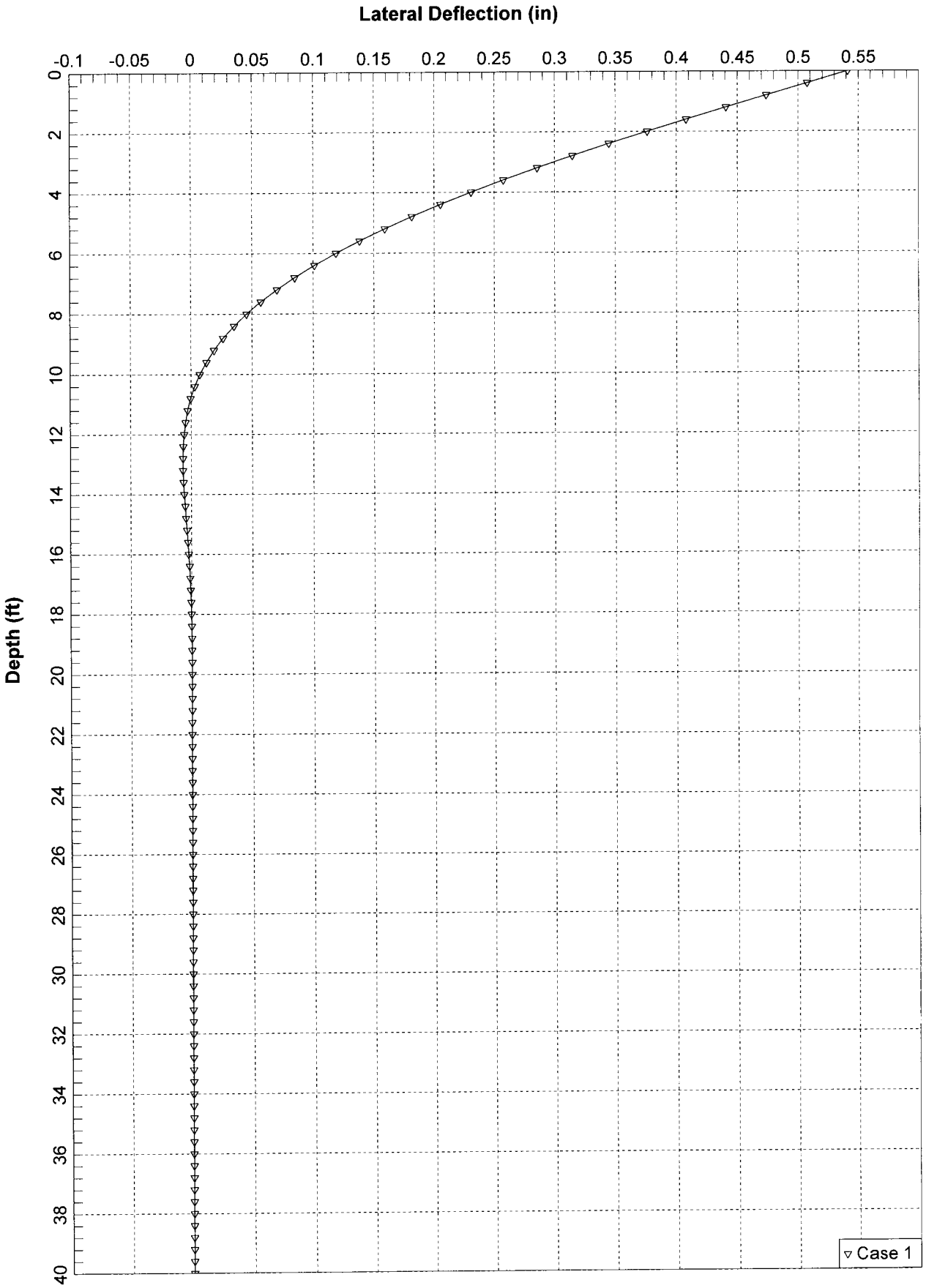
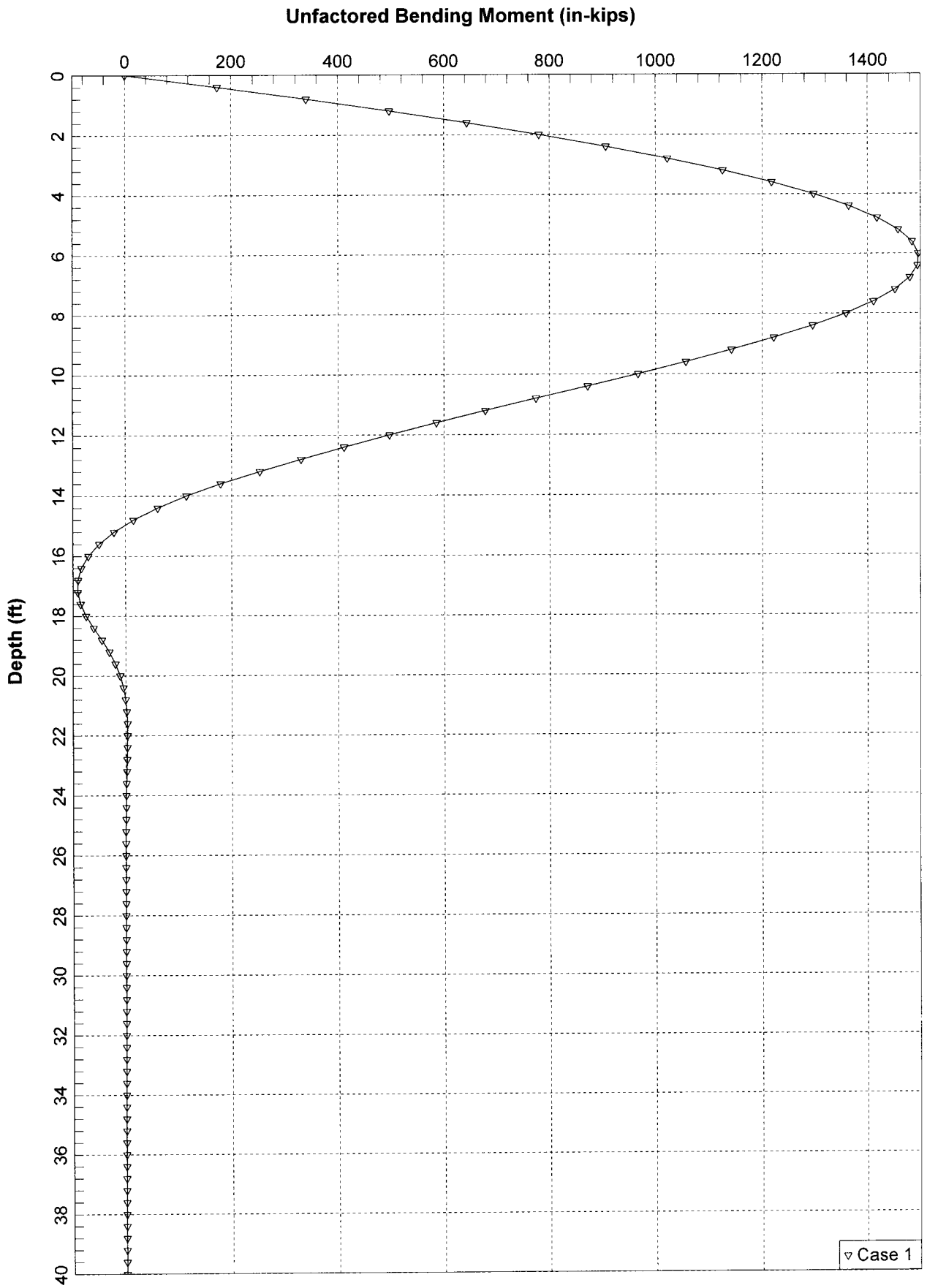
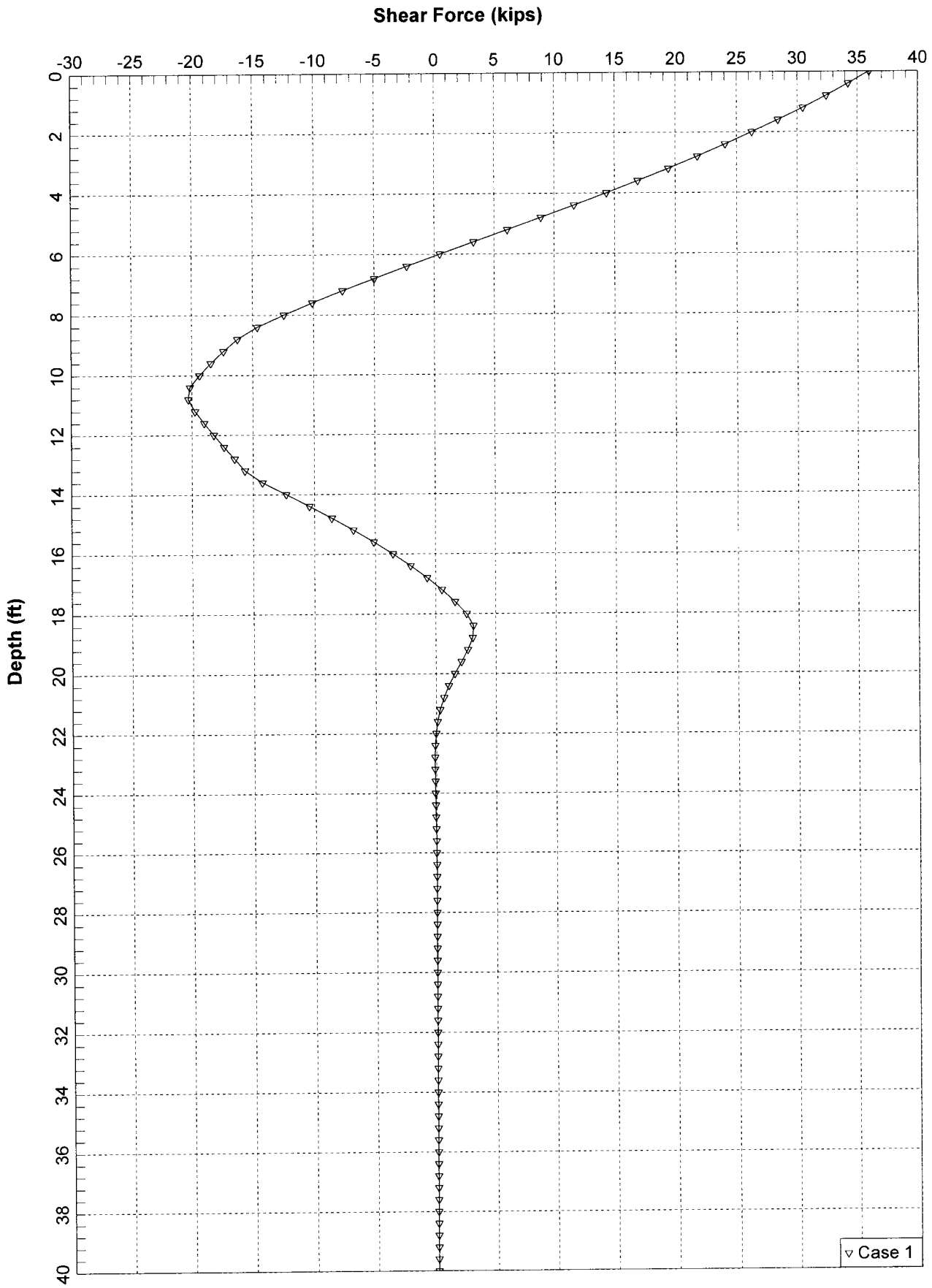

Depths -36 - 102 = Reese Sand
Depths 102 - 162 = Stiff Clay w/o free water
Depths 162 - 222 = Stiff Clay w/o free water
Depths 222 - 282 = Stiff Clay w/o free water
Depths 282 - 342 = Stiff Clay w/o free water
Depths 342 - 432 = Reese Sand
Depths 432 - 594 = Stiff Clay w/o free water
Depths 594 - 894 = Reese Sand


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
0.55
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

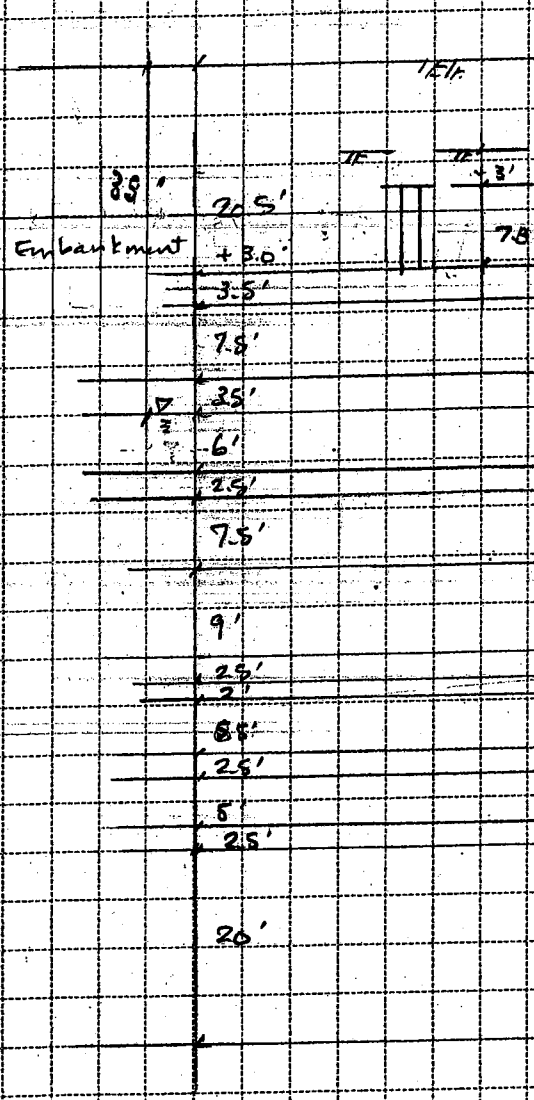

Unfactored Bending Moment (in-kips)
0
200
400
600
800
1000
1200
1400
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-30
-25
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

PARIKH
Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement - March Lane Ue Widen
SHEET NO ____ OF ____
CALCULATED BY A Lam DATE 12/09
CHECKED BY ____ DATE ____
SCALE ____

Lateral Pile Capacity Analyses March Lane UC (Widen) - CPT-09-001 Abutment 2-16" ϕ
Alt "W" pile - 94 tons service loads.



	Sand	ϕ' = 36°	γ' = 125 pcf
	k = 225 lb/in³		
ε50 = 0.007	Clay	c = 2000 psf	γ' = 115 pcf
ε50 = 0.005	Clay	c = 3750 psf	γ' = 120 pcf
ε50 = 0.005	Clay	c = 2750 psf	γsat = 115 pcf
ε50 = 0.005	Clay	c = 3000 psf	γsat = 130 pcf
k = 125 lb/in³	Sand	ϕ' = 36°	γsat = 135 pcf
ε50 = 0.005	Clay	c = 4000 psf	γsat = 130 pcf
ε50 = 0.005	Clay	c = 4000 psf	γsat = 130 pcf
	Clay	c = 2000 psf	γsat = 130 pcf
	Clay	c = 4000 psf	γsat = 130 pcf
	Clay	c = 1650 psf	γsat = 130 pcf
	Clay	c = 4000 psf	γsat = 130 pcf
	Clay	c = 2050 psf	γsat = 130 pcf
	Clay	c = 2750 psf	γsat = 130 pcf
	Clay	c = 4000 psf	γsat = 130 pcf

```
LPILEP5
I-5 Stockton-March-Abut-2
1       1       0       0       0       0
100     2       -36     480     0
0       16      731.94  24.74   30000000
480     16      731.94  24.74   30000000
8       16      16      0       2
4       -36     96      225     225
3       96      138     0       0
3       138     228     0       0
3       228     270     0       0
3       270     342     0       0
4       342     372     125     125
3       372     462     0       0
3       462     570     0       0
-36     0.072
96      0.072
96      0.067
138     0.067
138     0.069
228     0.069
228     0.067
270     0.067
270     0.039
342     0.039
342     0.042
372     0.042
372     0.039
462     0.039
462     0.039
570     0.039
-36     0       36      0       0
96      0       36      0       0
96      13.88   0       0.007   0
138     13.88   0       0.007   0
138     26      0       0.005   0
228     26      0       0.005   0
228     19.09   0       0.005   0
270     19.09   0       0.005   0
270     20.83   0       0.005   0
342     20.83   0       0.005   0
342     0       36      0       0
372     0       36      0       0
372     27.77   0       0.005   0
462     27.77   0       0.005   0
462     27.77   0       0.005   0
570     27.77   0       0.005   0
0       0.6     1
1206    0.6     1
0       1       1
1
1       36000   0       167000
0
1       1       0
100     1E-5    100
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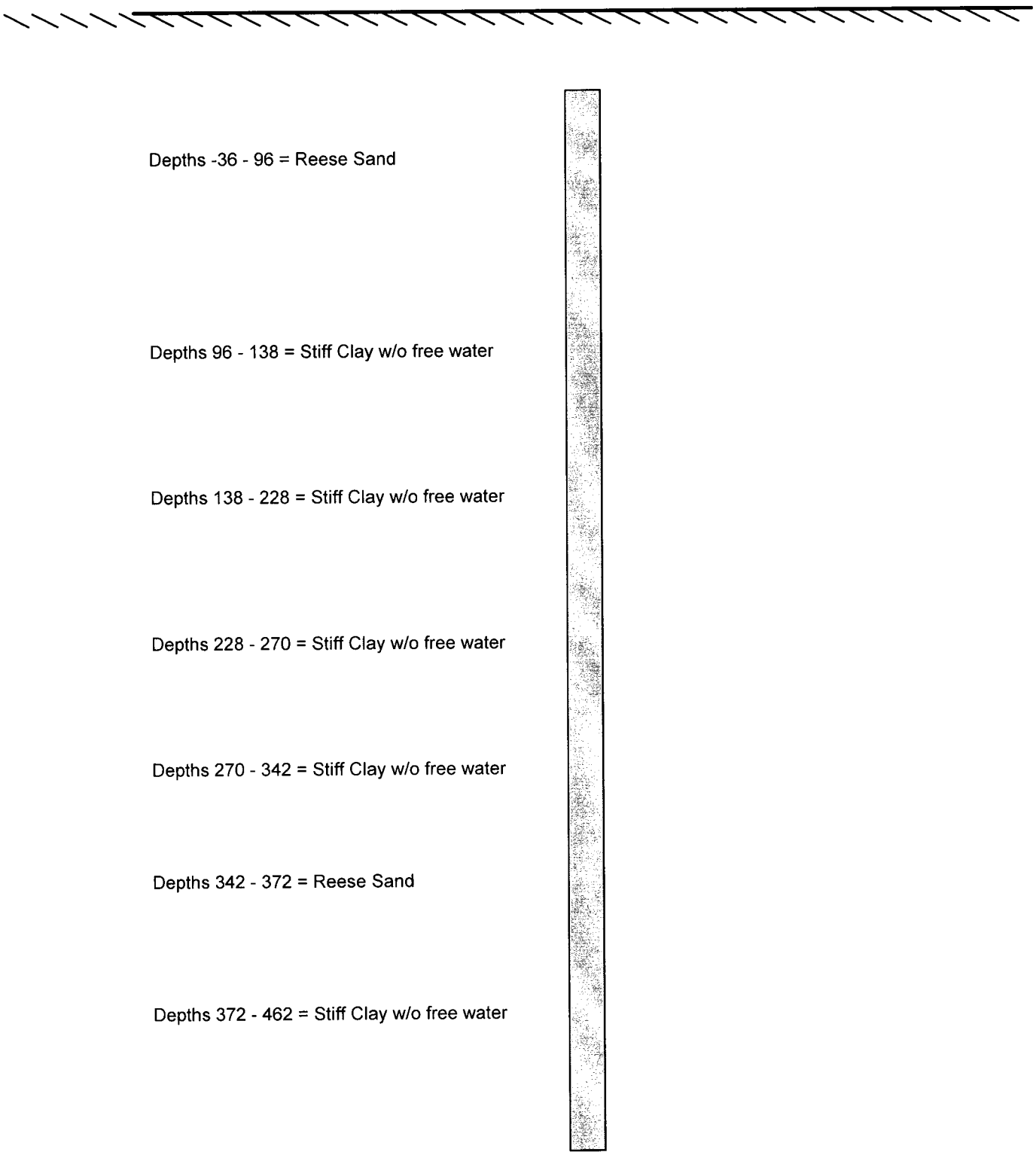
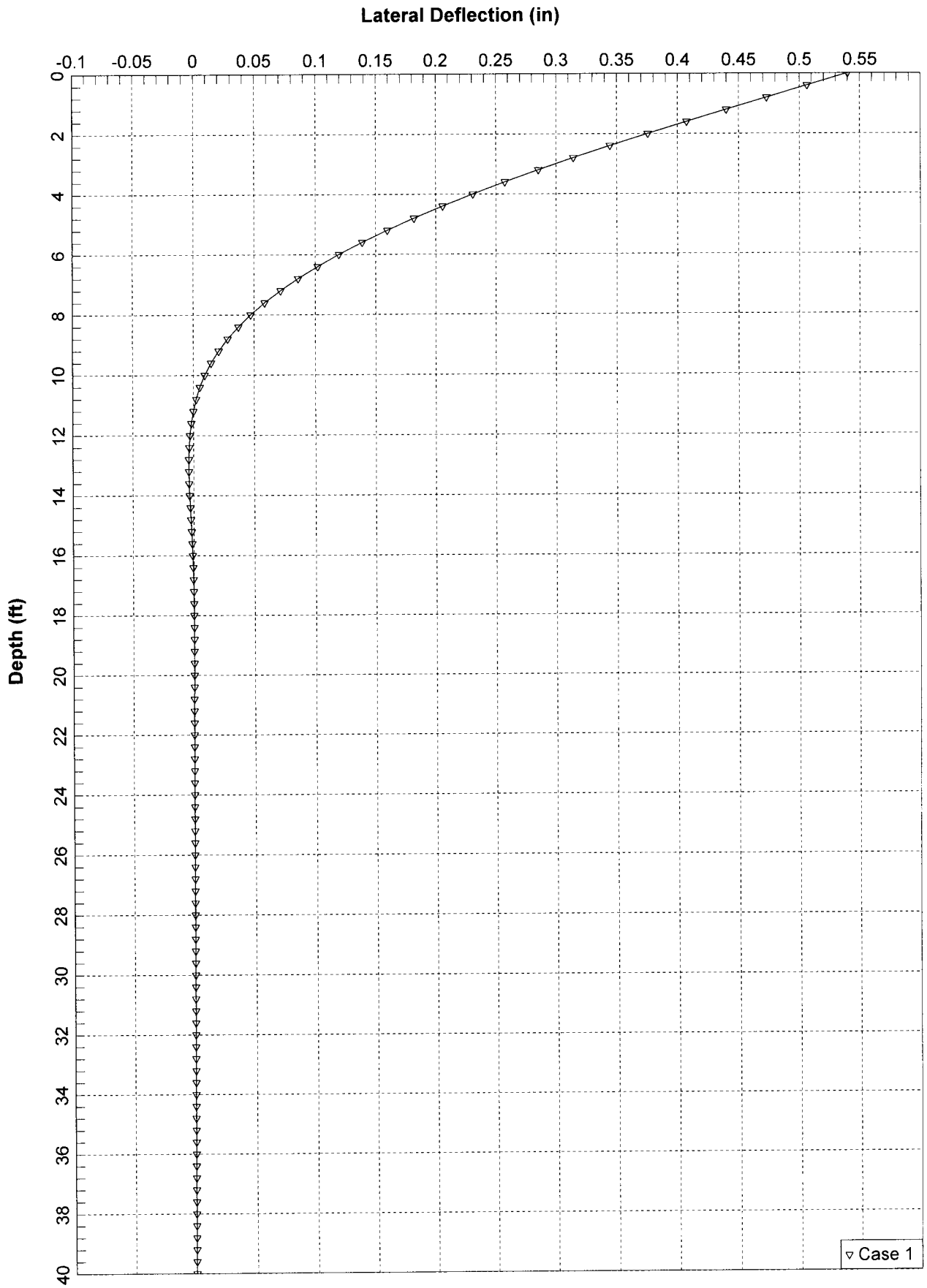
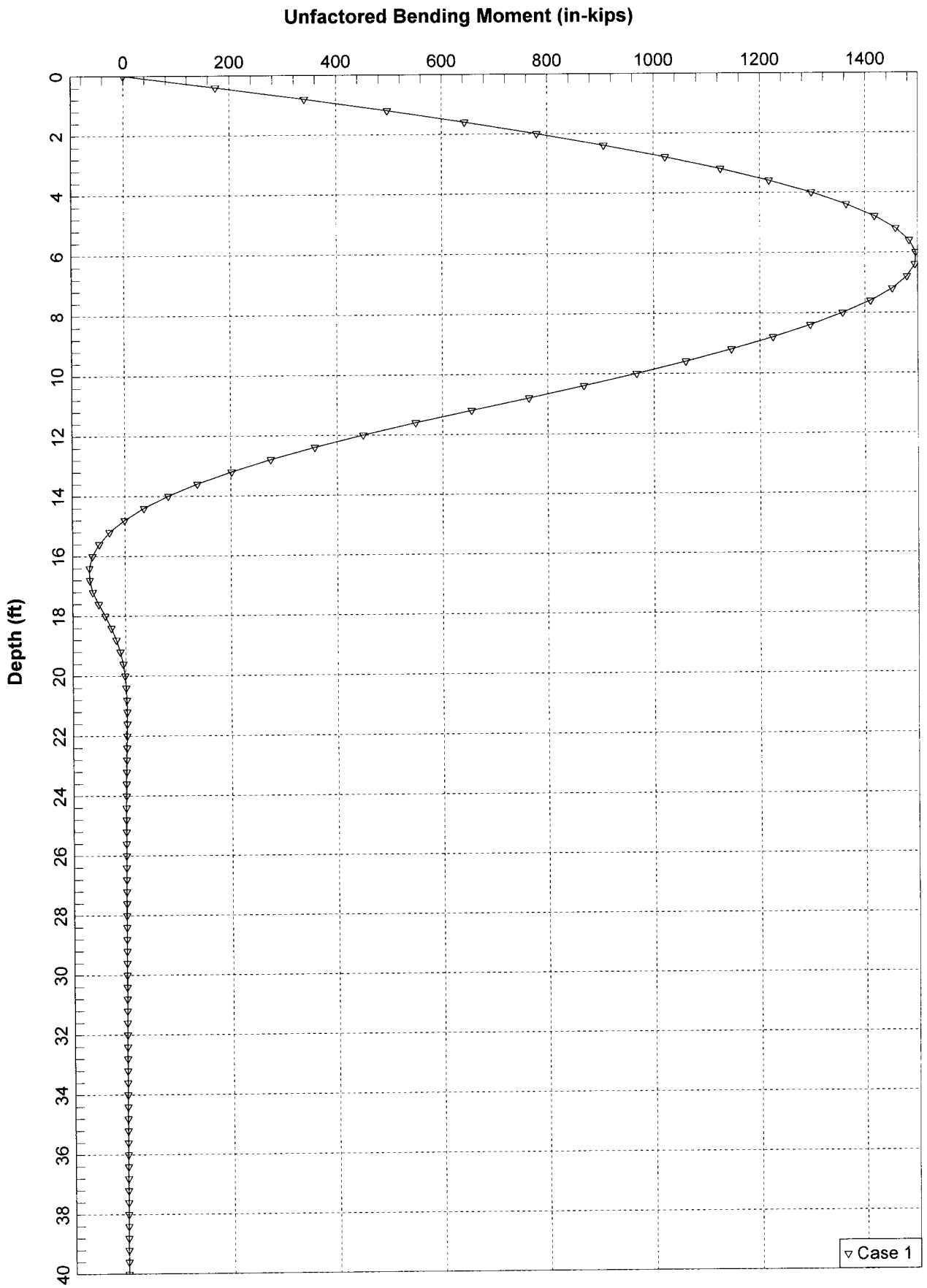
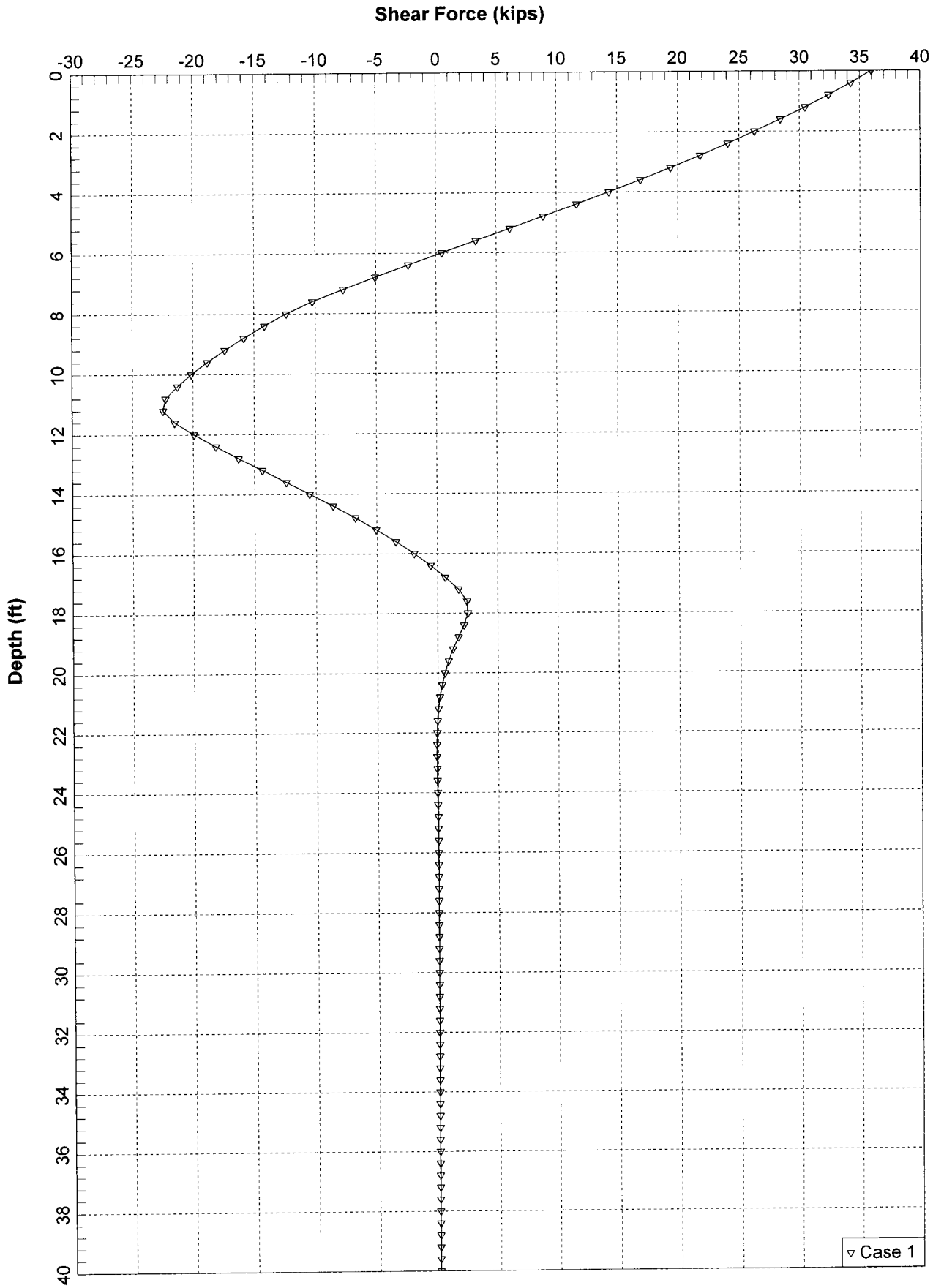

Depths -36 - 96 = Reese Sand
Depths 96 - 138 = Stiff Clay w/o free water
Depths 138 - 228 = Stiff Clay w/o free water
Depths 228 - 270 = Stiff Clay w/o free water
Depths 270 - 342 = Stiff Clay w/o free water
Depths 342 - 372 = Reese Sand
Depths 372 - 462 = Stiff Clay w/o free water
Depths 462 - 570 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
0.55
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Unfactored Bending Moment (in-kips)
0
200
400
600
800
1000
1200
1400
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-30
-25
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

Lateral Pile Capacity Analysis - March Lane UC (Widen)

Abutment 1

Depth of Second Point of Counterflexure = 18.8'

Design pile tip elevation = 11.80' - 18.8'

= -7.0 feet

Abutment 2

Depth of Second Point of Counterflexure = 18.0'

Design pile tip elevation = 10.87' - 18.0'

= -7.1 feet Say -7.5 feet

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
March Lane UC (Widen)

Boring A-09-ML-001 (Abutment 1) (16-inch dia. Alt "W" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
25.5* to 13.5	0 to 12	Sand (medium dense)	$\phi = 35°$; c= 0 psf	4, Sand	90	-	0.069	-
13.5 to 3.5	12 to 22	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.072	-
3.5 to -1.5	22 to 27	Clay (stiff)	$\phi = 0°$; c=1500 psf	3, Clay	500	0.007	0.067	-
-1.5 to -6.5	27 to 32	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.069	-
-6.5 to -11.5	32 to 37	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 32 feet deep below existing ground surface for design.
-11.5 to -16.5	37 to 42	Clay (stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-16.5 to -24.0	42 to 49.5	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.039	
-24.0 to -37.5	49.5 to 63	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-37.5 to -62.5	63 to 88	Sand (dense)	$\phi = 33°$; c= 0 psf	4, Sand	125	-	0.042	-
-62.5 to -84.5	88 to 110	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

*Finish ground elevation of +14.70 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
March Lane UC (Widen)

CPT-09-ML-001 (Abutment 2) (16-inch dia. Alt "W" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
23.5* to 3.0	0 to 20.5	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	225	-	0.072	-
3.0 to -0.5	20.5 to 24	Clay (stiff)	ϕ = 0°; c=2000 psf	3, Clay	500	0.007	0.067	-
-0.5 to -8.0	24 to 31.5	Clay (very stiff to hard)	ϕ = 0°; c=3750 psf	3, Clay	1000	0.005	0.069	-
-8.0 to -11.5	31.5 to 35	Clay (very stiff)	ϕ =0°; c= 2750 psf	3, Clay	1000	0.005	0.067	-
-11.5 to -17.5	35 to 41	Clay (very stiff)	ϕ = 0°; c= 3000 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 35 feet deep below existing ground surface for design.
-17.5 to -20.0	41 to 43.5	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	125	-	0.042	-
-20.0 to -27.5	43.5 to 51	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	
-27.5 to -36.5	51 to 60	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-36.5 to -39.0	60 to 62.5	Clay (very stiff)	ϕ = 0°; c=2000 psf	3, Clay	500	0.007	0.039	-
-39.0 to -41.0	62.5 to 64.5	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-41.0 to -46.5	64.5 to 70	Clay (stiff)	ϕ = 0°; c=1650 psf	3, Clay	500	0.007	0.039	-
-46.5 to -49.0	70 to 72.5	Clay (very stiff to hard)	ϕ =0°; c= 4000 psf	3, Clay	1000	0.005	0.039	-
-49.0 to -54.0	72.5 to 77.5	Clay (very stiff)	ϕ = 0°; c=2050 psf	3, Clay	1000	0.005	0.039	-
-54.0 to -56.5	77.5 to 80	Clay (very stiff)	ϕ = 0°; c=2750 psf	3, Clay	1000	0.005	0.039	-
-56.5 to -76.5	80 to 100	Clay (very stiff to hard)	ϕ =0°; c= 4000 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +13.85 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection __ 65% PS&E Unchecked Details _X PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 7/8/2010 **Structure Name*:** March Lane UC **Br No*:** 29-0193 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	_____	_____	_____	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 4th review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated June 11, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** March Lane UC **Br No*:** 29-0193 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	6/07/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	9	Please provide the Nominal Driving Resistance in Table 5.	Comment is incorporated. "Nominal Driving Resistance" is provided in Table 5.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** March Lane UC **Br No*:** 29-0193 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 EA: 0G4701 Project Name: I-5 North Stockton Widening OSFP Liaison: John Fujimoto Phone: 916-227-8757 e-mail: john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	Reviewer Name: Brandon Badeker Functional Unit: Geotechnical Services Cost Center: 59-323 Phone Number: 916-227-1046 e-mail: brandon_badeker@dot.ca.gov Date of Review: 12/16/2009 Structure Name*: March Lane UC Br No*: 29-0193 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	3	Under Section 5. As-Built LOTB, there is no information regarding the borings performed in 1997 for the bridge widening.	According to all the as-built LOTB that we received from Rajappan & Meyer, there is no available information for boring performed in 1997 for the bridge widening.	
3	FR	4, S.6	No 1997 As-Built subsurface information provided.	Same as #2 above, there is no available boring performed in 1997. Hence there is no as-built subsurface information provided in the report.	
4	FR	6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

5	FR	8	Tables 2 and 3, Although the pile tip appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	
6	FR	9	Table 4: Pile data table shows two different design tip elevations for the two abutments. It is recommended to keep the same pile length for ease of pile fabrication.	Comment is incorporated. Same pile length is used in the design of specified pile tip elevation.	
7	FR	LOTB	As is stated in the Soil Legend, the sample number or unconfined compressive strength is generally not located to the left of the graphic on the LOTB.	Comment is incorporated.	
8	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #5 above.	
9	FR	NA	C2 – Approved subject to OSFP Verification	NA	

FOUNDATION REPORT
FOURTEEN MILE SLOUGH BRIDGE (WIDEN)
(BRIDGE NO. 29-0175)
STOCKTON, CALIFORNIA
10-SJ-05, PM 31.00 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

PARIKH CONSULTANTS, INC.

2360 Qume Drive, Suite A, San Jose, CA 95131
(408) 452-9000

April 20, 2010 Job No. 206127.BR

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Log of Test Borings (Br. No. 29-0175)
As-built Log of Test Borings (Br. No. 29-0175 R/L)
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APPENDIX B

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Evaluation of Liquefaction Potential
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Geotechnical Parameters for LPILE (v. 5) Analyses

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Fourteen Mile Slough Bridge (Widen) General Plan

APPENDIX E

Office of Special Funded Projects Comment & Response Form (with PARIKH Consultants, Inc. Responses)

FOUNDATION REPORT
FOURTEEN MILE SLOUGH BRODGE (WIDEN)
(BRIDGE NO. 29-0175)
STOCKTON, CALIFORNIA
10-SJ-05 PM 31.00 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Fourteen Mile Slough Bridge (Widen) (Bridge No. 29-0175)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in two soil borings and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Fourteen Mile Slough Bridge (Widen) (Bridge No. 29-0175).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter

unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are seven-span structures. Based on the as-built general plan and foundation plan of the Fourteen Mile Slough Bridge, the structures were constructed in 1968. The existing left and right bridges (Bridge No. 29-0175 L/R) are approximately 214 feet long and 53 feet wide. The left and right bridges have not been widened since their construction in 1968.

The existing left and right bridge are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class II 45 tons) at the abutments and "Reinforced Concrete Raymond Step-Tapered Pile Extension" (Class I 45 tons) at the bents based on the as-built data.

Proposed Construction

The widening of the Fourteen Mile Slough Bridge is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. There will be outside widening in both northbound and southbound directions for the Fourteen Mile Slough Bridge.

According to Rajappan & Meyer Consulting Engineers Inc., the proposed Fourteen Mile Slough Bridge (Widen) will include both the median widening and outside widening to match the existing span with supports at abutments and bents. The widen structure will be PC/PS voided slab at the median and for the outside widening. The PC/PS voided slab will be supported on drop bent caps. The approximate length of the widened structure will be 214 feet measured along the "I-5" Line. The width of the inside widening will be 46 feet and 8 inches and the width of the

outside widening will be approximately 12 feet 6 inches. The abutments will incorporate diaphragm type abutments supported on piles.

Based on our discussions with the designer, the proposed bridge abutments and Bents will be supported on 24-inch diameter Cast-In-Drilled-Hole (CIDH) concrete piles.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The "Fourteen Mile Slough Bridge" is located at the intersection of Interstate 5 (I-5) and Fourteen Mile Slough. It is approximately 0.1 mile southeast of Swain Road Undercrossing (UC). The Project Location Map, Plate 1, shows the location of the site.

There are existing "levees" on both sides of the Fourteen Mile Slough. According to the designer, the gradient of the landside slope is flat and abutted by the I-5 embankments. Based on the "Elevation" in the "Fourteen Mile Slough Bridge (Widen) – General Plan" prepared by Biggs Cardosa Associates (a copy is include in Appendix D), the gradient of the waterside slope appears to be 4(H): 1(V) or flatter and there is a berm in the middle of the slope. The total height of the waterside slope is approximately 20 feet. The toes of the waterside slope are protected with rock slope protection.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the Fourteen Mile Slough Bridge (August 1963) have been used as a reference (Ref. 1). Based on the as-built LOTB, Borings B-1, B-2 and B-5 were drilled and Cone Penetration Test (CPT) at B-3 and B-4 were performed by Caltrans in August 1963. The soil borings were drilled to the approximate depths between 80 feet and 90 feet and to the bottom elevation between approximately -80 feet and -81 feet. The CPTs were pushed to the approximate depths between 54.5 feet and 69.5 feet and to the bottom elevation between approximately -53 feet and -61 feet.

Field Exploration in 2009

Additional field exploration was conducted in March and April 2009. Boring A-09-FM-001 was drilled on the south side and Boring R-09-FM-001 was drilled and CPT-09-FM-001 was performed on the north side of Fourteen Mile Slough. Boring R-09-FM-001 was drilled near the toe of the western slope of the southbound bridge abutment. The soil boring was drilled to the depth of 110 feet and the CPT was pushed to the approximate depth of 99.5 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content and dry unit weight of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built Borings and CPT, the subsurface soils generally consist of very soft to soft silty clay with peat seams, underlain by stiff to very stiff lean clays (with pocket/lens of loose silty sand in Boring B-2), underlain by medium dense to dense silty sands.

Based on Borings A-09-FM-001 and R-09-FM-001 and CPT-09-FM-001, the subsurface soils generally consist of soft to stiff lean clays/silt, underlain by loose to medium dense sands,

underlain by stiff to hard clay/silt with occasional pocket/lens/layer of dense silty sand.

Groundwater was recorded between approximate depths between 4 feet and 6 feet in the soil borings and between 5 feet and 5.5 feet in the CPTs in August 1963. Groundwater was measured at Elevation -2 feet (the depth of 22 feet below existing ground surface) during drilling for A-09-FM-001 in March 2009 and Elevation -4 feet (the depth of 17 feet below existing ground surface) during drilling for R-09-FM-001 in April 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock

accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of soft to stiff lean clays/silt, underlain by loose to medium dense sands, underlain by stiff to hard clay/silt with occasional pocket/lens/layer of dense silty sand. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g

3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Borings A-09-FM-001 and R-09-FM-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-FM-001 attached, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structures, different foundation systems have been evaluated with respect to their design capacity and construction considerations. 24-inch CIDH concrete piles are recommended to be the foundation system for the abutments and bents of the proposed bridge structures. Pertinent foundation design information provided by Biggs Cardosa Associates (Structural designer), including Foundation Design Data and Foundation Loads, are presented in the following tables. According to the structural designer, only one set of foundation design data and foundation design load will be used in the determination of the pile tip elevations for both the median and outside widening.

TABLE 2: FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	24” Dia. CIDH	17 ±	13±	2.5’	13.7+49.7 +13.3	1.00	10
Bent 2	LRFD	24” Dia. CIDH	10	10	N/A	N/A	1.00	10
Bent 3	LRFD	24” Dia. CIDH	2	2	N/A	N/A	1.00	10
Bent 4	LRFD	24” Dia. CIDH	1	1	N/A	N/A	1.00	10
Bent 5	LRFD	24” Dia. CIDH	1	1	N/A	N/A	1.00	10
Bent 6	LRFD	24” Dia. CIDH	2	2	N/A	N/A	1.00	10
Bent 7	LRFD	24” Dia. CIDH	10	10	N/A	N/A	1.00	10
Abut 8	WSD	24” Dia. CIDH	18.5±	14.5±	2.5	13.7+49.7 +13.3	1.00	10

TABLE 3: FOUNDATION DESIGN LOADS

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	170 + 430 + 170	105	105 + 285 + 105	N/A	N/A	N/A	N/A	N/A	-	N/A	-
Bent 2	300 + 835 + 300	178	155 + 450 + 155	445 + 1310 + 445	267	0	0	155+450 +155	210	0	75
Bent 3	325 + 900 + 325	194	175 + 510 + 175	490 + 1400 + 490	293	0	0	175+510 +175	240	0	70
Bent 4	325 + 900 + 325	194	175 + 510 + 175	490 + 1400 + 490	293	0	0	175+510 +175	240	0	70
Bent 5	325 + 900 + 325	194	175 + 510 + 175	490 + 1400 + 490	293	0	0	175+510 +175	240	0	70
Bent 6	325 + 900 + 325	194	175 + 510 + 175	490 + 1400 + 490	293	0	0	175+510 +175	240	0	70
Bent 7	300 + 835 + 300	178	155 + 450 + 155	445 + 1310 + 445	267	0	0	155+450 +155	210	0	75
Abut 8	170 + 430 + 170	105	105 + 285 + 105	N/A	N/A	N/A	N/A	N/A	-	N/A	-

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations and Load and Resistance Factor Design (LRFD) is used for the bent foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads". The Bent foundations are evaluated for the foundation design data and loading conditions using AASHTO LRFD Bridge Design Specifications–4th Edition, with interims through December 2008 and current Caltrans Amendments.

The evaluation of Load Demands on the piles, based upon WSD and LRFD are presented in Table 3 above. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Table 4 and Tale 5 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	24" Dia. CIDH	17.0 ±	770	495	105	210	-20.0 (a)	-20.0
Abut 8	24" Dia. CIDH	18.5 ±	770	495	105	210	-20.5 (a)	-20.5

Notes:

(i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively.

(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral.

TABLE 5: FOUDNATION RECOMMENDATION FOR BENTS

Location	Pile Type	Cut-off Elev. (feet)	Service-I Limit State Load (kips) per Support	Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev.[(i)] (ft)	Specified Tip Elev. (ft)[(ii)]
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1.0)	Tension (φ=1.0)		
Bent 2	24" Dia. CIDH	10	760	1	267	0	210	75	-45.0 (a), -29.5(c), -25.0 (d)	-45.0
Bent 3	24" Dia. CIDH	2	860	1	293	0	240	70	-54.5 (a), -39.5(c), -22.5 (d)	-54.5
Bent 4	24" Dia. CIDH	1	860	1	293	0	240	70	-56.0 (a), -39.5(c), -22.5 (d)	-56.0
Bent 5	24" Dia. CIDH	1	860	1	293	0	240	70	-56.0 (a), -39.5(c), -22.5 (d)	-56.0
Bent 6	24" Dia. CIDH	2	860	1	293	0	240	70	-54.5 (a), -39.5(c), -22.5 (d)	-54.5
Bent 7	24" Dia. CIDH	10	760	1	267	0	210	75	-51.5 (a), -28.5(c), -20.0 (d)	-51.5

Notes

(i) Design tip elevations are controlled by (a) Compression (Strength Limit), (b) Tension (Strength Limit), (c) Compression (Extreme Event), (d) Tension (Extreme Event), (e) Lateral Load, (f) Settlement respectively.

(ii) The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.

(iii) The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the penetrated soil layers, if any, which do not contribute to the design resistance.

TABLE 6: PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)
		Compression	Tension		
Abut 1	24" Dia. CIDH	210	0	-20.0 (a)	-20.0
Bent 2	24" Dia. CIDH	390	75	-45.0 (a), -25.0 (b)	-45.0
Bent 3	24" Dia. CIDH	420	70	-54.5 (a), -22.5 (b)	-54.5
Bent 4	24" Dia. CIDH	420	70	-56.0 (a), -22.5 (b)	-56.0
Bent 5	24" Dia. CIDH	420	70	-56.0 (a), -22.5 (b)	-56.0
Bent 6	24" Dia. CIDH	420	70	-54.5 (a), -22.5 (b)	-54.5
Bent 7	24" Dia. CIDH	390	70	-51.5 (a), -20.0 (b)	-51.5
Abut 8	24" Dia. CIDH	210	0	-20.0 (a)	-20.5

Notes

(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression (c) Lateral Load.
(ii) Design tip elevations for the ***Bents*** are controlled by (a) Compression, (b) Tension, (c) Lateral Load.
(iii) The design tip elevation due to the lateral loads are estimated by Biggs Cardosa Associates and not included in the pile data table above.
(iv) The specified tip elevation shall not be raised above the design tip elevations for tension and lateral load.

The estimation of the driven pile capacity in clay and sand are based on the adhesion factors per FHWA manual (O'Neil and Reese 1999). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 6, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles should not be spaced closer than 3 times the pile diameter measured center-to-center. For

piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter, there is no group effect for vertical pile capacity.

Caltrans standard specification for "Cast-in-Place Concrete Piling" should be used for the construction of CIDH piles. Groundwater is expected during pile construction. Due to presence of sand/gravel layers, and groundwater, raveling or caving may be expected, which may require additional drilling and cleaning effort and may increase the concrete volume for the piles. The CIDH holes are not expected to remain open without implementation of appropriate measures. Temporary steel casing and using slurry displacement method of construction should be used at all times to maintain the integrity of the piles. This should be consistent with any other special conditions required by the Reclamation District. Caltrans Standard Specifications and SSPs should be used for construction and quality assurance procedures. All pile excavations should be observed by the Geotechnical Engineer prior to the placement of reinforcement and concrete so that if conditions differ from those anticipated, appropriate recommendations can be made.

It is prudent to make the contractor aware of these conditions so that appropriate steps can be taken to comply with the standard and maintain the integrity of the pile foundations.

9.2.1 Scour

Based on the hydraulic study, the bent piles extension will have a future scour of one foot. This scour depth should be accounted for in the finish ground elevation in the pile capacity calculations.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. It is our understanding that the structural designer will perform lateral load analyses for the piles using LPILE Program (v. 5). “Generalized Vertical Soil Profile and Strength Parameters” based on A-09-FM-001, R-09-001 and CPT-09-FM-001 are included in Appendix C. To account for group effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D, respectively.

Generally, abutment piles are designed for service loads only, and batter piles are used

for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for the widening of the existing structures. Settlement of the existing embankment is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Borings A-09-FM-001 and R-09-FM-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
A-09-FM-001	21.0	960	7.5	54.6	24.0
R-09-FM-001	15.0	590	6.2	106.7	246.1

Based on the corrosion test results, the native subsurface soils in Borings A-09-FM-001 and R-09-FM-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Potential Impact on Existing Levees and Construction Consideration

The proposed project will be constructed over the existing levees of Reclamation District No. 1608. The District has requested that this report to address the potential impacts that the proposed project may have on the existing levees.

In our opinion, the pile foundation of the proposed structure widening should not impose significant additional horizontal or vertical loads onto the levees.

The slope stability and consolidation settlement of the existing levee may be impacted by the proposed project in the following ways.

a) Soil reaction of the pile may exert lateral load onto the levee. The extent of this load is localized and small in comparison with the overall length of the levee. This loading

should not be any different than the existing piles at the project site. The impact of the foundation piles on the slope stability of the existing levees is negligible.

b) Cast In-Drilled-Hole concrete piles were selected to eliminate the vibration (that would otherwise have resulted from pile driving) and impact on the stability of the levees.

c) The widened structure should not impose vertical load on the levee since it is supported on pile foundation. Hence, there should be no consolidation settlement of the existing levee due to the proposed structure. Based on the available boring information, the subsurface soils of the existing levees consist of soft to firm lean clays. Some localized settlement may result from construction equipments on the levees during construction. This impact should be minimized with the implementation of precautionary measures recommended below.

Construction Precautionary Measures and Construction Monitoring

Construction on the levees shall abide by Army Corps of Engineers and/or Department of Water Resources construction and backfill requirements.

The following precautionary measures and construction monitoring can be considered during construction to minimize the potential impact due to construction on the existing levees:

a) To minimize vibration due to the installation of temporary piles, steel pipe piles can be spun into the levees to support the temporary trestle, or temporary CIDH piles with steel pipe piles placed inside the CIDH can be used.

b) The maximum allowable load on the levee from the construction equipment should be in conformance with the regulatory agency's requirements. Normally a 200 to 250 pound per square foot (psf) traffic surcharge loading condition is used in the slope stability analyses, which generally represents maintenance equipment loads. This transient loading should not be exceeded by the total (dead + live) load of the construction equipment during construction. Parking and leaving equipment for a long term should be avoided. Also materials should not be stored at the levees.

c) No equipment shall be allowed within 10 feet from the edge of the levee regardless of

its weight. In order to further reduce the loading on the levees, timber planks or steel plates may also be considered to reduce point loads.

d) Contractor should be required to set horizontal and vertical monuments in levee section, which will be periodically checked for horizontal or vertical movements.

e) Construction monitoring may be considered to evaluate the settlement and movement in the levees. However, the level of sophisticated instrumentation may have to be further discussed, if it is deemed necessary.

In summary, it is our opinion that the impact on the existing levees due to the proposed project should be insignificant. However, precautionary measures should be taken and construction monitoring should be implemented during construction to minimize any potential impact on the integrity of the existing levees.

The above opinions are based on our understanding of the project and limited to the portion of the levee to be crossed by the proposed structure. Evaluation of the existing levee corridor has not been performed since it is beyond the scope of our work for this project.

9.8 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.9 Construction Observation

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

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

REGISTERED PROFESSIONAL ENGINEER
GAUTAM R. PARIKH
No. 666
EXP. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Fourteen Mile Slough Foundation Report (4-2010).rpt

REFERENCES

1. Fourteen Mile Slough Bridge As-built Log of Test Borings (August 1963).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

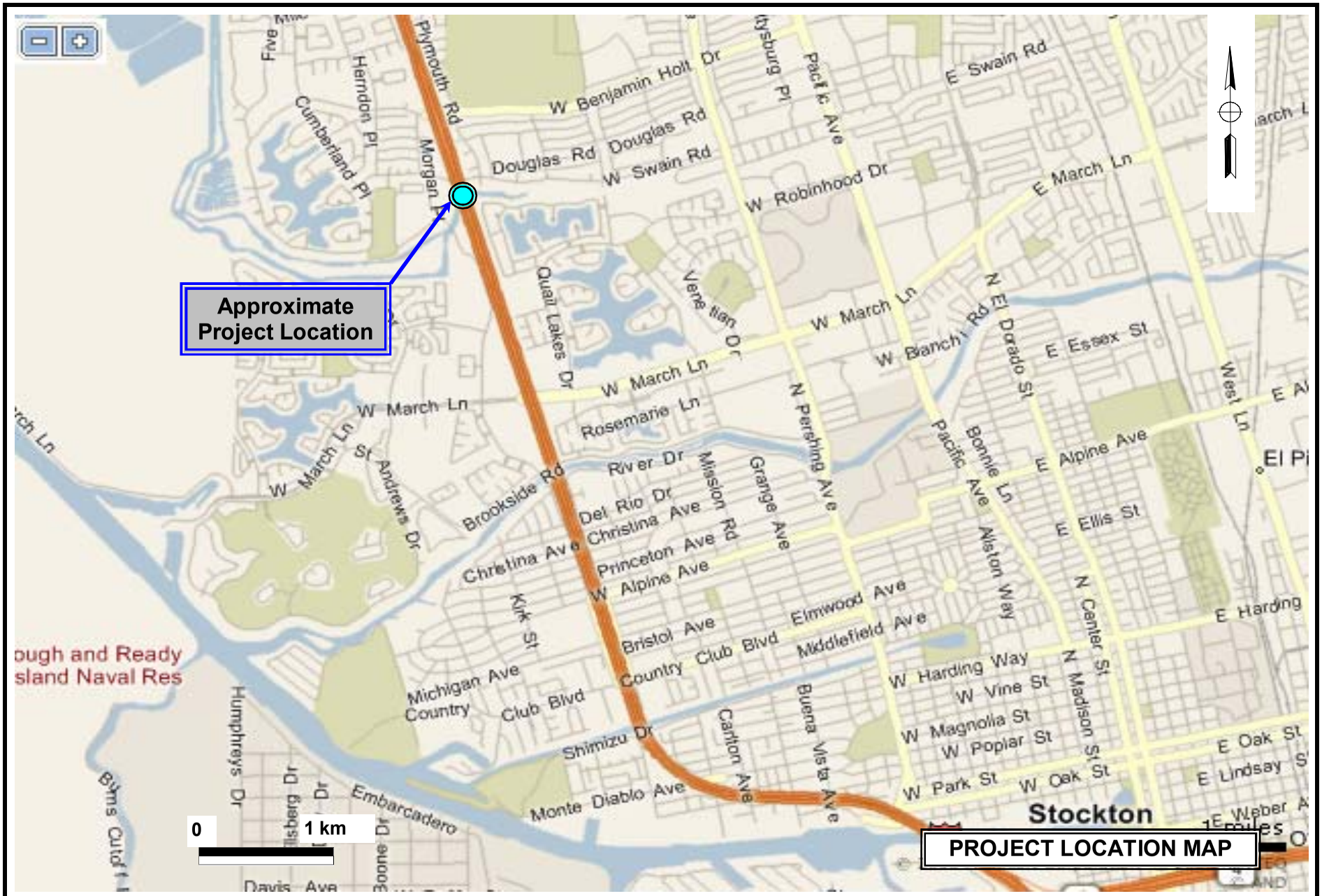
3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

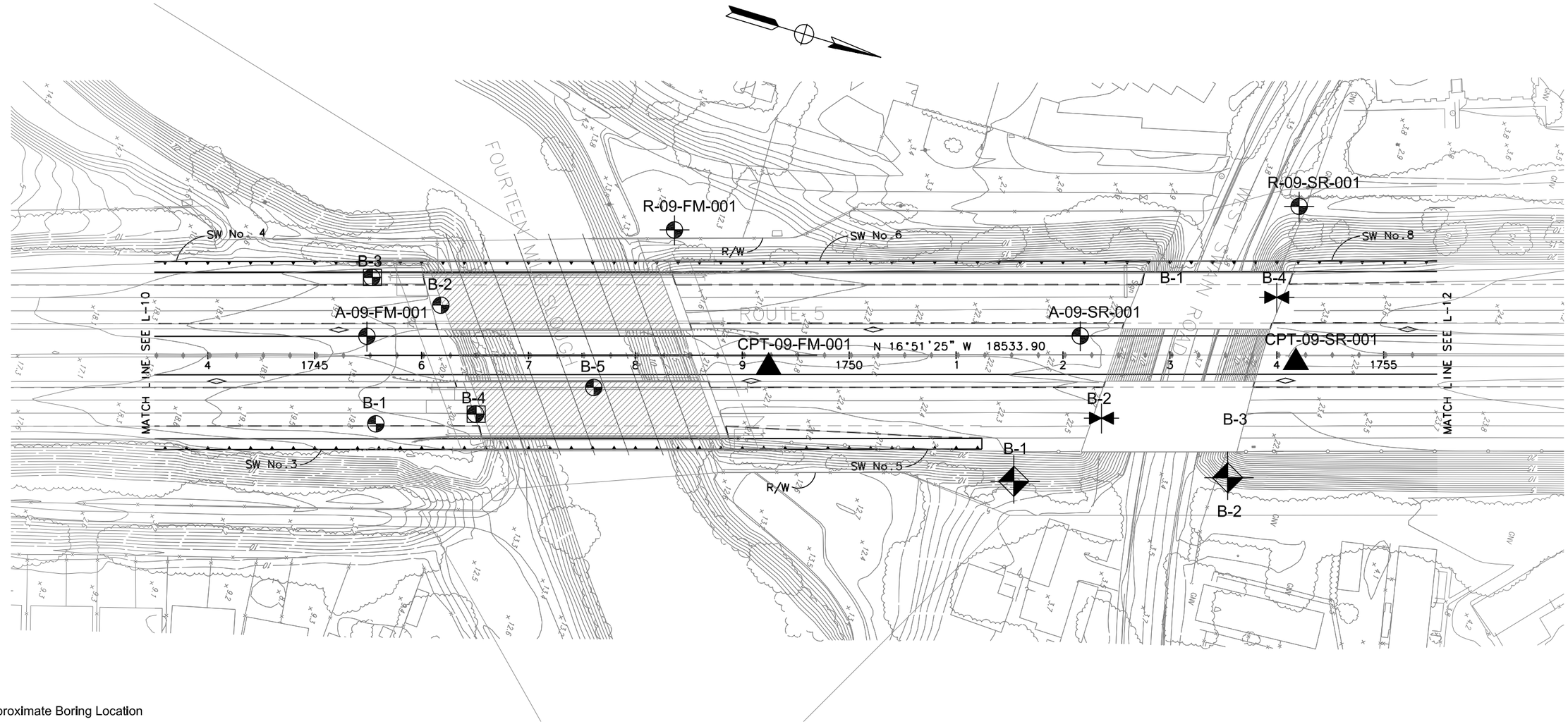
7. State of California Department of Transportation Standard Specifications May 2006.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0175)	PLATE NO.: 1



LEGEND

A-09-FM-001
Approximate Boring Location

B-4
Approximate CPT location performed in 1963

B-3
Approximate Boring location drilled in 1963

B-2
Approximate Boring location drilled in 1964

B-1
Approximate CPT location performed in 1964

B-1
Approximate Boring location drilled in 1997

CPT-09-FM-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

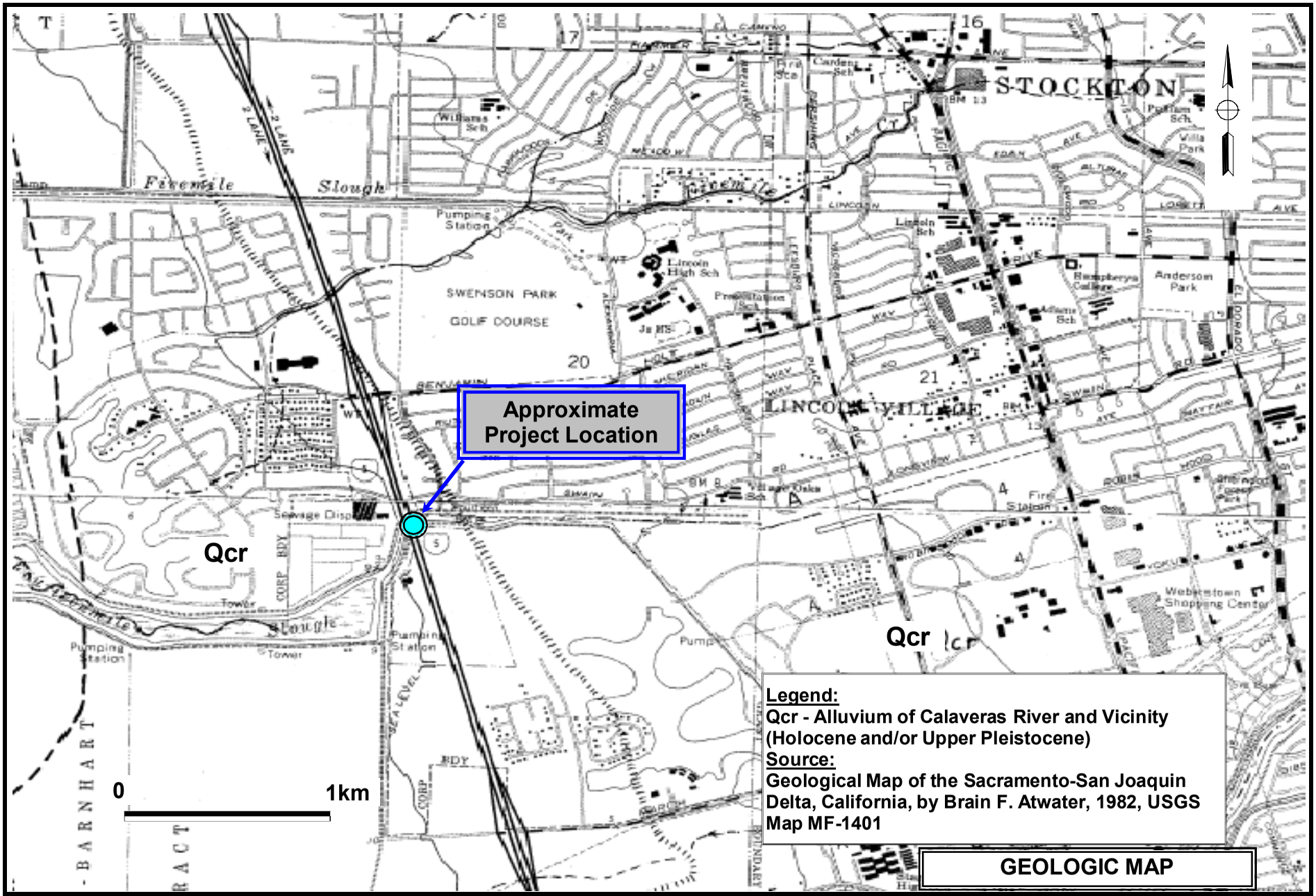
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

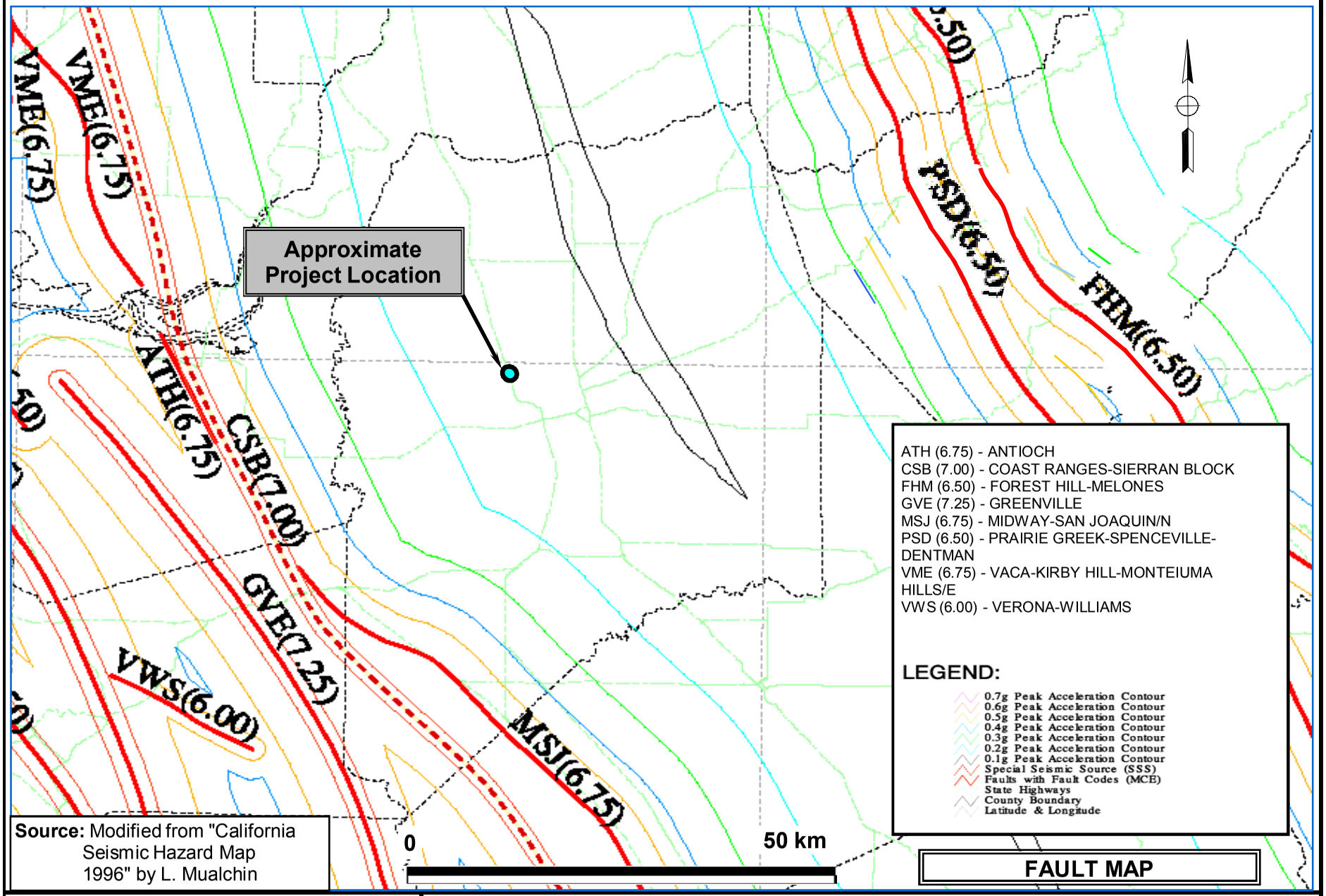
PLATE NO: 2



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0175)	PLATE NO.: 3



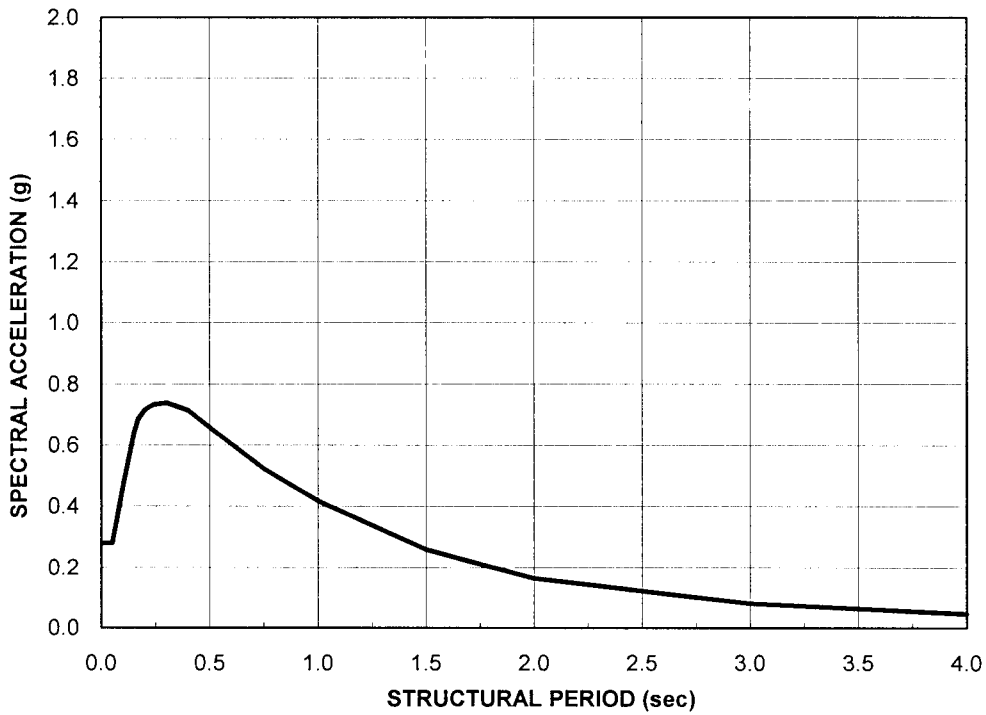
FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0175)	PLATE NO.: 4

ARS DESIGN CURVE
FOURTEEN MILE SLOUGH BRIDGE (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger and 5-inch diameter rotary wash drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

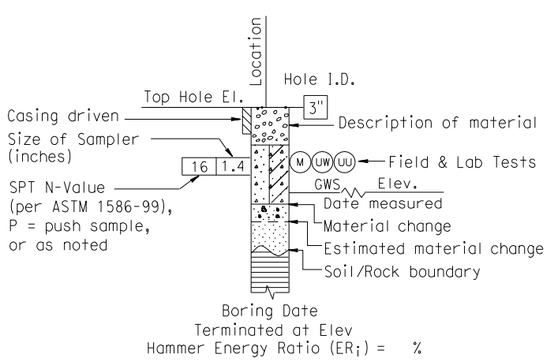
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

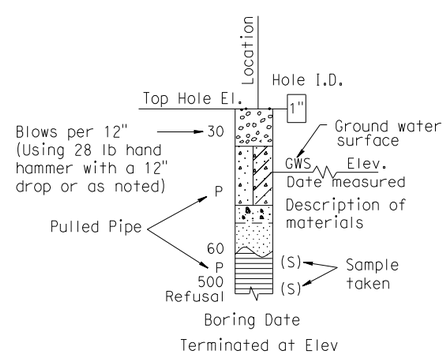
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

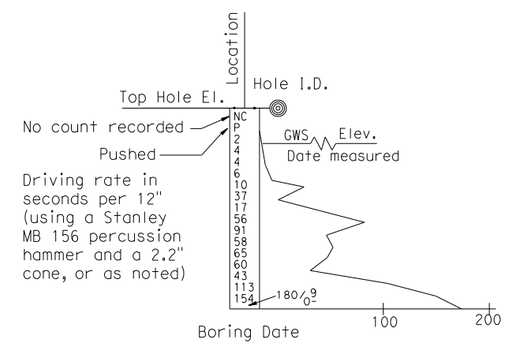
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



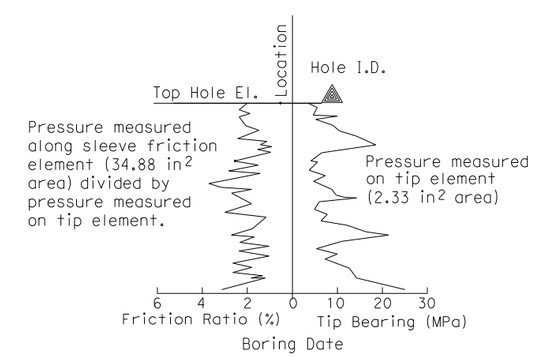
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY: O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER	BRIDGE NO. 29-0175 POST MILES 31.00
SIGN OFF DATE	CHECKED BY: D. WANG				

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 23 OF 26

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REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

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

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435)
- (CL) Collapse Potential (ASTM D 5333)
- (CP) Compaction Curve (CTM 216)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767)
- (DS) Direct Shear (ASTM D 3080)
- (EI) Expansion Index (ASTM D 4829)
- (M) Moisture Content (ASTM D 2216)
- (OC) Organic Content-% (ASTM D 2974)
- (P) Permeability (CTM 220)
- (PA) Particle Size Analysis (ASTM D 422)
- (PI) Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- (PL) Point Load Index (ASTM D 5731)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301)
- (SE) Sand Equivalent (CTM 217)
- (SG) Specific Gravity (AASHTO T 100)
- (SL) Shrinkage Limit (ASTM D 427)
- (SW) Swell Potential (ASTM D 4546)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850)
- (UW) Unit Weight (ASTM D 4767)
- (VS) Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

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STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0175
POST MILES 31.00

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)

SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 10/16/09 12/04/09 01/18/10 04/23/10 06/11/10

SHEET 24 OF 26

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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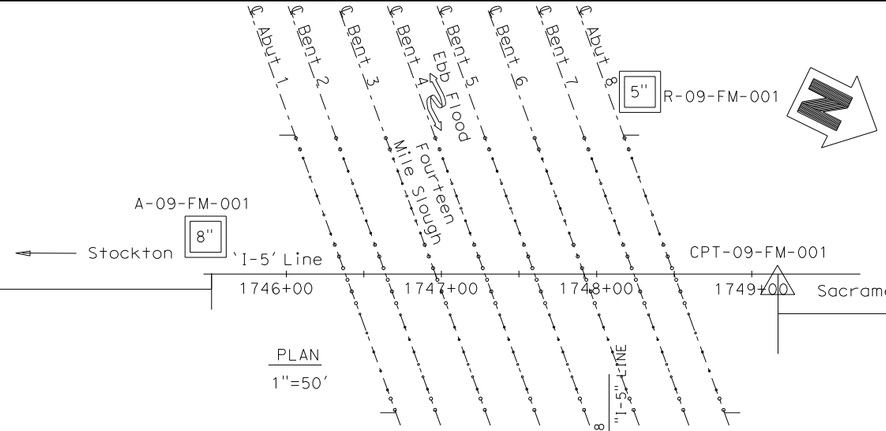
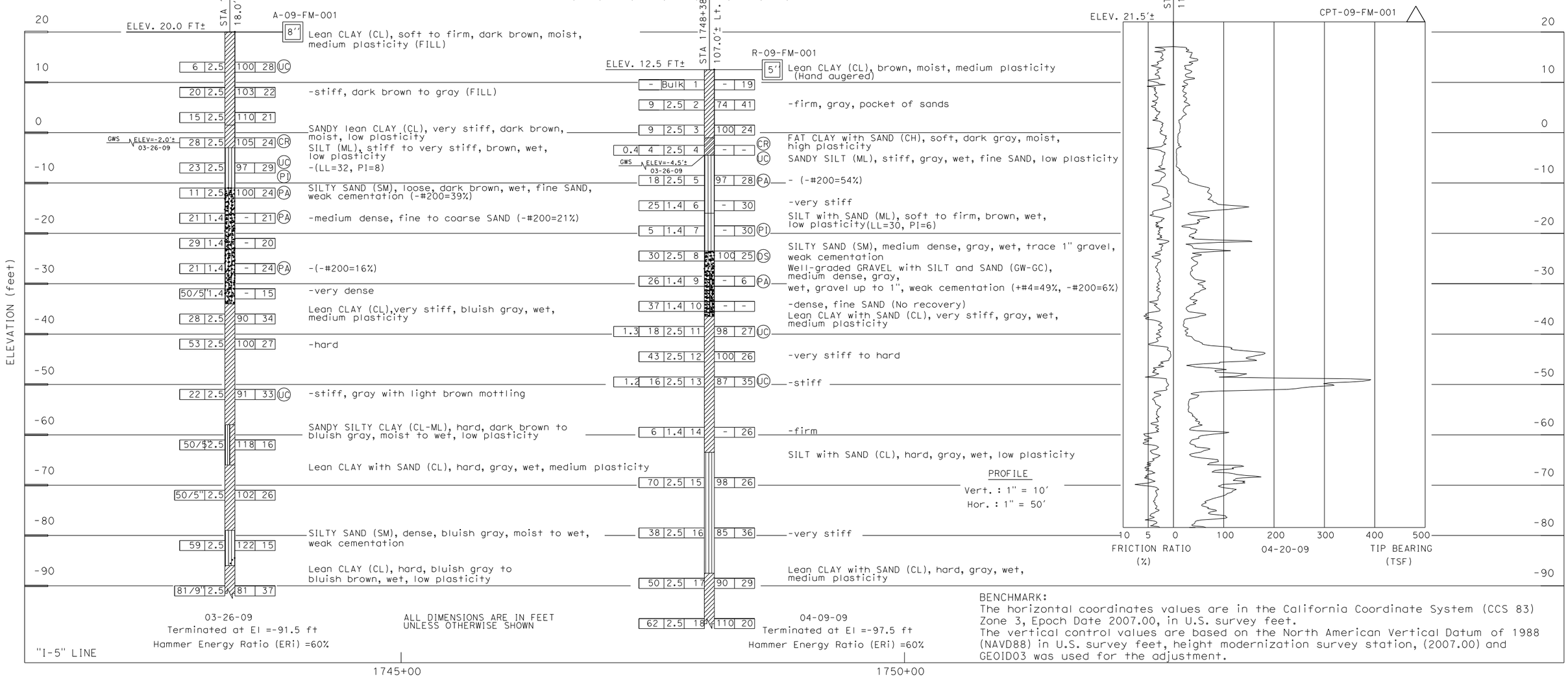
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131





BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.
The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	A. LAM	BRIDGE NO. 29-0175
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PROJECT ENGINEER	POST MILES 31.00

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)

LOG OF TEST BORINGS 1 OF 2

CU 06240
EA 0G4701

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

DISREGARD PRINTS BEARING EARLIER REVISION DATES 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 25 OF 26

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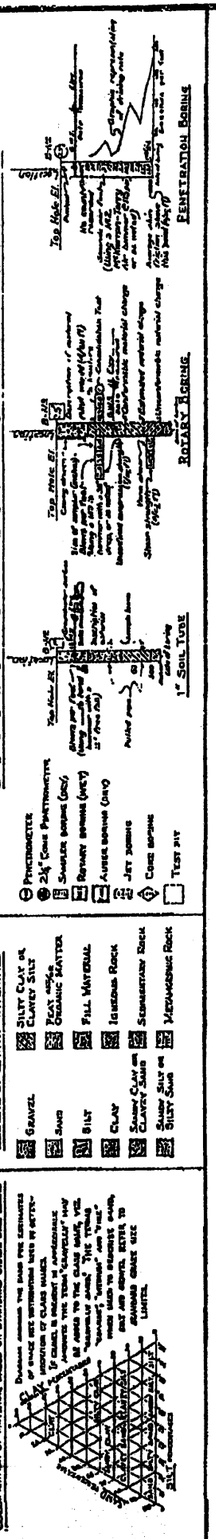
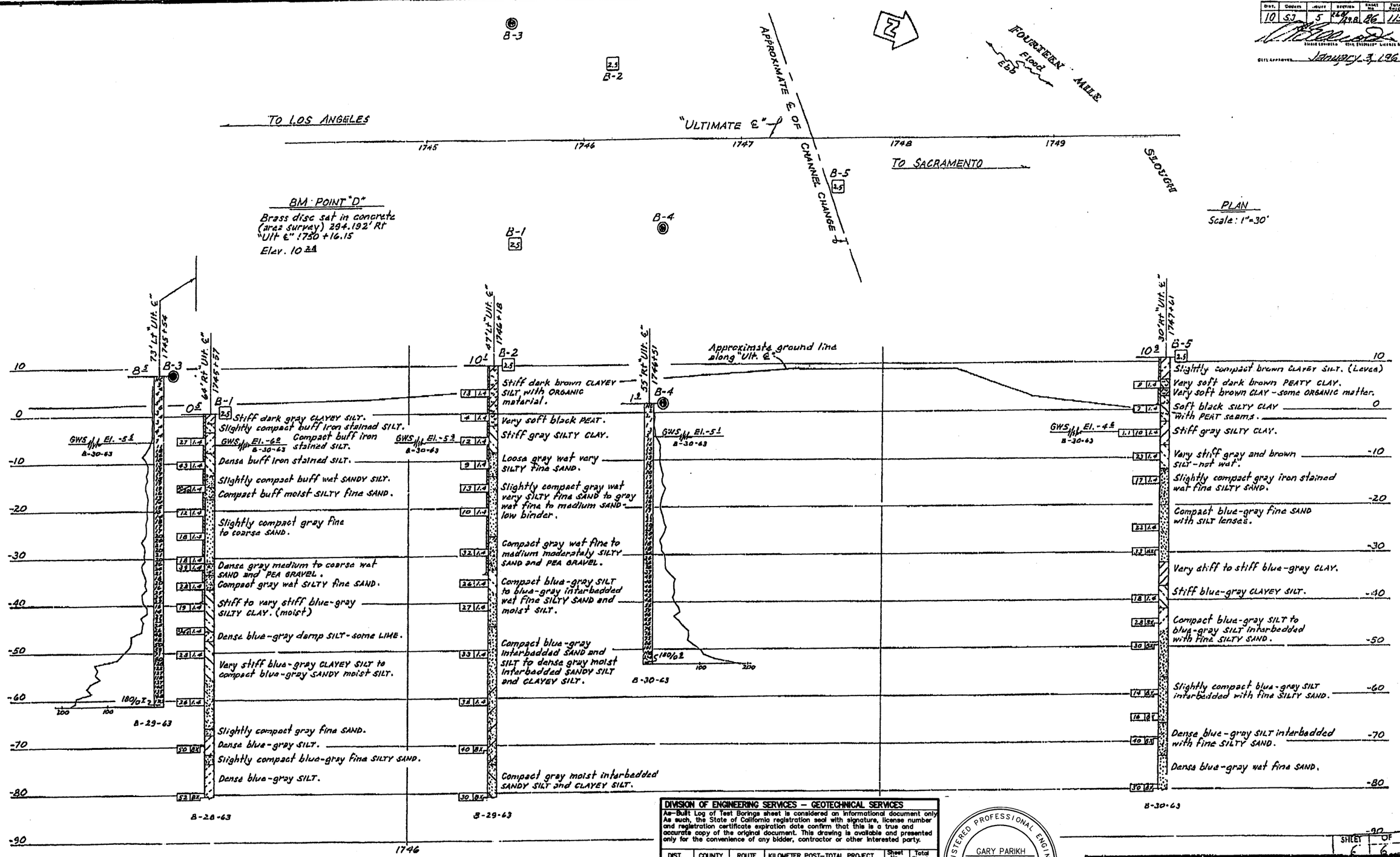
DIST.	COUNTY	ROUTE	SECTION	PHASE NO.	TOTAL SHEETS
10	SJ	5	22.6/29.8	86	113

CHIEF ENGINEER — CIVIL ENGINEER LICENSE NO.

CITY APPROVED January 3, 1967



NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.



DIVISION OF ENGINEERING SERVICES — GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5	—		

REGISTERED CIVIL ENGINEER — DATE

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)

LOG OF TEST BORINGS 2 OF 2

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0G4701

BRIDGE No. 29-0175

Sheet 26 of 26

REGISTERED PROFESSIONAL ENGINEER
GARY PARIKH
No. G.E. 666
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

AS BU
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SHEET 6 OF 6

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

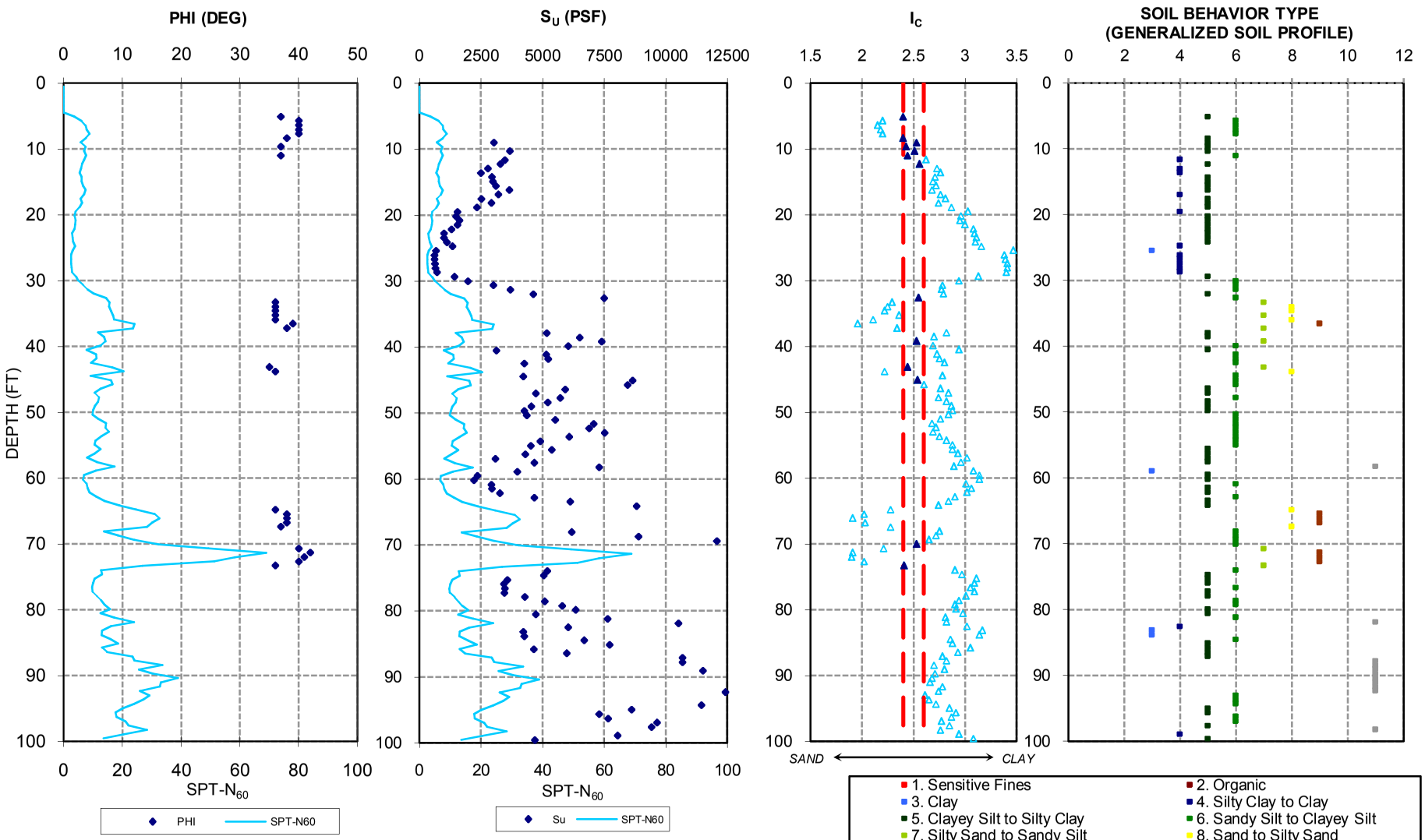
FOURTEEN MILE SLOUGH BR.

LOG OF TEST BORINGS

SCALE As Noted | BRIDGE 29-175 R/L | FILE | DRAWING 2.9175.09

CPT DATA CORRELATION

CPT-09-FM-008



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0175)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-FOURTEEN MILE SLOUGH BRIDGE (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0175)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-FM-008***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
5.085	5	Clayey Silt to Silty Clay	15.079	0.392	0.291		0.291	2.4	1.47552	0.99	0.129	2.400	1.48	22.25	2.312	51.449	0.09	1.00	1	
5.741	6	Sandy Silt to Clayey Silt	28.914	0.673	0.329		0.329	2.2	1.43885	0.99	0.129	2.200	1.44	41.60	1.667	69.342	0.11	1.00	1	
6.398	6	Sandy Silt to Clayey Silt	35.651	0.793	0.367		0.367	2.15	1.40396	0.99	0.128	2.150	1.40	50.05	1.553	77.721	0.12	1.00	1	
7.054	6	Sandy Silt to Clayey Silt	37	0.848	0.404		0.404	2.18	1.37157	0.99	0.128	2.180	1.37	50.75	1.619	82.170	0.13	1.00	1	
7.71	6	Sandy Silt to Clayey Silt	41.973	1.028	0.442		0.442	2.2	1.33983	0.98	0.128	2.200	1.34	56.24	1.667	93.734	0.16	1.00	1	
8.366	5	Clayey Silt to Silty Clay	32.605	1.068	0.479		0.479	2.4	1.3103	0.98	0.128	2.400	1.31	42.72	2.312	98.790	0.17	1.00	1	
9.022	5	Clayey Silt to Silty Clay	23.121	0.768	0.517		0.517	2.53	1.2813	0.98	0.128	2.530	1.28	29.63	2.925	86.646	0.14	1.00	1	
9.678	5	Clayey Silt to Silty Clay	31.382	0.959	0.555		0.555	2.43	1.25356	0.98	0.127	2.430	1.25	39.34	2.439	95.949	0.16	1.00	1	
10.335	5	Clayey Silt to Silty Clay	28.117	0.916	0.592		0.592	2.51	1.22768	0.98	0.127	2.510	1.23	34.52	2.820	97.326	0.17	1.00	1	
10.991	6	Sandy Silt to Clayey Silt	32.053	0.91	0.63		0.63	2.44	1.20219	0.98	0.127	2.440	1.20	38.53	2.483	95.684	0.16	1.00	1	
11.647	4	Silty Clay to Clay	26.539	1.053	0.667		0.667	2.62	1.17836	0.98	0.127							1.00	1	
12.303	5	Clayey Silt to Silty Clay	25.223	0.723	0.705		0.705	2.56	1.15486	0.97	0.127	2.560	1.15	29.13	3.091	90.025	0.15	1.00	1	
12.959	4	Silty Clay to Clay	21.554	0.848	0.742		0.742	2.73	1.13285	0.97	0.126							1.00	1	
13.615	4	Silty Clay to Clay	19.513	0.715	0.78		0.78	2.76	1.11111	0.97	0.126							1.00	1	
14.272	5	Clayey Silt to Silty Clay	22.756	0.802	0.818		0.818	2.71	1.09019	0.97	0.126							1.00	1	
14.928	5	Clayey Silt to Silty Clay	23.16	0.753	0.855		0.855	2.69	1.07056	0.97	0.126							1.00	1	
15.584	5	Clayey Silt to Silty Clay	24.099	0.851	0.893		0.893	2.72	1.05112	0.97	0.126							1.00	1	
16.24	5	Clayey Silt to Silty Clay	28.324	0.999	0.93		0.93	2.68	1.03286	0.97	0.126							1.00	1	
16.896	4	Silty Clay to Clay	24.929	0.957	0.968		0.968	2.76	1.01476	0.96	0.125							1.00	1	
17.552	5	Clayey Silt to Silty Clay	19.878	0.652	1.006		1.006	2.81	0.99728	0.96	0.125							1.00	1	
18.209	5	Clayey Silt to Silty Clay	22.914	0.675	1.043		1.043	2.74	0.98083	0.96	0.125							0.99	1	
18.865	5	Clayey Silt to Silty Clay	18.568	0.622	1.081		1.081	2.87	0.96449	0.96	0.125							0.98	1	
19.521	4	Silty Clay to Clay	12.546	0.406	1.118		1.118	3.03	0.94909	0.96	0.125							0.98	1	
20.177	5	Clayey Silt to Silty Clay	12.126	0.275	1.156		1.156	2.96	0.93379	0.96	0.124							0.97	1	
20.833	5	Clayey Silt to Silty Clay	13.179	0.305	1.194		1.194	2.95	0.91896	0.95	0.124							0.97	1	
21.49	5	Clayey Silt to Silty Clay	12.639	0.318	1.231		1.231	3	0.90498	0.95	0.124							0.96	1	
22.146	5	Clayey Silt to Silty Clay	10.919	0.273	1.269		1.269	3.08	0.89105	0.95	0.124							0.95	1	
22.802	5	Clayey Silt to Silty Clay	8.686	0.14	1.306		1.306	3.09	0.87789	0.95	0.123							0.95	1	
23.458	5	Clayey Silt to Silty Clay	8.67	0.148	1.344		1.344	3.11	0.86478	0.95	0.123							0.94	1	
24.114	5	Clayey Silt to Silty Clay	9.658	0.18	1.382		1.382	3.1	0.85205	0.94	0.123							0.94	1	
24.77	4	Silty Clay to Clay	11.405	0.347	1.419		1.419	3.16	0.84002	0.94	0.123							0.93	1	
25.427	3	Clay	6.628	0.238	1.456	0.013	1.443	3.47	0.83239	0.94	0.123							0.93	1	
26.083	4	Silty Clay to Clay	5.951	0.112	1.493	0.034	1.46	3.38	0.82707	0.94	0.125							0.93	1	
26.739	4	Silty Clay to Clay	5.891	0.12	1.531	0.054	1.477	3.39	0.82182	0.94	0.126							0.92	1	
27.395	4	Silty Clay to Clay	6.005	0.134	1.569	0.075	1.494	3.41	0.81663	0.93	0.127							0.92	1	
28.051	4	Silty Clay to Clay	6.186	0.147	1.606	0.095	1.511	3.41	0.81151	0.93	0.128							0.92	1	
28.707	4	Silty Clay to Clay	6.846	0.184	1.644	0.115	1.528	3.4	0.80645	0.93	0.130							0.92	1	
29.364	5	Clayey Silt to Silty Clay	12.098	0.32	1.681	0.136	1.545	3.13	0.80146	0.92	0.131							0.92	1	
30.02	6	Sandy Silt to Clayey Silt	15.953	0.318	1.719	0.156	1.562	2.94	0.79652	0.92	0.132							0.91	1	
30.676	6	Sandy Silt to Clayey Silt	23.536	0.458	1.756	0.177	1.58	2.78	0.79137	0.92	0.133							0.91	1	
31.332	6	Sandy Silt to Clayey Silt	29.258	0.747	1.794	0.197	1.597	2.77	0.78656	0.91	0.133							0.91	1	
31.988	5	Clayey Silt to Silty Clay	36.028	1.318	1.832	0.218	1.614	2.79	0.78181	0.91	0.134							0.91	1	
32.644	6	Sandy Silt to Clayey Silt	58.101	1.543	1.869	0.238	1.631	2.55	0.77711	0.91	0.135	2.550	0.78	45.15	3.034	136.998	0.32	0.91	1	2.82
33.301	7	Silty Sand to Sandy Silt	70.533	0.92	1.908	0.259	1.649	2.29	0.7722	0.90	0.136	2.290	0.77	54.47	1.917	104.413	0.19	0.90	1	1.63
33.957	8	Sand to Silty Sand	70.8	0.789	1.948	0.279	1.668	2.25	0.76709	0.90	0.136	2.250	0.77	54.31	1.798	97.662	0.17	0.90	1	1.46
34.613	8	Sand to Silty Sand	74.878	0.79	1.987	0.3	1.688	2.22	0.76177	0.89	0.137	2.220	0.76	57.04	1.717	97.948	0.17	0.90	1	1.46
35.269	7	Silty Sand to Sandy Silt	73.753	1.263	2.026	0.32	1.706	2.36	0.75705	0.89	0.137	2.360	0.76	55.84	2.156	120.403	0.24	0.90	1	2.11
35.925	8	Sand to Silty Sand	83.958	0.633	2.066	0.341	1.725	2.11	0.75214	0.88	0.138	2.110	0.75	63.15	1.473	93.027	0.15	0.90	1	1.34
36.581	9	Sand	124.725	0.911	2.106	0.361	1.745	1.96	0.74703	0.88	0.138	1.960	0.75	93.17	1.251	116.569	0.23	0.87	1	1.97
37.238	7	Silty Sand to Sandy Silt	104.355	2.451	2.145	0.382	1.763	2.34	0.74249	0.87	0.138	2.340	0.74	77.48	2.084	161.456		0.88	1	
37.894	5	Clayey Silt to Silty Clay	41.04	1.661	2.183	0.402	1.78	2.82	0.73826	0.87	0.139							0.89	1	
38.55	5	Clayey Silt to Silty Clay	50.944	1.764	2.22	0.423	1.798	2.7	0.73382	0.86	0.139							0.89	1	
39.206	7	Silty Sand to Sandy Silt	57.773	1.25	2.259	0.443	1.816	2.53	0.72944	0.86	0.139	2.530	0.73	42.14	2.925	123.256	0.25	0.89	1	2.18
39.862	6	Sandy Silt to Clayey Silt	47.668	1.404	2.296	0.464	1.833	2.69	0.72535	0.85	0.139							0.89	1	
40.518	5	Clayey Silt to Silty Clay	25.687	0.769	2.334	0.484	1.85	2.94	0.72131	0.85	0.139							0.88	1	
41.175	6	Sandy Silt to Clayey Silt	40.259	1.092	2.372	0.504	1.867	2.73	0.71731	0.84	0.139							0.88	1	
41.831	6	Sandy Silt to Clayey Silt	40.445	1.22	2.409	0.525	1.884	2.75	0.71336	0.83	0.139							0.88	1	
42.487	6	Sandy Silt to Clayey Silt	33.211	0.895	2.447	0.545	1.901	2.8	0.70945	0.83	0.139							0.88	1	
43.143	7	Silty Sand to Sandy Silt	69.61	1.332	2.485	0.566	1.92	2.44	0.70513	0.82	0.138	2.440	0.71	49.08	2.483	121.882	0.25	0.88	1	2.14
43.799	8	Sand to Silty Sand	94.544	1.125	2.525	0.586	1.939	2.22	0.70086	0.82	0.138	2.220	0.70	66.26	1.717	113.784	0.22	0.87	1	1.87
44.455	6	Sandy Silt to Clayey Silt	32.179	0.795	2.563	0.607	1.956	2.78	0.69708	0.81	0.138							0.87	1	
45.112	6	Sandy Silt to Clayey Silt	65.281	1.633	2.6	0.627	1.973	2.54	0.69335	0.80	0.137	2.540	0.69	45.26	2.979	134.836	0.31	0.87	1	2.67
45.768	6	Sandy Silt to Clayey Silt	65.106	1.911	2.638	0.648	1.99	2.6	0.68966	0.80	0.137							0.87	1	
46.424	5	Clayey Silt to Silty Clay	45.2	1.559	2.675	0.668	2.007	2.76	0.686	0.79	0.137							0.87	1	
47.08	5	Clayey Silt to Silty Clay	36.617	1.212	2.713	0.689	2.024	2.84	0.68238	0.78	0.136							0.87	1	
47.736	6	Sandy Silt to Clayey Silt	44.01	1.283	2.751	0.709	2.041	2.74	0.6788	0.78	0.136							0.87	1	
48.392	5	Clayey Silt to Silty Clay	41.1	1.383	2.788	0.73	2.059	2.82	0.67505	0.77	0.135							0.87	1	
49.049	5	Clayey Silt to Silty Clay	36.17	1.211	2.826	0.75	2.076	2.87	0.67155	0.76	0.135							0.86	1	
49.705	5	Clayey Silt to Silty Clay	33.975	1.079	2.863	0.771	2.093	2.88	0.66808	0.76	0.134							0.86	1	
50.361	6	Sandy Silt to Clayey Silt	34.188	0.973	2.901	0.791	2.11	2.84	0.66465	0.75	0.134							0.86	1	
51.017	6	Sandy Silt to Clayey Silt	42.454	1.222	2.939	0.812	2.127	2.76	0.66126	0.74	0.133							0.86	1	
51.673	6	Sandy Silt to Clayey Silt	54.312	1.608	2.976	0.832	2.144	2.68	0.65789	0.74	0.133							0.86	1	
52.329	6	Sandy Silt to Clayey Silt	52.789	1.702	3.014	0.852	2.161	2.72	0.65457	0.73	0.132							0.86	1	
52.986	6	Sandy Silt to Clayey Silt	57.452	1.854	3.051	0.873	2.178	2.69	0.65127	0.72	0.132							0.86	1	
53.642	6	Sandy Silt to Clayey Silt	46.674	1.412	3.089	0.893	2.196	2.75	0.64782	0.72	0.131							0.85	1	
54.298	6	Sandy Silt to Clayey Silt	37.677	1.118	3.127	0.914	2.213	2.82	0.64459	0.71	0.130							0.85	1	
54.954	6	Sandy Silt to Clayey Silt	35.373	1.132	3.164	0.934	2.23	2.88	0.6414	0.70	0.130							0.85	1	
55.61	5	Clayey Silt to Silty Clay	42.53	1.67	3.202	0.955	2.247	2.88	0.63824	0.70	0.129							0.85	1	
56.266	5	Clayey Silt to Silty Clay	34.27	1.229	3.239	0.975	2.264	2.93	0.6351	0.69	0.129							0.85	1	
56.923	5	Clayey Silt to Silty Clay	24.743	0.783	3.277	0.996	2.281	3.02	0.632	0.69	0.128							0.85	1	
57.579	5	Clayey Silt to Silty Clay	36.623	1.609	3.315	1.016	2.298	2.96	0.62893	0.68	0.127							0.85	1	
58.235	11	Sitff Fine-Grained	57.926	3.187	3.357	1.037	2.321	2.89	0.62482	0.67	0.127							0.85	1	
58.891	3	Clay	32.758	1.699	3.394	1.057	2.337	3.08	0.622	0.67	0.126							0.84	1	
59.547	5	Clayey Silt to Silty Clay	19.96	0.625	3.431	1.078	2.354	3.14	0.61902	0.66	0.126							0.84	1	
60.203	5	Clayey Silt to Silty Clay	19.125	0.528	3.469	1.098	2.371	3.14	0.61607	0.66	0.125							0.84	1	
60.86	6	Sandy Silt to Clayey Silt	24.148	0.623	3.507	1.119	2.388	3.01	0.61315	0.65	0.125							0.84	1	
61.516	5	Clayey Silt to Silty Clay	24.459	0.739	3.544	1.139	2.405	3.06	0.61026	0.65	0.124							0.84	1	
62.172	5	Clayey Silt to Silty Clay	27.177	0.8	3.582	1.16	2.422	3.02	0.6074	0.64	0.123							0.84	1	
62.828	6	Sandy Silt to Clayey Silt	37.016	1.175	3.619	1.18	2.439	2.9	0.60456	0.64	0.123							0.84	1	
63.484	5	Clayey Silt to Silty Clay	47.881	1.745	3.657	1.201	2.457	2.84	0.60159	0.63	0.122							0.84	1	
64.14	5	Clayey Silt to Silty Clay	69.501	2.624	3.695	1.221	2.474	2.74	0.5988	0.63	0.122							0.83	1	
64.797	8	Sand to Silty Sand	111.993	1.455	3.734	1.241	2.493	2.28	0.59572	0.62	0.121	2.280	0.60	66.72	1.886	125.839	0.27	0.83	1	2.61
65.453	9	Sand	155.937	1.179	3.775	1.262	2.513	2.02	0.59251	0.62	0.121	2.020	0.59	92.39	1.327	122.609	0.25	0.80	1	2.48
66.109	9	Sand	171.465	0.932	3.816	1.282	2.533	1.91	0.58934	0.61	0.120	1.910	0.59	101.05	1.198	121.100	0.25	0.79	1	2.43
66.765	9	Sand	151.941	1.123	3.857	1.303	2.554	2.03	0.58604	0.61	0.120	2.030	0.59	89.04	1.341	119.426	0.24	0.80	1	2.37
67.421	8	Sand to Silty Sand	128.268	1.983	3.896	1.323	2.573	2.28	0.58309	0.61	0.119	2.280	0.58	74.79	1.886	141.070	0.34	0.82	1	3.41
68.077	6	Sandy Silt to Clayey Silt	49.459	1.172	3.934	1.344	2.59	2.75	0.58047	0.60	0.119							0.83	1	
68.734	6	Sandy Silt to Clayey Silt	68.393	2.365	3.971	1.364	2.607	2.72	0.57788	0.60	0.119							0.83	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-FOURTEEN MILE SLOUGH BRIDGE (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0175)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-FM-008***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
69.39	6	Sandy Silt to Clayey Silt	92.562	3.524	4.009	1.385	2.624	2.65	0.57531	0.60	0.118							0.82	1	
70.046	6	Sandy Silt to Clayey Silt	130.441	4.705	4.047	1.405	2.641	2.53	0.57277	0.59	0.118	2.530	0.57	74.71	2.925	218.516		0.82	1	
70.702	7	Silty Sand to Sandy Silt	229.811	5.244	4.085	1.426	2.659	2.21	0.5701	0.59	0.117	2.210	0.57	131.01	1.692	221.626		0.76	1	
71.358	9	Sand	362.816	4.864	4.126	1.446	2.68	1.91	0.56701	0.58	0.117	1.910	0.57	205.72	1.198	246.537		0.69	1	
72.014	9	Sand	307.835	3.355	4.167	1.467	2.7	1.9	0.5641	0.58	0.117	1.900	0.56	173.65	1.189	206.443		0.71	1	
72.671	9	Sand	259.009	3.47	4.207	1.487	2.72	2.02	0.56122	0.58	0.116	2.020	0.56	145.36	1.327	192.897		0.73	1	
73.327	7	Silty Sand to Sandy Silt	114.657	2.273	4.246	1.508	2.738	2.41	0.55866	0.58	0.116	2.410	0.56	64.05	2.354	150.761	0.40	0.82	1	4.10
73.983	6	Sandy Silt to Clayey Silt	42.323	1.304	4.284	1.528	2.756	2.9	0.55612	0.57	0.116							0.82	1	
74.639	5	Clayey Silt to Silty Clay	41.908	1.548	4.321	1.549	2.773	2.97	0.55374	0.57	0.115							0.82	1	
75.295	5	Clayey Silt to Silty Clay	30.808	1.174	4.359	1.569	2.79	3.11	0.55138	0.57	0.115							0.81	1	
75.951	5	Clayey Silt to Silty Clay	29.329	0.993	4.396	1.589	2.807	3.09	0.54904	0.56	0.115							0.81	1	
76.608	6	Sandy Silt to Clayey Silt	29.405	0.846	4.434	1.61	2.824	3.05	0.54672	0.56	0.114							0.81	1	
77.264	5	Clayey Silt to Silty Clay	29.209	0.972	4.472	1.63	2.841	3.09	0.54442	0.56	0.114							0.81	1	
77.92	5	Clayey Silt to Silty Clay	35.596	1.193	4.509	1.651	2.858	3.01	0.54214	0.56	0.114							0.81	1	
78.576	6	Sandy Silt to Clayey Silt	41.133	1.346	4.547	1.671	2.875	2.94	0.53988	0.55	0.114							0.81	1	
79.232	6	Sandy Silt to Clayey Silt	46.079	1.571	4.584	1.692	2.893	2.9	0.5375	0.55	0.113							0.81	1	
79.888	5	Clayey Silt to Silty Clay	52.019	1.988	4.622	1.712	2.91	2.91	0.53528	0.55	0.113							0.81	1	
80.545	5	Clayey Silt to Silty Clay	39.981	1.281	4.66	1.733	2.927	2.98	0.53307	0.55	0.113							0.81	1	
81.201	6	Sandy Silt to Clayey Silt	60.847	2.018	4.697	1.753	2.944	2.81	0.53089	0.54	0.113							0.81	1	
81.857	11	Sitff Fine-Grained	83.723	4.113	4.74	1.774	2.966	2.82	0.52808	0.54	0.113							0.80	1	
82.513	4	Silty Clay to Clay	49.906	2.522	4.778	1.794	2.983	3.02	0.52594	0.54	0.112							0.80	1	
83.169	3	Clay	36.361	1.977	4.814	1.815	2.999	3.17	0.52393	0.54	0.112							0.80	1	
83.825	3	Clay	36.809	1.831	4.851	1.835	3.016	3.14	0.52182	0.54	0.112							0.80	1	
84.482	6	Sandy Silt to Clayey Silt	54.149	1.764	4.888	1.856	3.033	2.86	0.51973	0.53	0.112							0.80	1	
85.138	5	Clayey Silt to Silty Clay	62.502	2.573	4.926	1.876	3.05	2.88	0.51765	0.53	0.112							0.80	1	
85.794	5	Clayey Silt to Silty Clay	39.598	1.508	4.963	1.897	3.067	3.05	0.51558	0.53	0.112							0.80	1	
86.45	5	Clayey Silt to Silty Clay	48.246	1.719	5.001	1.917	3.084	2.93	0.51354	0.53	0.111							0.80	1	
87.106	5	Clayey Silt to Silty Clay	83.663	3.598	5.039	1.938	3.101	2.78	0.51151	0.53	0.111							0.80	1	
87.762	11	Sitff Fine-Grained	84.078	4.118	5.081	1.958	3.123	2.82	0.50891	0.52	0.111							0.80	1	
88.419	11	Sitff Fine-Grained	124.67	6.435	5.124	1.978	3.146	2.7	0.50621	0.52	0.111							0.80	1	
89.075	11	Sitff Fine-Grained	90.138	4.42	5.167	1.999	3.168	2.8	0.50366	0.52	0.111							0.79	1	
89.731	11	Sitff Fine-Grained	111.393	5.235	5.21	2.019	3.19	2.71	0.50114	0.52	0.110							0.79	1	
90.387	11	Sitff Fine-Grained	146.317	7.917	5.253	2.04	3.213	2.68	0.49853	0.52	0.110							0.79	1	
91.043	11	Sitff Fine-Grained	125.494	5.57	5.296	2.06	3.235	2.66	0.49605	0.52	0.110							0.79	1	
91.699	11	Sitff Fine-Grained	116.716	6.796	5.338	2.081	3.258	2.78	0.49349	0.52	0.110							0.79	1	
92.356	11	Sitff Fine-Grained	94.259	4.086	5.381	2.101	3.28	2.74	0.49107	0.51	0.110							0.79	1	
93.012	6	Sandy Silt to Clayey Silt	113.396	3.901	5.419	2.122	3.297	2.61	0.48922	0.51	0.109							0.79	1	
93.668	6	Sandy Silt to Clayey Silt	103.869	3.623	5.456	2.142	3.314	2.65	0.48737	0.51	0.109							0.79	1	
94.324	6	Sandy Silt to Clayey Silt	88.134	3.122	5.494	2.163	3.331	2.72	0.48554	0.51	0.109							0.79	1	
94.98	5	Clayey Silt to Silty Clay	68.354	2.653	5.532	2.183	3.348	2.85	0.48373	0.51	0.109							0.79	1	
95.636	5	Clayey Silt to Silty Clay	58.685	2.224	5.569	2.204	3.365	2.91	0.48193	0.51	0.109							0.78	1	
96.293	6	Sandy Silt to Clayey Silt	60.782	2.152	5.607	2.224	3.383	2.87	0.48003	0.51	0.109							0.78	1	
96.949	6	Sandy Silt to Clayey Silt	75.157	2.507	5.644	2.245	3.4	2.77	0.47826	0.50	0.109							0.78	1	
97.605	5	Clayey Silt to Silty Clay	75.179	3.127	5.682	2.265	3.417	2.85	0.4765	0.50	0.109							0.78	1	
98.261	11	Sitff Fine-Grained	101.652	4.369	5.725	2.286	3.439	2.76	0.47424	0.50	0.109							0.78	1	
98.917	4	Silty Clay to Clay	65.947	2.997	5.762	2.306	3.456	2.94	0.47251	0.50	0.108							0.78	1	
99.573	5	Clayey Silt to Silty Clay	40.516	1.461	5.8	2.326	3.473	3.08	0.47079	0.50	0.108							0.78	1	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The results of the test are presented on Plate B-2, "Plasticity Chart".

Grain Size Classification

Grain size classification test (ASTM Test Method D 420) was performed on selected sample of granular soil to aid in the classification. The result is presented on Plate B-3, "Grain Size Distribution Curves".

Unconfined Compression Tests

Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-6A through B-6F.

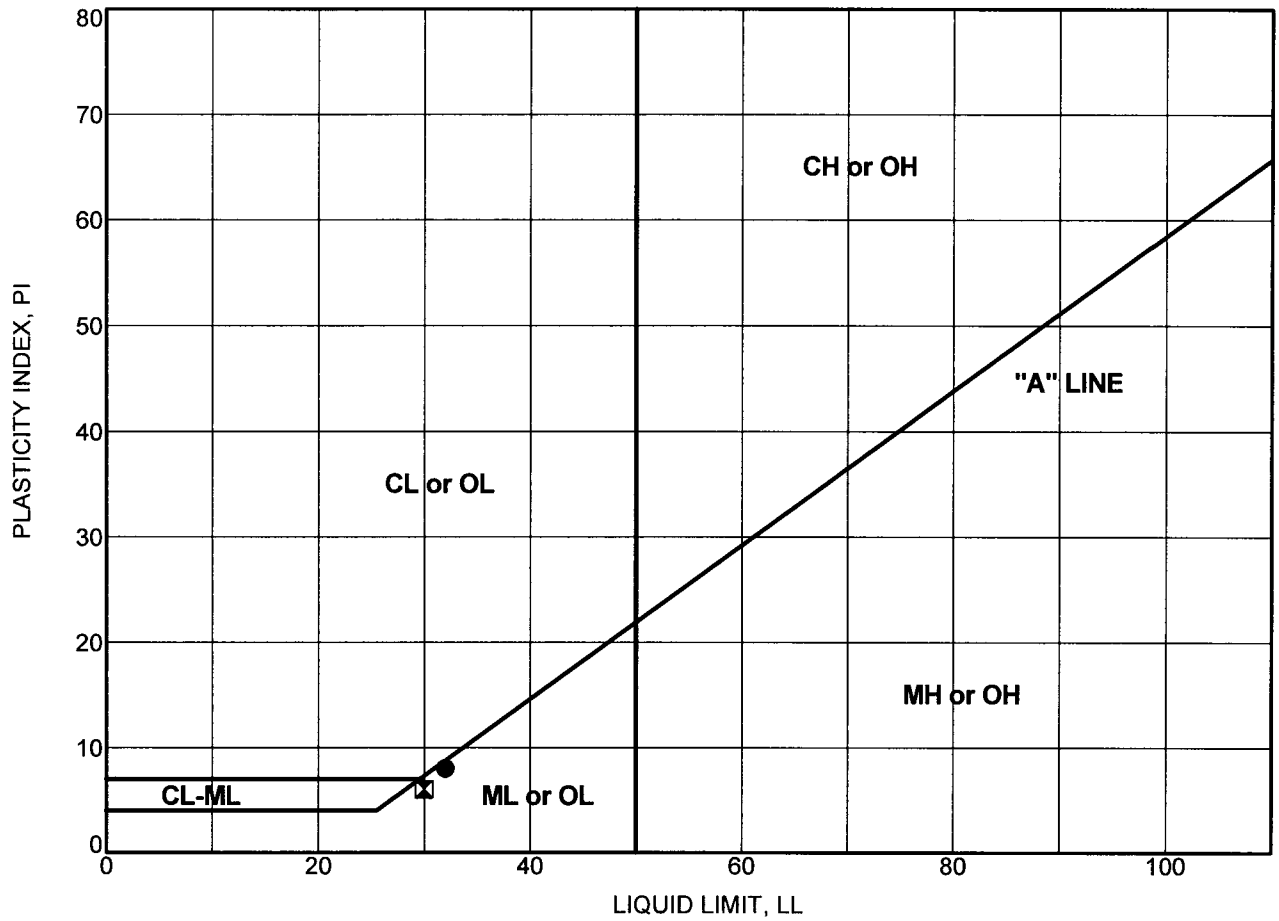
Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-4A and Plate B-4B.

LABORATORY TESTS
(Continued)

Direct Shear Tests

Direct Shear test was performed on selected relatively undisturbed sample to determine the shear strength of a soil material in direct shear. The test was performed according to ASTM Test Method D 3080 by Cooper . The test result is presented on Plate B-5.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A-09-001	MC-5	26.0	●	29	32	24	8	SILT (ML)
R-09-001	SPT-7	31.0	⊠	30	30	24	6	SILT (ML)

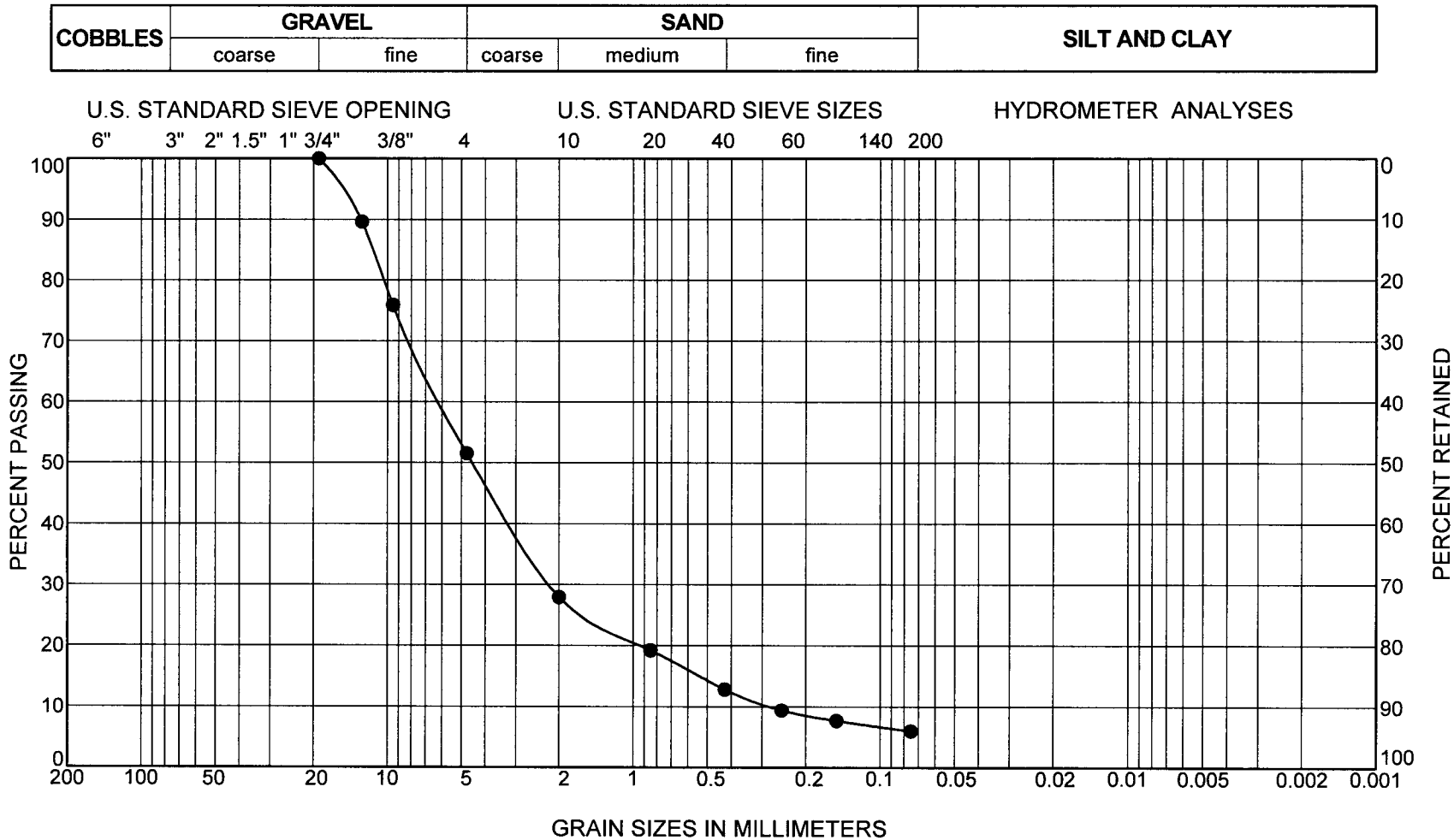
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Fourteen Mile Slough Bridge (Widen)
Stockton, California

JOB NO: 206127.BR (FMB)

PLATE NO: B-2

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
R-09-001	SPT-9	41.0	●			WELL GRADED GRAVEL WITH SAND AND CLAY (GW-GC)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

JOB NO: 206127.BR (FMB)

Fourteen Mile Slough Bridge (Widen)
Stockton, California

PLATE NO: B-3

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/06/2009
Date Submitted 05/01/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR I-5 NORTH Site ID : A-09-FM-001#4@21.0'
Thank you for your business.

* For future reference to this analysis please use SUN # 55616-112072.

EVALUATION FOR SOIL CORROSION

Soil pH	7.47	
Minimum Resistivity	0.96 ohm-cm (x1000)	
Chloride	54.6 ppm	00.00546 %
Sulfate	24.0 ppm	00.00240 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 8 2009
By

PLATE NO. : B-4A

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/29/2009
Date Submitted 04/24/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR Site ID : R-09-FM-001#4@15.0'
Thank you for your business.

* For future reference to this analysis please use SUN # 55554-111853.

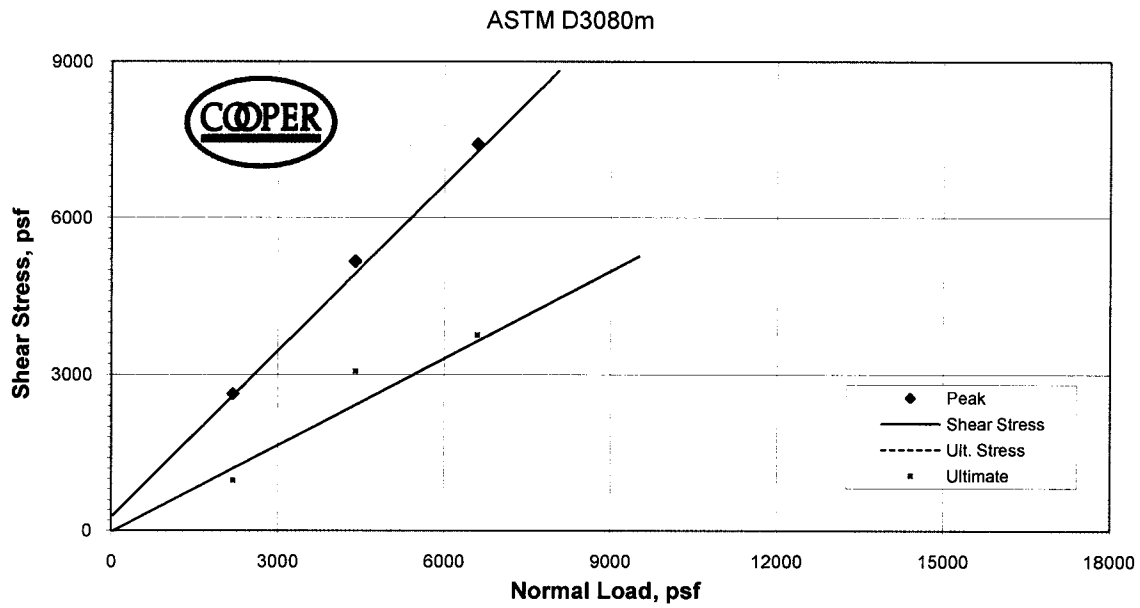
EVALUATION FOR SOIL CORROSION

Soil pH	6.18	
Minimum Resistivity	0.59 ohm-cm (x1000)	
Chloride	106.7 ppm	00.01067 %
Sulfate	246.1 ppm	00.02461 %

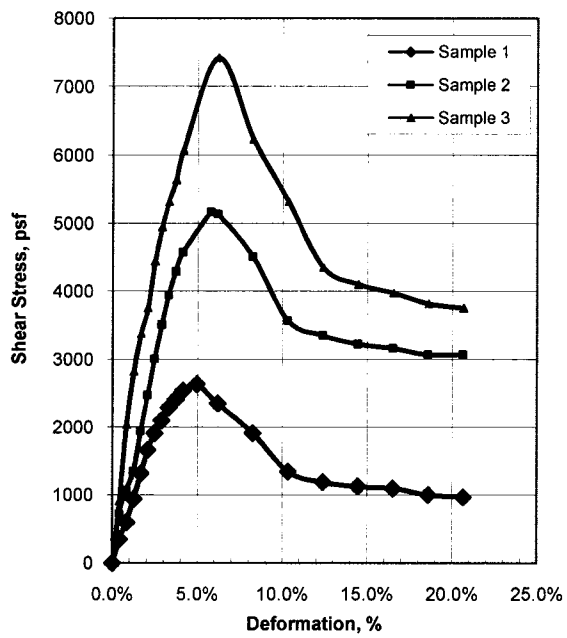
METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 1 2009
By

Direct Shear



P. Phi (degrees)	45.5	Ult. Phi (degrees)	27.5
P. Cohesion(psf)	300	Ult. Cohesion (psf)	0



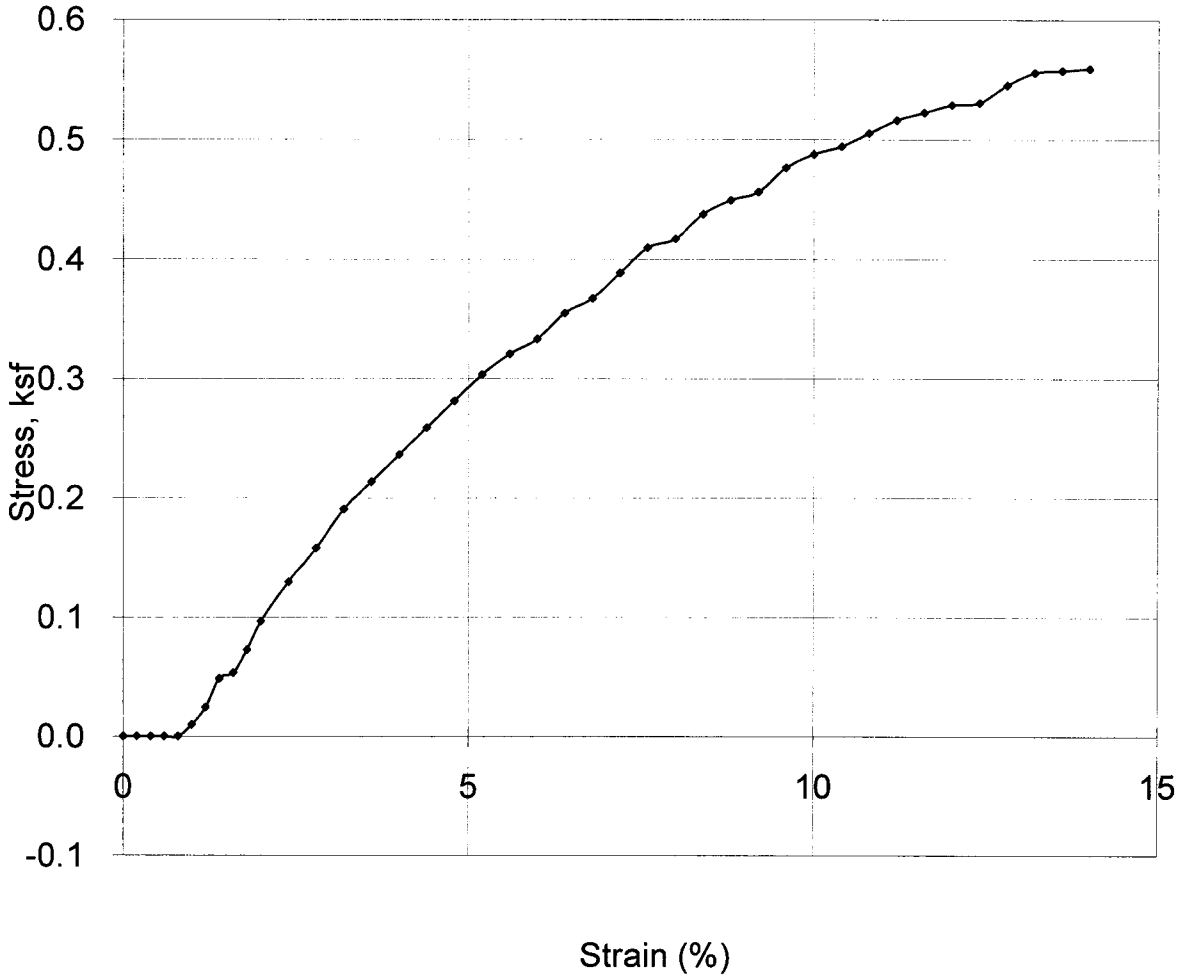
	Sample Data: Initial			
	1	2	3	4
Moisture %	26.1	24.6	25.5	
Dry Dens., pcf	99.2	100.5	99.1	
Void Ratio	0.730	0.707	0.732	
Saturation %	98.4	95.7	95.9	
Diameter	2.42	2.42	2.42	
Height	1.00	1.00	1.01	
	Sample Data: At Test			
Moisture %	26.1	24.1	24.1	
Dry Dens., pcf	99.4	102.1	102.9	
Void Ratio	0.727	0.681	0.668	
Saturation %	98.8	97.4	99.2	
Diameter	2.42	2.42	2.42	
Height	1.00	0.98	0.97	
Normal Stress, psf	2200	4400	6600	
Shear Stress, psf	2630	5166	7420	
Strengths picked at	5.0%	5.8%	6.2%	
Ult. Stress, psf	971	3068	3757	
Strain Rate, in/min	0.020	0.020	0.020	

CTL #	157-272
Client:	Parikh Consultants, Inc.
Project	206127.GDR
Tested By:	MD
Reduced By:	RU
Date:	4/21/2009

Specimen #	Boring:	Sample:	Depth, ft.	Visual Soil Classification
1	R-09-FM-001	#8	36.0	Brown Silty SAND
2	R-09-FM-001	#8	36.0	Brown Silty SAND
3	R-09-FM-001	#8	36.0	Brown Silty SAND
4				

Remarks: *DS-CU* A fully undrained condition may not be attained in this test.

UNCONFINED COMPRESSION TEST



Boring No.: A-09-FM-001

Sample No. : MC-1

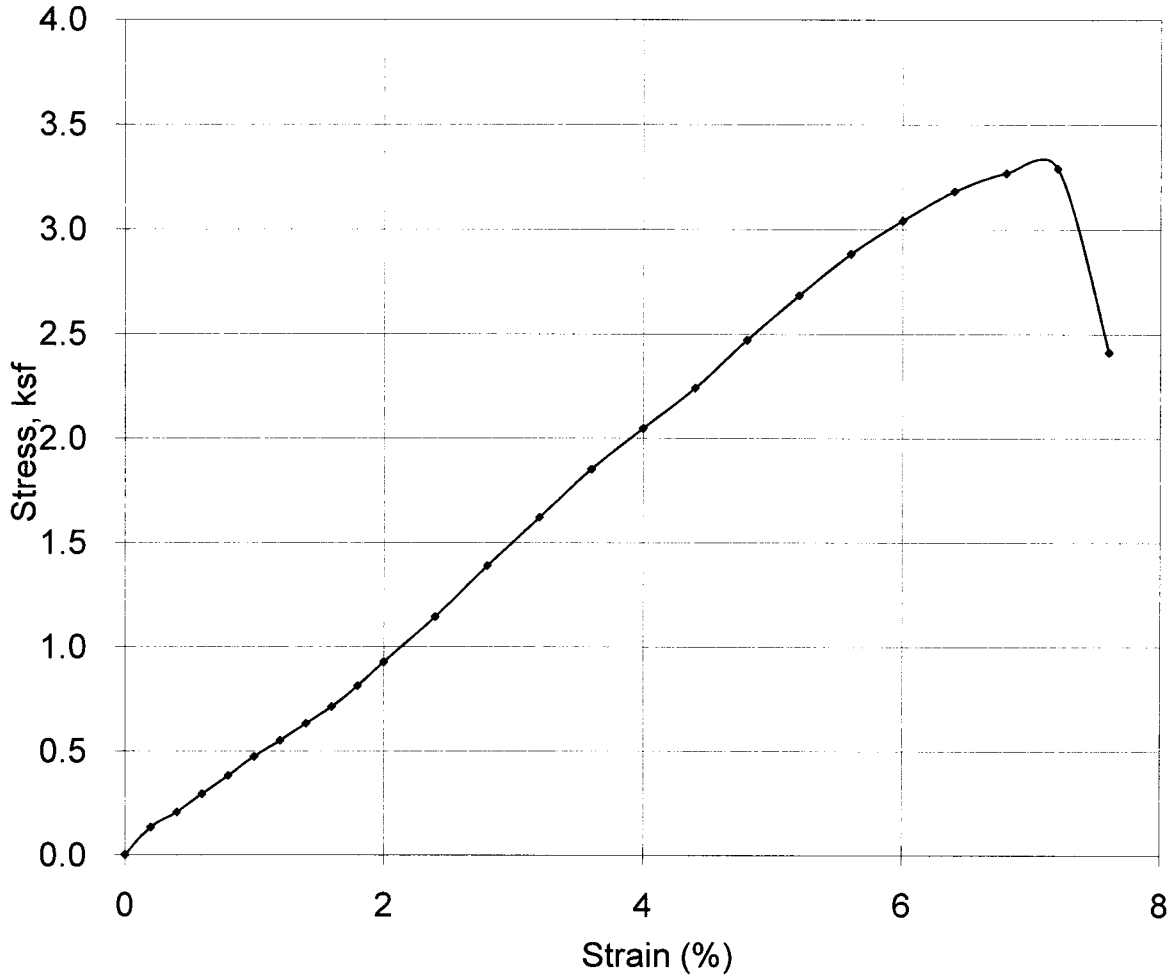
Depth (feet): 6

Material Description:
Lean Clay

Unconfined Compressive Strength: 0.6 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**FOURTEEN MILE SLOUGH BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-6A

UNCONFINED COMPRESSION TEST



Boring No.: A-09-FM-001

Sample No. : MC-5

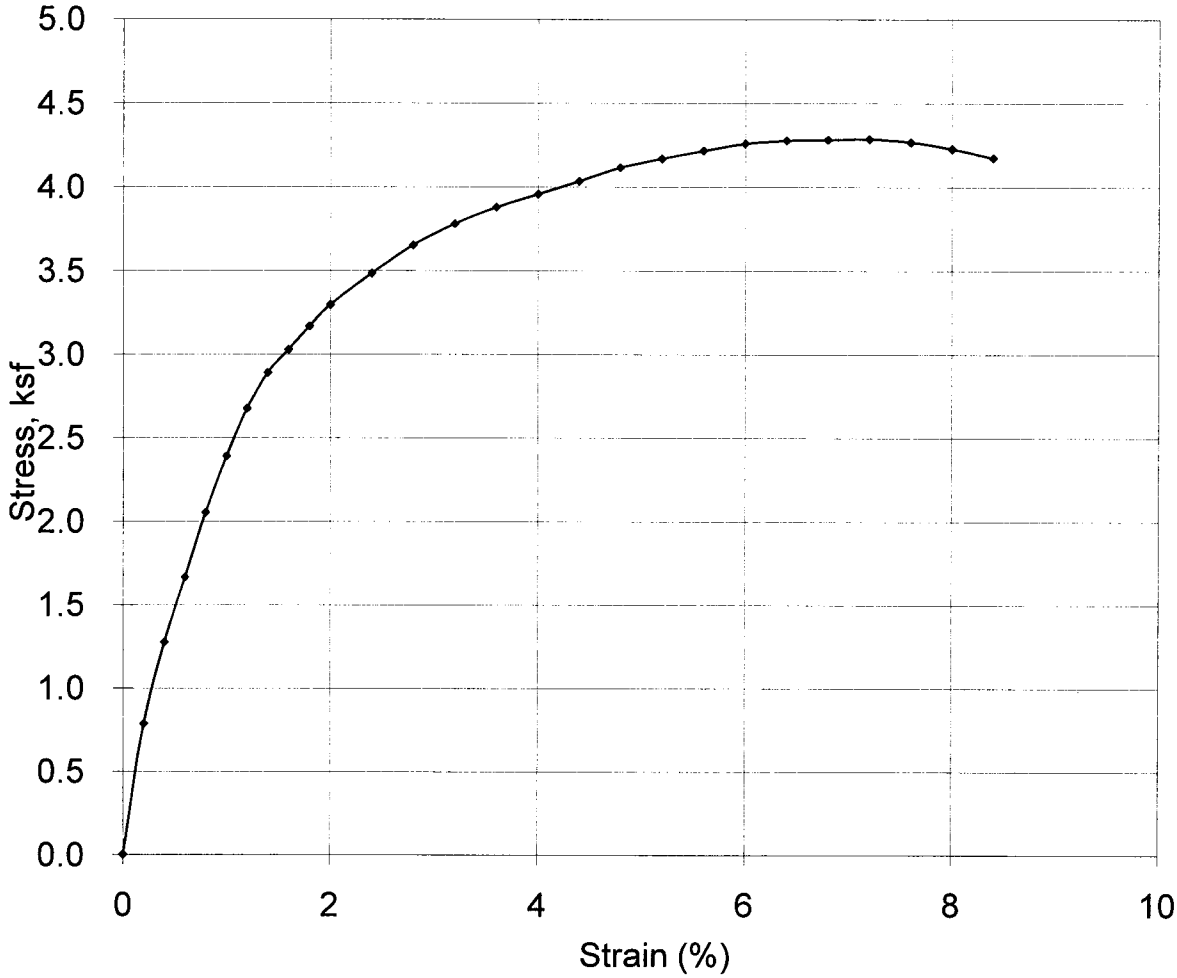
Depth (feet): 26

Material Description:
Lean Clay

Unconfined Compressive Strength: 3.3 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**FOURTEEN MILE SLOUGH BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-6B

UNCONFINED COMPRESSION TEST



Boring No.: A-09-FM-001

Sample No. : MC-13

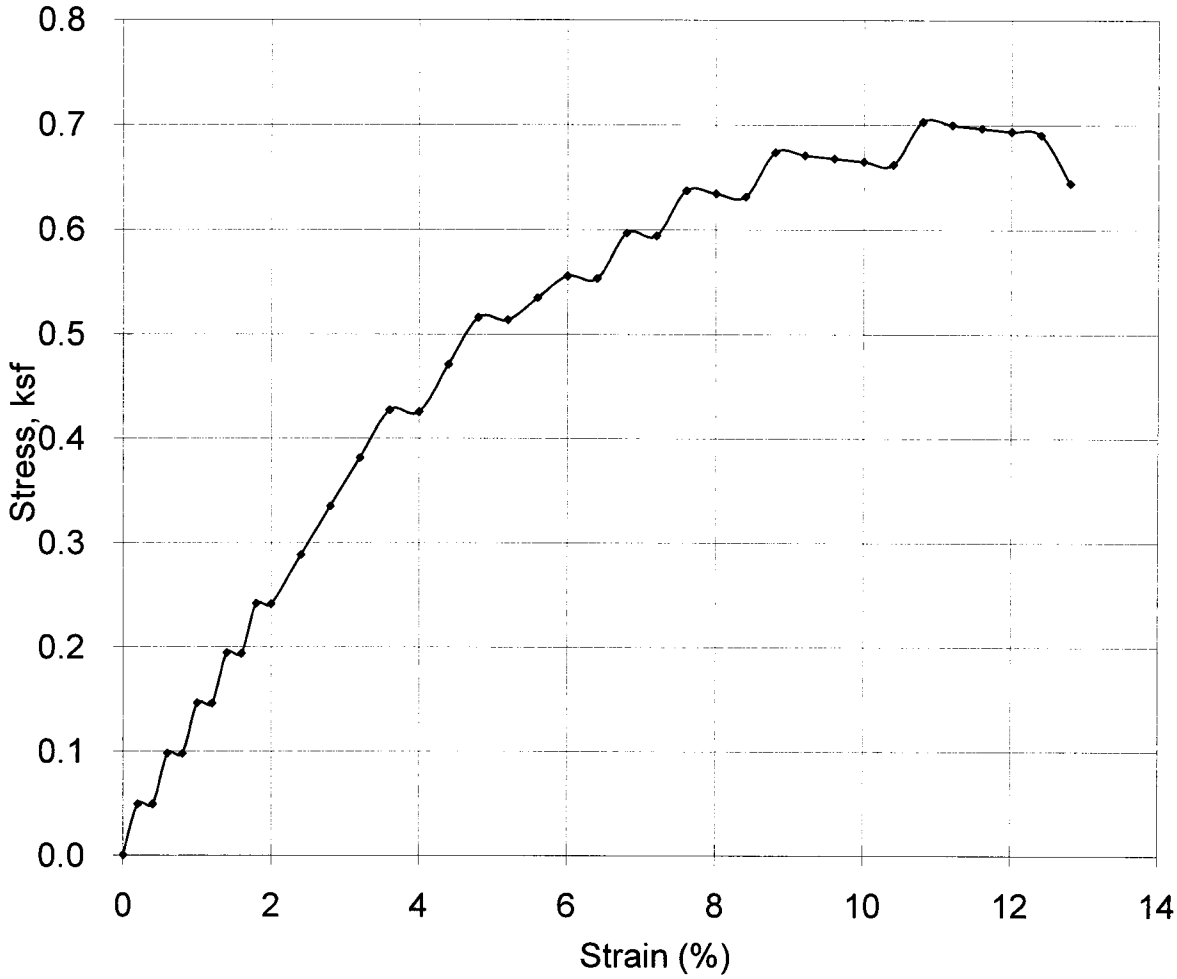
Depth (feet): 71

Material Description:
Lean Clay

Unconfined Compressive Strength: 4.3 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**FOURTEEN MILE SLOUGH BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-6C

UNCONFINED COMPRESSION TEST



Boring No.: R-09-FM-001

Sample No. : MC-4

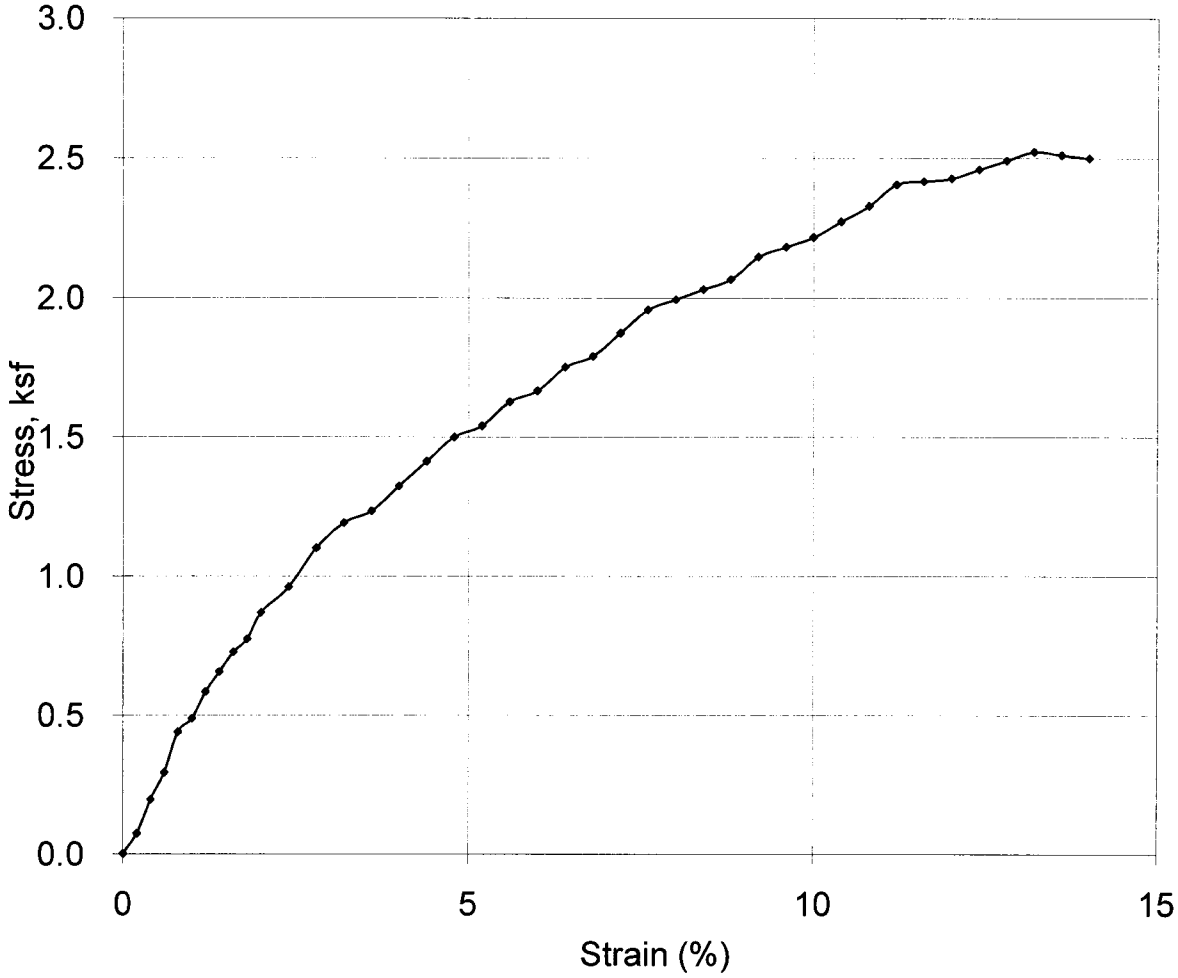
Depth (feet): 15

Material Description:
Fat Clay

Unconfined Compressive Strength: 0.7 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**FOURTEEN MILE SLOUGH BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	**PLATE NO.: B-6D**

UNCONFINED COMPRESSION TEST



Boring No.: R-09-FM-001

Sample No. : MC-11

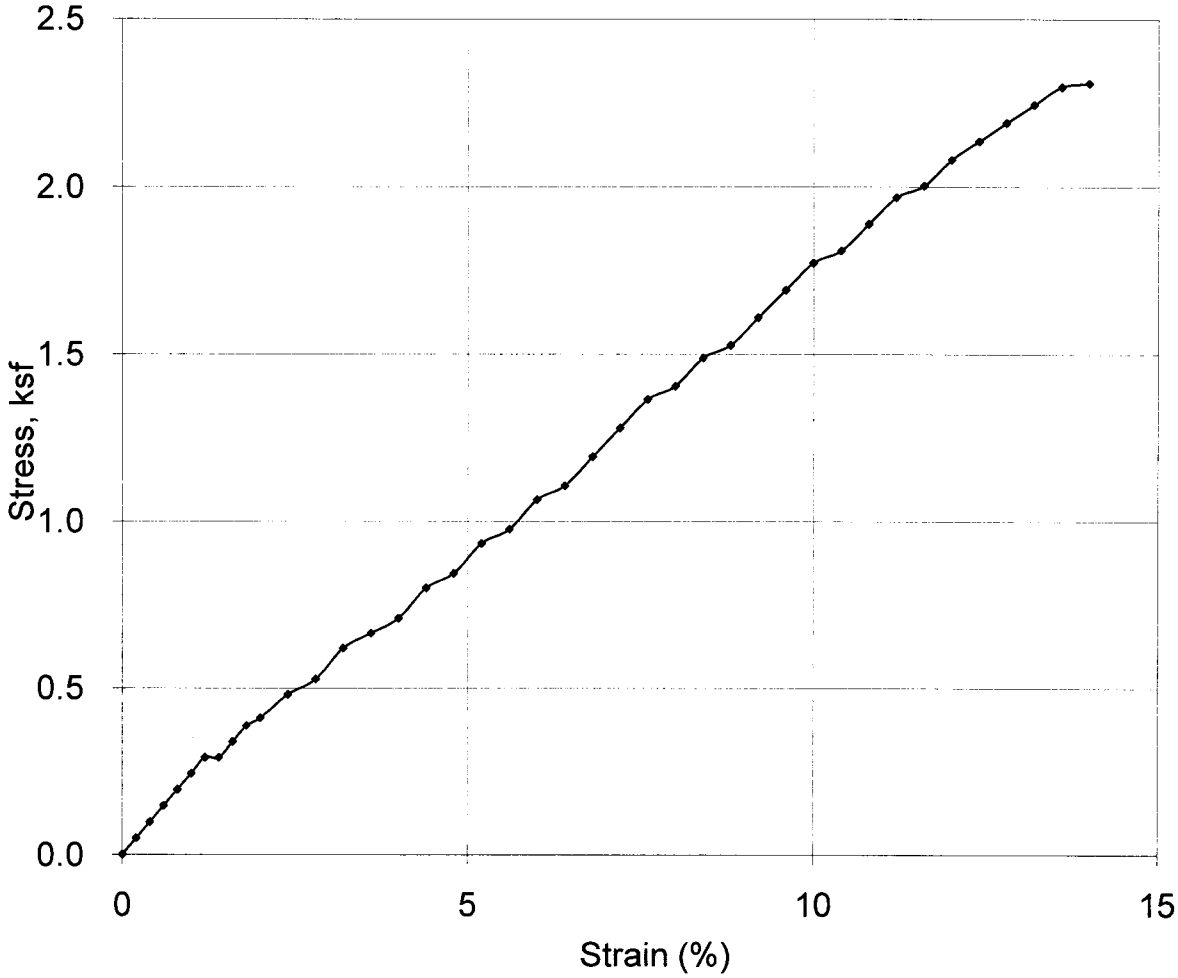
Depth (feet): 51

Material Description:
Lean Clay

Unconfined Compressive Strength: 2.5 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**FOURTEEN MILE SLOUGH BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	**PLATE NO.: B-6E**

UNCONFINED COMPRESSION TEST



Boring No.: R-09-FM-001

Sample No. : MC-13

Depth (feet): 61

Material Description:
Lean Clay

Unconfined Compressive Strength: 2.3 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**FOURTEEN MILE SLOUGH BRIDGE (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-6F

APPENDIX C

Evaluation of Liquefaction Potential

Evaluation of Liquefaction Potential - Eastern Mile Slough (Boring A-09-FM-001)

Depth	Description	SPT N	γ' / γ_{sat}
0.5'	Lean Clay	SPT N=4	γ' = 110 pcf
1.0'	Lean Clay	Avg SPT N=12	γ' = 115 pcf
12.5'	Lean Clay/Silt	Avg SPT N=17	γ' = 115 pcf, γ_{sat} = 130 pcf
3'	(Loose) Silty Sand	SPT N=9	γ_{sat} = 130 pcf
14'	Poorly Graded Sand (Medium dense)	Avg SPT N=24	γ_{sat} = 130 pcf
6'	Silty Sand	SPT N=50/5"	γ_{sat} = 130 pcf
4.5'	Lean Clay	SPT N=19	γ_{sat} = 130 pcf
7.5'	Lean Clay	SPT N=35	γ_{sat} = 130 pcf
12'	Lean Clay	SPT N=15	γ_{sat} = 130 pcf
20.5'	Sandy Lean Clay/Lean Clay/Silt	Avg SPT N>50	γ_{sat} = 130 pcf
7'	Silty Sand	SPT N=39	γ_{sat} = 135 pcf
4.5'	Lean Clay	SPT N>50	γ_{sat} = 125 pcf

22'

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-FM-001 (Fourteen Mile Slough Bridge)(Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 22

SOIL GROUPS

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR$_{7.0}$/CSR)*MSF*Kσ*Kα*

Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	*CYCLIC STRESS RATIO (CSR)* σ_v (psf)	σ_v' (psf)	γ_d	CSR	*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)* SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	8.5	2	4	SPT	935.0	935.0	0.96		5	1.46	1	0.75	1.2	1.15	6.99	55%			1.00	1	
2	18.5	2	12	SPT	2085.0	2085.0	0.92		14	0.98	1	0.80	1.2	1.15	14.97	55%			0.99	1	
3	31.0	2	17	SPT	3657.5	3095.0	0.83		20	0.80	1	0.80	1.2	1.15	17.41	55%			0.88	1	
4	34.0	1	9	SPT	4047.5	3297.5	0.81	0.13	10	0.78	1	0.80	1.2	1.15	8.93	39%	15.71	0.17	0.86	1	1.33
5	48.0	1	24	SPT	5867.5	4242.5	0.67	0.12	28	0.69	1	0.80	1.2	1.15	20.99	21%	26.58	0.33	0.80	1	2.57
6	54.0	1	50	SPT	6647.5	4647.5	0.62	0.12	58	0.66	1	0.80	1.2	1.15	41.78	15%	46.29		0.78	1	
7	58.5	2	19	SPT	7232.5	4951.3	0.59		22	0.64	1	0.80	1.2	1.15	15.38	55%			0.76	1	
8	66.0	2	35	SPT	8207.5	5457.5	0.55		40	0.61	1	0.80	1.2	1.15	26.99	55%			0.74	1	
9	78.0	2	15	SPT	9767.5	6267.5	0.50		17	0.56	1	0.80	1.2	1.15	10.79	55%			0.71	1	
10	98.5	2	50	SPT	12432.5	7651.3	0.47		58	0.51	1	0.80	1.2	1.15	32.56	55%			0.67	1	
11	105.5	1	39	SPT	13377.5	8158.8	0.46	0.10	45	0.50	1	0.80	1.2	1.15	24.60	15%	28.28	0.38	0.66	1	3.00
12	110.0	2	50	SPT	13767.5	8267.5	0.46		58	0.49	1	0.80	1.2	1.15	31.33	55%			0.65	1	

PARIKH

Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement 206127.01

SHEET NO OF

CALCULATED BY A Lam DATE 5/2009

CHECKED BY DATE

SCALE

Evaluation of Liquefaction Potential - Fountain Mile Slough (Boring R-09-FM-001) Toe of the slope

FILL

FILL

13.5 — Lean Clay Avg SPT N = 7 $\gamma' = 110$ pcf

3.5 — Fat Clay SPT N = 4 $\gamma' = 105$ pcf

11.5' — Poorly Graded Sand / Silty Sand Avg SPT N = 19 $\gamma_{sat} = 130$ pcf

7.5' — Lean Clay SPT N = 5 $\gamma_{sat} = 120$ pcf

13.5' — Silty Sand Avg SPT N = 25 $\gamma_{sat} = 130$ pcf

27' — Lean Clay Avg SPT N = 14 $\gamma_{sat} = 130$ pcf

12' — Silt Avg SPT N = 46 $\gamma_{sat} = 125$ pcf

12' — Silt SPT N = 25 $\gamma_{sat} = 120$ pcf

10' — Lean Clay SPT N = 37 $\gamma_{sat} = 130$ pcf

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***R-09-FM-001 (Fourteen Mile Slough Bridge)(Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 17

SOIL GROUPS

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR 7.0/CSR)*MSF*Kσ*Kα*

Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	CYCLIC STRESS RATIO (CSR) σ_v (psf)	σ_v' (psf)	γ_d	CSR	LIQUEFACTION RESISTANCE ($CRR_{7.0}$) SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,cs}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	13.5	2	7	SPT	1485.0	1485.0	0.94		8	1.16	1	0.85	1.2	1.15	11.00	55%			1.00	1	
2	17.0	2	4	SPT	1852.5	1852.5	0.92		5	1.04	1	0.80	1.2	1.15	5.29	55%			1.00	1	
3	28.5	1	19	SPT	3347.5	2628.8	0.85	0.14	22	0.87	1	0.80	1.2	1.15	21.11	15%	24.62	0.28	0.92	1	2.21
4	36.0	2	5	SPT	4247.5	3060.0	0.79		6	0.81	1	0.80	1.2	1.15	5.15	55%			0.88	1	
5	49.0	1	25	SPT	5937.5	3937.5	0.66	0.13	29	0.71	1	0.80	1.2	1.15	22.70	15%	26.29	0.32	0.82	1	2.39
6	76.0	2	14	SPT	9447.5	5760.0	0.51		16	0.59	1	0.80	1.2	1.15	10.51	55%			0.73	1	
7	88.0	2	46	SPT	10947.5	6510.0	0.48		53	0.55	1	0.80	1.2	1.15	32.48	55%			0.70	1	
8	100.0	2	25	SPT	12327.5	7140.0	0.47		29	0.53	1	0.80	1.2	1.15	16.85	55%			0.68	1	
9	110.0	2	37	SPT	13807.5	7995.0	0.46		43	0.50	1	0.80	1.2	1.15	23.57	55%			0.66	1	

Vertical Pile Capacity Calculations

PROJECT: FOURTEEN MILE SLOUGH BRIDGE (WIDEN)
BRIDGE NO: 29-0175 R/L

FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft) [1]	Pile Cut-off Elev. (ft) [1]	Pile Cap Size (ft)		Permissible Settlement (in.)	No. of Piles per Support Per Bridge
					B	L[4]		
Abutment 1	WSD	2'-0" Dia CIDH	17 ±	N/A	2.5'	13.3+49.7+13.3	1"	10
Bent 2	LRFD	2'-0" Dia CIDH	10	10	N/A	N/A	1"	10
Bent 3	LRFD	2'-0" Dia CIDH	2	2	N/A	N/A	1"	10
Bent 4	LRFD	2'-0" Dia CIDH	1	1	N/A	N/A	1"	10
Bent 5	LRFD	2'-0" Dia CIDH	1	1	N/A	N/A	1"	10
Bent 6	LRFD	2'-0" Dia CIDH	2	2	N/A	N/A	1"	10
Bent 7	LRFD	2'-0" Dia CIDH	10	10	N/A	N/A	1"	10
Abutment 8	WSD	2'-0" Dia CIDH	18.5±	N/A	2.5	13.3+49.7+13.3	1"	10

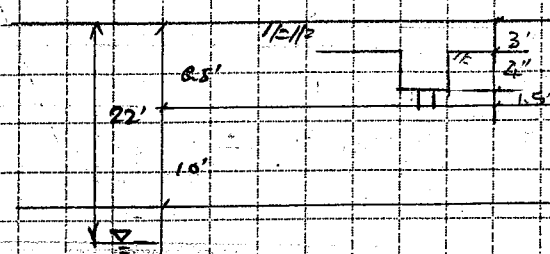
FOUNDATION DESIGN LOADS

Support No.	Service-I Limit State (kips)[2]			Strength Limit State[3] (Controlling Group, kips)				Extreme Limit State[4] (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per[6] Support	Max. Per Pile	Per[6] Support	Per[6] Support	Max. Per Pile	Per Support	Max. Per Pile	Per[6] Support	Max. Per Pile	Per Support	Max. Per Pile
Abutment 1	170 + 430 + 170	105	105 + 285 + 105	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	300 + 835 + 300	178	155 + 450 + 155	445 + 1310 + 445	267	0	0	155 + 450 + 155	210	0	75
Bent 3	325 + 900 + 325	194	175 + 510 + 175	490 + 1400 + 490	293	0	0	175 + 510 + 175	240	0	70
Bent 4	325 + 900 + 325	194	175 + 510 + 175	490 + 1400 + 490	293	0	0	175 + 510 + 175	240	0	70
Bent 5	325 + 900 + 325	194	175 + 510 + 175	490 + 1400 + 490	293	0	0	175 + 510 + 175	240	0	70
Bent 6	325 + 900 + 325	194	175 + 510 + 175	490 + 1400 + 490	293	0	0	175 + 510 + 175	240	0	70
Bent 7	300 + 835 + 300	178	155 + 450 + 155	445 + 1310 + 445	267	0	0	155 + 450 + 155	210	0	75
Abutment 8	170 + 430 + 170	105	105 + 285 + 105	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1. Elevations at abutments vary with the cross slope of the superstructure, but can be assumed at approximately 4'± below finish grade elevation.
2. Total Load for Service-I Limit State @ Abutments 1 & 8 = DL + LL (HL93) + Earth
3. Strength Limit State = Assumes LRFD at Bent 2 - 7 using 1.25DC + 1.50DW + 1.75LL (HL-93) or 1.25DC + 1.50DW + 1.35LL (P15).
4. Extreme Limit State = DL + Seismic
5. There are three pile cap lengths given. The first is from the left outside widening the second is from the center widening and the third is from the right outside widening.
6. There are three loads given Per Support. The first is from the left outside widening the second is from the center widening and the third is from the right outside widening.

Pile Capacity Analyses - Fourteen Mile Slough (A-09-008) (Hwy 5) Abutment 1
2'Φ CIDH concrete pile - 102 tons service load



Lean Clay SPT N=4 $\gamma' = 110$ pcf
c = 500 psf

Lean Clay Avg SPT N=12 $\gamma' = 115$ pcf
c = 1500 psf

Lean Clay/Silt Avg SPT N=17 $\gamma' = 115$ pcf
c = 2000 psf $\gamma_{sat} = 130$ pcf

3' (Loose) Silty Sand SPT N=9 $\gamma_{sat} = 130$ pcf

14' Poorly Graded Sand Avg SPT N=24
(Medium dense) $\gamma_{sat} = 130$ pcf

6' Silty Sand SPT N=50/5" $\gamma_{sat} = 130$ pcf

4.5' Lean Clay SPT N=19 $\gamma_{sat} = 130$ pcf

7.5' Lean Clay SPT N=35 $\gamma_{sat} = 130$ pcf

12' Lean Clay SPT N=15 $\gamma_{sat} = 130$ pcf

20.5' Sandy Lean Clay/Lean Clay/Silt Avg SPT N>50
$\gamma_{sat} = 130$ pcf

7' Silty Sand SPT N=39 $\gamma_{sat} = 135$ pcf

4.5' Lean Clay SPT N>50 $\gamma_{sat} = 125$ pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Foureteen Mile SLough Bridge (Widen) Abutment 1
206127.FMS

ALLOWABLE PILE CAPACTY

12/01/09
206127.BR

Abutment 1 (Based on Boring A-09-FM-001)

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

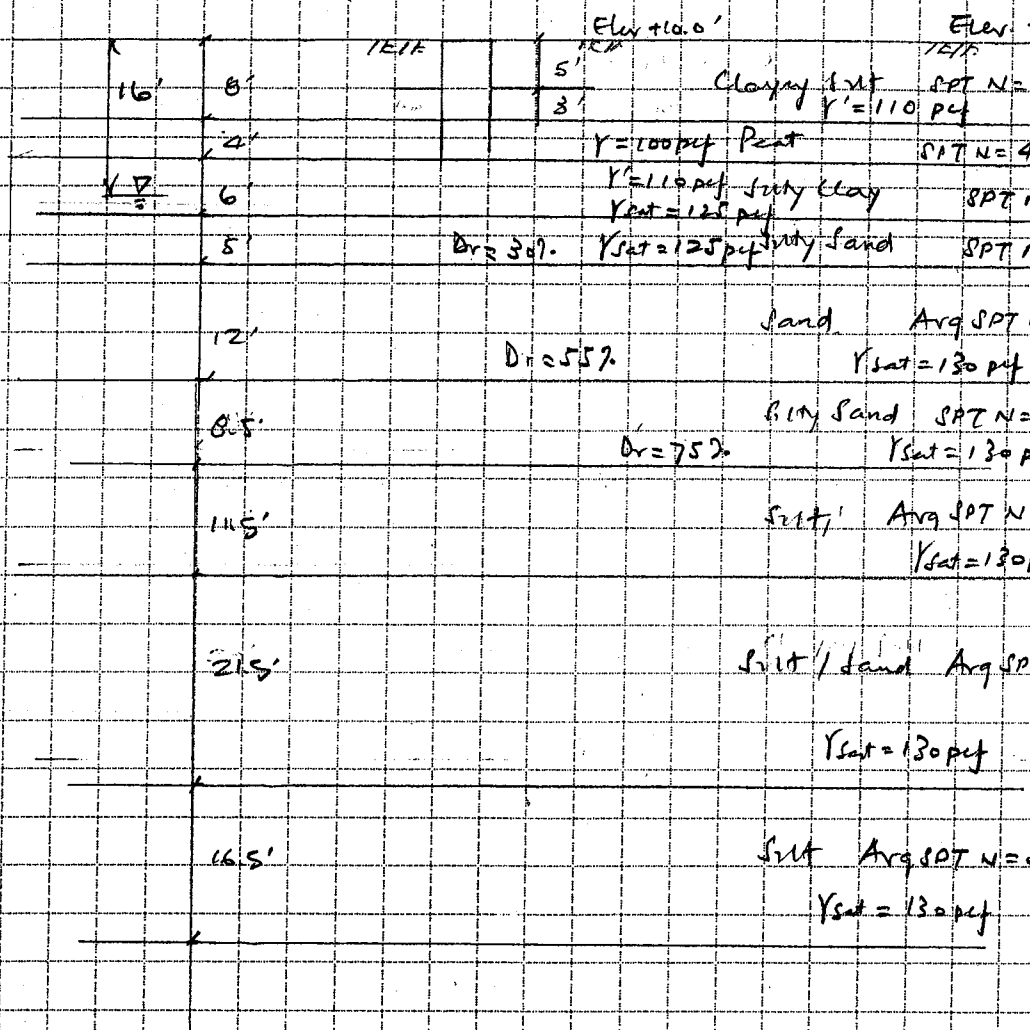
F.S. FOR COMPRESSION = 1
F.S. FOR UPLIFT = 1
PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.5	2	CLAY	110	605	302.5	4	SOFT	0.55	500			0.00	0.00	0.00	0.00
5.5	9.0	2	CLAY	115	402.5	806.3	12	STIFF	0.55	1500			0.00	0.00	0.00	0.00
9.0	12.0	2	CLAY	115	345	1180.0	12	STIFF	0.55	1500			7.38	7.38	5.17	5.17
12.0	15.5	2	CLAY	115	402.5	1553.8	12	STIFF	0.55	1500			8.61	16.00	6.03	11.20
15.5	19.0	2	CLAY	115	402.5	1956.3	17	V.STIFF	0.55	2125			12.20	28.20	8.54	19.74
19.0	23.5	2	CLAY	68	306	2310.5	17	V.STIFF	0.55	2125			15.69	43.89	10.98	30.72
23.5	28.0	2	CLAY	68	306	2616.5	17	V.STIFF	0.55	2125			15.69	59.58	10.98	41.70
28.0	31.0	1	SAND	68	204	2871.5	9	MED DENSE			33	0.46	11.82	71.40	5.91	47.61
31.0	35.0	1	SAND	68	272	3109.5	24	DENSE			37	0.72	26.88	98.28	13.44	61.05
35.0	37.0	1	SAND	68	136	3313.5	24	DENSE			36	0.69	13.64	111.92	6.82	67.87
37.0	45.0	1	SAND	68	544	3653.5	24	DENSE			36	0.64	55.41	167.33	27.71	95.58
45.0	51.0	1	SAND	68	408	4129.5	50	V. DENSE			39	0.56	41.74	209.07	20.87	116.45
51.0	55.5	2	CLAY	68	306	4486.5	19	V.STIFF	0.55	2375			17.53	226.60	12.27	128.72
55.5	59.5	2	CLAY	68	272	4775.5	35	STIFF	0.48	4375			25.12	251.73	17.59	146.31
59.5	63.0	2	CLAY	68	238	5030.5	35	STIFF	0.48	4375			21.98	273.71	15.39	161.70
63.0	69.0	2	CLAY	68	408	5353.5	15	STIFF	0.55	1875			18.46	292.17	12.92	174.62
69.0	75.0	2	CLAY	68	408	5761.5	15	STIFF	0.55	1875			18.46	310.62	12.92	187.54
75.0	80.0	2	CLAY	68	340	6135.5	50	STIFF	0.45	6250			41.95	352.57	29.36	216.90
80.0	85.0	2	CLAY	68	340	6475.5	50	STIFF	0.45	6250			41.95	394.52	29.36	246.27
85.0	90.0	2	CLAY	68	340	6815.5	50	STIFF	0.45	6250			41.95	436.47	29.36	275.63
90.0	95.5	2	CLAY	68	374	7172.5	50	STIFF	0.45	6250			46.14	482.61	32.30	307.93
95.5	99.0	1	SAND	73	255.5	7487.3	39	DENSE			36	0.25	19.54	502.15	9.77	317.70
99.0	102.5	1	SAND	73	255.5	7742.8	39	DENSE			36	0.25	20.21	522.36	10.10	327.81
102.5	107.0	2	CLAY	63	283.5	8012.3	50	STIFF	0.45	6250			37.75	560.12	26.43	354.23

Abutment 1 Pile cut-off elevation is +13.0 feet. Finished grade elevation is +17.0 feet
105 kips/pile (Service Load). Required pile capacity (comp) = 105 kips /(0.5) = 210 kips (105 tons)
Pile Length @ Abutment 1 = 37.0' - 4.0' (Finish grade to pile cut-off) = 33.0'.
Pile tip at 33.0' (comp) below pile cut-off (pile tip @ Elev. -20.0' (comp), pile cut-off @ Elev. +13.0'), Ult. Pile Capacity =111.9 tons (Friction) > required pile capacity (210 k/105 tons), OK

Sheet 2 of 16

Pile Capacity Analyses - Fourteen Mile Slough Bridge Widen (Boring B-2) Bent 2
24" ACIDH concrete pile - Nominal Resistance = 171.5 tons



CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Foureteen Mile SLough Bridge (Widen) Bent 2
206127.FMS

Bent 2 (Based on Boring B-2)

ALLOWABLE PILE CAPACTY 12/01/09

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1
F.S. FOR UPLIFT = 1
PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

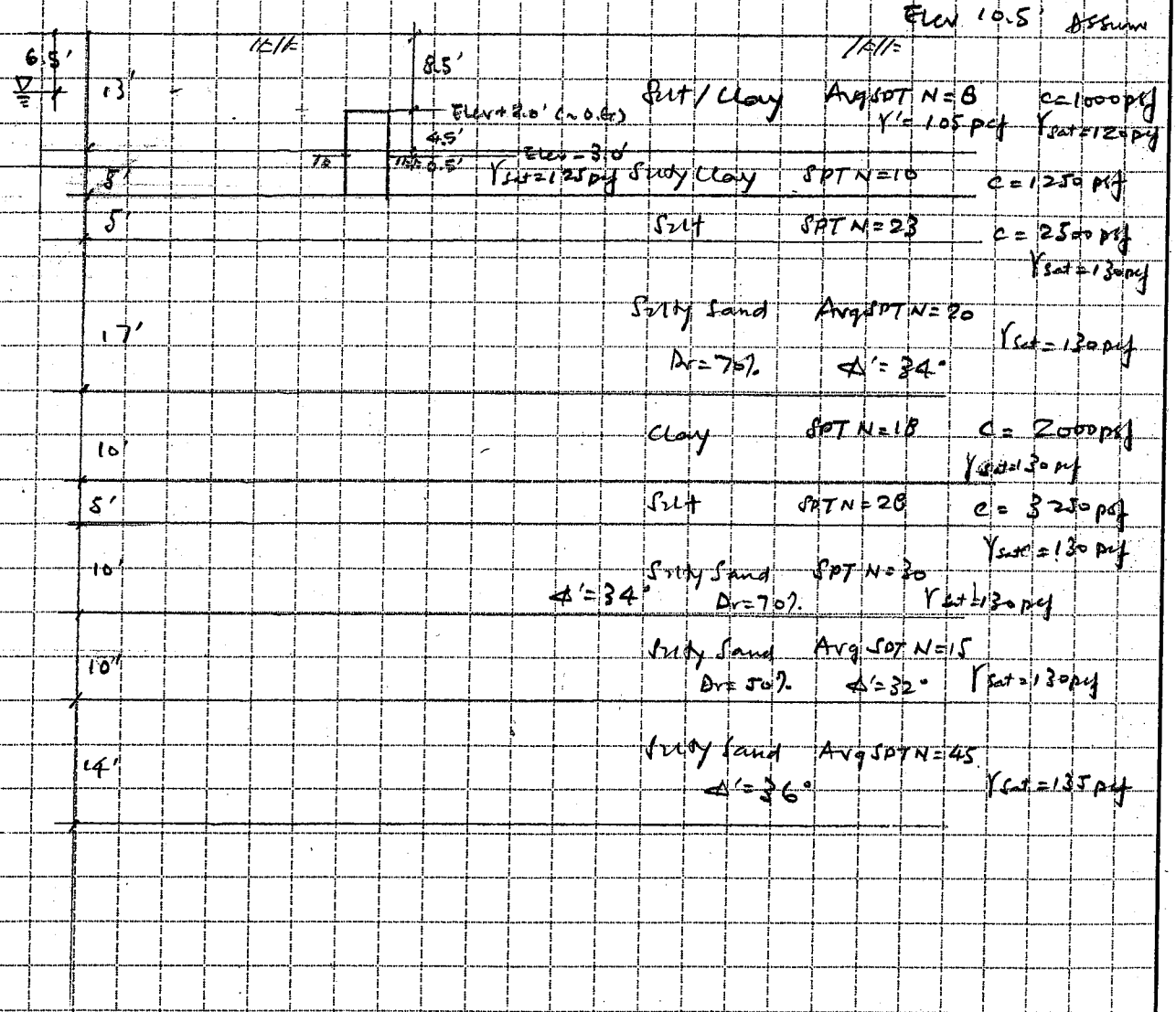
DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	88	440	220.0	13	STIFF	0.55	1625			0.00	0.00	0.00	0.00
5.0	8.0	2	CLAY	110	330	605.0	13	STIFF	0.55	1625			8.00	8.00	5.60	5.60
8.0	12.0	2	CLAY	100	400	970.0	4	SOFT	0.55	500			3.28	11.28	2.30	7.90
12.0	16.0	2	CLAY	110	440	1390.0	12	STIFF	0.55	1500			9.84	21.12	6.89	14.79
16.0	18.0	2	CLAY	63	126	1673.0	12	STIFF	0.55	1500			4.92	26.05	3.45	18.23
18.0	23.0	1	SAND	63	315	1893.5	5	LOOSE			32	0.30	8.37	34.41	4.18	22.42
23.0	29.0	1	SAND	68	408	2255.0	12	MED DENSE			34	0.65	26.21	60.62	13.10	35.52
29.0	35.0	1	SAND	68	408	2663.0	12	MED DENSE			34	0.59	28.08	88.69	14.04	49.56
35.0	39.5	1	SAND	68	306	3020.0	32	DENSE			38	0.68	27.41	116.10	13.70	63.26
39.5	43.5	1	SAND	68	272	3309.0	32	DENSE			38	0.63	24.89	140.99	12.44	75.70
43.5	51.0	2	CLAY	68	510	3700.0	27	V.STIFF	0.53	3375			40.11	181.10	28.08	103.78
51.0	55.0	2	CLAY	68	272	4091.0	27	V.STIFF	0.53	3375			21.39	202.49	14.98	118.76
55.0	60.0	2	CLAY	68	340	4397.0	34	STIFF	0.49	4250			30.90	233.40	21.63	140.39
60.0	65.0	2	CLAY	68	340	4737.0	34	STIFF	0.49	4250			30.90	264.30	21.63	162.02
65.0	70.0	2	CLAY	68	340	5077.0	34	STIFF	0.49	4250			30.90	295.20	21.63	183.65
70.0	76.5	2	CLAY	68	442	5468.0	34	STIFF	0.49	4250			40.17	335.37	28.12	211.77
76.5	81.5	2	CLAY	68	340	5859.0	35	STIFF	0.48	4375			31.40	366.78	21.98	233.75
81.5	86.5	2	CLAY	68	340	6199.0	35	STIFF	0.48	4375			31.40	398.18	21.98	255.74
86.5	93.0	2	CLAY	68	442	6590.0	35	STIFF	0.48	4375			40.82	439.00	28.58	284.31

Bent 2 Pile cut-off elevation is +10.0 feet. Finished grade elevation is +10.0 feet - 1.0 foot (scour depth) = +9.0 feet
267 kips/pile (Strength Limit State). Required pile capacity (comp)= 267 kips /(0.7) = 382 kips (191 tons). 75kips/pile (Extreme Limit State). Required pile capacity (tens) = 75 kips (38 tons)
Pile Length @ Bent 2 = 55.0'.
Pile tip at 55.0'(comp)& 35'(tens)below pile cut-off (pile tip @ Elev.-45.0'(comp)&-25.0'(tens), pile cut-off @ Elev. +10.0'), Ult. Pile Capacity =202.5 T (Friction) > required pile capacity (382k/191 tons), OK

Sheet 4 of 16

Pile Capacity Analyses - Fourteen Mile Slough Bridge (Boring B-5) Bent 3

24" ø CIDH concrete pile nominal resistance = 171.5 tons



CAST-IN-DRILLED-HOLE PILE CAPACITY

Design per O'Neil and Reese (1999)

I-5 Foureteen Mile SLough Bridge (Widen) Bent 3

206127.FMS

ALLOWABLE PILE CAPACTY

10/15/09

206127.BR

Bent 3 (Based on Boring B-5)

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION

FACTOR (CLAY) = 0.95

FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS

1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES

GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.

AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	$\Delta\sigma'$ (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	ϕ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0	4.5	2	CLAY	45	203	101.5	8	MEDIUM	0.55	1000			0.00	0.00	0.00	0.00
4.5	5	2	CLAY	63	31.5	218.8	8	MEDIUM	0.55	1000			0.82	0.82	0.57	0.57
5	9.5	2	CLAY	63	283.5	376.3	10	STIFF	0.55	1250			9.23	10.05	6.46	7.03
9.5	14.5	2	CLAY	68	340	688.0	23	V.STIFF	0.55	2875			23.58	33.63	16.51	23.54
14.5	19.5	1	SAND	68	340	1028.0	20	DENSE			38	0.94	14.46	48.10	7.23	30.78
19.5	24.5	1	SAND	68	340	1368.0	20	DENSE			37	0.87	17.69	65.78	8.84	39.62
24.5	29.5	1	SAND	68	340	1708.0	20	DENSE			37	0.80	20.34	86.13	10.17	49.79
29.5	31.5	1	SAND	68	136	1946.0	20	DENSE			37	0.75	8.76	94.88	4.38	54.17
31.5	36.5	2	CLAY	68	340	2184.0	18	V.STIFF	0.55	2250			18.46	113.34	12.92	67.09
36.5	41.5	2	CLAY	68	340	2524.0	18	V.STIFF	0.55	2250			18.46	131.80	12.92	80.01
41.5	46.5	2	CLAY	68	340	2864.0	28	V.STIFF	0.53	3500			27.41	159.21	19.18	99.19
46.5	51	1	SAND	68	306	3187.0	30	DENSE			38	0.56	23.85	183.05 ←	11.92	111.12
51	56.5	1	SAND	68	374	3527.0	30	DENSE			37	0.51	29.53	212.58	14.76	125.88
56.5	61.5	1	SAND	68	340	3884.0	15	MED DENSE			34	0.46	26.82	239.40	13.41	139.29
61.5	66.5	1	SAND	68	340	4224.0	15	MED DENSE			34	0.42	26.46	265.86	13.23	152.52
66.5	71.5	1	SAND	73	365	4576.5	45	DENSE			38	0.38	25.84	291.71	12.92	165.44
71.5	76.5	1	SAND	73	365	4941.5	45	DENSE			38	0.34	24.96	316.67	12.48	177.92
76.5	80.5	1	SAND	73	292	5270.0	45	DENSE			38	0.30	19.11	335.78	9.55	187.48

Bent 3 Pile cut-off elevation is +2.0 feet. Finished grade elevation is +2.0 feet - 1.0 foot (scour depth) = +1.0 foot

240 kips/pile (Strength Limit State). Required pile capacity = 240 kips /(0.7) = 350 kips (175 tons)

Pile Length @ Bent 3 = 51.0'.

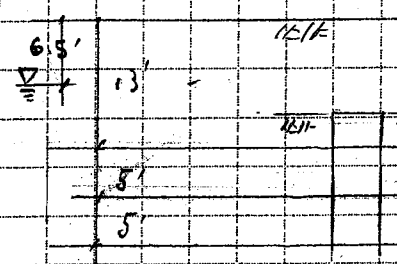
Pile tip at approxiamtely 51.0' below pile cut-off (pile tip @ Elev. -49.0', pile cut-off @ Elev. +2.0'), available Ult. Pile Capacity =183.1 tons (Friction) > required pile capacity (350k/175 tons), OK

Sheet 6 of 16

Pile Capacity Analysis - Fourteen Mile Slough Bridge (Boring B-5) Bent 4

24" ϕ CIDH concrete pile - Nominal resistance = 171.5 tons

Elev 10.5' Assume



	Silt / Clay	Avg SPT N = 8, γ' = 105 pcf	c = 1000 psf, γsat = 120 pcf
	Silty Clay	γsat = 125 pcf, SPT N = 10	c = 1250 psf
	Silt	SPT N = 23	c = 2500 psf, γsat = 130 pcf
17'	Silty Sand	Avg SPT N = 20, Dr = 70%, φ' = 34°	γsat = 130 pcf
10'	Clay	SPT N = 18	c = 2000 psf, γsat = 130 pcf
5'	Silt	SPT N = 28	c = 3250 psf, γsat = 130 pcf
10'	Silty Sand	φ' = 34°, Dr = 70%, SPT N = 30	γsat = 130 pcf
10'	Silty Sand	Avg SPT N = 15, Dr = 50%, φ' = 32°	γsat = 130 pcf
14'	Silty Sand	Avg SPT N = 45, φ' = 36°	γsat = 135 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Foureteen Mile SLough Bridge (Widen) Bent 4
206127.FMS

Bent 4 (Based on Boring B-5)

ALLOWABLE PILE CAPACTY

12/01/09
206127.BR

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.5	2	CLAY	41	145	72.5	8	MEDIUM	0.55	1000			0.00	0.00	0.00	0.00
3.5	5.0	2	CLAY	63	94.5	192.3	10	STIFF	0.55	1250			0.00	0.00	0.00	0.00
5.0	8.5	2	CLAY	63	220.5	349.8	10	STIFF	0.55	1250			7.18	7.18	5.02	5.02
8.5	13.5	2	CLAY	68	340	630.0	23	V.STIFF	0.55	2875			23.58	30.76	16.51	21.53
13.5	18.5	1	SAND	68	340	970.0	20	DENSE			38	0.96	13.89	44.65	6.94	28.48
18.5	23.5	1	SAND	68	340	1310.0	20	DENSE			37	0.88	17.22	61.87	8.61	37.09
23.5	28.5	1	SAND	68	340	1650.0	20	DENSE			37	0.81	19.97	81.85	9.99	47.08
28.5	30.5	1	SAND	68	136	1888.0	20	DENSE			37	0.77	8.64	90.48	4.32	51.39
30.5	35.5	2	CLAY	68	340	2126.0	18	V.STIFF	0.55	2250			18.46	108.94	12.92	64.31
35.5	40.5	2	CLAY	68	340	2466.0	18	V.STIFF	0.55	2250			18.46	127.40	12.92	77.23
40.5	45.5	2	CLAY	68	340	2806.0	28	V.STIFF	0.53	3500			27.41	154.80	19.18	96.42
45.5	51.0	1	SAND	68	374	3163.0	30	DENSE			38	0.56	29.18	183.98	14.59	111.01
51.0	57.0	1	SAND	68	408	3554.0	30	DENSE			37	0.51	32.31	216.29	16.16	127.16
57.0	60.5	1	SAND	68	238	3877.0	15	MED DENSE			34	0.47	18.83	235.12	9.42	136.58
60.5	65.5	1	SAND	68	340	4166.0	15	MED DENSE			34	0.43	26.62	261.75	13.31	149.89
65.5	70.5	1	SAND	73	365	4518.5	45	V. DENSE			39	0.39	26.07	287.81	13.03	162.92
70.5	75.5	1	SAND	73	365	4883.5	45	DENSE			38	0.35	25.24	313.05	12.62	175.54
75.5	79.5	1	SAND	73	292	5212.0	45	DENSE			38	0.31	19.37	332.43	9.69	185.23

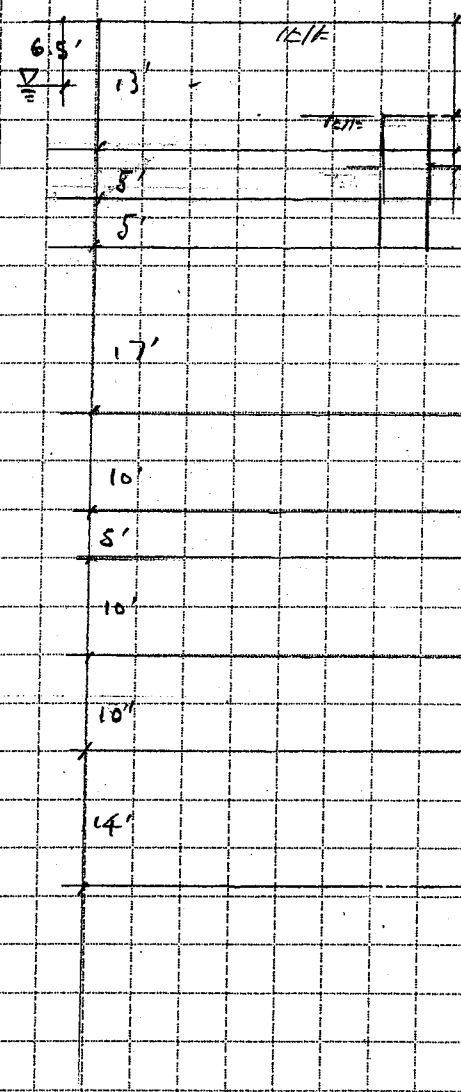
Bent 4 Pile cut-off elevation is +1.0 feet. Finished grade elevation is +1.0 feet - 1.0 foot (scour depth) = 0.0 foot
293 kips/pile (Strength Limit State). Required pile capacity (comp)= 293 kips /(0.7) = 419 kips (210 tons). 70 kips/pile (Extreme Limit State). Required pile capacity (tens)= 70 kips (35 tons)
Pile Length @ Bent 4 = 57.0'.
Pile tip at 57.0'(comp)&23.5'(tens) below pile cut-off (pile tip @Elev. -56.0'(comp)&@Elev.-22.5'(tens), pile cut-off @ Elev. +1.0'),Ult. Pile Capacity =216.3T (Friction)>required pile capacity (419k/210 tons)

Sheet 8 of 16

Pile Capacity Analyses - Fourteen Mile Slough Bridge (Boring B-5) Bent 5

24" ϕ CIDH concrete pile - Nominal Resistance = 171.5 tons

Elev 10.5' Assume



Silt/Clay	Avg SPT N=8, γ'=105 pcf	c=1000 psf	γ_{sat}=120 pcf
Silty Clay	SPT N=10	c=1250 psf	
Silt	SPT N=23	c=2500 psf	γ_{sat}=130 pcf
Silty Sand, Dr=70%	Avg SPT N=20, ϕ'=34°		γ_{sat}=130 pcf
Clay	SPT N=18	c=2000 psf	γ_{sat}=130 pcf
Silt	SPT N=28	c=3250 psf	γ_{sat}=130 pcf
Silty Sand, Dr=70%	SPT N=30, ϕ'=34°		γ_{sat}=130 pcf
Silty Sand, Dr=50%	Avg SPT N=15, ϕ'=32°		γ_{sat}=130 pcf
Silty Sand, ϕ'=36°	Avg SPT N=45		γ_{sat}=135 pcf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Foureteen Mile SLough Bridge (Widen) Bent 5
206127.FMS

Bent 5 (Based on Boring B-5)

ALLOWABLE PILE CAPACTY

12/01/09
206127.BR

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

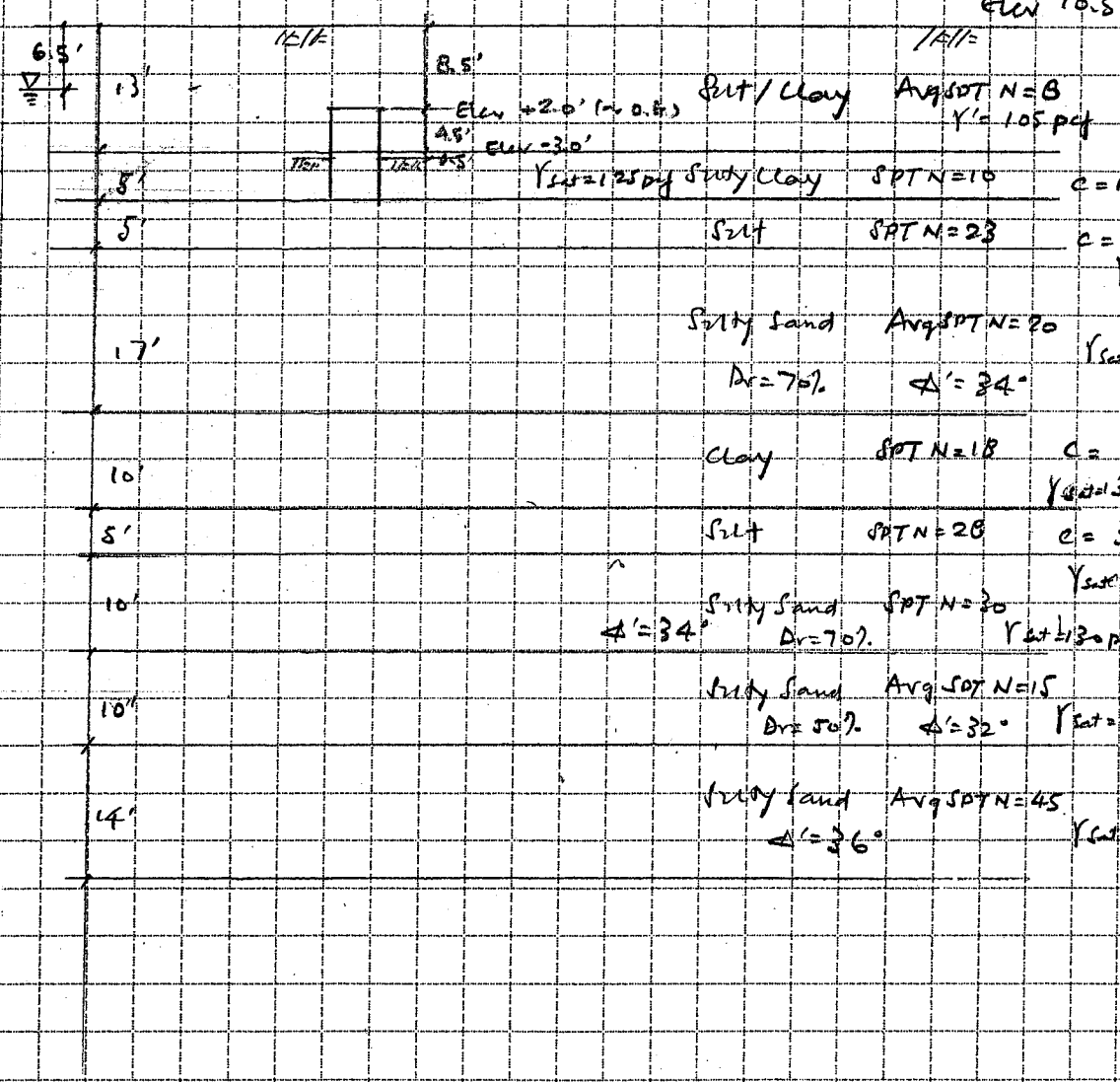
PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.5	2	CLAY	41	145	72.5	8	MEDIUM	0.55	1000			0.00	0.00	0.00	0.00
3.5	5.0	2	CLAY	63	94.5	192.3	10	STIFF	0.55	1250			0.00	0.00	0.00	0.00
5.0	8.5	2	CLAY	63	220.5	349.8	10	STIFF	0.55	1250			7.18	7.18	5.02	5.02
8.5	13.5	2	CLAY	68	340	630.0	23	V.STIFF	0.55	2875			23.58	30.76	16.51	21.53
13.5	18.5	1	SAND	68	340	970.0	20	DENSE			38	0.96	13.89	44.65	6.94	28.48
18.5	23.5	1	SAND	68	340	1310.0	20	DENSE			37	0.88	17.22	61.87	8.61	37.09
23.5	28.5	1	SAND	68	340	1650.0	20	DENSE			37	0.81	19.97	81.85	9.99	47.08
28.5	30.5	1	SAND	68	136	1888.0	20	DENSE			37	0.77	8.64	90.48	4.32	51.39
30.5	35.5	2	CLAY	68	340	2126.0	18	V.STIFF	0.55	2250			18.46	108.94	12.92	64.31
35.5	40.5	2	CLAY	68	340	2466.0	18	V.STIFF	0.55	2250			18.46	127.40	12.92	77.23
40.5	45.5	2	CLAY	68	340	2806.0	28	V.STIFF	0.53	3500			27.41	154.80	19.18	96.42
45.5	51.0	1	SAND	68	374	3163.0	30	DENSE			38	0.56	29.18	183.98	14.59	111.01
51.0	57.0	1	SAND	68	408	3554.0	30	DENSE			37	0.51	32.31	216.29	16.16	127.16
57.0	60.5	1	SAND	68	238	3877.0	15	MED DENSE			34	0.47	18.83	235.12	9.42	136.58
60.5	65.5	1	SAND	68	340	4166.0	15	MED DENSE			34	0.43	26.62	261.75	13.31	149.89
65.5	70.5	1	SAND	73	365	4518.5	45	V. DENSE			39	0.39	26.07	287.81	13.03	162.92
70.5	75.5	1	SAND	73	365	4883.5	45	DENSE			38	0.35	25.24	313.05	12.62	175.54
75.5	79.5	1	SAND	73	292	5212.0	45	DENSE			38	0.31	19.37	332.43	9.69	185.23

Bent 5 Pile cut-off elevation is +1.0 feet. Finished grade elevation is +1.0 feet - 1.0 foot (scour depth) = 0.0 foot
293 kips/pile (Strength Limit State). Required pile capacity (comp)= 293 kips /(0.7) = 419 kips (210 tons). 70 kips/pile (Extreme Limit State). Required pile capacity (tens)= 70 kips (35 tons)
Pile Length @ Bent 5 = 57.0'.
Pile tip at 57.0'(comp)&23.5'(tens) below pile cut-off (pile tip @Elev. -56.0'(comp)&@Elev.-22.5'(tens), pile cut-off @ Elev. +1.0'),Ult. Pile Capacity =216.3T (Friction)>required pile capacity (419k/210 tons)

Sheet 10 of 16

Pile Capacity Analyses - Fourteen Mile Slough Bridge (Boring B-5) Bent 6
24" ø CIDH concrete pile - Nominal Resistance = 171.5 tons



CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Foureteen Mile SLough Bridge (Widen) Bent 6
206127.FMS

Bent 6 (Based on Boring B-5)

ALLOWABLE PILE CAPACTY

12/02/09
206127.BR

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

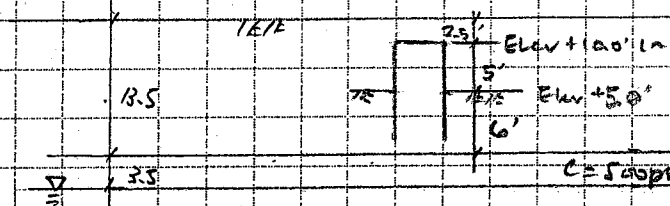
PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	4.5	2	CLAY	45	203	101.5	8	MEDIUM	0.55	1000			0.00	0.00	0.00	0.00
4.5	5.0	2	CLAY	63	31.5	218.8	8	MEDIUM	0.55	1000			0.82	0.82	0.57	0.57
5.0	9.5	2	CLAY	63	283.5	376.3	10	STIFF	0.55	1250			9.23	10.05	6.46	7.03
9.5	14.5	2	CLAY	68	340	688.0	23	V.STIFF	0.55	2875			23.58	33.63	16.51	23.54
14.5	19.5	1	SAND	68	340	1028.0	20	DENSE			38	0.94	14.46	48.10	7.23	30.78
19.5	24.5	1	SAND	68	340	1368.0	20	DENSE			37	0.87	17.69	65.78	8.84	39.62
24.5	29.5	1	SAND	68	340	1708.0	20	DENSE			37	0.80	20.34	86.13	10.17	49.79
29.5	31.5	1	SAND	68	136	1946.0	20	DENSE			37	0.75	8.76	94.88	4.38	54.17
31.5	36.5	2	CLAY	68	340	2184.0	18	V.STIFF	0.55	2250			18.46	113.34	12.92	67.09
36.5	41.5	2	CLAY	68	340	2524.0	18	V.STIFF	0.55	2250			18.46	131.80	12.92	80.01
41.5	46.5	2	CLAY	68	340	2864.0	28	V.STIFF	0.53	3500			27.41	159.21	19.18	99.19
46.5	51.0	1	SAND	68	306	3187.0	30	DENSE			38	0.56	23.85	183.05	11.92	111.12
51.0	56.5	1	SAND	68	374	3527.0	30	DENSE			37	0.51	29.53	212.58	14.76	125.88
56.5	61.5	1	SAND	68	340	3884.0	15	MED DENSE			34	0.46	26.82	239.40	13.41	139.29
61.5	66.5	1	SAND	68	340	4224.0	15	MED DENSE			34	0.42	26.46	265.86	13.23	152.52
66.5	71.5	1	SAND	73	365	4576.5	45	DENSE			38	0.38	25.84	291.71	12.92	165.44
71.5	76.5	1	SAND	73	365	4941.5	45	DENSE			38	0.34	24.96	316.67	12.48	177.92
76.5	80.5	1	SAND	73	292	5270.0	45	DENSE			38	0.30	19.11	335.78	9.55	187.48

Bent 6 Pile cut-off elevation is +2.0 feet. Finished grade elevation is +2.0 feet - 1.0 foot (scour depth) = +1.0 foot
293 kips/pile (Strength Limit State). Required pile capacity (comp)= 293 kips /(0.7) = 419 kips (210 tons). 70 kips/pile (Extreme Limit State). Required pile capacity (tens)= 70 kips (35 tons)
Pile Length @ Bent 5 = 56.5'.
Pile tip at 56.5'(comp)&24.5'(tens) below pile cut-off (pile tip @Elev. -54.5'(comp)&@Elev.-22.5'(tens), pile cut-off @ Elev. +2.0'),Ult. Pile Capacity =212.6T (Friction)>required pile capacity (419k/210 tons)

Sheet 12 of 16

Pile Capacity Analyses - Fountain Mill Slough (Boring R-09-009) (Toe of the slope)
24" ø CIDH concrete pile Bent 7
Nominal Resistance = 171.5 tons



13.5 — Lean Clay Avg SPT N = 7 — γ' = 110 pcf, c = 875 psf

3.5 — c = 500 psf — Fat Clay SPT N = 4 — γ' = 105 pcf

11.5' — Poorly Graded Sand / Silty Sand — Dr = 65% — φ' = 34° — Avg SPT N = 19 — γsat = 130 pcf

7.5' — c = 625 psf — Lean Clay SPT N = 5 — γsat = 120 pcf

13.5' — Dr = 69% — Silty Sand Avg SPT N = 25 — φ' = 35° — γsat = 130 pcf

27' — Lean Clay Avg SPT N = 14 — γsat = 130 pcf, c = 1500 psf

12' — Silt Avg SPT N = 46 — γsat = 125 pcf, c = 4000 psf

12' — Silt SPT N = 25 — γsat = 120 pcf, c = 2750 psf

10' — Lean Clay SPT N = 37 — γsat = 130 pcf, c = 3500 psf

CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Foureteen Mile SLough Bridge (Widen) Bent 7
206127.FMS

Bent 7 (Based on Boring R-09-FM-001)

ALLOWABLE PILE CAPACTY

12/02/09
206127.BR

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

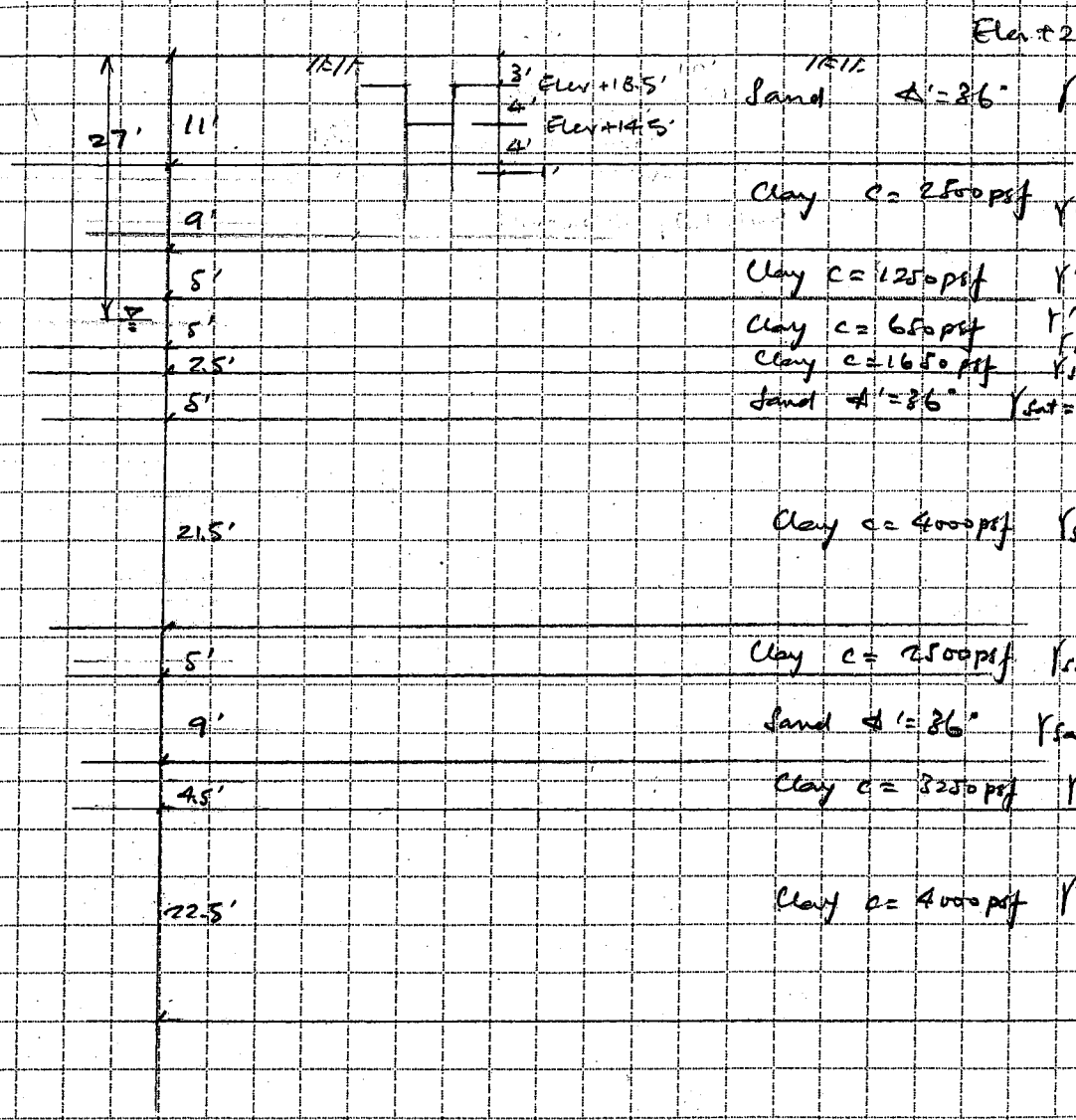
DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	5.0	2	CLAY	88	440	220.0	7	MEDIUM	0.55	875			0.00	0.00	0.00	0.00
5.0	11.0	2	CLAY	110	660	770.0	7	MEDIUM	0.55	875			8.61	8.61	6.03	6.03
11.0	14.5	2	CLAY	105	367.5	1283.8	4	SOFT	0.55	500			2.87	11.48	2.01	8.04
14.5	20.5	1	SAND	68	408	1671.5	19	DENSE			37	0.94	27.98	39.46	13.99	22.03
20.5	26.0	1	SAND	68	374	2062.5	19	DENSE			36	0.85	28.73	68.19	14.37	36.39
26.0	30.0	2	CLAY	58	232	2365.5	5	MEDIUM	0.55	625			4.10	72.30	2.87	39.27 ←
30.0	33.5	2	CLAY	58	203	2583.0	5	MEDIUM	0.55	625			3.59	75.89	2.51	41.78
33.5	38.5	1	SAND	68	340	2854.5	25	DENSE			37	0.69	29.38	105.26	14.69	56.47
38.5	43.5	1	SAND	68	340	3194.5	25	DENSE			37	0.64	30.28	135.54	15.14	71.61
43.5	46.5	1	SAND	68	204	3466.5	25	DENSE			37	0.59	18.44	153.98	9.22	80.83
46.5	51.5	2	CLAY	68	340	3738.5	14	STIFF	0.55	1750			14.36	168.34	10.05	90.88
51.5	57.0	2	CLAY	68	374	4095.5	14	STIFF	0.55	1750			15.79	184.13	11.05	101.93
57.0	61.5	2	CLAY	68	306	4435.5	14	STIFF	0.55	1750			12.92	197.05 ←	9.04	110.97
61.5	67.5	2	CLAY	68	408	4792.5	14	STIFF	0.55	1750			17.23	214.28	12.06	123.03
67.5	73.5	2	CLAY	68	408	5200.5	14	STIFF	0.55	1750			17.23	231.50	12.06	135.09
73.5	79.5	2	CLAY	63	378	5593.5	46	STIFF	0.45	5750			46.31	277.82	32.42	167.51
79.5	85.5	2	CLAY	63	378	5971.5	46	STIFF	0.45	5750			46.31	324.13	32.42	199.93
85.5	91.5	2	CLAY	58	348	6334.5	25	V.STIFF	0.54	3125			30.41	354.54	21.29	221.22
91.5	97.5	2	CLAY	58	348	6682.5	25	V.STIFF	0.54	3125			30.41	384.95	21.29	242.50
97.5	102.5	2	CLAY	68	340	7026.5	37	STIFF	0.47	4625			32.34	417.29	22.63	265.14
102.5	107.5	2	CLAY	68	340	7366.5	37	STIFF	0.47	4625			32.34	449.62	22.63	287.77

Bent 7 Pile cut-off elevation is +10.0 feet. Finished grade elevation is +10.0 feet - 1.0 foot (scour depth) = +9.0 feet
267 kips/pile (Strength Limit State). Required pile capacity (comp)= 267 kips /(0.7) = 382 kips (191 tons). 75kips/pile (Extreme Limit State). Required pile capacity (tens) = 75 kips (38 tons)
Pile Length @ Bent 7 = 61.5'.
Pile tip at 61.5'(comp)& 30'(tens)below pile cut-off (pile tip @ Elev.-51.5'(comp)&-20.0'(tens), pile cut-off @ Elev. +10.0'), Ult. Pile Capacity =197.1 T (Friction) > required pile capacity (382k/191 tons), OK

Sheet 14 of 16

Pile Capacity Analyses - Fourteen Mile Slough Bridge (Widen) (CPT-09-003) - Abutment 6

24" ϕ CIDH concrete pile - Service Load 102 tons



CAST-IN-DRILLED-HOLE PILE CAPACITY
Design per O'Neil and Reese (1999)

I-5 Foureteen Mile SLough Bridge (Widen) Abut 8
206127.FMS

ALLOWABLE PILE CAPACTY

12/02/09
206127.BR

Abut 8 (Based on Boring CPT-09-FM-001)

PILE DIA, B (ft) = 2.00

SIDE MOBILIZATION
FACTOR (CLAY) = 0.95
FACTOR (SAND) = 0.95

INCREAMENTS (FT)= 5

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS
3. IGM (COHESIONLESS MATERIAL)

SOIL TYPES
GRAVEL, SAND, NSILT, CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) = 6.28 SQ. FT.
AREA OF THE PILE TIP 3.14 SQ. FT.

DEPTH FROM	DEPTH TO	SOIL GROUP	SOIL TYPE	γ' (pcf)	Δσ' (psf)	σ' (psf)	S.P.T - N	CONSISTENCY	SHEAR STRENGTH PARAMETERS α	c(psf)	φ (°)	β	ULTIMATE COMPRESSION (PER SECTION) (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	ULTIMATE UPLIFT (PER SECTION) (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	8.0	1	SAND	125	1000	500.0	10	MED DENSE			35	0.82	0.00	0.00	0.00	0.00
8.0	9.0	2	CLAY	120	120	1060.0	20	V.STIFF	0.55	2500			0.00	0.00	0.00	0.00
9.0	17.0	2	CLAY	120	960	1600.0	20	V.STIFF	0.55	2500			32.81	32.81	22.97	22.97
17.0	22.0	2	CLAY	115	575	2367.5	10	STIFF	0.55	1250			10.25	43.07	7.18	30.15
22.0	24.0	2	CLAY	105	210	2760.0	5	MEDIUM	0.55	625			2.05	45.12	1.44	31.58
24.0	27.0	2	CLAY	58	174	2952.0	5	MEDIUM	0.55	625			3.08	48.19	2.15	33.74
27.0	29.5	2	CLAY	63	157.5	3117.8	13	STIFF	0.55	1625			6.67	54.86	4.67	38.40
29.5	34.5	1	SAND	68	340	3366.5	13	MED DENSE			34	0.64	32.04	86.90	16.02	54.42
34.5	39.0	2	CLAY	68	306	3689.5	32	V.STIFF	0.50	4000			26.85	113.75 ←	18.79	73.22
39.0	44.5	2	CLAY	68	374	4029.5	32	V.STIFF	0.50	4000			32.81	146.56	22.97	96.18
44.5	50.5	2	CLAY	68	408	4420.5	32	V.STIFF	0.50	4000			35.80	182.36	25.06	121.24
50.5	56.0	2	CLAY	68	374	4811.5	32	V.STIFF	0.50	4000			32.81	215.17	22.97	144.21
56.0	61.0	2	CLAY	68	340	5168.5	20	V.STIFF	0.55	2500			20.51	235.68	14.36	158.57
61.0	66.0	1	SAND	68	340	5508.5	28	DENSE			36	0.42	34.85	270.53	17.43	175.99
66.0	70.0	1	SAND	68	272	5814.5	28	DENSE			36	0.39	26.83	297.37	13.42	189.41
70.0	74.5	2	CLAY	68	306	6103.5	26	V.STIFF	0.54	3250			23.45	320.81	16.41	205.82
74.5	79.5	2	CLAY	68	340	6426.5	32	V.STIFF	0.50	4000			29.83	350.64	20.88	226.71
79.5	84.5	2	CLAY	68	340	6766.5	32	V.STIFF	0.50	4000			29.83	380.47	20.88	247.59
84.5	90.5	2	CLAY	68	408	7140.5	32	V.STIFF	0.50	4000			35.80	416.27	25.06	272.64
90.5	97.0	2	CLAY	68	442	7565.5	32	V.STIFF	0.50	4000			38.78	455.05	27.15	299.79

Abutment 8 Pile cut-off elevation is +14.5 feet. Finished grade elevation is +18.5 feet
105 kips/pile (Service Load). Required pile capacity (comp) = 105 kips /(0.5) = 210 kips (105 tons)
Pile Length @ Abutment 8 = 39.0' - 4.0' (Finish grade to pile cut-off) = 35.0'.
Pile tip at 35.0' (comp) below pile cut-off (pile tip @ Elev. -20.5' (comp), pile cut-off @ Elev. +14.5'), Ult. Pile Capacity =113.8 tons (Friction) > required pile capacity (210 k/105 tons), OK

Sheet 16 of 16

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Fourteen Mile Slough Bridge (Widen)

Boring A-09-FM-001 (Abutment 1) (24-inch dia. CIDH concrete pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
20.0* to 11.5	0 to 8.5	Clay (soft)	$\phi = 0°$; c=500 psf	3, Clay	100	0.01	0.064	-
11.5 to 1.5	8.5 to 18.5	Clay (stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.067	-
1.5 to -2.0	18.5 to 22	Clay (stiff to very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.067	-
-2.0 to -11.0	22 to 31	Clay (stiff to very stiff)	$\phi = 0°$; c=2000 psf	3, Clay	500	0.007	0.039	Assumed GWL at 22 feet deep below existing ground surface for design.
-11.0 to -14.0	31 to 34	Sand (loose)	$\phi = 30°$; c= 0 psf	4, Sand	20	-	0.039	-
-14.0 to -28.0	34 to 48	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.039	-
-28.0 to -34.0	48 to 54	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-
-34.0 to -38.5	54 to 58.5	Clay (very stiff)	$\phi = 0°$; c=2250 psf	3, Clay	500	0.007	0.039	-
-38.5 to -46.0	58.5 to 66	Clay (very stiff to hard)	$\phi = 0°$; c=3500 psf	3, Clay	1000	0.005	0.039	-
-46.0 to -58.0	66 to 78	Clay (stiff)	$\phi = 0°$; c=1750 psf	3, Clay	500	0.007	0.039	-
-58.0 to -78.5	78 to 98.5	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-78.5 to -85.5	98.5 to 105.5	Sand (dense)	$\phi = 34°$; c= 0 psf	4, Sand	125	-	0.042	-
-85.5 to -90.0	105.5 to 110	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.036	-

* Finish ground elevation of + 17.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Fourteen-Mile Slough Bridge (Widen)

As-built Boring B-2 (Bent 2) (24-inch diameter Cast-In-Drilled Hole)

10-14-2009

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
10.0* to 2.0	0 to 8	Clay (stiff)	$\phi = 0°$; c= 1500 psf	3, Clay	500	0.007	0.064	-
2.0 to -2.0	8 to 12	Clay (soft)	$\phi = 0°$; c= 500 psf	3, Clay	100	0.01	0.058	-
-2.0 to -6.0	12 to 16	Clay (stiff)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.064	-
-6.0 to -8.0	16 to 18	Clay (stiff)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.036	Assumed GWL at 16 feet deep below existing ground surface for design.
-8.0 to -13.0	18 to 23	Sand (loose)	$\phi = 30°$; c= 0 psf	4, Sand	20	-	0.036	-
-13.0 to -25.0	23 to 35	Sand (loose to medium dense)	$\phi = 32°$; c= 0 psf	4, Sand	60	-	0.039	-
-25.0 to -33.5	35 to 43.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-
-33.5 to -45.0	43.5 to 55	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	-
-45.0 to -66.5	55 to 76.5	Clay (hard)	$\phi = 0°$; c= 3750 psf	3, Clay	1000	0.005	0.039	-
-66.5 to -93.0	76.5 to 93	Clay (hard)	$\phi = 0°$; c= 3750 psf	3, Clay	1000	0.005	0.039	-

*Finish ground elevation of +10.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Fourteen-Mile Slough Bridge (Widen)

As-built Boring B-5 (Bent 3) (24-inch diameter CIDH concrete pile) **10-14-2009**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
10.5* to 4.0	0 to 6.5	Clay (firm to stiff)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.061	-
4.0 to -2.5	6.5 to 13	Clay (firm to stiff)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.033	Assumed GWL at 6.5 feet deep below existing ground surface for design.
-2.5 to -7.5	13 to 18	Clay (firm)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.036	
-7.5 to -12.5	18 to 23	Clay (stiff)	$\phi = 0°$; c= 2500 psf	3, Clay	500	0.007	0.039	-
-12.5 to -29.5	23 to 40	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-29.5 to -39.5	40 to 50	Clay (very stiff)	$\phi = 0°$; c= 2000 psf	3, Clay	1000	0.005	0.039	-
-39.5 to -44.5	50 to 55	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	-
-44.5 to -54.5	55 to 65	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-54.5 to -64.5	65 to 75	Sand (medium dense)	$\phi = 32°$; c= 0 psf	4, Sand	60	-	0.039	-
-64.5 to -78.5	75 to 89	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.042	-

*Finish ground elevation of +2.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Fourteen-Mile Slough Bridge (Widen)

As-built Boring B-5 (Bent 4) (24-inch diameter CIDH concrete pile) **10-14-2009**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
10.5* to 4.0	0 to 6.5	Clay (firm to stiff)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.061	-
4.0 to -2.5	6.5 to 13	Clay (firm to stiff)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.033	Assumed GWL at 6.5 feet deep below existing ground surface for design.
-2.5 to -7.5	13 to 18	Clay (firm)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.036	
-7.5 to -12.5	18 to 23	Clay (stiff)	$\phi = 0°$; c= 2500 psf	3, Clay	500	0.007	0.039	-
-12.5 to -29.5	23 to 40	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-29.5 to -39.5	40 to 50	Clay (very stiff)	$\phi = 0°$; c= 2000 psf	3, Clay	1000	0.005	0.039	-
-39.5 to -44.5	50 to 55	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	-
-44.5 to -54.5	55 to 65	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-54.5 to -64.5	65 to 75	Sand (medium dense)	$\phi = 32°$; c= 0 psf	4, Sand	60	-	0.039	-
-64.5 to -78.5	75 to 89	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.042	-

*Finish ground elevation of +1.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Fourteen-Mile Slough Bridge (Widen)

As-built Boring B-5 (Bent 5) (24-inch diameter CIDH concrete pile) **10-14-2009**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
10.5* to 4.0	0 to 6.5	Clay (firm to stiff)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.061	-
4.0 to -2.5	6.5 to 13	Clay (firm to stiff)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.033	Assumed GWL at 6.5 feet deep below existing ground surface for design.
-2.5 to -7.5	13 to 18	Clay (firm)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.036	
-7.5 to -12.5	18 to 23	Clay (stiff)	$\phi = 0°$; c= 2500 psf	3, Clay	500	0.007	0.039	-
-12.5 to -29.5	23 to 40	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-29.5 to -39.5	40 to 50	Clay (very stiff)	$\phi = 0°$; c= 2000 psf	3, Clay	1000	0.005	0.039	-
-39.5 to -44.5	50 to 55	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	-
-44.5 to -54.5	55 to 65	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-54.5 to -64.5	65 to 75	Sand (medium dense)	$\phi = 32°$; c= 0 psf	4, Sand	60	-	0.039	-
-64.5 to -78.5	75 to 89	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.042	-

*Finish ground elevation of +1.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Fourteen-Mile Slough Bridge (Widen)

As-built Boring B-5 (Bent 6) (24-inch diameter CIDH concrete pile) **10-14-2009**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
10.5* to 4.0	0 to 6.5	Clay (firm to stiff)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.061	-
4.0 to -2.5	6.5 to 13	Clay (firm to stiff)	$\phi = 0°$; c= 1000 psf	3, Clay	100	0.01	0.033	Assumed GWL at 6.5 feet deep below existing ground surface for design.
-2.5 to -7.5	13 to 18	Clay (firm)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.036	
-7.5 to -12.5	18 to 23	Clay (stiff)	$\phi = 0°$; c= 2500 psf	3, Clay	500	0.007	0.039	-
-12.5 to -29.5	23 to 40	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-29.5 to -39.5	40 to 50	Clay (very stiff)	$\phi = 0°$; c= 2000 psf	3, Clay	1000	0.005	0.039	-
-39.5 to -44.5	50 to 55	Clay (very stiff)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	-
-44.5 to -54.5	55 to 65	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	-
-54.5 to -64.5	65 to 75	Sand (medium dense)	$\phi = 32°$; c= 0 psf	4, Sand	60	-	0.039	-
-64.5 to -78.5	75 to 89	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.042	-

*Finish ground elevation of +2.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Fourteen-Mile Slough Bridge (Widen)

Boring R-09-FM-001 (Bent 7) (24-inch diameter CIDH concrete pile) **10-14-2009**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
12.5* to -1.0	0 to 13.5	Clay (soft to firm)	$\phi = 0°$; c= 875 psf	3, Clay	100	0.01	0.061	-
-1.0 to -4.5	13.5 to 17	Clay (soft)	$\phi = 0°$; c= 500 psf	3, Clay	100	0.01	0.061	-
-4.5 to -16.0	17 to 28.5	Sand (medium dense)	$\phi = 34°$; c= 0 psf	4, Sand	60	-	0.039	Assumed GWL at 6.5 feet deep below existing ground surface for design.
-16.0 to -23.5	28.5 to 36	Clay (stiff to very stiff)	$\phi = 0°$; c= 625 psf	3, Clay	500	0.007	0.036	
-23.5 to -36.5	36 to 49	Sand (medium dense)	$\phi = 35°$; c= 0 psf	4, Sand	60	-	0.039	-
-36.5 to -63.5	49 to 76	Clay (stiff to very stiff)	$\phi = 0°$; c= 1500 psf	3, Clay	1000	0.005	0.039	-
-63.5 to -75.5	76 to 88	Clay (hard)	$\phi = 0°$; c= 4000 psf	3, Clay	1000	0.005	0.039	-
-75.5 to -87.5	88 to 100	Clay (very stiff)	$\phi = 0°$; c= 2750 psf	3, Clay	1000	0.005	0.039	-
-87.5 to -97.5	100 to 110	Clay (very stiff to hard)	$\phi = 0°$; c= 3500 psf	3, Clay	1000	0.005	0.039	-

*Finish ground elevation of +10.00 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Fourteen-Mile Slough Bridge (Widen)

CPT-09-FM-001 (Abutment 8) (24-inch diameter CIDH concrete pile) **10-14-2009**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
21.5* to 10.5	0 to 11	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.072	
10.5 to 1.5	11 to 20	Clay (very stiff)	$\phi = 0°$; c= 2500 psf	3, Clay	1000	0.005	0.069	-
1.5 to -3.5	20 to 25	Clay (firm to stiff)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.067	-
-3.5 to -5.5	25 to 27	Clay (soft)	$\phi = 0°$; c= 650 psf	3, Clay	100	0.01	0.061	-
-5.5 to -8.5	27 to 30	Clay (soft)	$\phi = 0°$; c= 650 psf	3, Clay	100	0.01	0.033	Assumed GWL at 27 feet deep below existing ground surface for design.
-8.5 to -11.0	30 to 32.5	Clay (stiff)	$\phi = 0°$; c= 1250 psf	3, Clay	500	0.007	0.036	
-11.0 to -16.0	32.5 to 37.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-
-16.0 to -37.5	37.5 to 59	Clay (very stiff to hard)	$\phi = 0°$; c= 4000 psf	3, Clay	1000	0.005	0.039	-
-37.5 to -42.5	59 to 64	Clay (very stiff)	$\phi = 0°$; c= 2500 psf	3, Clay	1000	0.005	0.039	-
-42.5 to -51.5	64 to 73	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.039	-
-51.5 to -56.0	73 to 77.5	Clay (very stiff to hard)	$\phi = 0°$; c= 3250 psf	3, Clay	1000	0.005	0.039	-
-56.0 to -78.5	77.5 to 100	Clay (very stiff to hard)	$\phi = 0°$; c= 4000 psf	3, Clay	1000	0.005	0.039	-

*Finish ground elevation of +18.50 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	25.0/32.7		

REGISTERED CIVIL ENGINEER DATE

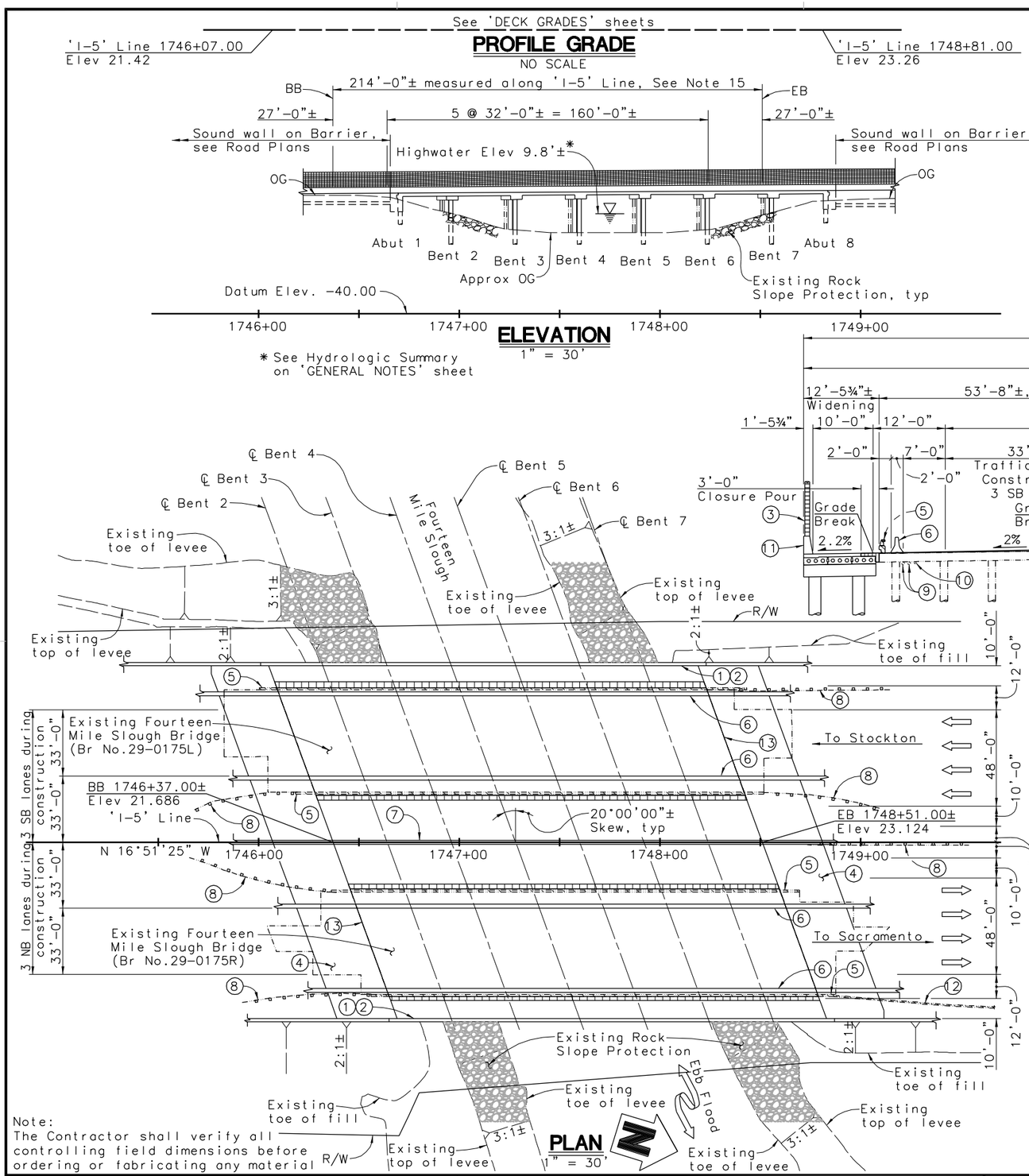
PLANS APPROVAL DATE

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

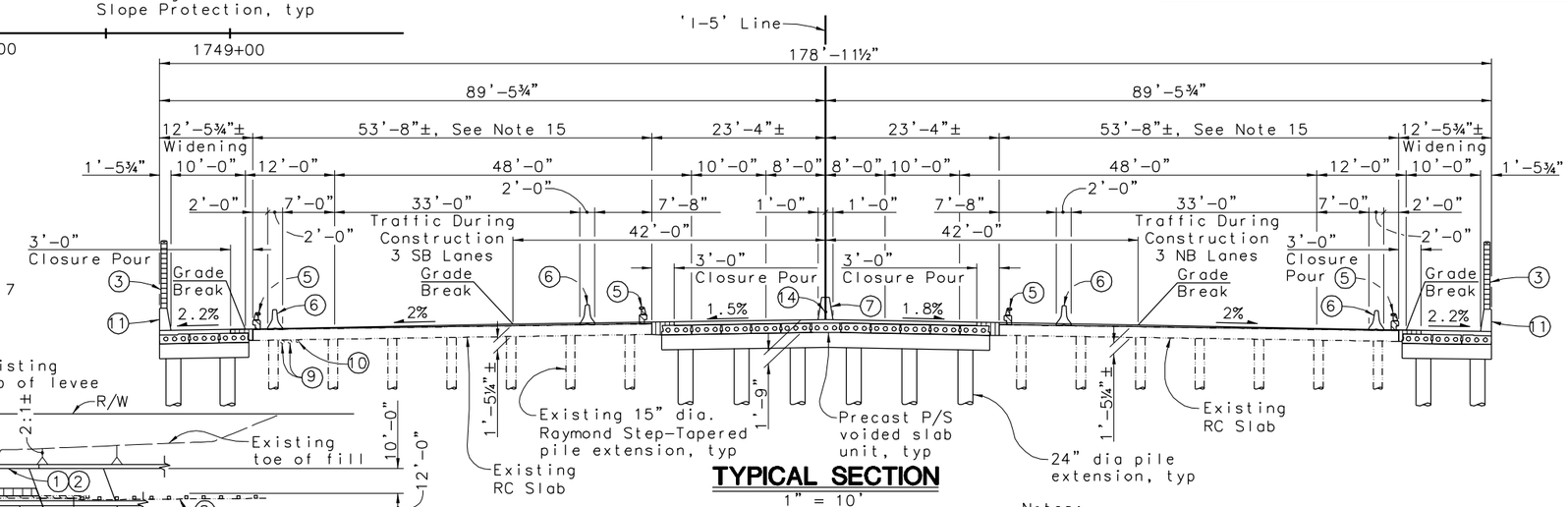
REGISTERED PROFESSIONAL ENGINEER RONALD L OEN No. 46945 Exp. 9/30/11 CIVIL STATE OF CALIFORNIA

BIGGS CARDOSA ASSOCIATES INC.
865 THE ALAMEDA
SAN JOSE, CALIFORNIA 95126

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 E. WEBER AVENUE
STOCKTON, CA 95202



Note:
The Contractor shall verify all controlling field dimensions before ordering or fabricating any material



TYPICAL SECTION
1" = 10'

Legend:

- Bridge Removal (Portion)
- Closure Pour
- Indicates existing structure
- Indicates direction of traffic

Notes:

1. Paint 'Bridge No. 29-0175 R or L'
2. Paint 'Fourteen Mile Slough Bridge'
3. Soundwall (Barrier) (Masonry Block)
4. Structure Approach Type N(30D) & R(30D)
5. Salvage existing Type 9 Barrier Railing
6. Temporary Railing Type K for Stage Construction, see Road Plans
7. Concrete Barrier Type 60A (Mod)
8. Remove existing Bridge Approach Guard Railing, See Road Plans
9. Exist 22kv electrical cable in 6" conduit, total 2, see Road Plans
10. Exist neutral ground in 4" conduit, see Road Plans
11. Concrete Barrier Type 736 (Mod)
12. Exist sound wall to be removed, see Road Plans
13. Remove and replace existing joint seal, full width
14. 4" dia FO (Future), see Electrical Plans
15. Indicates limits of Clean Bridge Deck and Furnish and Place Polyester Concrete Overlay
16. For General Notes, Index to Bridge Plans and Quantities, see 'GENERAL NOTES' sheet
17. For Pile Data Table, see 'FOUNDATION PLAN' sheet

PLAN CHECK SET/NOT FOR CONSTRUCTION (1/15/10)

DESIGN OVERSIGHT	DESIGN	BY S. SUMPTER	CHECKED A. REJAIE	LOAD & RESISTANCE FACTOR DESIGN	LIVE LOADING: HL93 W/"LOW-BOY"; PERMIT DESIGN VEHICLE
SIGN OFF DATE	DETAILS	BY S. SUMPTER	CHECKED A. NOTARO	LAYOUT BY B. GURUNG	CHECKED A. REJAIE
	QUANTITIES	BY S. SUMPTER	CHECKED S. OGHUMU	SPECIFICATIONS BY F. ARON	PLANS AND SPECS COMPARED T. NOTARO

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

JOHN A. ALCIATI
PROJECT ENGINEER

BRIDGE NO. 29-0175 R/L
POST MILES 31.00

FOURTEEN MILE SLOUGH BRIDGE (WIDEN)

GENERAL PLAN

DESIGN GENERAL PLAN SHEET (ENGLISH) (REV. 6-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 6/02/09 9/4/09 10/16/09 12/4/09 1/15/10

SHEET 1 OF 26

FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

2006048 (2006048S1D)

APPENDIX E

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** Fourteen Mile Slough Bridge **Br No*:** 29-0175 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection _X 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Fourteen Mile Slough Bridge **Br No*:** 29-0175 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 12/16/2009 **Structure Name*:** Fourteen Mile Slough Bridge **Br No*:** 29-0175 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	p.6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
3	FR	p.8	Tables 2 and 3, Although the pile tips appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	
4	FR	p.10	Table 4 needs to be updated to our standard LRFD Pile Data Table showing the resistance factor used.	Comment is incorporated. Caltrans standard LRFD Pile Data Table showing the resistance factor used is shown in Table 4.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

5	FR	p.13, S.9.7	A construction consideration may be to abide by Army Corps of Engineers and/or Department of Water Resources construction and backfill requirements, please respond.	This statement is correct. This will be included in the construction consideration. The details provided in this section were discussed with the design team and the agency. Some of these will be incorporated in the specifications.	
6	FR	LOTB	As is stated in the Soil Legend, the sample number or unconfined compressive strength is generally not located to the left of the graphic on the LOTB.	Comment is incorporated.	
7	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #3 above.	
8	FR	NA	C3 – Not Approved (Resubmittal to GS Required)	NA	

FOUNDATION REPORT
SWAIN ROAD UC (WIDEN)
(BRIDGE NO. 29-0215)
STOCKTON, CALIFORNIA
10-SJ-05, PM 31.11 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

PARIKH CONSULTANTS, INC.

2360 Qume Drive, Suite A, San Jose, CA 95131
(408) 452-9000

June 11, 2010

Job No. 206127.BR

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CPT Correlation Data (CPT-09-SR-001)

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Office of Special Funded Projects Comment & Responses Form (with PARKH Consultant, Inc. Responses)

FOUNDATION REPORT
SWAIN ROAD UC (WIDEN)
(BRIDGE NO. 29-0215)
STOCKTON, CALIFORNIA
10-SJ-05 PM 31.11 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Swain Road Undercrossing (Widen) (Bridge No. 29-0215)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in two soil borings and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Swain Road Undercrossing (UC) (Widen) (Bridge No. 29-0215).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter

unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are single-span structures. Based on the as-built general plan and foundation plan of the Swain Road UC, the structures were constructed in 1970. The existing left and right bridges (Bridge No. 29-0215 L/R) are approximately 104 feet long. The left bridge (Bridge No. 29-0215 L) is approximately 53 feet wide and the right bridge (Bridge No. 29-215 R) is approximately 65 feet wide. The right bridge was widened in 1998.

The abutments of the existing left and right bridge are supported on "Reinforced Concrete Piles/Caltrans Class 625" piles based on the as-built data. The specified tip elevation is -52.5 feet (-16 m) (NAVD 29).

Proposed Construction

The widening of the Swain Road UC is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. The left bridge (along southbound I-5) will be widened in this project.

According to Rajappan & Meyer Consulting Engineers Inc., the proposed Swain Road UC (Widen) will include both the median widening and outside widening to match the existing span with supports at Abutments 1 and 2. The widen structure will be a cast-in-place prestressed concrete box girder with approximate length of 104 feet measured along the "I-5" Line. The width of the median widening will be 48 feet and outside widening will be 12 feet 6 inches.

Based on our discussions with the designer, the proposed bridge abutments are to be supported on Caltrans Standard Precast Prestressed Class 200 Alternate "X" (Modified) (14-inch square) piles for the median and outside widening.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The Swain Road UC is located at the intersection of Interstate 5 (I-5) and Swain Road. It is approximately 0.35 miles southeast of Benjamin Holt UC. The Project Location Map, Plate 1, shows the location of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the Swain Road UC (September 1964) (Ref. 1) and Swain Road Undercrossing Widen (March 1997) (Ref. 2) have been used as a reference. Based on the as-built LOTB, Borings B-2 and B-4 were drilled by Caltrans in September 1964 and Cone Penetration Test (CPT) at B-1 and B-3 were performed by Caltrans in 1953 and 1954. Borings B-1 and B-2 were drilled in March 1997 for the Swain Road Undercrossing (Widen). The soil borings in 1964 were drilled to the depths of 65.5 feet and 71 feet and to the bottom elevation approximately -65.5 feet and -71 feet. The CPTs were pushed to the approximate depths of 55 feet and 65 feet and to the bottom elevation approximately -55 feet and -65 feet. The soil borings in 1997 were drilled to the depths of 37 feet and 70.5 feet and to the bottom elevation approximately -37 feet and -65.5 feet.

Field Exploration in 2009

Additional field exploration was conducted in March and April 2009. Boring A-09-SR-001 was drilled on the south side and Boring R-09-SR-001 was drilled and CPT-09-SR-001 was performed on the north side of West Swain Road. Boring R-09-SR-001 was drilled near the toe of the western slope of the southbound bridge abutment. The soil boring was drilled to the depth of

111.5 feet and the CPT was pushed to the approximate depth of 109.5 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit weight, Plasticity Index and grain size classification of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built Borings and CPT, the subsurface soils generally consist of loose to medium dense sands, underlain by interbedded layers of loose to medium dense sands and stiff to hard sandy/silty clays.

Based on Borings A-09-SR-001, R-09-SR-001 and CPT-09-SR-001, the subsurface soils generally consist of medium dense silty/poorly graded sand (embankment fill), underlain by firm to very stiff lean clays, underlain by pockets/lenses/layers of medium dense silty sand/poorly graded sand, underlain by stiff to hard silts/clays/siltstone/claystone and very dense sands.

Groundwater was recorded between Elevation -5 feet and -5.5 feet in September 1964. Groundwater was either not encountered or measured in the soil borings in 1997. Groundwater was measured at Elevation -4.5 feet (the depth of 27 feet below existing ground surface) during

drilling for A-09-SR-001 in March 2009 and Elevation -29.5 feet (the depth of 33 feet below existing ground surface) during drilling for R-09-SR-001 in April 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 3).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 4) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since

there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 5), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 6). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of medium dense sands, underlain by firm to very stiff lean clays, underlain by medium dense sands, underlain by stiff to hard silts/clays and very dense sands. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 7). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Borings A-09-SR-001 and R-09-SR-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-SR-001 attached, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the

project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure (inside and outside), different foundation systems have been evaluated with respect to their design capacity and construction considerations. Caltrans standard Class 200 Alternative "X" (Mod) Piles are recommended to be the foundation system for the abutments of the proposed bridge structure. These piles may be designed for service load of 75 tons at the abutments to the indicated pile tip elevations. Pertinent foundation design information provided by Rajappan & Meyer Consulting Engineers, Inc. including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2A: FOUNDATION DESIGN DATA (MEDIAN WIDEN)

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 200 Alt X (Mod)	13.20	10.20	2.5	52.0	1.00	11
Abut 2	WSD	Class 200 Alt X (Mod)	13.40	10.40	2.5	52.0	1.00	11

TABLE 2B: FOUNDATION DESIGN DATA (OUTSIDE WIDEN)

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 200 Alt X (Mod)	12.15	9.00	2.5	13.0	1.00	4
Abut 2	WSD	Class 200 Alt X (Mod)	12.35	9.20	2.5	13.0	1.00	4
Abut 1 WW	WSD	Class 200 Alt X (Mod)	19.20	16.20	3.0	6.0	1.00	2
Abut 2 WW	WSD	Class 200 Alt X (Mod)	19.42	16.42	3.0	6.0	1.00	2

TABLE 3A: FOUNDATION DESIGN LOADS (MEDIAN WIDEN)

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	1569	143	1177	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1569	143	1177	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

TABLE 3B: FOUNDATION DESIGN LOADS (OUTSIDE WIDEN)

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	599	150	418	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	599	150	418	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Wingwall	38.4	19.2	19.2	N/A	N/A	N/A	N/A	N/A	56	N/A	19

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads", based upon WSD is presented in Table 3 above.

The evaluation of Load Demands on the piles, based upon WSD is presented in Tables 3A and 3B above. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Tables 4A and 4B below. The pile cut-off elevations are shown in Tables 2A and 2B.

TABLE 4A: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS (MEDIAN WIDEN)

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	Class 200	10.20	1,569	1,177	150	300	-45.0 (a), -9.0 (b)	-45.7
Abut 2	Class 200	10.40	1,569	1,177	150	300	-45.5 (a), -6.5 (b)	-45.5

Notes: (i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

TABLE 4B: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS (OUTSIDE WIDEN)

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	Class 200 Alt X (Mod)	9.00	599	418	150	300	-44.0 (a), -11.0 (d)	-44.0
Abut 2	Class 200 Alt X (Mod)	9.20	599	418	150	300	-32.0 (a), -10.0 (d)	-43.8
Abut 1 WW	Class 200 Alt X (Mod)	16.20	38.4	19.2	20	40	-7.5 (a), -12.5 (b), -7.5 (c), -8.0 (d)	-12.5
Abut 2 WW	Class 200 Alt X (Mod)	16.42	38.4	19.2	20	40	-4.5 (a), -9.5 (b), -4.5(c), -6.0 (d)	-12.3

Notes: (i) Design tip elevations are controlled by (a) Compression (Service Limit), (b) Compression (Extreme Limit), (c) Tension (Extreme Limit), (d) Lateral Load.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

TABLE 5A: PILE DATA TABLE (MEDIAN WIDEN)

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 200 Alt X (Mod)	260	0	-45.0 (a), -9.0 (c)	-45.7	260
Abut 2	Class 200 Alt X (Mod)	260	0	-45.5 (a), -6.5 (c)	-45.5	260

Notes

(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression), (b) Tension, (c) Lateral Load.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iii) Based on Caltrans review comment, it was recommended to keep the same pile length (specified tip elevation) for "ease of pile fabrication".

TABLE 5B: PILE DATA TABLE (OUTSIDE WIDEN)

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.[1] (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 200 Alt X (Mod)	300	0	-44.0 (a), -11.0 (d)	-44.0	300
Abut 2	Class 200 Alt X (Mod)	300	0	-32.0 (a), -10.0 (d)	-43.8	300
Abut 1 WW	Class 200 Alt X (Mod)	40	20	-7.5 (a), -12.5 (b), -7.5 (c), -8.0 (d)	-12.5	40
Abut 2 WW	Class 200 Alt X (Mod)	40	20	-4.5 (a), -9.5 (b), -4.5(c), -6.0 (d)	-12.3	40

Notes

(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression (Service Limit), (b) Compression (Extreme Limit), (c) Tension (Extreme Limit), (d) Lateral Load.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iii) Based on Caltrans review comment, it was recommended to keep the same pile length (specified tip elevation) for "ease of pile fabrication".

The estimation of the driven pile capacity in clay is based on the adhesion factor per FHWA manual (Tomlinson 1980). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Tables 5A and 5B, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap

(measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles should not be spaced closer than 3 times the pile diameter measured center-to-center. For piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter, there is no group effect for vertical pile capacity.

It is anticipated that the pile capacity will develop after driving as a result of soil "freeze" and dissipation of excess pore water pressures. The final pile capacity can be verified and the gain of pile capacity after initial driving may be evaluated based on "re-driving" after 24-hour (min.) set up. Caltrans Standard Specifications (Section 49-1.08) may be used as the pile driving acceptance criteria. As an option, Pile Driving Analyzer (PDA) may be used to evaluate the pile capacity of the driven piles when unanticipated conditions arise. Typical applications of the PDA include capacity evaluation (for both during driving and re-driving) and integrity testing for piles that have experienced hard driving.

It is prudent to make the contractor aware of these conditions so that appropriate steps can be taken to comply with the standards and maintain the integrity of the piles.

Based on the available boring/CPT information, medium dense sands were encountered in the embankment fill. It is recommended to follow Caltrans standard specifications (Section 49-1.06) (Ref. 8) for oversize predrill through the existing embankment.

Due to the hard driving condition that may be encountered during pile driving, pile driving should be allowed to terminate short of the specified tip elevation provided the following conditions are satisfied:

- Other requirements including tension and lateral demands are met;
- Pile attaining its capacity and refusal within 5 feet above the specified tip elevation.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. Lateral load analyses for the piles were performed using LPILE Program (v. 5). "Generalized Vertical Soil Profile and Strength Parameters" based on A-

09-SR-001, R-09-SR-001 and CPT-09-SR-001 are included in Appendix C. To account for group effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D, respectively.

Generally, abutment piles are designed for service loads only, and batter piles are used for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for

the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for new structures. Settlement is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Borings A-09-SR-001 and R-09-SR-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
A-09-SR-001	24.5	540	7.2	114.0	88.2
R-09-SR-001	21.0	880	7.4	55.5	38.8

Based on the corrosion test results, the native subsurface soils in Borings A-09-SR-001 and R-09-SR-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.8 Construction Observation

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

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Swain Road UC (Widen) Foundation Report (6-2010).rpt

REFERENCES

1. Swain Road UC As-built Log of Test Borings (September 1964).

2. Swain Road UC Widen As-built Log of Test Borings (March 1997).

3. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

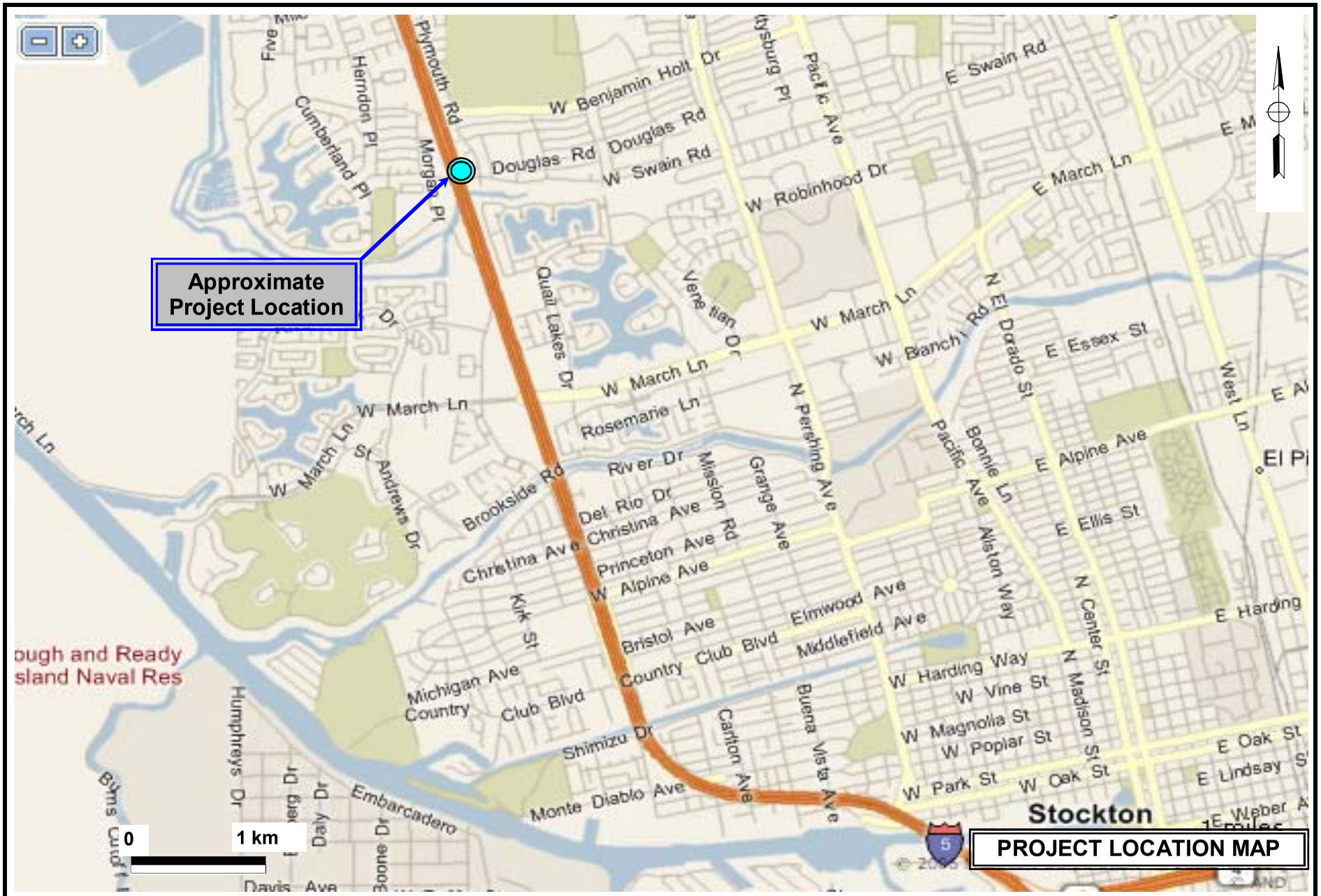
4. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

5. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

6. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

7. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

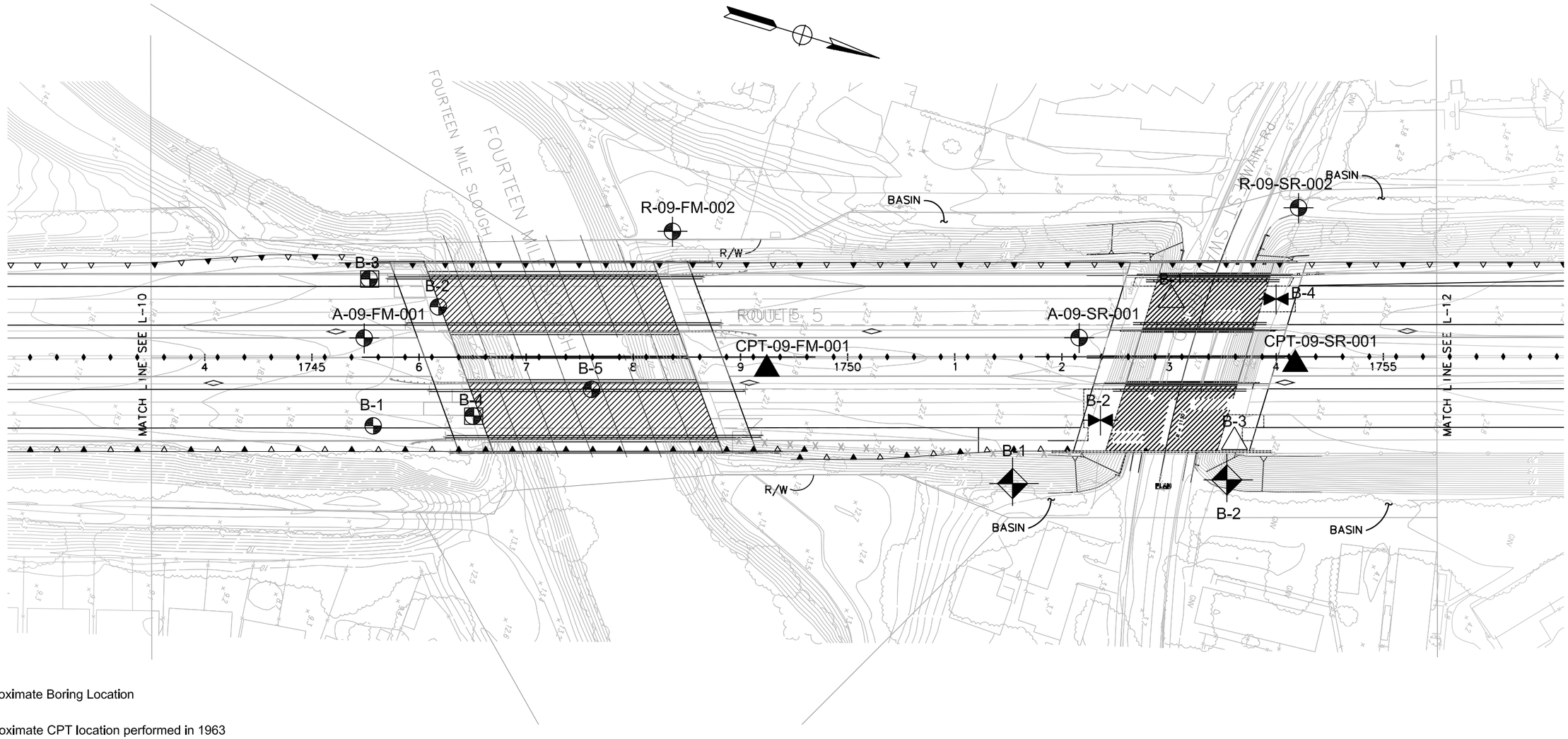
8. State of California Department of Transportation Standard Specifications May 2006.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

SWAIN ROAD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0215)	PLATE NO.: 1



LEGEND

A-09-SR-001
Approximate Boring Location

B-4
Approximate CPT location performed in 1963

B-3
Approximate Boring location drilled in 1963

B-2
Approximate Boring location drilled in 1964

B-1
Approximate CPT location performed in 1964

B-1
Approximate Boring location drilled in 1997

CPT-09-SR-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

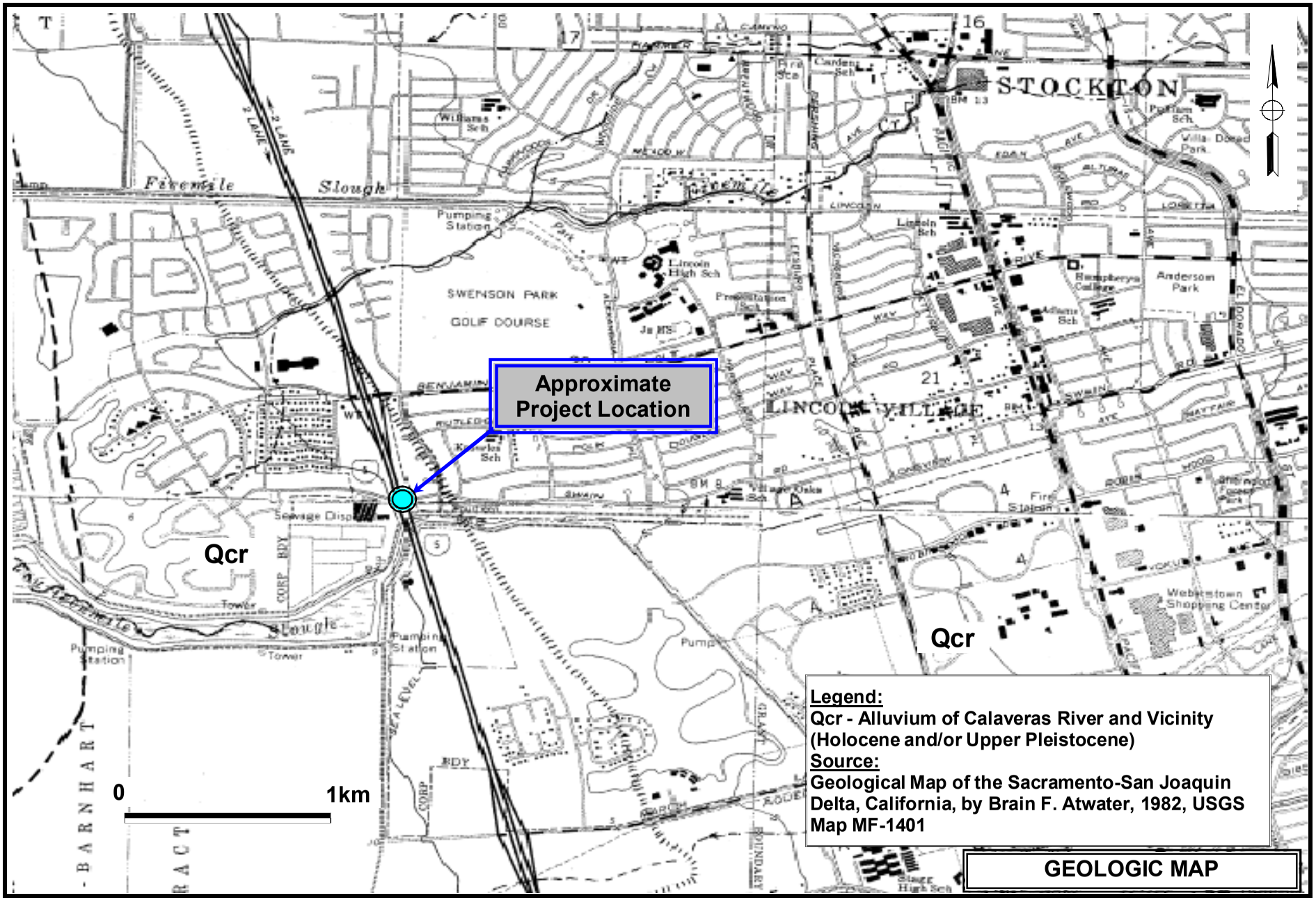
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

SWAIN ROAD UC (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

PLATE NO: 2

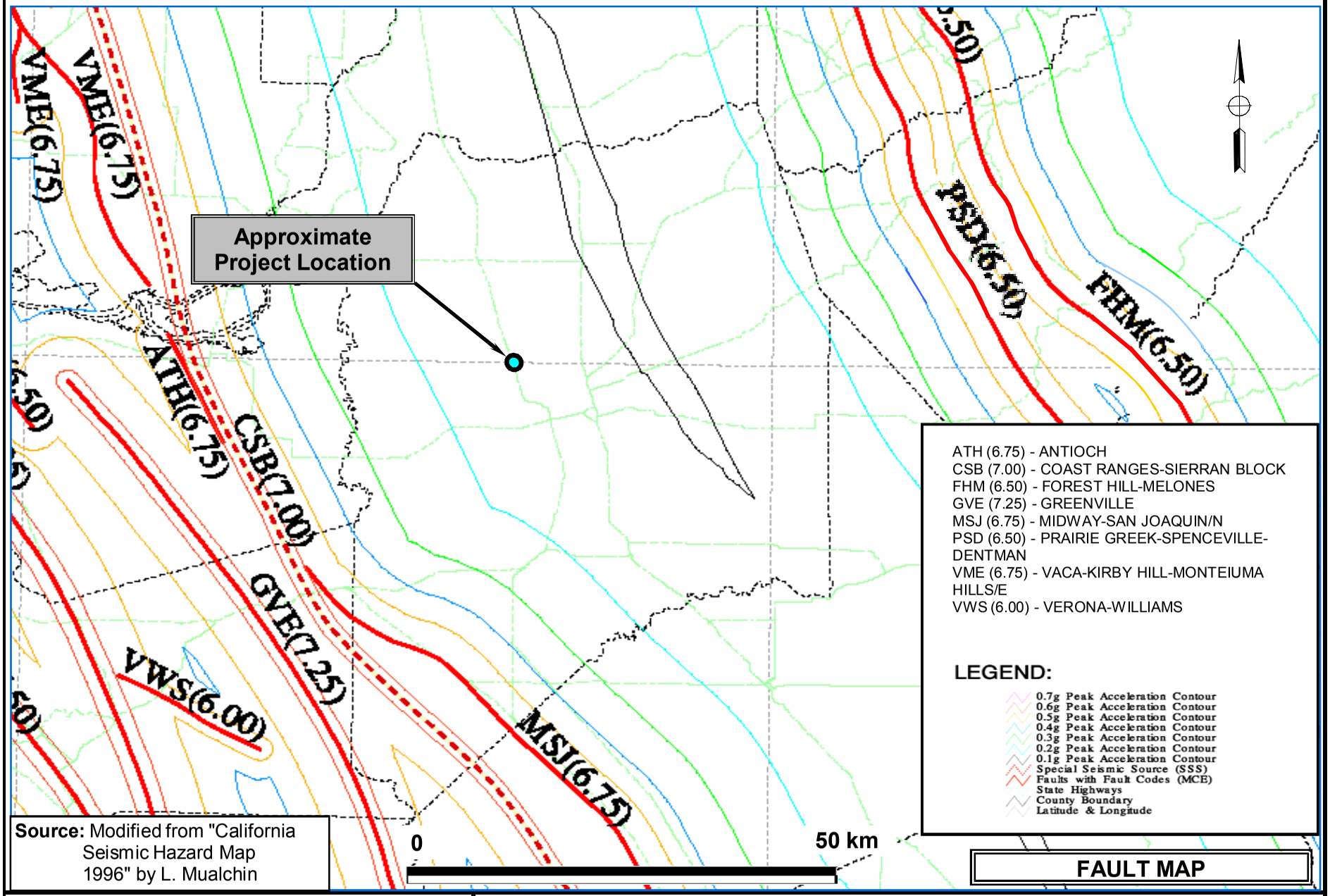


GEOLOGIC MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

SWAIN ROAD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0215) | PLATE NO.: 3



Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

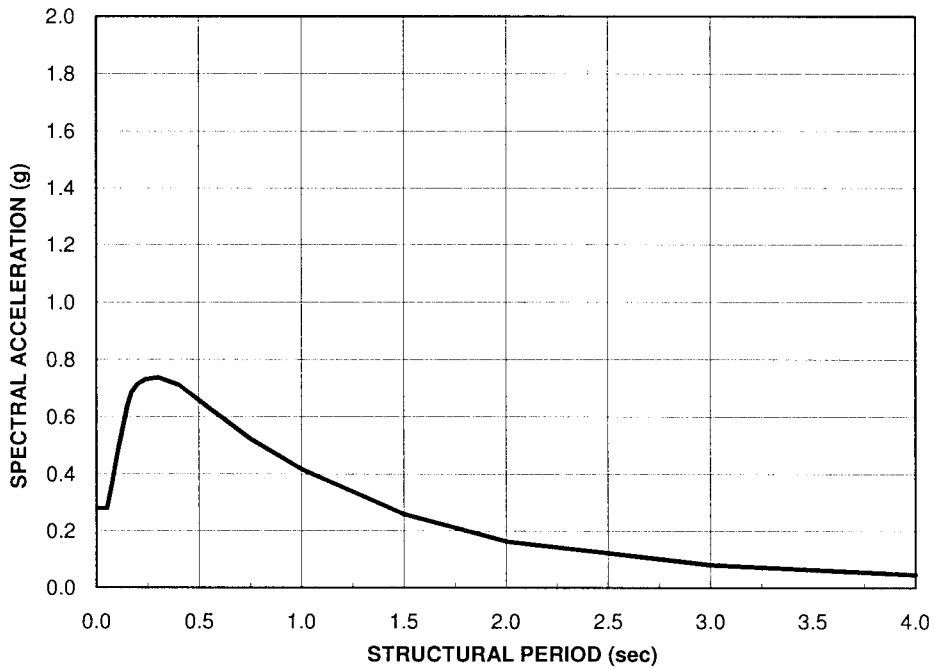
FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

SWAIN ROAD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0215) | **PLATE NO.: 4**

ARS DESIGN CURVE
SWAIN ROAD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

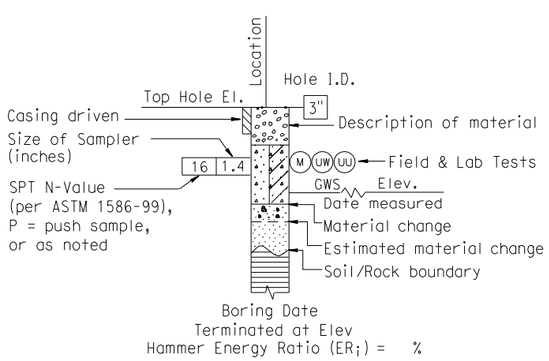
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

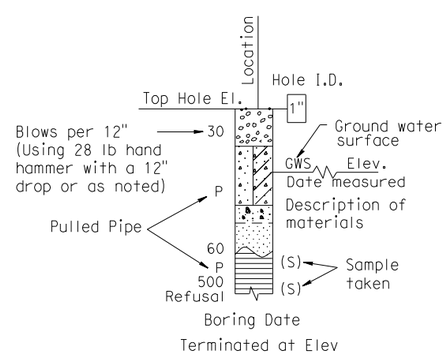
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

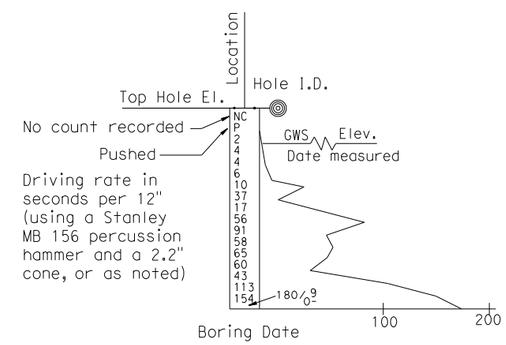
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



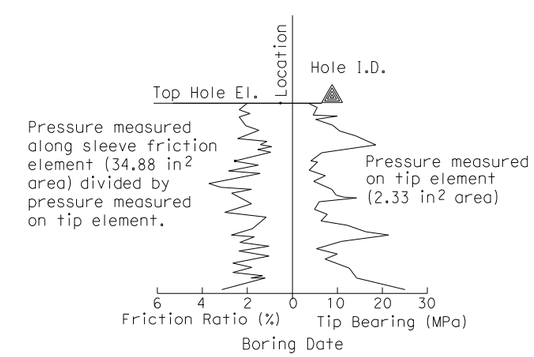
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY: O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER	BRIDGE NO. 29-0215 POST MILES 31.11
SIGN OFF DATE	CHECKED BY: D. WANG				

SWAIN ROAD UC (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 23 OF 27

...\29-0215-z-ltb01.dgn 6/8/2010 9:54:49 AM

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 5333)
CP	Compaction Curve (CTM 216)
CR	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content-% (ASTM D 2974)
P	Permeability (CTM 220)
PA	Particle Size Analysis (ASTM D 422)
PI	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (CTM 217)
SG	Specific Gravity (AASHTO T 100)
SL	Shrinkage Limit (ASTM D 427)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 4767)
VS	Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0215
POST MILES 31.11

SWAIN ROAD UC (WIDEN)
SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 24 OF 27

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL — 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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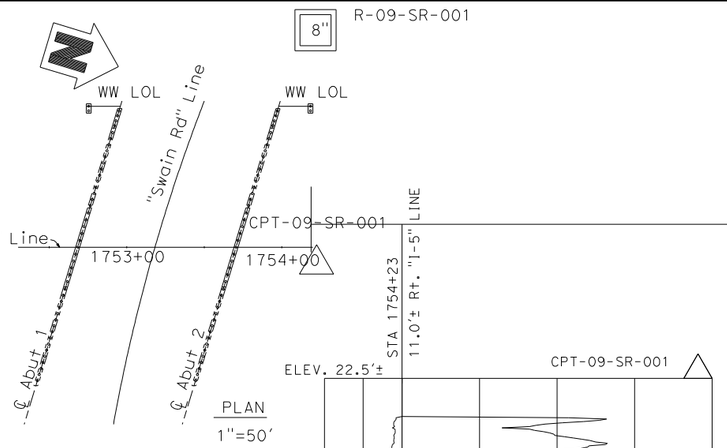
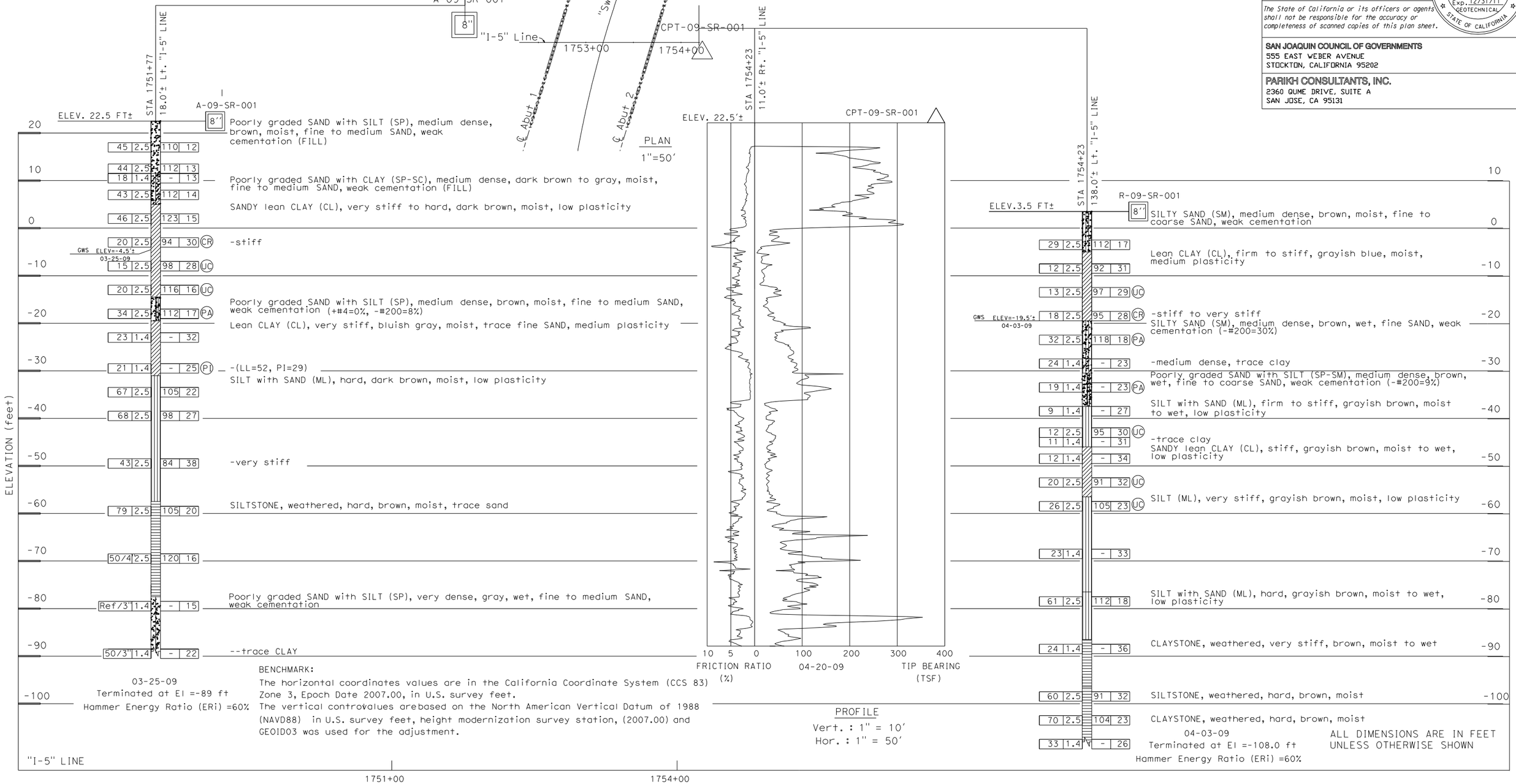
SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131





BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.
The vertical controvalues arebased on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER

BRIDGE NO. 29-0215 — POST MILES 31.11

SWAIN ROAD UC (WIDEN)
LOG OF TEST BORINGS 1 OF 3

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES — REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 25 OF 27

...\29-0215-z-ltb03.dgn 6/8/2010 9:50:58 AM

BENCHMARK

CALTRANS 57.2 mm BRASS DISK "1746", EL 5.431 m, STATION "I-5" 532+20.25, NORTHING 666617.712, EASTING 1925473.269 AND CALTRANS 57.2 mm BRASS DISK "1755", EL 6.352 m, STATION "I-5" 534+92.71, NORTHING 666878.451, EASTING 1925394.210. ELEVATIONS ARE BASED ON THE CITY OF STOCKTON BENCHMARK LIST DATED OCTOBER, 1992, WHICH REFERS TO NGVD SEA LEVEL DATUM OF 1929.

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

As such, the State of California registration seal with signature, license number

DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5			

REGISTERED CIVIL ENGINEER _____ DATE _____

SWAIN ROAD UC (WIDEN)

LOG OF TEST BORINGS 2 OF 3

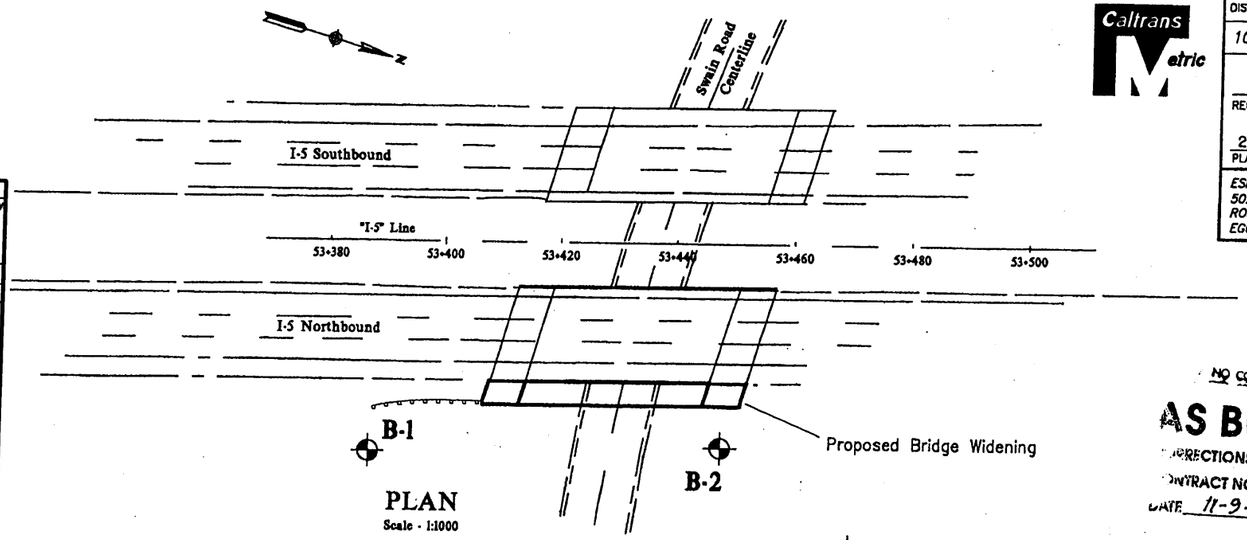
NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0G4701

BRIDGE No. 29-0215

Sheet 26 of 27

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA



PLAN
Scale - 1:1000

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
10	SJ	5	49.9/52.9	298	299

REGISTERED PROFESSIONAL ENGINEER

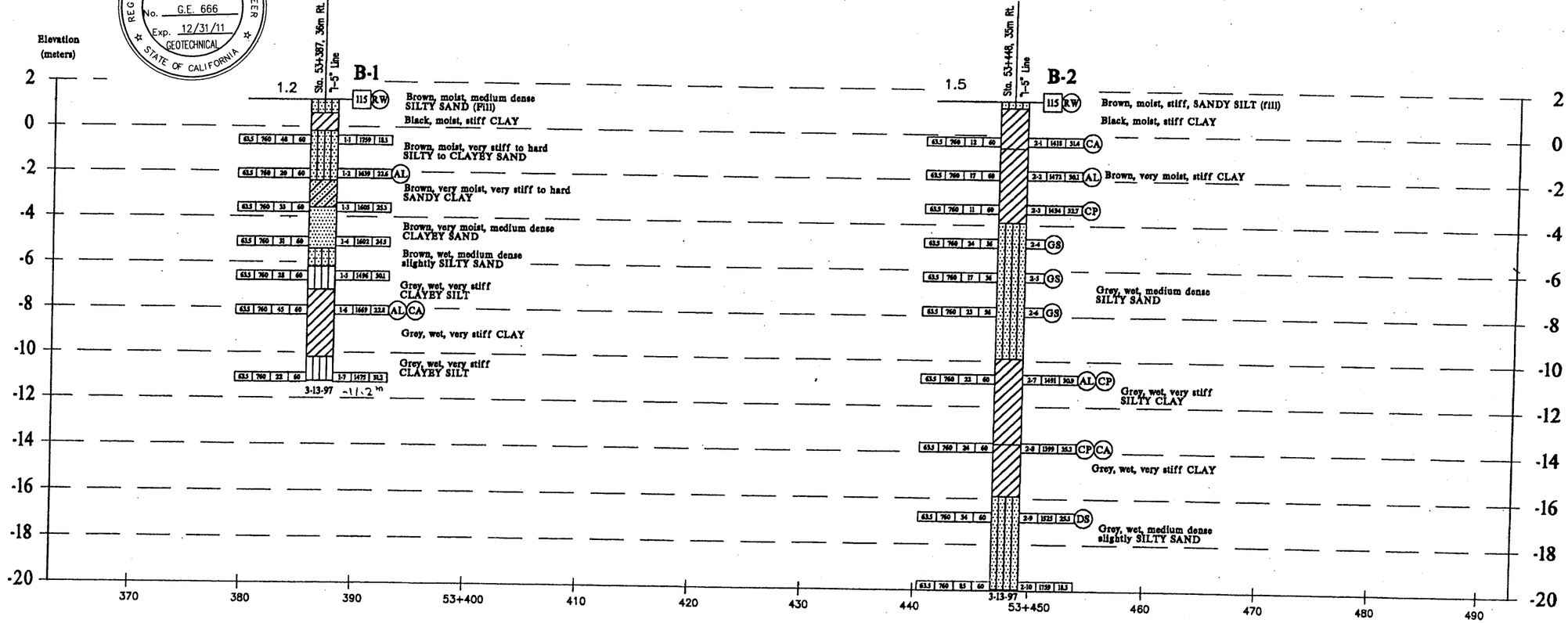
2-1-99
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER CARLOS ESPAÑA No. 298 Exp. 9/30/01 GEOTECHNICAL STATE OF CALIFORNIA

ESPAÑA GEOTECHNICAL CONSULTING
502 GIUSEPPE CT, SUITE 11
ROSEVILLE, CA 95678
EGC PROJECT NO. F187-5

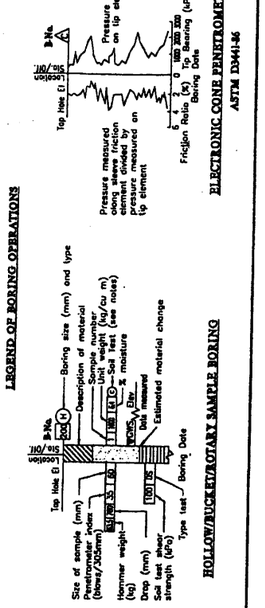
NO CORRECTIONS THIS SHEET

AS BUILT
CORRECTIONS BY D.R. SMALL RHT
CONTRACT NO. 10-44B3U4
DATE 11-9-00 7-18-01



PROFILE
Stationing: "I-5" Line, meters
1:200 (Horizontal)
1:100 (Vertical)

LEGEND OF BORING OPERATIONS



NOTES:

1. TYPES OF BORINGS
   - Rotary Wash
   - Cone Penetrometer
   - Hollow Stem Auger
   - Test Pit
2. SOIL SAMPLER
   - 50mm Sampler, 76mm OD by 60mm ID, split barrel sampler, 63.5 kg hammer, rope and cathead
3. SOIL TEST DESIGNATIONS
   - Atterberg limits
   - Chemical analysis
   - Max. dry density
   - R-value
   - Direct shear
   - Consolidation
   - Sieve analysis
   - Pocket penetrometer
   - Unconfined compression
   - Unconsolidated undrained triaxial

LEGEND OF EARTH MATERIALS

- CL, Lean Clay
- ML, Silt
- OL, Organic Clay or Silt
- CH, Fat Clay
- MH, Elastic Silt
- OH, Organic Clay or Silt
- Organic Material, Sawdust, etc.
- GP, Poorly Graded Gravel
- GM, Silty Gravel
- GC, Clayey Gravel
- SW, Well-Graded Sand
- SP, Poorly-Graded Sand
- SM, Silty Sand
- SC, Clayey Sand
- Sandstone
- Granitic Rock
- Claystone
- Siltstone

all soil classification according to Unified Soil Classification System and ASTM D2488

RELATIVE DENSITY AND CONSISTENCY CLASSIFICATION

Sand and Gravels — ASTM D1586 76mm OD x 60mm ID

Consistency	Blows/305mm (SPT)
Very loose	0 – 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very dense	Over 50

Clays and Silts

Consistency	Blows/305mm
Very soft	0 – 2
Soft	2 – 5
Firm	5 – 9
Stiff	9 – 18
Very stiff	18 – 30
Hard	Over 30

DESIGN OVERSIGHT 10-7-98 SIGN OFF DATE	DRAWN BY D. KOELZER	S. HARVEY FIELD INVESTIGATOR	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	CARLOS ESPAÑA PROJECT ENGINEER
	CHECKED BY C. ESPANA	DATE 3/97		

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 20 40 60 80

CU 10115
EA 4483U1

BRIDGE NO. 29-215R/L
POST KILOMETER 47.9

SWAIN ROAD UNDERCROSSING (WIDEN)
LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 5/8/97 11/3/97 5/4/98 5/5/98 8/12/98

SHEET 15 OF 16

ClibPDF - www.fastio.com

DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5			

DIVISION OF ENGINEERING SERVICES – GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

REGISTERED CIVIL ENGINEER — DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

SWAIN ROAD UC (WIDEN)

LOG OF TEST BORINGS 3 OF 3

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0G4701

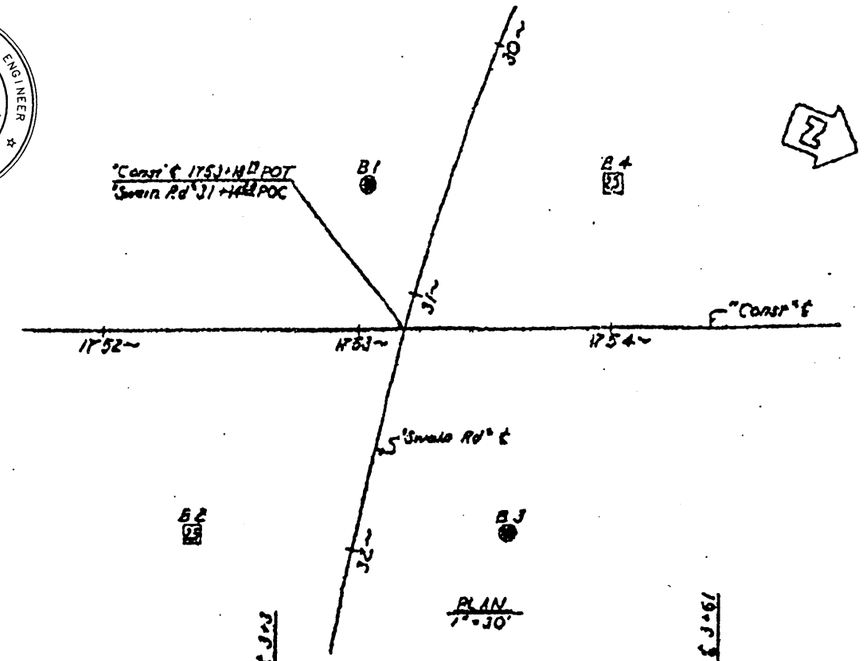
BRIDGE No. 29-0215

Sheet 27 of 27

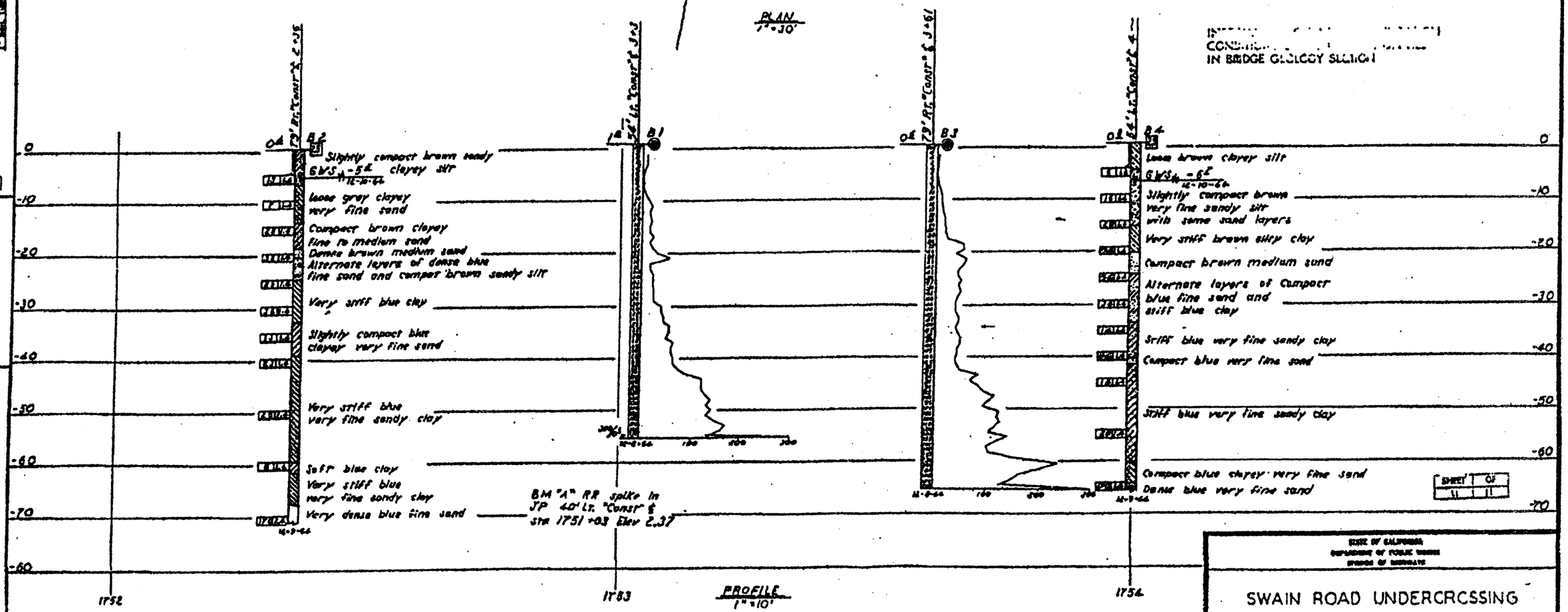
District	County	Route	Section	Sheet	Total Sheets
10	SJ	5		364	372

September 5, 1967

AS BUILT PLANS
Contract No. 10-0C924
Date Completed
Document No. 00000629



PLAN
1" = 30'



PROFILE
1" = 10'

SHEET 1 OF 11

IN BRIDGE GEOLOGY SECTION

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

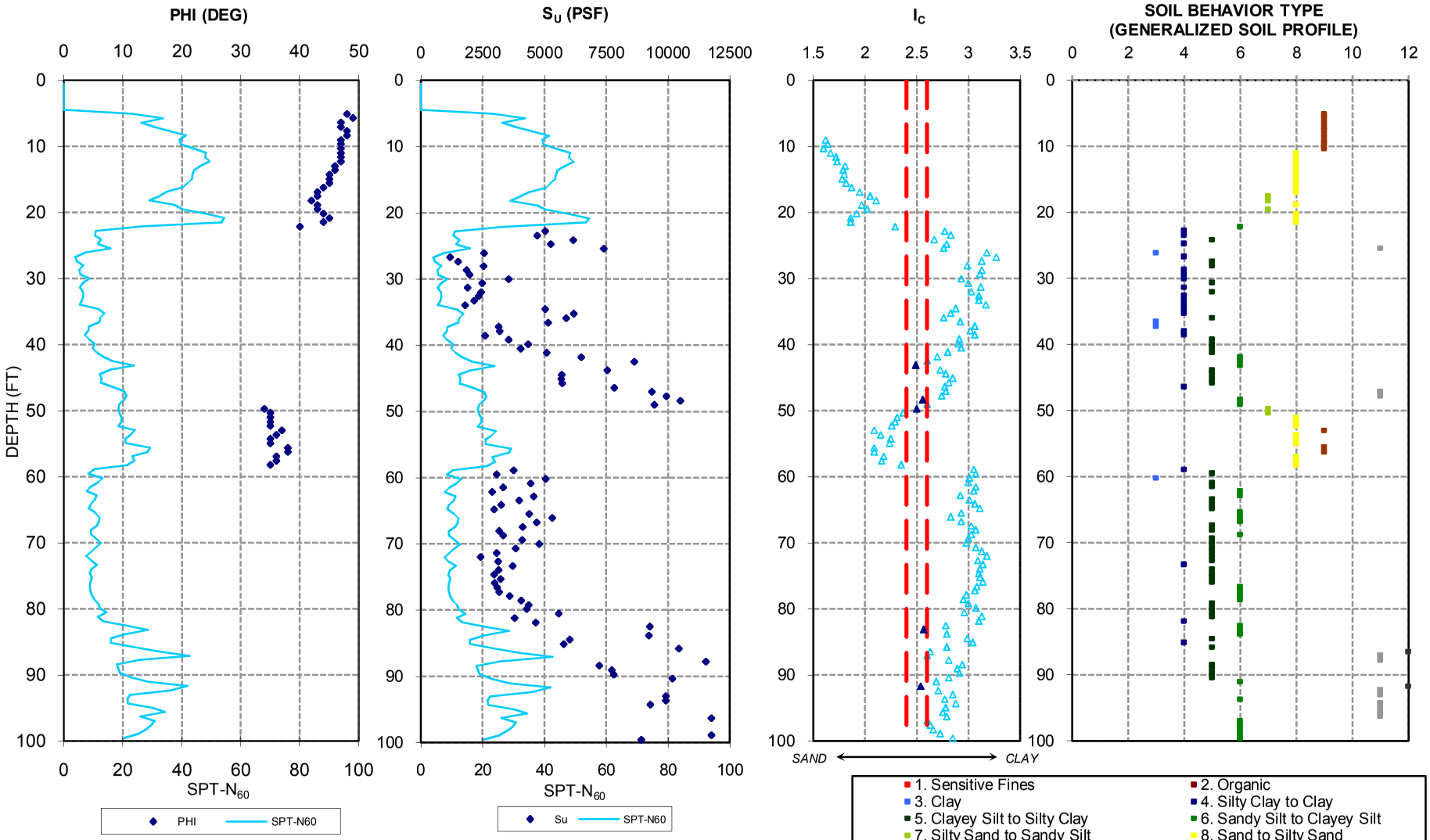
SWAIN ROAD UNDERCROSSING

LOG OF TEST BORINGS

SCALE As shown | BRIDGE 29-0215 | FILE | DRAWING 29215-7

CPT DATA CORRELATION

CPT-09-SR-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

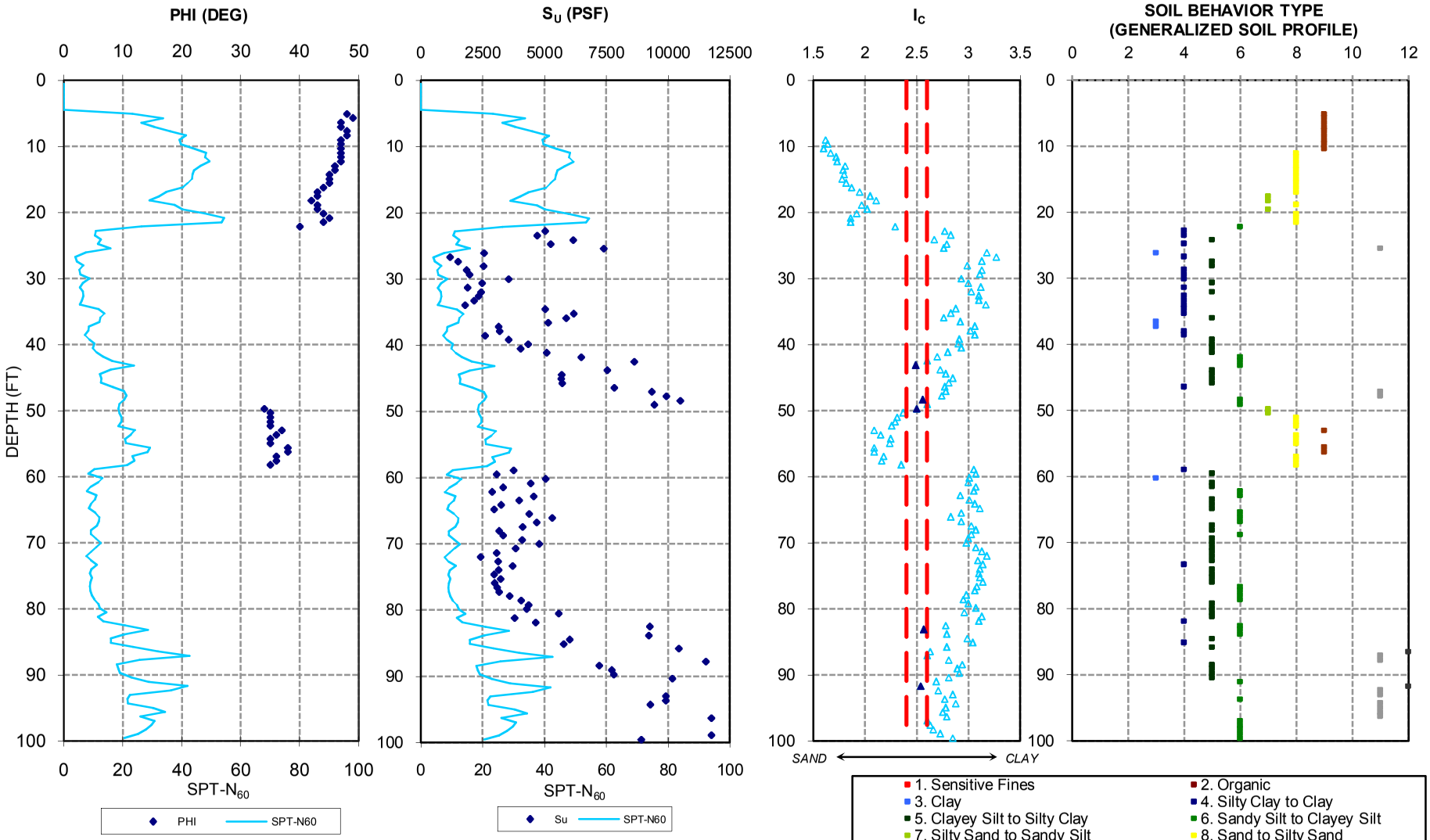
SWAIN ROAD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0215)

PLATE NO.:

CPT DATA CORRELATION

CPT-09-SR-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

SWAIN ROAD UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0215)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT- SWAIN ROAD UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0215)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-SR-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
5.085	9	Sand	146.683	1.17	0.316		0.316	1.38	1.45119	0.99	0.129	1.380	1.45	212.86	1.000	212.865		1.00	1	
5.741	9	Sand	210.506	2.102	0.356		0.356	1.41	1.41388	0.99	0.129	1.410	1.41	297.63	1.000	297.631		1.00	1	
6.398	9	Sand	160.267	1.596	0.397		0.397	1.49	1.37758	0.99	0.128	1.490	1.38	220.78	1.000	220.781		1.00	1	
7.054	9	Sand	187.238	1.756	0.438		0.438	1.46	1.3431	0.99	0.128	1.460	1.34	251.48	1.000	251.480		1.00	1	
7.71	9	Sand	220.279	2.276	0.479		0.479	1.48	1.3103	0.98	0.128	1.480	1.31	288.63	1.000	288.632		1.00	1	
8.366	9	Sand	252.092	2.82	0.519		0.519	1.49	1.27981	0.98	0.128	1.490	1.28	322.63	1.000	322.631		1.00	1	
9.022	9	Sand	228.485	3.283	0.56		0.56	1.62	1.25	0.98	0.128	1.620	1.25	285.61	1.000	285.606		1.00	1	
9.678	9	Sand	229.599	3.313	0.601		0.601	1.64	1.22154	0.98	0.127	1.640	1.22	280.47	1.000	280.465		1.00	1	
10.335	9	Sand	256.973	3.384	0.641		0.641	1.6	1.195	0.98	0.127	1.600	1.20	307.08	1.000	307.083		1.00	1	
10.991	8	Sand to Silty Sand	275.978	4.449	0.681		0.681	1.67	1.16959	0.98	0.127	1.670	1.17	322.78	1.017	328.390		1.00	1	
11.647	8	Sand to Silty Sand	270.059	4.722	0.721		0.721	1.72	1.14524	0.98	0.127	1.720	1.15	309.28	1.051	325.156		1.00	1	
12.303	8	Sand to Silty Sand	276.633	4.974	0.76		0.76	1.73	1.12245	0.97	0.127	1.730	1.12	310.51	1.058	328.534		1.00	1	
12.959	8	Sand to Silty Sand	253.321	5.085	0.8		0.8	1.81	1.1	0.97	0.126	1.810	1.10	278.65	1.114	310.422		1.00	1	
13.615	8	Sand to Silty Sand	242.926	4.338	0.84		0.84	1.79	1.07843	0.97	0.126	1.790	1.08	261.98	1.099	288.031		1.00	1	
14.272	8	Sand to Silty Sand	239.339	4.225	0.88		0.88	1.8	1.05769	0.97	0.126	1.800	1.06	253.15	1.107	280.147		1.00	1	
14.928	8	Sand to Silty Sand	239.775	3.856	0.919		0.919	1.78	1.03823	0.97	0.126	1.780	1.04	248.94	1.092	271.929		1.00	1	
15.584	8	Sand to Silty Sand	228.092	3.835	0.959		0.959	1.82	1.01899	0.97	0.126	1.820	1.02	232.42	1.122	260.666		1.00	1	
16.24	8	Sand to Silty Sand	216.479	3.932	0.999		0.999	1.87	1.00045	0.97	0.126	1.870	1.00	216.58	1.162	251.628		1.00	1	
16.896	8	Sand to Silty Sand	180.118	3.538	1.038		1.038	1.95	0.98302	0.96	0.125	1.950	0.98	177.06	1.240	219.530		0.99	1	
17.552	7	Silty Sand to Sandy Silt	160.595	3.654	1.077		1.077	2.05	0.96618	0.96	0.125	2.050	0.97	155.16	1.371	212.738		0.98	1	
18.209	7	Silty Sand to Sandy Silt	141.12	3.436	1.116		1.116	2.11	0.94991	0.96	0.125	2.110	0.95	134.05	1.473	197.480		0.97	1	
18.865	8	Sand to Silty Sand	192.664	3.858	1.155		1.155	1.97	0.93418	0.96	0.125	1.970	0.93	179.98	1.263	227.269		0.95	1	
19.521	7	Silty Sand to Sandy Silt	202.792	4.736	1.194		1.194	2.02	0.91896	0.96	0.125	2.020	0.92	186.36	1.327	247.300		0.94	1	
20.177	8	Sand to Silty Sand	250.962	5.022	1.234		1.234	1.92	0.90386	0.96	0.124	1.920	0.90	226.84	1.208	274.079		0.92	1	
20.833	8	Sand to Silty Sand	291.188	5.417	1.273		1.273	1.86	0.88961	0.95	0.124	1.860	0.89	259.04	1.153	298.766		0.91	1	
21.49	8	Sand to Silty Sand	286.417	5.184	1.313		1.313	1.86	0.87545	0.95	0.124	1.860	0.88	250.74	1.153	289.193		0.90	1	
22.146	6	Sandy Silt to Clayey Silt	118.998	3.555	1.351		1.351	2.29	0.86241	0.95	0.124	2.290	0.86	102.62	1.917	196.737		0.92	1	
22.802	4	Silty Clay to Clay	38.943	1.7	1.388		1.388	2.77	0.85008	0.95	0.123							0.94	1	
23.458	4	Silty Clay to Clay	36.508	1.757	1.426		1.426	2.83	0.83778	0.95	0.123							0.93	1	
24.114	5	Clayey Silt to Silty Clay	47.373	1.754	1.463		1.463	2.67	0.82614	0.94	0.123							0.93	1	
24.77	4	Silty Clay to Clay	40.81	1.848	1.501		1.501	2.79	0.81451	0.94	0.123							0.92	1	
25.427	11	Sitff Fine-Grained	56.922	3.302	1.544	0.013	1.531	2.76	0.80557	0.94	0.123							0.92	1	
26.083	3	Clay	20.708	1.495	1.58	0.034	1.547	3.18	0.80087	0.94	0.125							0.92	1	
26.739	4	Silty Clay to Clay	10.362	0.348	1.618	0.054	1.564	3.27	0.79595	0.94	0.126							0.91	1	
27.395	5	Clayey Silt to Silty Clay	12.89	0.358	1.655	0.075	1.581	3.13	0.79108	0.93	0.127							0.91	1	
28.051	5	Clayey Silt to Silty Clay	20.621	0.708	1.693	0.095	1.598	2.99	0.78628	0.93	0.128							0.91	1	
28.707	4	Silty Clay to Clay	15.385	0.553	1.731	0.115	1.615	3.13	0.78153	0.93	0.129							0.91	1	
29.364	4	Silty Clay to Clay	16.351	0.608	1.768	0.136	1.632	3.11	0.77684	0.92	0.130							0.91	1	
30.02	4	Silty Clay to Clay	28.183	1.168	1.806	0.156	1.649	2.93	0.7722	0.92	0.131							0.90	1	
30.676	5	Clayey Silt to Silty Clay	20.244	0.659	1.843	0.177	1.667	3	0.76735	0.92	0.132							0.90	1	
31.332	4	Silty Clay to Clay	15.882	0.541	1.881	0.197	1.684	3.12	0.76283	0.91	0.133							0.90	1	
31.988	5	Clayey Silt to Silty Clay	19.949	0.692	1.919	0.218	1.701	3.03	0.75836	0.91	0.133							0.90	1	
32.644	4	Silty Clay to Clay	19.322	0.812	1.956	0.238	1.718	3.1	0.75394	0.91	0.134							0.90	1	
33.301	4	Silty Clay to Clay	18.093	0.671	1.994	0.259	1.735	3.1	0.74957	0.90	0.135							0.90	1	
33.957	4	Silty Clay to Clay	15.238	0.55	2.031	0.279	1.752	3.17	0.74526	0.90	0.135							0.89	1	
34.613	4	Silty Clay to Clay	39.255	1.903	2.069	0.3	1.769	2.88	0.74099	0.89	0.136							0.89	1	
35.269	4	Silty Clay to Clay	47.837	2.424	2.107	0.32	1.786	2.83	0.73677	0.89	0.136							0.89	1	
35.925	5	Clayey Silt to Silty Clay	44.452	1.737	2.144	0.341	1.803	2.76	0.7326	0.88	0.137							0.89	1	
36.581	3	Clay	39.812	2.256	2.181	0.361	1.82	2.92	0.72848	0.88	0.137							0.89	1	
37.238	3	Clay	25.556	1.231	2.217	0.382	1.836	3.06	0.72464	0.87	0.137							0.89	1	
37.894	4	Silty Clay to Clay	25.891	1.114	2.255	0.402	1.853	3.02	0.7206	0.87	0.137							0.88	1	
38.55	4	Silty Clay to Clay	21.576	0.799	2.292	0.423	1.87	3.06	0.71661	0.86	0.138							0.88	1	
39.206	5	Clayey Silt to Silty Clay	28.417	0.904	2.33	0.443	1.887	2.91	0.71267	0.86	0.138							0.88	1	
39.862	5	Clayey Silt to Silty Clay	34.308	1.397	2.368	0.464	1.904	2.9	0.70876	0.85	0.138							0.88	1	
40.518	5	Clayey Silt to Silty Clay	31.906	1.273	2.405	0.484	1.921	2.93	0.7049	0.85	0.138							0.88	1	
41.175	5	Clayey Silt to Silty Clay	39.588	1.33	2.443	0.504	1.938	2.8	0.70108	0.84	0.138							0.88	1	
41.831	6	Sandy Silt to Clayey Silt	49.027	1.578	2.48	0.525	1.955	2.7	0.69731	0.83	0.138							0.87	1	
42.487	6	Sandy Silt to Clayey Silt	64.674	2.026	2.518	0.545	1.973	2.6	0.69335	0.83	0.137							0.87	1	
43.143	6	Sandy Silt to Clayey Silt	97.869	3.162	2.556	0.566	1.99	2.49	0.68966	0.82	0.137	2.490	0.69	67.50	2.718	183.480		0.87	1	
43.799	5	Clayey Silt to Silty Clay	58.232	2.336	2.593	0.586	2.007	2.73	0.686	0.82	0.137							0.87	1	
44.455	5	Clayey Silt to Silty Clay	43.639	1.529	2.631	0.607	2.024	2.78	0.68238	0.81	0.137							0.87	1	
45.112	5	Clayey Silt to Silty Clay	43.906	1.875	2.668	0.627	2.041	2.85	0.6788	0.80	0.136							0.87	1	
45.768	5	Clayey Silt to Silty Clay	44.141	1.685	2.706	0.648	2.058	2.81	0.67526	0.80	0.136							0.87	1	
46.424	4	Silty Clay to Clay	59.258	2.779	2.744	0.668	2.075	2.77	0.67176	0.79	0.136							0.86	1	
47.08	11	Sitff Fine-Grained	72.138	4.079	2.786	0.689	2.098	2.78	0.66707	0.78	0.135							0.86	1	
47.736	11	Sitff Fine-Grained	77.374	3.974	2.829	0.709	2.12	2.74	0.66265	0.78	0.135							0.86	1	
48.392	6	Sandy Silt to Clayey Silt	81.632	2.427	2.867	0.73	2.137	2.56	0.65927	0.77	0.134	2.560	0.66	53.82	3.091	166.327		0.86	1	
49.049	6	Sandy Silt to Clayey Silt	73.667	2.184	2.904	0.75	2.154	2.6	0.65593	0.76	0.134							0.86	1	
49.705	7	Silty Sand to Sandy Silt	76.063	1.641	2.943	0.771	2.172	2.5	0.65243	0.76	0.133	2.500	0.65	49.63	2.768	137.386	0.32	0.86	1	2.88
50.361	7	Silty Sand to Sandy Silt	81.954	1.17	2.982	0.791	2.191	2.37	0.64878	0.75	0.133	2.370	0.65	53.17	2.194	116.657	0.23	0.85	1	2.05
51.017	8	Sand to Silty Sand	87.55	1.108	3.021	0.812	2.21	2.31	0.64516	0.74	0.132	2.310	0.65	56.48	1.981	111.908	0.21	0.85	1	1.90
51.673	8	Sand to Silty Sand	87.774	1.013	3.061	0.832	2.229	2.29	0.64159	0.74	0.131	2.290	0.64	56.31	1.917	107.958	0.20	0.85	1	1.79
52.329	8	Sand to Silty Sand	84.58	0.803	3.101	0.852	2.248	2.26	0.63805	0.73	0.131	2.260	0.64	53.97	1.827	98.585	0.17	0.85	1	1.54
52.986	9	Sand	118.949	0.91	3.141	0.873	2.268	2.09	0.63437	0.72	0.130	2.090	0.63	75.46	1.437	108.427	0.20	0.84	1	1.82
53.642	8	Sand to Silty Sand	109.493	0.94	3.181	0.893	2.288	2.15	0.63073	0.72	0.129	2.150	0.63	69.06	1.553	107.237	0.19	0.84	1	1.79
54.298	8	Sand to Silty Sand	95.549	1.002	3.221	0.914	2.307	2.25	0.62732	0.71	0.129	2.250	0.63	59.94	1.798	107.786	0.20	0.85	1	1.82
54.954	8	Sand to Silty Sand	96.755	0.987	3.26	0.934	2.326	2.24	0.62394	0.70	0.128	2.240	0.62	60.37	1.770	106.882	0.19	0.84	1	1.80
55.61	9	Sand	143.32	1.426	3.301	0.955	2.346	2.09	0.62042	0.70	0.128	2.090	0.62	88.92	1.437	127.769	0.27	0.81	1	2.56
56.266	9	Sand	139.515	1.298	3.342	0.975	2.367	2.09	0.61676	0.69	0.127	2.090	0.62	86.05	1.437	123.644	0.26	0.82	1	2.40
56.923	8	Sand to Silty Sand	109.836	0.996	3.382	0.996	2.386	2.18	0.6135	0.69	0.126	2.180	0.61	67.38	1.619	109.106	0.20	0.84	1	1.90
57.579	8	Sand to Silty Sand	114.734	1.026	3.421	1.016	2.405	2.16	0.61026	0.68	0.126	2.160	0.61	70.02	1.574	110.227	0.20	0.83	1	1.94
58.235	8	Sand to Silty Sand	93.873	1.332	3.461	1.037	2.424	2.35	0.60706	0.67	0.125	2.350	0.61	56.99	2.120	120.792	0.24	0.84	1	2.33
58.891	4	Silty Clay to Clay	31.382	1.283	3.499	1.057	2.441	3.05	0.60423	0.67	0.125							0.84	1	
59.547	5	Clayey Silt to Silty Clay	25.005	0.827	3.536	1.078	2.459	3.07	0.60126	0.66	0.124							0.84	1	
60.203	3	Clay	40.445	2.088	3.573	1.098	2.475	3.01	0.59864	0.66	0.123							0.83	1	
60.86	5	Clayey Silt to Silty Clay	36.508	1.499	3.61	1.119	2.492	3	0.59588	0.65	0.123							0.83	1	
61.516	5	Clayey Silt to Silty Clay	27.495	0.987	3.648	1.139	2.509	3.07	0.59315	0.65	0.122							0.83	1	
62.172	6	Sandy Silt to Clayey Silt	23.542	0.643	3.686	1.16	2.526	3.05	0.59045	0.64	0.122							0.83	1	
62.828	6	Sandy Silt to Clayey Silt	36.377	1.144	3.723	1.18	2.543	2.92	0.58776	0.64	0.121							0.83	1	
63.484	5	Clayey Silt to Silty Clay	33.031	1.186	3.761	1.201	2.56	3.01	0.58511	0.63	0.121							0.83	1	
64.14	5	Clayey Silt to Silty Clay	27.451	0.875	3.798	1.221	2.577	3.06	0.58247	0.63	0.120							0.83	1	
64.797	5	Clayey Silt to Silty Clay	24.858	0.84	3.836	1.241	2.594	3.11	0.57986	0.62	0.120							0.83	1	
65.453	6	Sandy Silt to Clayey Silt	34.925	1.032	3.873	1.262	2.612	2.93	0.57712	0.62	0.119							0.83	1	
66.109	6	Sandy Silt to Clayey Silt	42.137	1.091	3.911	1.282	2.629	2.83	0.57456	0.61	0.119							0.82	1	
66.765	6	Sandy Silt to Clayey Silt	38.676	1.212	3.949	1.303	2.646	2.93	0.57202	0.61	0.118							0.82	1	
67.421	5	Clayey Silt to Silty Clay	34.232	1.307	3.986	1.323	2.663	3.03	0.56951	0.61	0.118							0.82	1	
68.077	5	Clayey Silt to Silty Clay	27.189	0.816	4.024	1.344	2.68	3.07	0.56701	0.60	0.118							0.82	1	
68.734	6	Sandy Silt to Clayey Silt	28.051	0.752	4.061	1.364	2.697	3.03	0.56454	0.60	0.117							0.82	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

	I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT- SWAIN ROAD		
PROJECT NAME	***UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0215)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-SR-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
69.39	5	Clayey Silt to Silty Clay	33.784	1.117	4.099	1.385	2.714	3	0.56208	0.60	0.117							0.82	1	
70.046	5	Clayey Silt to Silty Clay	39.937	1.458	4.137	1.405	2.731	2.98	0.55965	0.59	0.117							0.82	1	
70.702	5	Clayey Silt to Silty Clay	32.921	1.213	4.174	1.426	2.749	3.07	0.5571	0.59	0.116							0.82	1	
71.358	5	Clayey Silt to Silty Clay	26.872	0.889	4.212	1.446	2.766	3.13	0.55472	0.58	0.116							0.82	1	
72.014	5	Clayey Silt to Silty Clay	21.494	0.611	4.249	1.467	2.783	3.18	0.55235	0.58	0.115							0.81	1	
72.671	5	Clayey Silt to Silty Clay	26.785	0.801	4.287	1.487	2.8	3.09	0.55	0.58	0.115							0.81	1	
73.327	4	Silty Clay to Clay	32.163	1.444	4.325	1.508	2.817	3.14	0.54767	0.58	0.115							0.81	1	
73.983	5	Clayey Silt to Silty Clay	27.424	0.863	4.362	1.528	2.834	3.11	0.54536	0.57	0.114							0.81	1	
74.639	5	Clayey Silt to Silty Clay	25.622	0.713	4.4	1.549	2.851	3.1	0.54308	0.57	0.114							0.81	1	
75.295	5	Clayey Silt to Silty Clay	27.735	0.901	4.437	1.569	2.868	3.11	0.54081	0.57	0.114							0.81	1	
75.951	5	Clayey Silt to Silty Clay	25.786	0.809	4.475	1.589	2.885	3.14	0.53856	0.56	0.114							0.81	1	
76.608	6	Sandy Silt to Clayey Silt	26.484	0.693	4.513	1.61	2.903	3.08	0.53619	0.56	0.113							0.81	1	
77.264	6	Sandy Silt to Clayey Silt	27.189	0.689	4.55	1.63	2.92	3.06	0.53398	0.56	0.113							0.81	1	
77.92	6	Sandy Silt to Clayey Silt	30.137	0.673	4.588	1.651	2.937	2.98	0.53179	0.56	0.113							0.81	1	
78.576	6	Sandy Silt to Clayey Silt	33.615	0.784	4.625	1.671	2.954	2.95	0.52961	0.55	0.113							0.81	1	
79.232	5	Clayey Silt to Silty Clay	36.64	1.15	4.663	1.692	2.971	3	0.52745	0.55	0.112							0.80	1	
79.888	5	Clayey Silt to Silty Clay	36.514	1.438	4.701	1.712	2.988	3.07	0.52531	0.55	0.112							0.80	1	
80.545	5	Clayey Silt to Silty Clay	46.227	1.709	4.738	1.733	3.005	2.96	0.52319	0.55	0.112							0.80	1	
81.201	5	Clayey Silt to Silty Clay	33.173	1.338	4.776	1.753	3.022	3.13	0.52108	0.54	0.112							0.80	1	
81.857	4	Silty Clay to Clay	39.358	1.873	4.813	1.774	3.04	3.1	0.51887	0.54	0.112							0.80	1	
82.513	6	Sandy Silt to Clayey Silt	73.983	2.697	4.851	1.794	3.057	2.78	0.5168	0.54	0.111							0.80	1	
83.169	6	Sandy Silt to Clayey Silt	113.522	3.453	4.889	1.815	3.074	2.57	0.51474	0.54	0.111	2.570	0.51	58.43	3.148	183.949		0.80	1	
83.825	6	Sandy Silt to Clayey Silt	72.771	2.692	4.926	1.835	3.091	2.79	0.5127	0.54	0.111							0.80	1	
84.482	5	Clayey Silt to Silty Clay	49.961	2.186	4.964	1.856	3.108	2.99	0.51068	0.53	0.111							0.80	1	
85.138	4	Silty Clay to Clay	48.263	2.387	5.001	1.876	3.125	3.04	0.50867	0.53	0.111							0.80	1	
85.794	5	Clayey Silt to Silty Clay	83.123	3.432	5.039	1.897	3.142	2.79	0.50668	0.53	0.111							0.80	1	
86.45	12	Sand to Clayey Sand	122.858	4.793	5.079	1.917	3.162	2.63	0.50436	0.53	0.110							0.79	1	
87.106	11	Sitff Fine-Grained	166.687	8.021	5.121	1.938	3.184	2.6	0.50182	0.53	0.110							0.79	1	
87.762	11	Sitff Fine-Grained	89.931	4.459	5.164	1.958	3.206	2.81	0.49932	0.52	0.110							0.79	1	
88.419	5	Clayey Silt to Silty Clay	58.254	2.6	5.202	1.978	3.223	2.94	0.4974	0.52	0.110							0.79	1	
89.075	5	Clayey Silt to Silty Clay	62.125	2.464	5.239	1.999	3.24	2.89	0.4955	0.52	0.110							0.79	1	
89.731	5	Clayey Silt to Silty Clay	62.687	2.74	5.277	2.019	3.258	2.91	0.49349	0.52	0.109							0.79	1	
90.387	5	Clayey Silt to Silty Clay	80.354	3.342	5.315	2.04	3.275	2.81	0.49162	0.52	0.109							0.79	1	
91.043	6	Sandy Silt to Clayey Silt	108.079	4.205	5.352	2.06	3.292	2.69	0.48976	0.52	0.109							0.79	1	
91.699	12	Sand to Clayey Sand	169.346	6.497	5.392	2.081	3.311	2.54	0.4877	0.52	0.109	2.540	0.49	82.59	2.979	246.032		0.79	1	
92.356	11	Sitff Fine-Grained	134.29	7.002	5.435	2.101	3.333	2.71	0.48533	0.51	0.109							0.79	1	
93.012	11	Sitff Fine-Grained	77.073	3.566	5.477	2.122	3.356	2.85	0.48288	0.51	0.109							0.78	1	
93.668	6	Sandy Silt to Clayey Silt	76.833	2.681	5.515	2.142	3.373	2.77	0.48108	0.51	0.109							0.78	1	
94.324	11	Sitff Fine-Grained	73.246	3.438	5.558	2.163	3.395	2.88	0.47878	0.51	0.108							0.78	1	
94.98	11	Sitff Fine-Grained	107.827	5.383	5.601	2.183	3.418	2.78	0.4764	0.51	0.108							0.78	1	
95.636	11	Sitff Fine-Grained	124.261	6.497	5.644	2.204	3.44	2.75	0.47414	0.51	0.108							0.78	1	
96.293	11	Sitff Fine-Grained	91.754	4.052	5.686	2.224	3.462	2.79	0.4719	0.51	0.108							0.78	1	
96.949	6	Sandy Silt to Clayey Silt	119.817	3.864	5.724	2.245	3.479	2.6	0.47019	0.50	0.108							0.78	1	
97.605	6	Sandy Silt to Clayey Silt	114.925	3.827	5.762	2.265	3.496	2.63	0.46848	0.50	0.108							0.78	1	
98.261	6	Sandy Silt to Clayey Silt	105.584	3.569	5.799	2.286	3.514	2.66	0.46669	0.50	0.108							0.78	1	
98.917	6	Sandy Silt to Clayey Silt	91.962	3.215	5.837	2.306	3.531	2.73	0.46502	0.50	0.108							0.78	1	
99.573	6	Sandy Silt to Clayey Silt	70.745	2.615	5.874	2.326	3.548	2.85	0.46335	0.50	0.107							0.78	1	
100.23	4	Silty Clay to Clay	60.645	2.823	5.912	2.347	3.565	2.98	0.4617	0.50	0.107							0.78	2	
100.886	5	Clayey Silt to Silty Clay	88.49	3.754	5.949	2.367	3.582	2.82	0.46006	0.50	0.107							0.77	3	
101.542	11	Sitff Fine-Grained	87.343	4.151	5.992	2.388	3.604	2.86	0.45795	0.50	0.107							0.77	4	
102.198	6	Sandy Silt to Clayey Silt	76.991	2.483	6.03	2.408	3.622	2.8	0.45624	0.49	0.107							0.77	5	
102.854	4	Silty Clay to Clay	38.25	1.67	6.067	2.429	3.639	3.16	0.45464	0.49	0.107							0.77	6	
103.511	7	Silty Sand to Sandy Silt	145.362	3.321	6.106	2.449	3.657	2.46	0.45295	0.49	0.107	2.460	0.45	65.84	2.574	169.496		0.77	7	
104.167	9	Sand	325.519	3.322	6.147	2.47	3.677	1.97	0.4511	0.49	0.107	1.970	0.45	146.84	1.263	185.419		0.77	8	
104.823	9	Sand	246.949	1.538	6.188	2.49	3.697	1.95	0.44925	0.49	0.107	1.950	0.45	110.94	1.240	137.554	0.32	0.77	9	3.60
105.479	9	Sand	158.809	1.031	6.228	2.511	3.718	2.13	0.44734	0.49	0.107	2.130	0.45	71.04	1.512	107.396	0.20	0.77	10	2.19
106.135	8	Sand to Silty Sand	130.108	1.376	6.268	2.531	3.737	2.32	0.44561	0.49	0.106	2.320	0.45	57.98	2.015	116.801	0.23	0.77	11	2.56
106.791	6	Sandy Silt to Clayey Silt	54.454	1.788	6.306	2.552	3.754	2.94	0.44409	0.49	0.106							0.77	12	
107.448	6	Sandy Silt to Clayey Silt	64.09	2.119	6.343	2.572	3.771	2.87	0.44257	0.49	0.106							0.77	13	
108.104	6	Sandy Silt to Clayey Silt	76.778	2.479	6.381	2.593	3.788	2.79	0.44106	0.49	0.106							0.77	14	
108.76	5	Clayey Silt to Silty Clay	77.723	3.365	6.418	2.613	3.805	2.89	0.43956	0.48	0.106							0.77	15	
109.416	4	Silty Clay to Clay	47.832	2.159	6.456	2.634	3.822	3.1	0.43807	0.48	0.106							0.76	16	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The results of the test are presented on Plate B-2, "Plasticity Chart".

Grain Size Classification

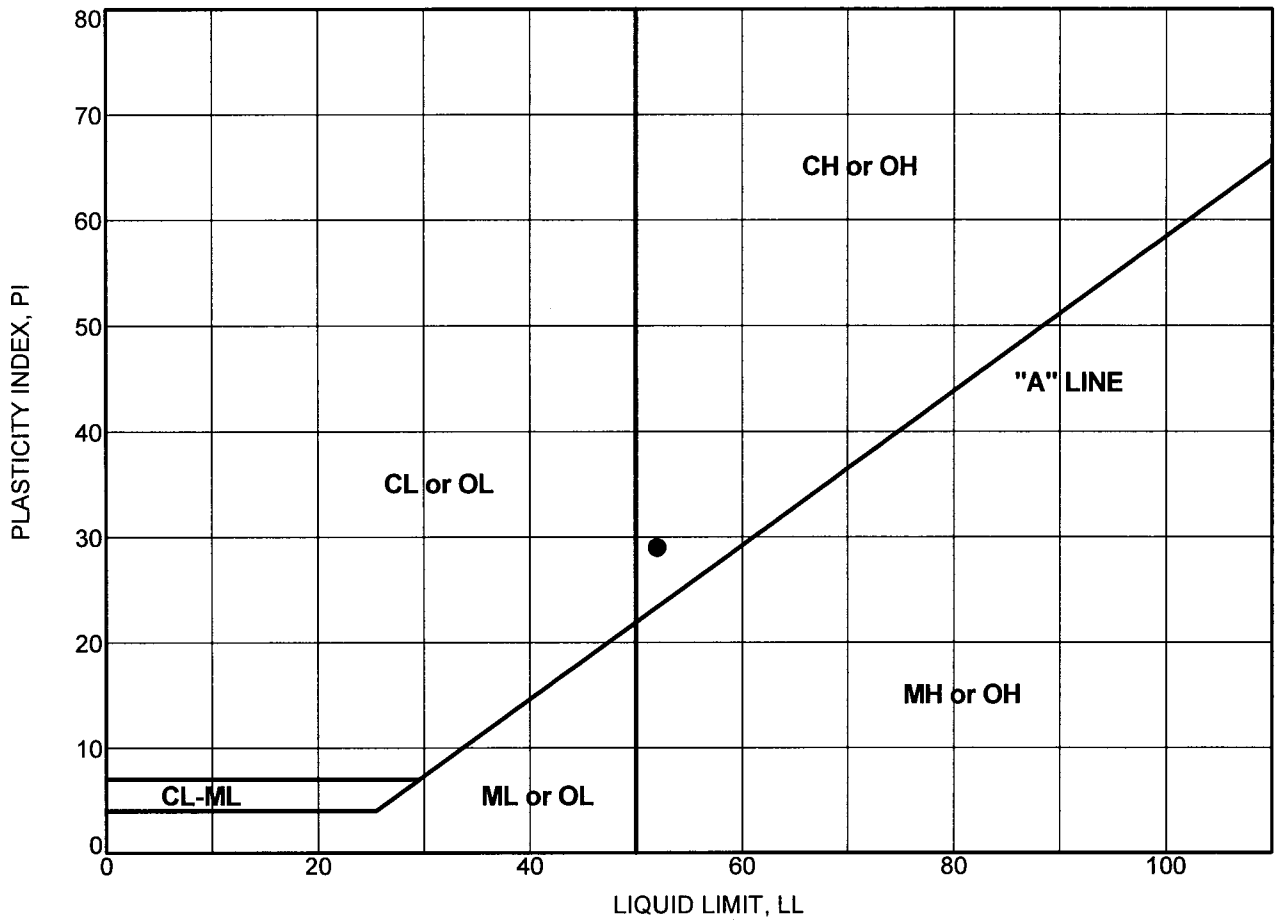
Grain size classification test (ASTM Test Method D 420) was performed on selected sample of granular soil to aid in the classification. The result is presented on Plate B-3, "Grain Size Distribution Curves".

Unconfined Compression Tests

Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-5A through B-5F.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plates B-4A and 4B.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A-09-001	SPT-11	51.0	●	25	52	23	29	FAT CLAY (CH)

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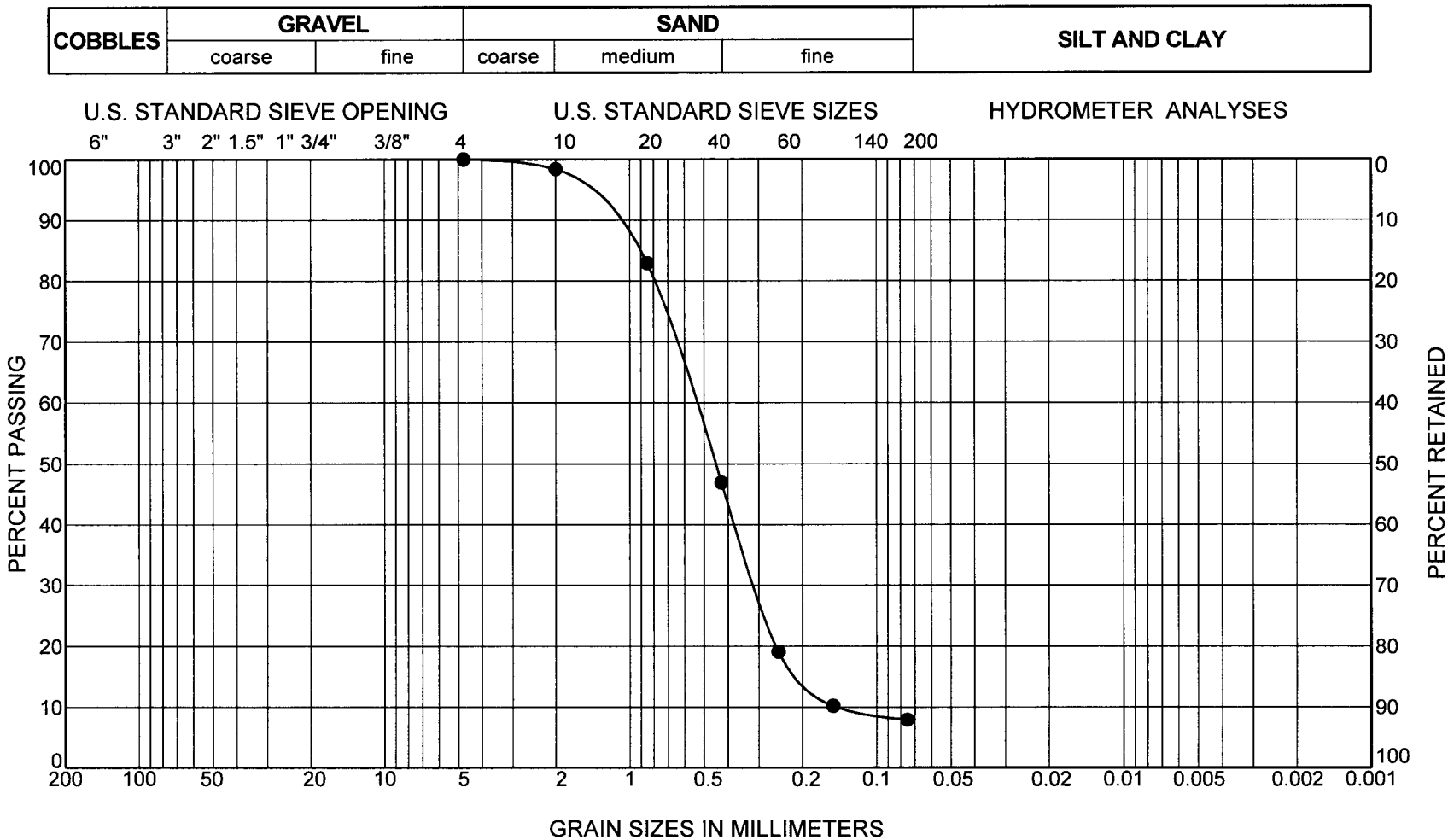
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Swain Road UC (Widen)
Stockton, California

JOB NO: 206127.BR (SRB)

PLATE NO: B-2

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
A-09-001	MC-9	39.5	●			POORLY GRADED SAND (SP)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

JOB NO: 206127.BR (SRB)

Swain Road UC (Widen)
Stockton, California

PLATE NO: B-3

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 04/29/2009
Date Submitted 04/24/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR Site ID : A-09-SR-001#6@24.5'
Thank you for your business.

* For future reference to this analysis please use SUN # 55554-111855.

EVALUATION FOR SOIL CORROSION

Soil pH	7.22		
Minimum Resistivity	0.54 ohm-cm (x1000)		
Chloride	114.0 ppm	00.01140	%
Sulfate	88.2 ppm	00.00882	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

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MAY 1 2009
By

RECEIVED
MAY 1 2009
By

PLATE NO. : B-4A

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/06/2009
Date Submitted 05/01/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR I-5 NORTH Site ID : R-09-SR-001#4@21.0'
Thank you for your business.

* For future reference to this analysis please use SUN # 55616-112073.

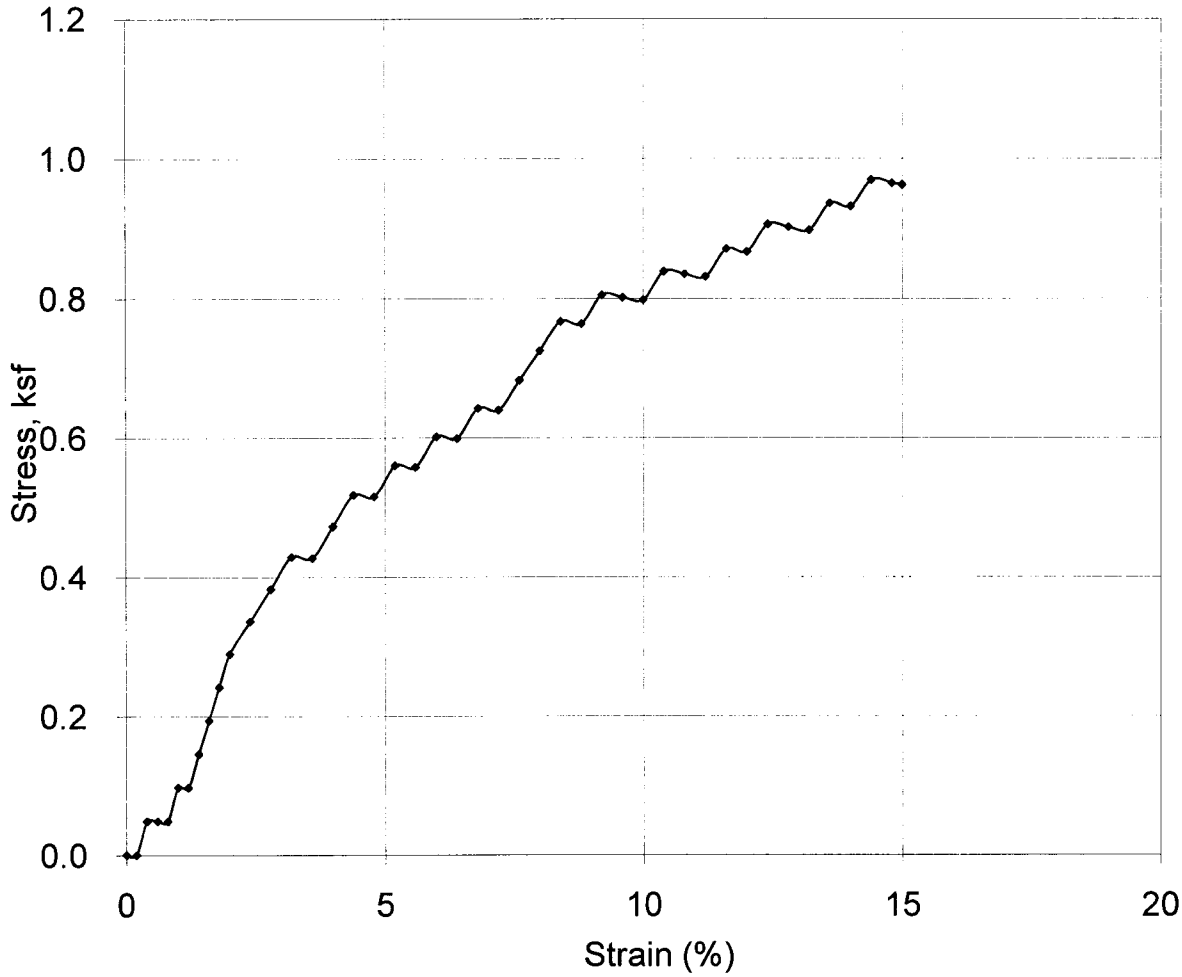
EVALUATION FOR SOIL CORROSION

Soil pH	7.36		
Minimum Resistivity	0.88 ohm-cm (x1000)		
Chloride	55.5 ppm	00.00555	%
Sulfate	38.8 ppm	00.00388	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 8 2009
BY

UNCONFINED COMPRESSION TEST



Boring No.: A-09-SR-001

Sample No. : MC-7

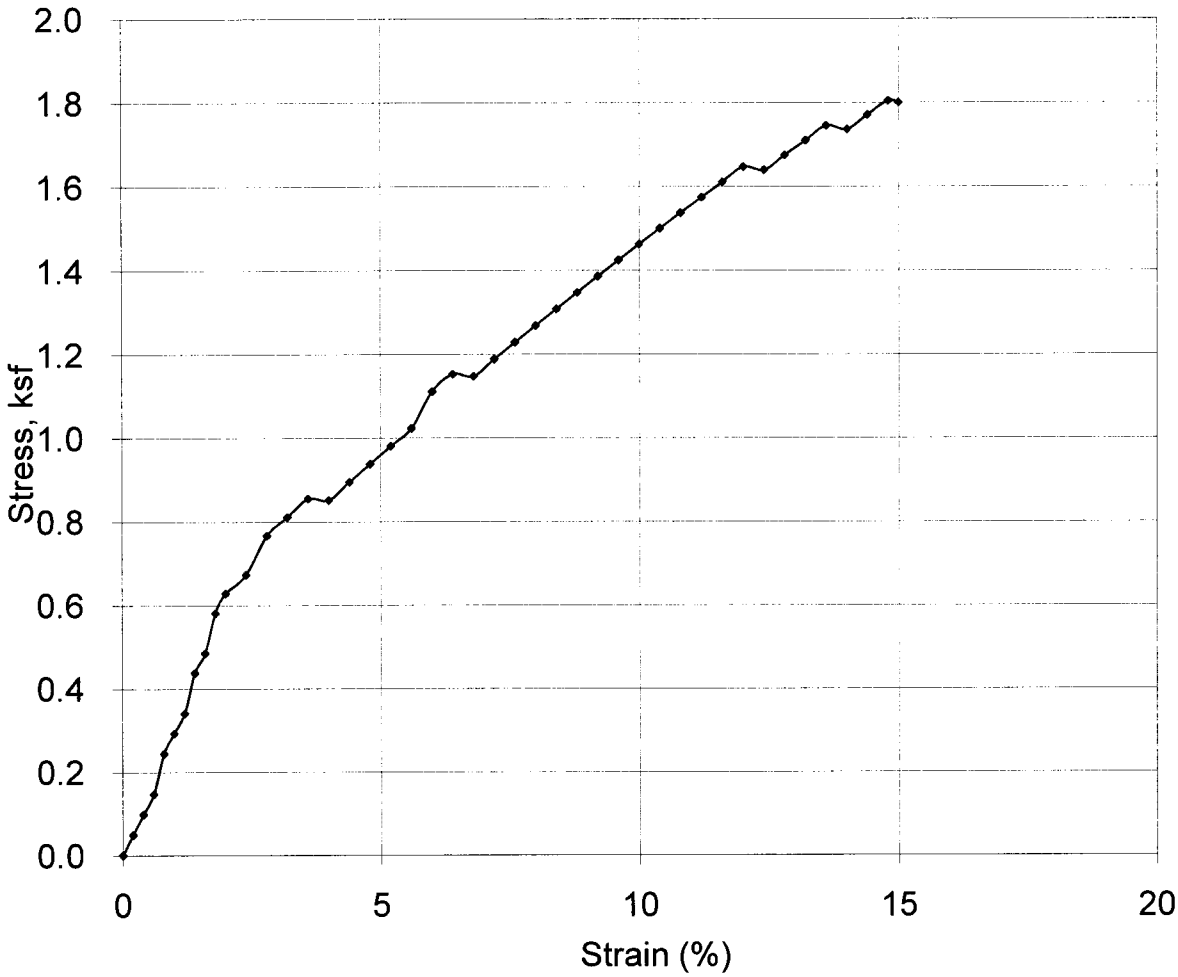
Depth (feet): 29.5

Material Description:
Silty Clay

Unconfined Compressive Strength: 1.0 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**SWAIN ROAD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5A

UNCONFINED COMPRESSION TEST



Boring No.: A-09-SR-001

Sample No. : MC-8

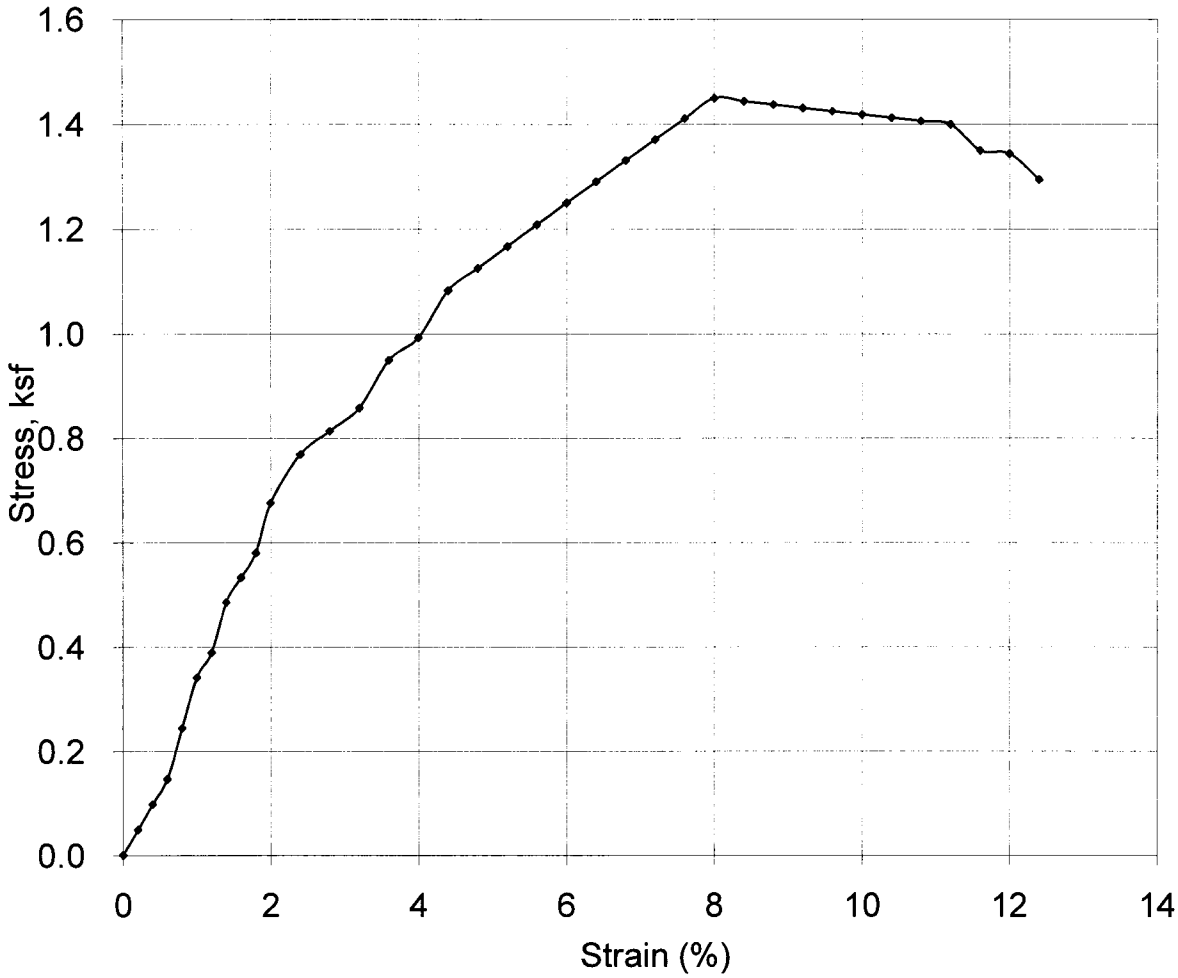
Depth (feet): 34.5

Material Description:
Silty Clay

Unconfined Compressive Strength: 1.8 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**SWAIN ROAD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5B

UNCONFINED COMPRESSION TEST



Boring No.: R-09-SR-001

Sample No. : MC-3

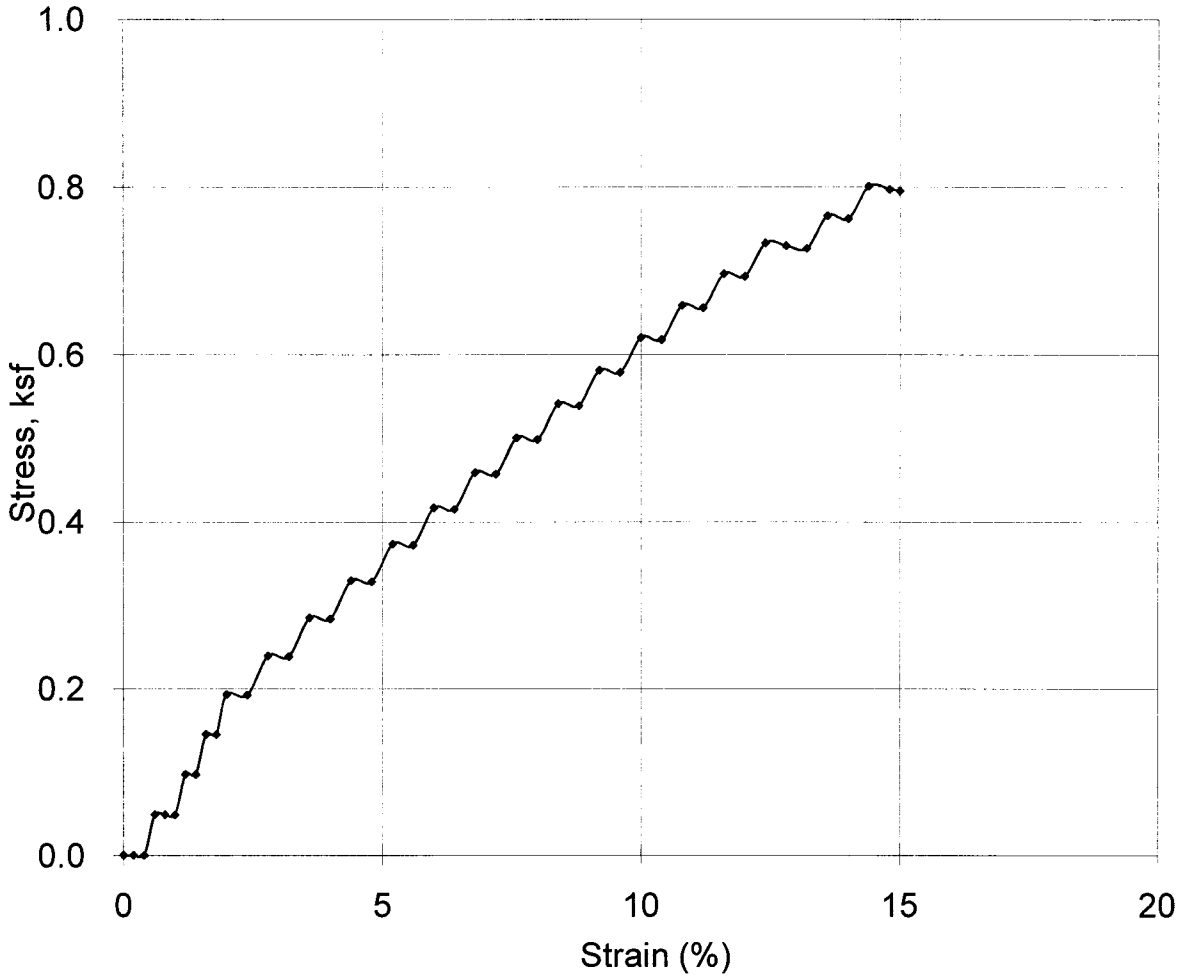
Depth (feet): 16

Material Description:
Silty with Clay

Unconfined Compressive Strength: 1.4 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**SWAIN ROAD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5C

UNCONFINED COMPRESSION TEST



Boring No.: R-09-SR-001

Sample No. : MC-9

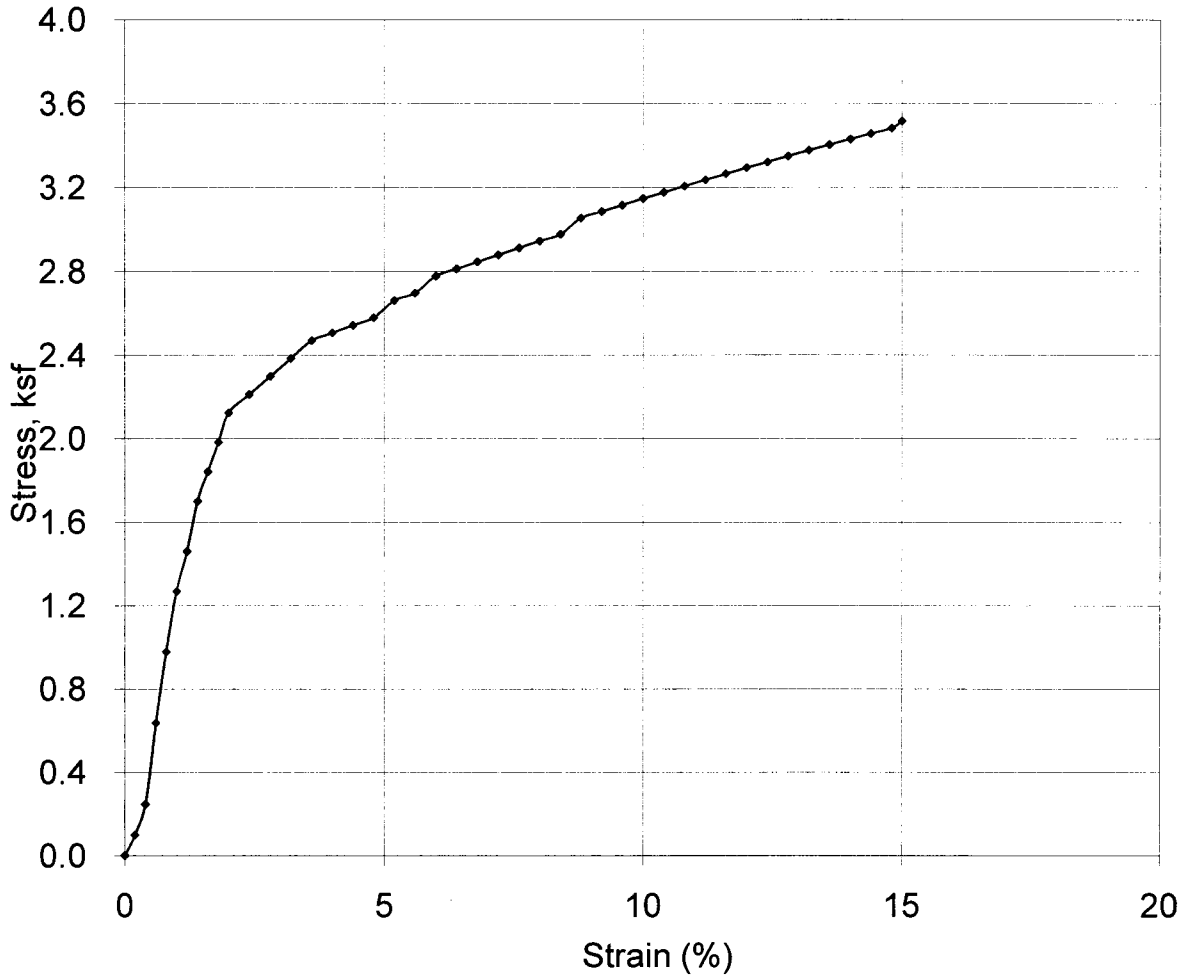
Depth (feet): 46

Material Description:
Silty Clay

Unconfined Compressive Strength: 0.8 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**SWAIN ROAD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	**PLATE NO.: B-5D**

UNCONFINED COMPRESSION TEST



Boring No.: R-09-SR-001

Sample No. : MC-12

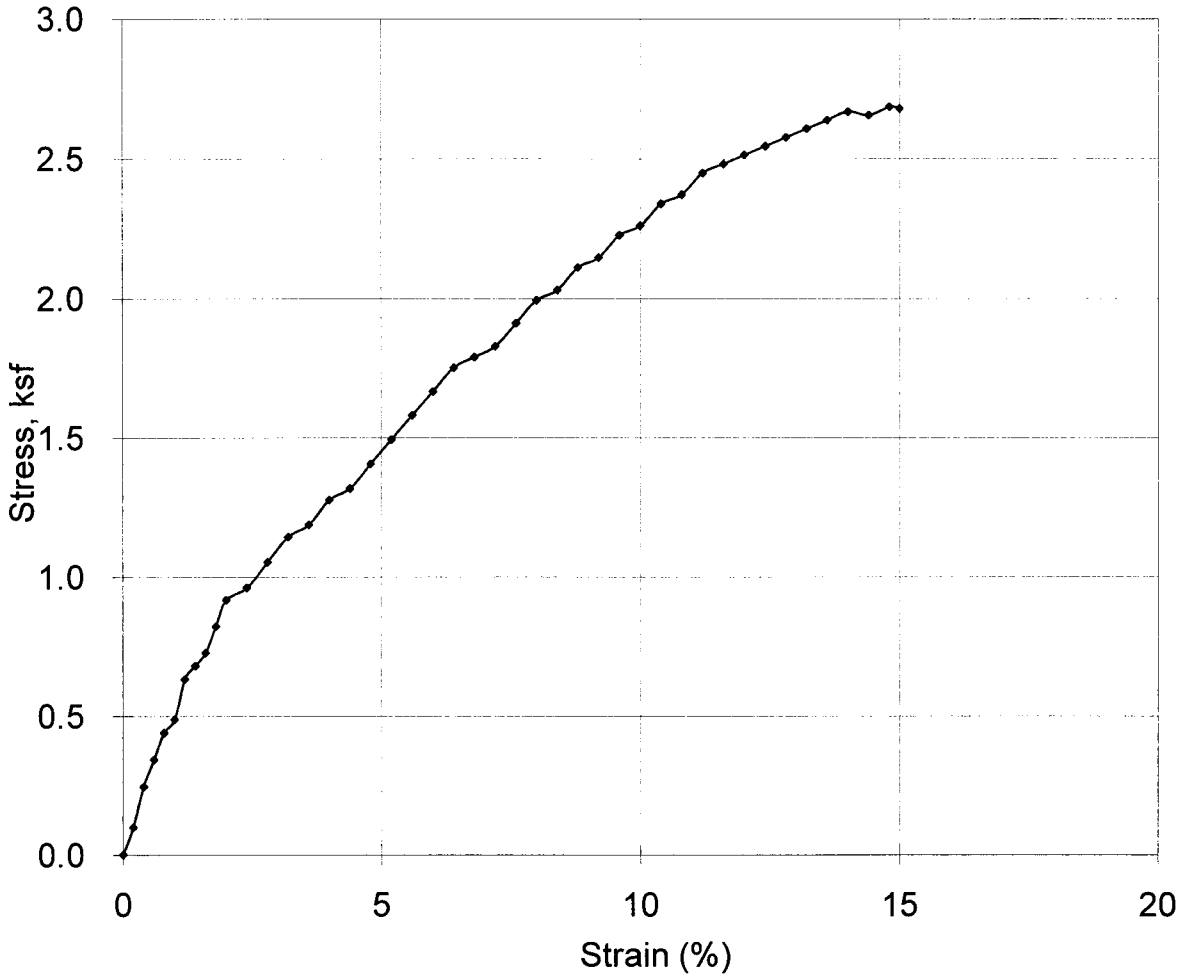
Depth (feet): 56

Material Description:
Lean Clay

Unconfined Compressive Strength: 3.5 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**SWAIN ROAD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5E

UNCONFINED COMPRESSION TEST



Boring No.: R-09-SR-001

Sample No. : MC-13

Depth (feet): 61

Material Description:
Clayey Silt

Unconfined Compressive Strength: 2.7 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**SWAIN ROAD. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	**PLATE NO.: B-5F**

APPENDIX C

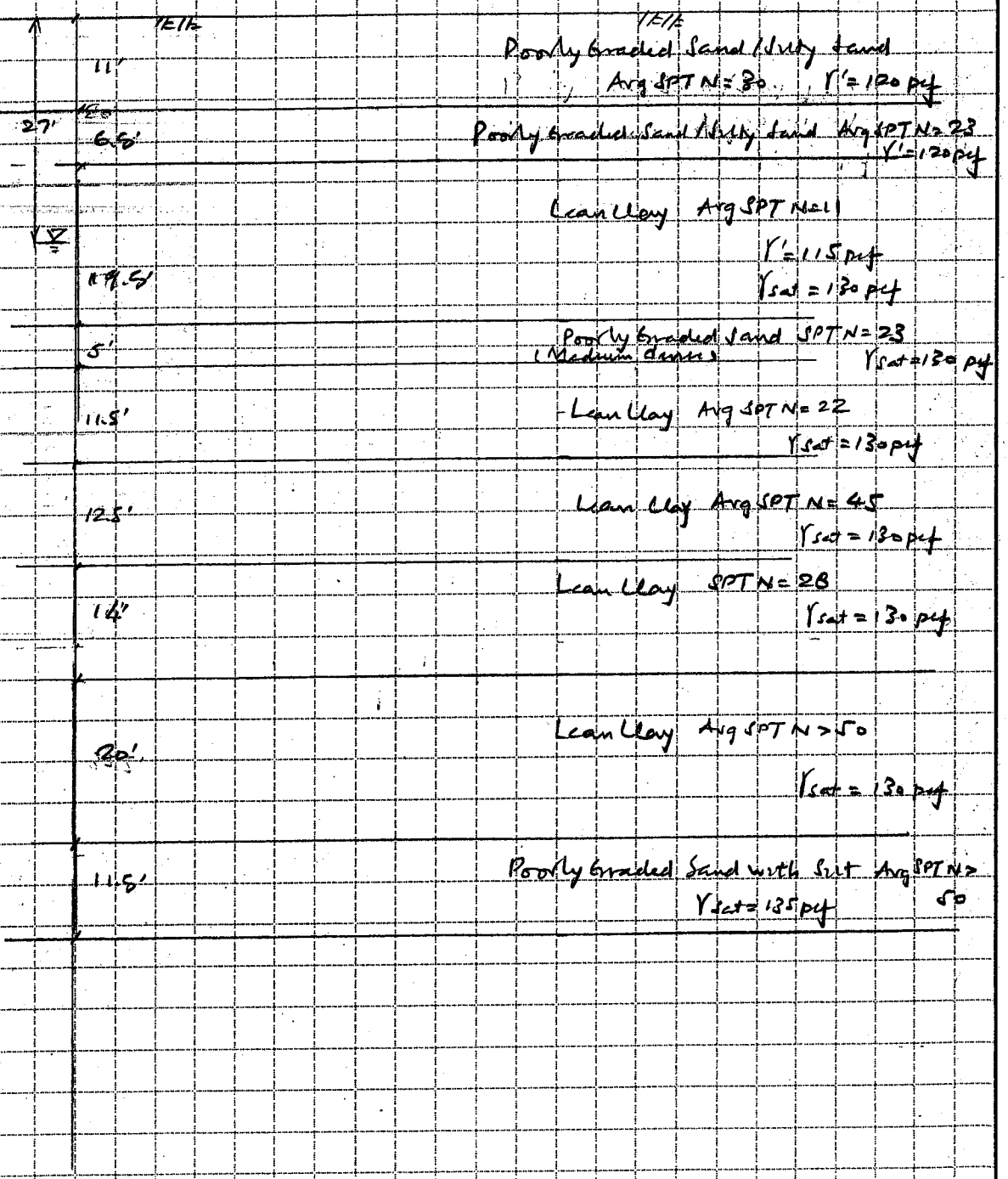
Evaluation of Liquefaction Potential

PARIKH

Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement
SHEET NO OF
CALCULATED BY A Lam DATE 5/2009
CHECKED BY DATE
SCALE

Evaluation of Liquefaction Potential - Swain Road UC (Boring A-09-SW-001)



LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-SW-001 (Swain Road UC)(Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 27

SOIL GROUPS

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR$_{7.0}$/CSR)*MSF*Kσ*Kα*

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	11.0	1	30	SPT	1320.0	1320.0	0.95	0.12	35	1.23	1	0.80	1.2	1.15	47.04	15%	51.80		1.00	1	
2	17.5	1	23	SPT	2100.0	2100.0	0.92	0.12	27	0.98	1	0.85	1.2	1.15	30.38	15%	34.34		0.99	1	
3	37.0	2	11	SPT	4460.0	3835.0	0.78		13	0.72	1	0.80	1.2	1.15	10.12	55%			0.82	1	
4	42.0	1	23	SPT	5110.0	4172.5	0.73	0.12	27	0.69	1	0.80	1.2	1.15	20.28	15%	23.76	0.27	0.80	1	2.21
5	53.5	2	22	SPT	6605.0	4948.8	0.62		25	0.64	1	0.80	1.2	1.15	17.82	55%			0.76	1	
6	66.0	2	45	SPT	8230.0	5792.5	0.55		52	0.59	1	0.80	1.2	1.15	33.68	55%			0.73	1	
7	80.0	2	28	SPT	10050.0	6737.5	0.50		32	0.54	1	0.80	1.2	1.15	19.43	55%			0.69	1	
8	100.0	2	50	SPT	12650.0	8087.5	0.47		58	0.50	1	0.80	1.2	1.15	31.67	55%			0.66	1	
9	111.5	1	50	SPT	14145.0	8863.8	0.46	0.09	58	0.48	1	0.80	1.2	1.15	30.25	15%	34.21		0.64	1	

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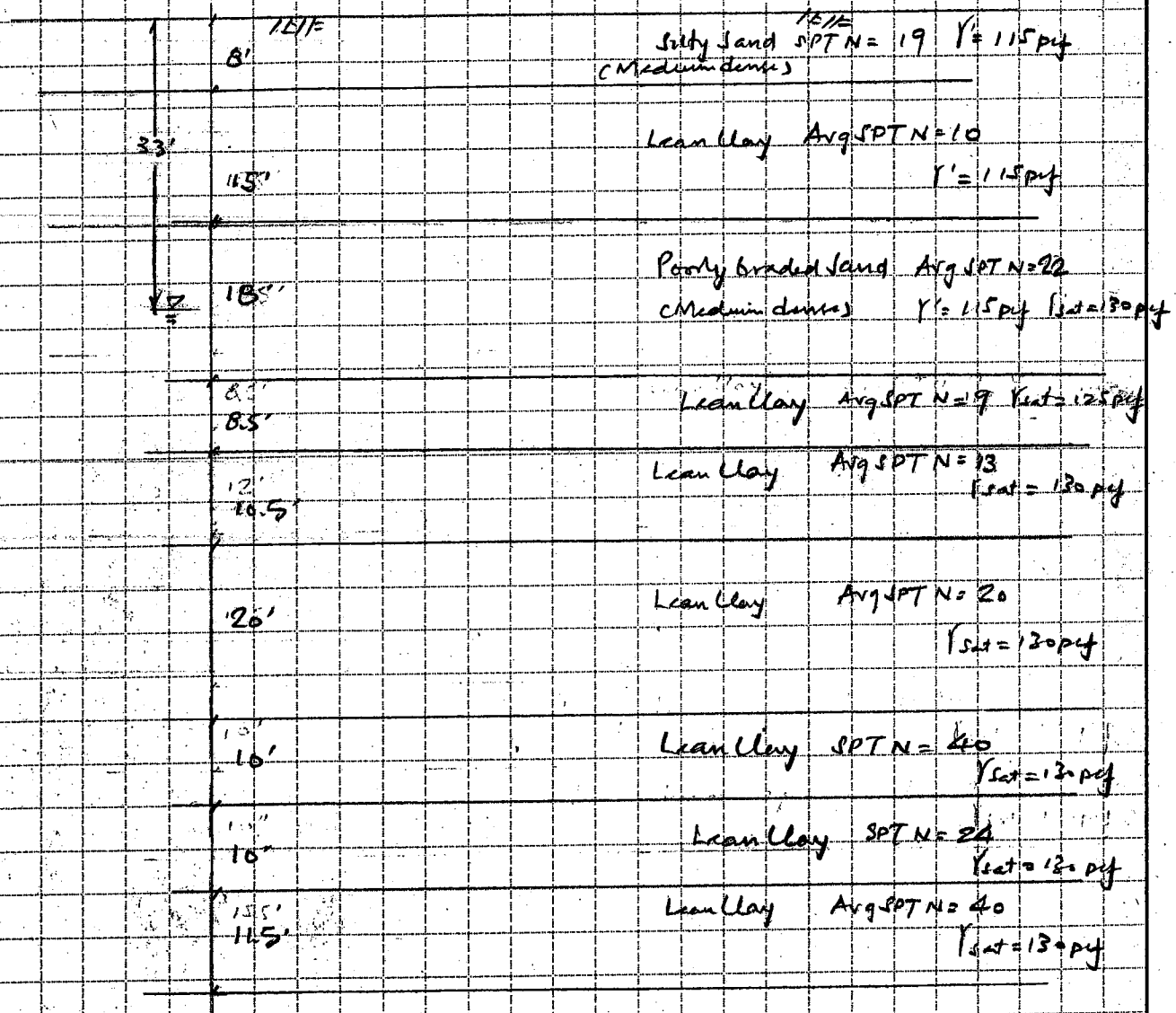
Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement 216(2)
SHEET NO ____ OF ____
CALCULATED BY A. Lam DATE 5/2009
CHECKED BY ____ DATE ____
SCALE ____

Evaluation of Liquefaction Potential - Swain Road UC (Boring R-09-SW-001) (Toe of the slope)



LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***R-09-SW-001 (Swain Road UC) (Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 33

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} (g)= 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR$_{7.0}$/CSR)*MSF*Kσ*Kα*

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	8.0	1	19	SPT	920.0	920.0	0.96	0.13	22	1.47	1	0.75	1.2	1.15	33.46	15%	37.56		1.00	1	
2	23.0	2	10	SPT	2645.0	2645.0	0.89		12	0.87	1	0.80	1.2	1.15	11.08	55%			0.92	1	
3	41.0	1	22	SPT	4835.0	4335.0	0.74	0.11	25	0.68	1	0.80	1.2	1.15	19.04	15%	22.45	0.25	0.79	1	2.18
4	49.5	2	9	SPT	5897.5	4866.3	0.66		10	0.64	1	0.80	1.2	1.15	7.35	55%			0.77	1	
5	60.0	2	13	SPT	7262.5	5575.0	0.58		15	0.60	1	0.80	1.2	1.15	9.92	55%			0.74	1	
6	80.0	2	20	SPT	9862.5	6925.0	0.50		23	0.54	1	0.80	1.2	1.15	13.69	55%			0.69	1	
7	90.0	2	40	SPT	11162.5	7600.0	0.48		46	0.51	1	0.80	1.2	1.15	26.14	55%			0.67	1	
8	100.0	2	24	SPT	12462.5	8275.0	0.47		28	0.49	1	0.80	1.2	1.15	15.03	55%			0.65	1	
9	111.5	2	40	SPT	13957.5	9051.3	0.46		46	0.47	1	0.80	1.2	1.15	23.95	55%			0.64	1	

Vertical Pile Capacity Calculations/Lateral Pile Capacity Analyses

Rajappan & Meyer
CONSULTING ENGINEERS INC

Project Name: ***I-5 North Stockton Corridor Improvement***
Bridge Name: ***SWAIN RD UC (Widen) (Median Widening)***
Br No: ***29-0215 R/L***
Date: ***April 17, 2010***

FOUNDATION DESIGN INFORMATION FROM BRIDGE DESIGNER TO GEOTECHNICAL ENGINEER

SWAIN RD UC (WIDEN) (MEDIAN WIDENING)

Table 1. Foundation Design Data Sheet

Location	Design Method	Pile Type	Finish Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		No. of Piles Per Support
					B	L	
Abut 1	WSD	Class 200 Alt X (Mod)	13.20	10.20	2.50	52.00	11
Abut 2	WSD	Class 200 Alt X (Mod)	13.40	10.40	2.50	52.00	11

Note: Finish Grade is at the front face of abutment diaphragm; Pile Cut-off Elevations are the Bottom of Footing Elevations

Table 2. Foundation Design Loads

Location	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent	Compression		Tension		Compression		Tension	
	Per Support	Max per Pile	Per Support	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile
Abut 1	1569	143	1177	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1569	143	1177	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 3. Lateral Load on Piles

Per Support	Service Limit (kips/pile)	Max per Pile
Abut 1	Negligible	30
Abut 2	Negligible	30

PARIKH

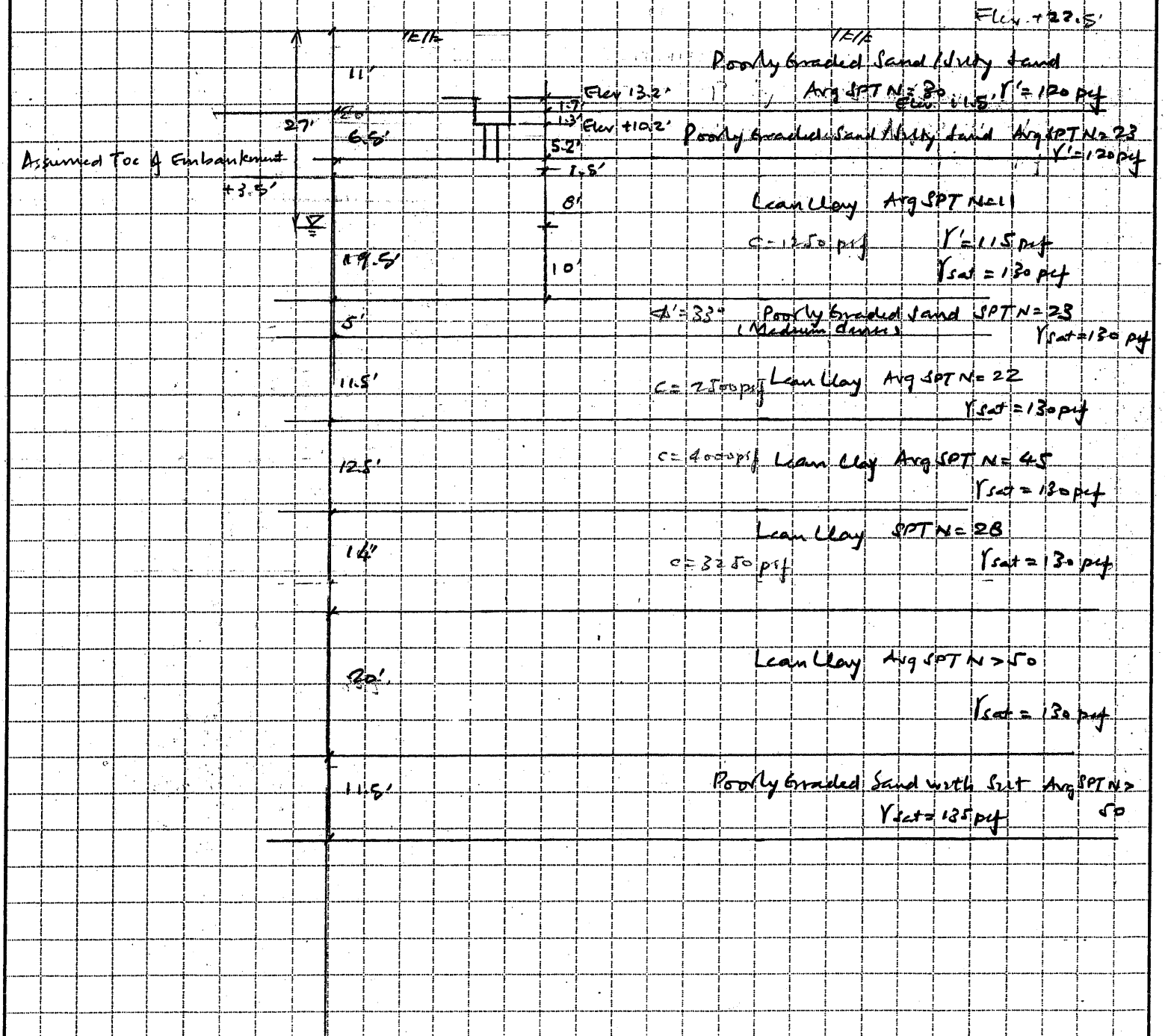
Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement
SHEET NO 1 OF 4
CALCULATED BY A Lam DATE 4/2010
CHECKED BY DATE
SCALE

(Inside Widening)

Pile Capacity Analyses - Swain Road UC (Boring A-09-001) (Hwy 5) Abutment 1

14" □ precast concrete pile - 159 tons service load



(TEMPLATE)

DRIVEN PILE CAPACITY - *I-5 Swain Road UC (Widen) (median)* *Abut 1 - 75 tons service load (150 tons)* **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/06/10

Design per FHWA Manual — 206127.BR

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2
(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1
(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1
(1=DRIVEN ,2=DRILLED)

GROUPS — INCREAMENTS
1- GRAVELS, SANDS AND NONPLASTIC SILTS (FT)= 5
2- CLAYS AND PLASTIC SILTS

SOIL TYPES
GRAVEL, SAND, NSILT Ref Boring : A-09-SW-001 (PARIKH 2009)
CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 200 Alt. "X" - 14" square precast prestressed

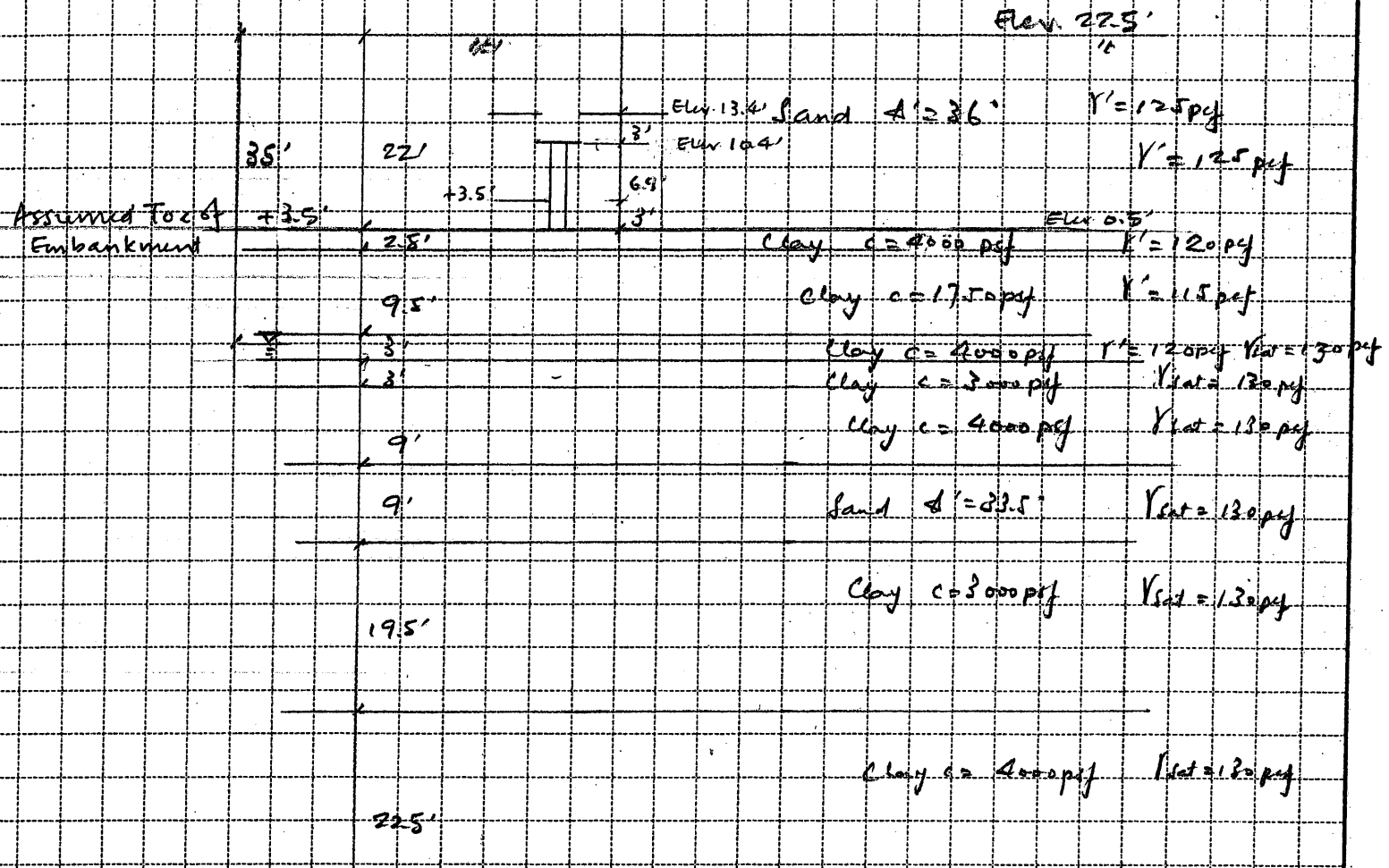
DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	125	375	188	SAND	V. DENSE	0	41	0.00	0.00	0.00	0.00	0.00
3.0	8.0		1	120	600	675	SAND	V. DENSE	0	39	0.00	11.37	11.37	3.81	3.81
8.0	9.5		2	115	172.5	1061	CLAY	STIFF	1250	0	0.85	3.59	14.95	3.59	7.39
9.5	17.5		2	115	920	1608	CLAY	STIFF	1250	0	0.85	19.13	34.08	19.13	26.52
17.5	22.5		2	68	340	2238	CLAY	STIFF	1250	0	0.90	12.66	46.74	12.66	39.18
22.5	27.5		2	68	340	2578	CLAY	STIFF	1250	0	0.95	13.36	60.10	13.36	52.54
27.5	32.5		1	68	340	2918	SAND	DENSE	0	37	0.00	25.05	85.15	8.39	60.93
32.5	37.5		2	68	340	3258	CLAY	V.STIFF	2500	0	0.65	18.28	103.43	18.28	79.21
37.5	44.0		2	68	442	3649	CLAY	V.STIFF	2500	0	0.65	23.77	127.19	23.77	102.97
44.0	48.0		2	68	272	4006	CLAY	V.STIFF	4000	0	0.40	14.40	141.59	14.40	117.37
48.0	52.0		2	68	272	4278	CLAY	V.STIFF	4000	0	0.40	14.40	155.99	14.40	131.77
52.0	58.0		2	68	408	4618	CLAY	V.STIFF	4000	0	0.40	21.60	177.59	21.60	153.37
58.0	63.5		2	68	374	5009	CLAY	V.STIFF	3250	0	0.25	10.05	187.65	10.05	163.43
63.5	70.5		2	68	476	5434	CLAY	V.STIFF	3250	0	0.25	12.80	200.44	12.80	176.23
70.5	75.5		2	68	340	5842	CLAY	V.STIFF	4000	0	0.25	11.25	211.69	11.25	187.48
75.5	80.5		2	68	340	6182	CLAY	V.STIFF	4000	0	0.25	11.25	222.94	11.25	198.73
80.5	85.5		2	68	340	6522	CLAY	V.STIFF	4000	0	0.25	11.25	234.19	11.25	209.98
85.5	90.5		2	68	340	6862	CLAY	V.STIFF	4000	0	0.25	11.25	245.44	11.25	221.23
90.5	96.5		1	68	408	7236	SAND	DENSE	0	38	0.00	31.16	276.60	10.44	231.66
96.5	102.0		1	68	374	7627	SAND	MED DEN	0	37	0.00	28.39	304.99	9.51	241.17

Abutment 1 --> Pile cut-off elevation is +10.20 feet. Assumed finished grade elevation is +13.20 feet
Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +3.5 feet
Pile Length @ Abutment 1 = 58.0' - 3.0' = 55.0 '
Pile tip at 55.0' below pile cut-off (pile tip @ Elev. -45.0', pile cut-off @ Elev. +10.2'), Ult. Pile Capacity =177.6 tons (Friction)-15.0 tons (Predrilled)=162.6 tons > 300k/150T (Service Limit State), OK

I-5 Swain Road (median) Abut 1 Class 200 Alt X (4-06-10)

Sheet 2 of 4

Pile Capacity Analysis Swain Road UC (Widen) - CPT-09-001 Abutment 2 Inside Widening
14" □ precast concrete pile 159 tons service load



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Swain Road UC (Widen) (Inside Widening)*** ***Abut 2 - 75 tons service load (150 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/06/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.17 — 1- GRAVELS, SANDS (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 2 — AND NONPLASTIC SILTS

(1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 1 — SOIL TYPES

(1=CONCRETE ,2=STEEL) — GRAVEL, SAND, NSILT Ref Boring : CPT-09-SW-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

DRIVEN/DRILLED = 1 — CLAY, PSILT — AREA OF THE PILE TIP 1.35 SQ. FT.

(1=DRIVEN ,2=DRILLED)

Class 200 Alt. "X" - 14" square precast prestressed

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	125	375	188	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	10.0		1	125	875	813	SAND	DENSE	0	36	0.00	17.58	17.58	5.89	5.89
10.0	13.0		1	125	375	1438	SAND	DENSE	0	36	0.00	13.04	30.61	4.37	10.26
13.0	15.5		2	120	300	1775	CLAY	V.STIFF	4000	0	0.25	5.63	36.24	5.63	15.88
15.5	20.5		2	115	575	2213	CLAY	STIFF	1750	0	0.75	14.77	51.00	14.77	30.65
20.5	25.0		2	115	517.5	2759	CLAY	STIFF	1750	0	0.75	13.29	64.29	13.29	43.93
25.0	26.0		2	120	120	3078	CLAY	V.STIFF	4000	0	0.25	2.25	66.54	2.25	46.18
26.0	28.0		2	68	136	3206	CLAY	V.STIFF	4000	0	0.25	4.50	71.04	4.50	50.68
28.0	31.0		2	68	204	3376	CLAY	V.STIFF	3000	0	0.30	6.08	77.12	6.08	56.76
31.0	36.0		2	68	340	3648	CLAY	V.STIFF	4000	0	0.25	11.25	88.37	11.25	68.01
36.0	40.0		2	68	272	3954	CLAY	V.STIFF	4000	0	0.25	9.00	97.37	9.00	77.01
40.0	45.0		1	68	340	4260	SAND	MED DEN	0	34	0.00	20.37	117.74	6.82	83.83
45.0	49.0		1	68	272	4566	SAND	MED DEN	0	34	0.00	16.22	133.95	5.43	89.27
49.0	58.5		2	68	646	5025	CLAY	V.STIFF	3000	0	0.75	48.09	182.05 ←	48.09	137.36
58.5	60.0		2	68	102	5399	CLAY	V.STIFF	3000	0	0.75	7.59	189.64	7.59	144.95
60.0	64.0		2	68	272	5586	CLAY	V.STIFF	3000	0	0.40	10.80	200.44	10.80	155.75
64.0	68.5		2	68	306	5875	CLAY	V.STIFF	3000	0	0.40	12.15	212.59	12.15	167.90
68.5	73.5		2	68	340	6198	CLAY	V.STIFF	4000	0	0.40	18.00	230.59	18.00	185.90
73.5	78.5		2	68	340	6538	CLAY	V.STIFF	4000	0	0.40	18.00	248.59	18.00	203.90
78.5	84.5		2	68	408	6912	CLAY	V.STIFF	4000	0	0.40	21.60	270.19	21.60	225.50
84.5	91.0		2	68	442	7337	CLAY	V.STIFF	4000	0	0.40	23.40	293.59	23.40	248.90

Abutment 2 --> Pile cut-off elevation is +10.40 feet. Assumed finished grade elevation is +13.40 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +3.5 feet

Pile Length @ Abutment 2 = 58.5' - 3.0' = 55.5 '

Pile tip at 55.5' below pile cut-off (pile tip @ Elev. -45.5', pile cut-off @ Elev. +10.4'), Ult. Pile Capacity =182.1 tons (Friction)-17.6 (Predrilled)=164.5 tons > Required capacity 300k/150T (Service Limit State), OK

I-5 Swain Road (median) Abut 2 Class 200 Alt X (4-06-10)

Sheet 4 of 4

Rajappan & Meyer
CONSULTING ENGINEERS INC

Project Name: ***I-5 North Stockton Corridor Improvement***
Bridge Name.: ***SWAIN RD UC (Widen) (Outside Widening)***
Br No.: ***29-0215 R/L***
Date: ***April 17, 2010***

FOUNDATION DESIGN INFORMATION FROM BRIDGE DESIGNER TO GEOTECHNICAL ENGINEER

SWAIN RD UC (WIDEN) (OUTSIDE WIDENING)

Table 1. Foundation Design Data Sheet

Location	Design Method	Pile Type	Finish Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		No. of Piles Per Support
					B	L	
Abut 1	WSD	Class 200 Alt X (Mod)	12.15	9.00	2.50	13.00	4
Abut 2	WSD	Class 200 Alt X (Mod)	12.35	9.20	2.50	13.00	4
Abut 1 WW	WSD	Class 200 Alt X (Mod)	19.2	16.20	3	6	2
Abut 2 WW	WSD	Class 200 Alt X (Mod)	19.42	16.42	3	6	2

Note: Finish Grade is at the front face of abutment diaphragm; Pile Cut-off Elevations are the Bottom of Footing Elevations

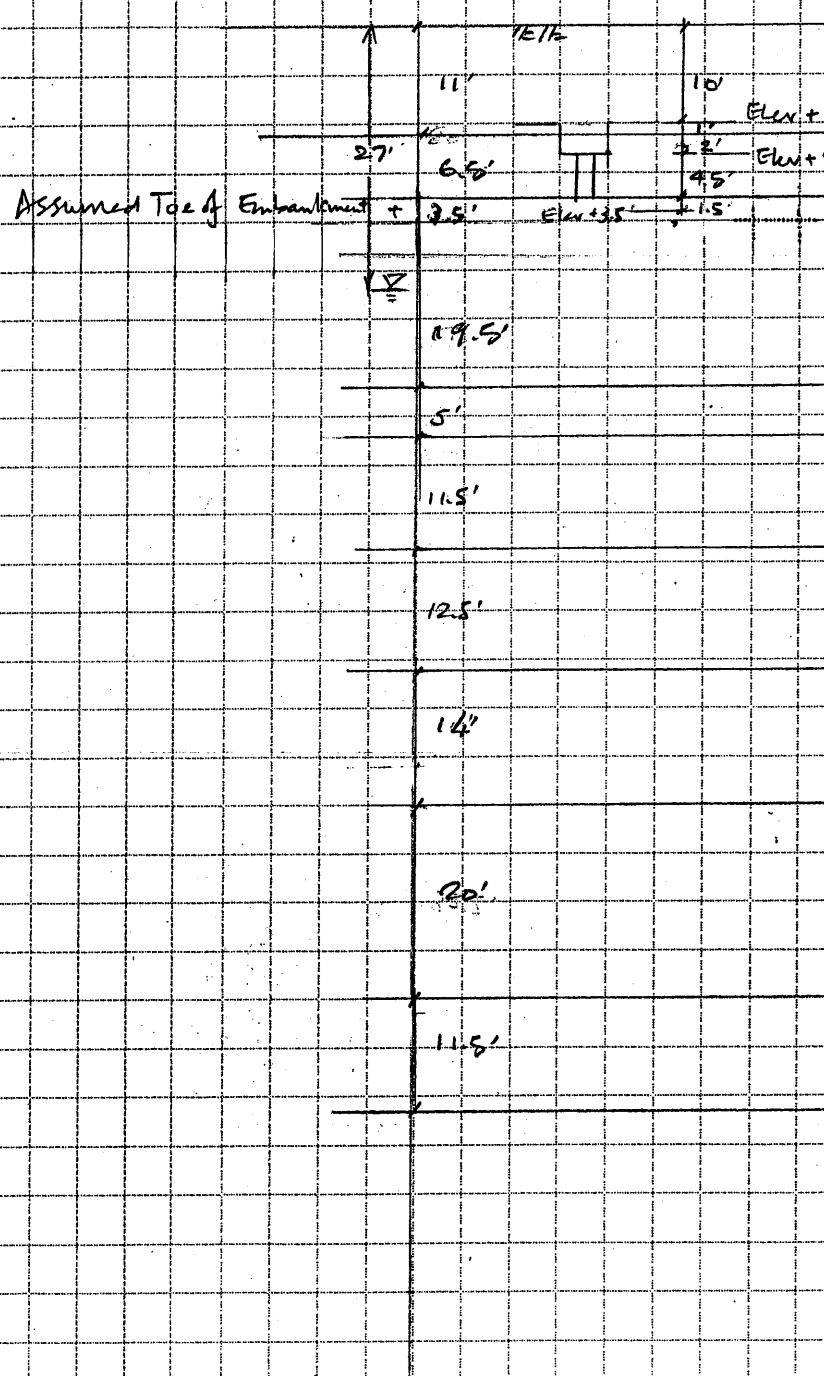
Table 2. Foundation Design Loads

Location	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State			
	Total Load		Permanent	Compression		Tension		Compression		Tension	
	Per Support	Max per Pile	Per Support	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile
Abut 1	599	150	418	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	599	150	418	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Wingwall	38.4	19.2	38.4	N/A	N/A	N/A	N/A	N/A	56	N/A	19

Table 3. Lateral Load on Piles

Per Support	Service Limit (kips/pile)	Max per Pile
Abut 1	Negligible	29
Abut 2	Negligible	29
Abut 1&2 WW	Negligible	14

Pile Capacity Analyses - Swain Road UC (Boring A-09-010) (Hwy 5) (Outside widening Abutment 1 14" ⌀ precast concrete pile · 154 tons service load



Poorly Graded Sand / Silty Sand Avg SPT N = 30, γ' = 120 pcf

Poorly Graded Sand / Silty Sand Avg SPT N = 23, γ' = 120 pcf

Lean Clay Avg SPT N = 11, γ' = 115 pcf, γsat = 130 pcf

Poorly Graded Sand SPT N = 23 (Medium dense) γsat = 130 pcf

Lean Clay Avg SPT N = 22, γsat = 130 pcf

Lean Clay Avg SPT N = 45, γsat = 130 pcf

Lean Clay SPT N = 28, γsat = 130 pcf

Lean Clay Avg SPT N > 50, γsat = 130 pcf

Poorly Graded Sand with Silt Avg SPT N > 50, γsat = 135 pcf

(TEMPLATE)

DRIVEN PILE CAPACITY - *I-5 Swain Road UC (Widen) (outside)* *Abut 1 - 75 tons service load (150 tons)* **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/06/10

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS INCREAMENTS

1- GRAVELS, SANDS (FT)= 5

AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : A-09-SW-001 (PARIKH 2009)

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 200 Alt. "X" - 14" square precast prestressed

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	φ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	125	375	188	SAND	V. DENSE	0	41	0.00	0.00	0.00	0.69	0.69
3.0	7.5		1	120	540	645	SAND	V. DENSE	0	39	0.00	9.79	9.79	3.28	3.97
7.5	9.0		2	115	172.5	1001	CLAY	STIFF	1250	0	0.85	3.59	13.38	3.59	7.55
9.0	17.0		2	115	920	1548	CLAY	STIFF	1250	0	0.85	19.13	32.50	19.13	26.68
17.0	22.0		2	68	340	2178	CLAY	STIFF	1250	0	0.90	12.66	45.16	12.66	39.33
22.0	27.0		2	68	340	2518	CLAY	STIFF	1250	0	0.95	13.36	58.52	13.36	52.69
27.0	32.0		1	68	340	2858	SAND	DENSE	0	37	0.00	24.15	82.67	8.09	60.78
32.0	37.0		2	68	340	3198	CLAY	V.STIFF	2500	0	0.65	18.28	100.95	18.28	79.06
37.0	43.5		2	68	442	3589	CLAY	V.STIFF	2500	0	0.65	23.77	124.72	23.77	102.83
43.5	47.5		2	68	272	3946	CLAY	V.STIFF	4000	0	0.40	14.40	139.12	14.40	117.23
47.5	51.5		2	68	272	4218	CLAY	V.STIFF	4000	0	0.40	14.40	153.52	14.40	131.63
51.5	56.0		2	68	306	4507	CLAY	V.STIFF	4000	0	0.40	16.20	169.72 ←	16.20	147.83
56.0	61.0		2	68	340	4830	CLAY	V.STIFF	3250	0	0.25	9.14	178.86	9.14	156.97
61.0	70.0		2	68	612	5306	CLAY	V.STIFF	3250	0	0.25	16.45	195.31	16.45	173.42
70.0	75.0		2	68	340	5782	CLAY	V.STIFF	4000	0	0.25	11.25	206.56	11.25	184.67
75.0	80.0		2	68	340	6122	CLAY	V.STIFF	4000	0	0.25	11.25	217.81	11.25	195.92
80.0	85.0		2	68	340	6462	CLAY	V.STIFF	4000	0	0.25	11.25	229.06	11.25	207.17
85.0	90.0		2	68	340	6802	CLAY	V.STIFF	4000	0	0.25	11.25	240.31	11.25	218.42
90.0	96.0		1	68	408	7176	SAND	DENSE	0	38	0.00	30.02	270.33	10.06	228.48
96.0	101.5		1	68	374	7567	SAND	MED DEN	0	37	0.00	27.35	297.69	9.16	237.64

Abut 1 - Pile cut-off elevation is +9.00 feet. Assumed finished grade elevation is +12.15 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +3.5 feet

Pile Length @ Abutment 1 = 56.0' - 3.0' = 53.0 '

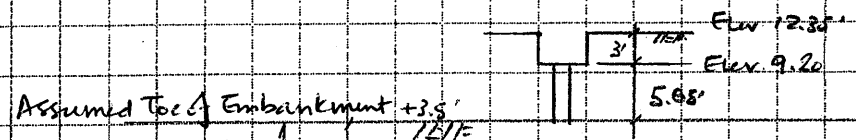
Pile tip at 53.0' below pile cut-off (pile tip @ Elev. -44.0', pile cut-off @ Elev. +9.00'),Ult. Pile Capacity =169.7 tons (Friction)-13.4 tons (Predrilled)=156.3 tons > Required capacity 300k/150T (Service Limit State),OK

I-5 Swain Road (outside) Abut 1 Class 200 Alt X (4-6-10)

Sheet 2 of 5

Simplified Soil Profile - Swain Road UC (Boring R-09-001) Outside Widening

About 2 14" □ precast concrete piles 154 tons service load



8' — Silty Sand (Medium dense) SPT N = 19 $\gamma' = 115$ pcf

33' — 15' — Lean Clay Avg SPT N = 10 $\alpha = 0.85$ c = 1250 psf $\gamma' = 115$ pcf

18' — Poorly Graded Sand (Medium dense) Avg SPT N = 22 $\gamma' = 115$ pcf $\gamma_{sat} = 130$ pcf

8.5' — Lean Clay Avg SPT N = 9 $\gamma_{sat} = 125$ pcf c = 1625 psf $\alpha = 1.00$

10.5' — Lean Clay Avg SPT N = 13 $\gamma_{sat} = 130$ pcf c = 1625 psf $\alpha = 1.0$

20' — Lean Clay Avg SPT N = 20 $\gamma_{sat} = 130$ pcf c = 2500 psf $\alpha = 0.65, 0.50$

10' — Lean Clay SPT N = 40 c = 4000 psf $\alpha = 0.25$ $\gamma_{sat} = 130$ pcf

10' — Lean Clay SPT N = 24 c = 3000 psf $\alpha = 0.25$ $\gamma_{sat} = 130$ pcf

11.5' — Lean Clay Avg SPT N = 40 c = 4000 psf $\alpha = 0.25$ $\gamma_{sat} = 130$ pcf

(TEMPLATE)

DRIVEN PILE CAPACITY - *I-5 Swain Road UC (Widen) (Outside Widening)* *Abut 2 - 75 tons service load (150 tons)* **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/06/10

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS INCREAMENTS

1- GRAVELS, SANDS (FT)= 5

AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : CPT-09-SR-001 (PARIKH 2009)

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 200 Alt. "X" - 14" square precast prestressed

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	115	345	173	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	9.0		1	115	690	690	SAND	DENSE	0	38	0.00	13.49	13.49	4.52	4.52
9.0	17.0		1	115	920	1495	SAND	DENSE	0	37	0.00	37.76	51.25	12.65	17.17
17.0	24.5		2	115	862.5	2386	CLAY	STIFF	1250	0	0.85	17.93	69.18	17.93	35.10
24.5	32.0		2	115	862.5	3249	CLAY	STIFF	1250	0	0.85	17.93	87.11	17.93	53.03
32.0	37.0		1	115	575	3968	SAND	DENSE	0	36	0.00	36.07	123.18	12.08	65.11
37.0	42.0		1	115	575	4543	SAND	MED DEN	0	35	0.00	35.67	158.85	11.95	77.06
42.0	44.0		1	68	136	4898	SAND	MED DEN	0	35	0.00	14.18	173.03	4.75	81.81
44.0	50.0		1	68	408	5170	SAND	MED DEN	0	35	0.00	42.35	215.38	14.19	96.00
50.0	54.0		2	68	272	5510	CLAY	STIFF	1125	0	1.00	10.13	225.50	10.13	106.12
54.0	58.5		2	68	306	5799	CLAY	STIFF	1125	0	1.00	11.39	236.89	11.39	117.51
58.5	63.5		2	68	340	6122	CLAY	STIFF	1625	0	1.00	18.28	255.17	18.28	135.79
63.5	69.0		2	68	374	6479	CLAY	STIFF	1625	0	1.00	20.11	275.28	20.11	155.90
69.0	74.0		2	68	340	6836	CLAY	V.STIFF	2500	0	0.65	18.28	293.57	18.28	174.19
74.0	79.0		2	68	340	7176	CLAY	V.STIFF	2500	0	0.65	18.28	311.85	18.28	192.47
79.0	84.0		2	68	340	7516	CLAY	V.STIFF	2500	0	0.50	14.06	325.91	14.06	206.53
84.0	89.0		2	68	340	7856	CLAY	V.STIFF	2500	0	0.50	14.06	339.97	14.06	220.59
89.0	94.0		2	68	340	8196	CLAY	V.STIFF	4000	0	0.25	11.25	351.22	11.25	231.84
94.0	99.0		2	68	340	8536	CLAY	V.STIFF	4000	0	0.25	11.25	362.47	11.25	243.09
99.0	104.0		2	68	340	8876	CLAY	V.STIFF	3000	0	0.25	8.44	370.91	8.44	251.53
104.0	109.0		2	68	340	9216	CLAY	V.STIFF	3000	0	0.25	8.44	379.35	8.44	259.97
109.0	115.0		2	68	408	9590	CLAY	V.STIFF	4000	0	0.25	13.50	392.85	13.50	273.47
115.0	120.5		2	68	374	9981	CLAY	V.STIFF	4000	0	0.25	12.38	405.22	12.38	285.84

Abutment 2 --> Pile cut-off elevation is +9.20 feet. Assumed finished grade elevation is +12.35 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +3.5 feet

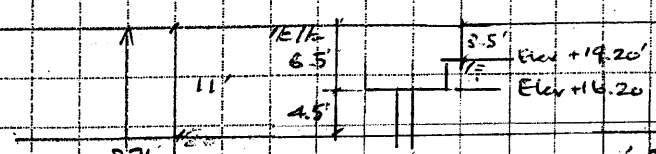
Pile Length @ Abutment 2 = 44.0'- 3.0' = 41.0 '

Pile tip at 41.0' below pile cut-off (pile tip @ Elev. -32.0', pile cut-off @ Elev. +9.20'), Ult. Pile Capacity =173.0 tons (Friction)-13.5 (Predrilled)=159.5 tons > Required capacity 300k/150T (Service Limit State), OK

I-5 Swain Road (outside) Abut 2 Class 200 Alt X (4-06-10)

Sheet 4 of 5

Pile Capacity Analyses - Swain Road UC (Boring A-09-010) (Hwy 5) (Outside widening Abutment 1 WW1) 14" ø precast concrete pile · 154 tons service load



Assumed Toe of Embankment

Elev +22.5'

Poorly Graded Sand / Silty Sand
Avg SPT N = 30 γ' = 120 pcf

Poorly Graded Sand / Silty Sand Avg SPT N = 23
γ' = 120 pcf

Lean Clay Avg SPT N = 11
γ' = 115 pcf
γsat = 130 pcf

19.5'

Poorly Graded Sand SPT N = 23
(Medium dense)
γsat = 130 pcf

5'

Lean Clay Avg SPT N = 22
γsat = 130 pcf

11.5'

Lean Clay Avg SPT N = 45
γsat = 130 pcf

12.5'

Lean Clay SPT N = 28
γsat = 130 pcf

14'

Lean Clay Avg SPT N > 50
γsat = 130 pcf

20'

Poorly Graded Sand with Silt Avg SPT N > 50
γsat = 135 pcf

11.5'

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Swain Road UC (Widen) (outside)*** ***Abut 1 Wing Wall - 28 tons Extreme Limit I*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/06/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.17 — 1- GRAVELS, SANDS — (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 2 — AND NONPLASTIC SILTS

(1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 1 — SOIL TYPES

(1=CONCRETE ,2=STEEL) — GRAVEL, SAND, NSILT Ref Boring : A-09-SW-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) : 4.50 SQ. FT.

DRIVEN/DRILLED = 1 — CLAY, PSILT — AREA OF THE PILE TIP 1.35 SQ. FT.

(1=DRIVEN ,2=DRILLED)

Class 200 Alt. "X" - 14" square precast prestressed c

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	125	375	188	SAND	V. DENSE	0	41	0.00	0.00	0.00	0.69	0.69
3.0	14.0		1	120	1320	1035	SAND	DENSE	0	38	0.00	37.74	37.74	12.64	13.33
14.0	15.5		2	115	172.5	1781	CLAY	STIFF	1250	0	0.85	3.59	41.32	3.59	16.91
15.5	26.5		2	115	1265	2500	CLAY	STIFF	1250	0	0.85	26.30	67.62 ←	26.30	43.21 ←
26.5	31.5		2	68	340	3303	CLAY	STIFF	1250	0	0.90	12.66	80.27 ←	12.66	55.87
31.5	33.5		2	68	136	3541	CLAY	STIFF	1250	0	0.95	5.34	85.62	5.34	61.21
33.5	38.5		1	68	340	3779	SAND	DENSE	0	36	0.00	38.18	123.80	12.79	74.00
38.5	43.5		2	68	340	4119	CLAY	V.STIFF	2500	0	0.65	18.28	142.08	18.28	92.28
43.5	50.0		2	68	442	4510	CLAY	V.STIFF	2500	0	0.65	23.77	165.85	23.77	116.05
50.0	54.0		2	68	272	4867	CLAY	V.STIFF	4000	0	0.40	14.40	180.25	14.40	130.45
54.0	58.0		2	68	272	5139	CLAY	V.STIFF	4000	0	0.40	14.40	194.65	14.40	144.85
58.0	62.5		2	68	306	5428	CLAY	V.STIFF	4000	0	0.40	16.20	210.85	16.20	161.05
62.5	67.5		2	68	340	5751	CLAY	V.STIFF	3250	0	0.25	9.14	219.99	9.14	170.19
67.5	76.5		2	68	612	6227	CLAY	V.STIFF	3250	0	0.25	16.45	236.44	16.45	186.64
76.5	81.5		2	68	340	6703	CLAY	V.STIFF	4000	0	0.25	11.25	247.69	11.25	197.89
81.5	86.5		2	68	340	7043	CLAY	V.STIFF	4000	0	0.25	11.25	258.94	11.25	209.14
86.5	91.5		2	68	340	7383	CLAY	V.STIFF	4000	0	0.25	11.25	270.19	11.25	220.39
91.5	96.5		2	68	340	7723	CLAY	V.STIFF	4000	0	0.25	11.25	281.44	11.25	231.64
96.5	102.5		1	68	408	8097	SAND	DENSE	0	37	0.00	47.83	329.27	16.02	247.66
102.5	108.0		1	68	374	8488	SAND	MED DEN	0	37	0.00	43.61	372.88	14.61	262.27

Abut 1 WW--> Pile cut-off elevation is +16.2 feet. Assumed finished grade elevation is +19.2 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +3.5 feet

Pile Length @ Abutment 1 = 31.5' - 3.0' = 28.5' (Extreme Limit Comp), 26.5'-3.0'=23.5' (Service Limit), 26.5'-3.0' = 23.5' (Extreme Limit Tension)

Pile tip at 28.5' below pile cut-off(pile tip @Elev. -12.5', pile cut-off@ Elev. +16.2'), Ult. Pile Capacity=80.3 tons (Friction)-41.3 tons (Predrilled)=39.0 tons > Required capacity 56K/28T (Extreme Limit Comp), OK

Pile tip at 23.5' below pile cut-off(pile tip@Elev. -7.5', pile cut-off @Elev. +16.2'), Ult. Pile Capacity =67.6 tons (Friction)-41.3 tons (Predrilled)=26.3 tons >Required capacity 38.4K/19.2T (Service Limit Comp), OK

Pile tip at 23.5' below pile cut-off (pile tip@Elev. -7.5', pile cut-off @Elev. +16.2'), Ult. Pile Capacity =43.2 tons (Friction)-16.9 tons(Predrilled)=26.3 tons >Required capacity 19k/9.5T (Extreme Limit Tens), OK

I-5 Swain Road (outside) Abut 1 Wing Wall Class 200 Alt X (4-06-10)

Sheet 2 of 5

Pile Capacity Analyses - Swain Road use (Boring R-09-001) Outside Widening

Abut 2 rw 14" □ precast concrete pile 154 tons service load

Elev 19.42'

Elev 16.42'

~13'

Assumed Toe of Embankment +3.5'

Elev 3.5'

33'

8' — Silty Sand SPT N = 19 γ = 115 pcf (Medium dense)

15' — Lean Clay Avg SPT N = 10, α = 0.85, c = 1250 psf, γ' = 115 pcf

18' — Poorly Graded Sand Avg SPT N = 22, γ' = 115 pcf, γsat = 130 pcf (Medium dense)

8.5' — Lean Clay Avg SPT N = 9, γsat = 125 pcf, c = 1125 psf, α = 1.00

10.5' — Lean Clay Avg SPT N = 13, γsat = 130 pcf, c = 1625 psf, α = 1.0

20' — Lean Clay Avg SPT N = 20, γsat = 130 pcf, c = 2500 psf, α = 0.65, 0.50

10' — Lean Clay SPT N = 40, c = 4000 psf, α = 0.25, γsat = 130 pcf

10' — Lean Clay SPT N = 24, c = 3000 psf, α = 0.25, γsat = 130 pcf

11.5' — Lean Clay Avg SPT N = 40, c = 4000 psf, α = 0.25, γsat = 130 pcf

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Swain Road UC (Widen) (Outside Widening)*** ***Abut 2 Wing Wall- 28 tons Exreme Limt loa*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/06/10

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 2

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 1

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS INCREAMENTS

1- GRAVELS, SANDS (FT)= 5

AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : CPT-09-SR-001 (PARIKH 2009)

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) 4.50 SQ. FT.

AREA OF THE PILE TIP 1.35 SQ. FT.

Class 200 Alt. "X" - 14" square precast prestressed

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	125	375	188	SAND	V. DENSE	0	39	0.00	0.00	0.00	0.00	0.00
3.0	16.0		1	125	1625	1188	SAND	DENSE	0	37	0.00	49.28	49.28	16.51	16.51
16.0	24.0		1	125	1000	2500	SAND	DENSE	0	36	0.00	80.55	129.83 ←	26.98	43.49 ←
24.0	29.0		2	115	575	3288	CLAY	STIFF	1250	0	0.85	11.95	141.78 ←	11.95	55.45
29.0	34.0		2	115	575	3863	CLAY	STIFF	1250	0	0.85	11.95	153.73	11.95	67.40
34.0	39.0		2	115	575	4438	CLAY	STIFF	1250	0	0.85	11.95	165.69	11.95	79.35
39.0	44.0		1	115	575	5013	SAND	MED DEN	0	35	0.00	48.74	214.43	16.33	95.68
44.0	49.0		1	115	575	5588	SAND	MED DEN	0	35	0.00	36.72	251.15	12.30	107.98
49.0	53.0		1	68	272	6011	SAND	MED DEN	0	35	0.00	29.19	280.35	9.78	117.76
53.0	57.0		1	68	272	6283	SAND	MED DEN	0	35	0.00	29.08	309.43	9.74	127.51
57.0	61.0		2	68	272	6555	CLAY	STIFF	1125	0	1.00	10.13	319.56	10.13	137.63
61.0	65.5		2	68	306	6844	CLAY	STIFF	1125	0	1.00	11.39	330.95	11.39	149.02
65.5	70.5		2	68	340	7167	CLAY	STIFF	1625	0	1.00	18.28	349.23	18.28	167.30
70.5	76.0		2	68	374	7524	CLAY	STIFF	1625	0	0.65	13.07	362.30	13.07	180.37
76.0	81.0		2	68	340	7881	CLAY	V.STIFF	2500	0	0.65	18.28	380.58	18.28	198.66
81.0	86.0		2	68	340	8221	CLAY	V.STIFF	2500	0	0.50	14.06	394.64	14.06	212.72
86.0	91.0		2	68	340	8561	CLAY	V.STIFF	2500	0	0.50	14.06	408.71	14.06	226.78
91.0	96.0		2	68	340	8901	CLAY	V.STIFF	2500	0	0.25	7.03	415.74	7.03	233.81
96.0	101.0		2	68	340	9241	CLAY	V.STIFF	4000	0	0.25	11.25	426.99	11.25	245.06
101.0	106.0		2	68	340	9581	CLAY	V.STIFF	4000	0	0.25	11.25	438.24	11.25	256.31
106.0	111.0		2	68	340	9921	CLAY	V.STIFF	3000	0	0.25	8.44	446.67	8.44	264.75
111.0	126.0		2	68	1020	10601	CLAY	V.STIFF	3000	0	0.25	25.31	471.99	25.31	290.06

Abut 2 WW--> Pile cut-off elevation is +16.42 feet. Assumed finished grade elevation is +19.42 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +3.5 feet

Pile Length @ Abutment 2 = 29.0' - 3.0' = 26.0' (Extreme Limit Comp), 24.0'-3.0'=21.0' (Service Limit), 24.0'-3.0' = 21.0' (Extreme Limit Tension)

Pile tip at 26.0' below pile cut-off(pile tip @Elev. -9.5', pile cut-off @Elev. +16.42'), Ult. Pile Capacity =141.8 tons (Friction)-49.3 tons (Predrilled)=92.5 tons >Required capacity 56K/28T (Extreme Limit Comp), OK

Pile tip at 21.0' below pile cut-off(pile tip @Elev. -4.5', pile cut-off@Elev. +16.42'), Ult. Pile Capacity =129.8 tons(Friction)-49.3 tons (Predrilled)=80.5 tons >Required capacity 38.4K/19.2T (Service Limit Comp), OK

Pile tip at 21.0' below pile cut-off(pile tip @Elev. -4.5', pile cut-off @Elev. +16.42'), Ult. Pile Capacity =43.5 tons (Friction)-16.5 tons (Predrilled)=27.0 tons >Required capacity 19k/9.5T (Extreme Limit Tens), OK

I-5 Swain Road (outside) Abut 2 Wing Wall Class 200 Alt X (4-06-10)

Sheet 4 of 5

SWAIN ROAD UC (WIDEN)

Sheet 5 of 5

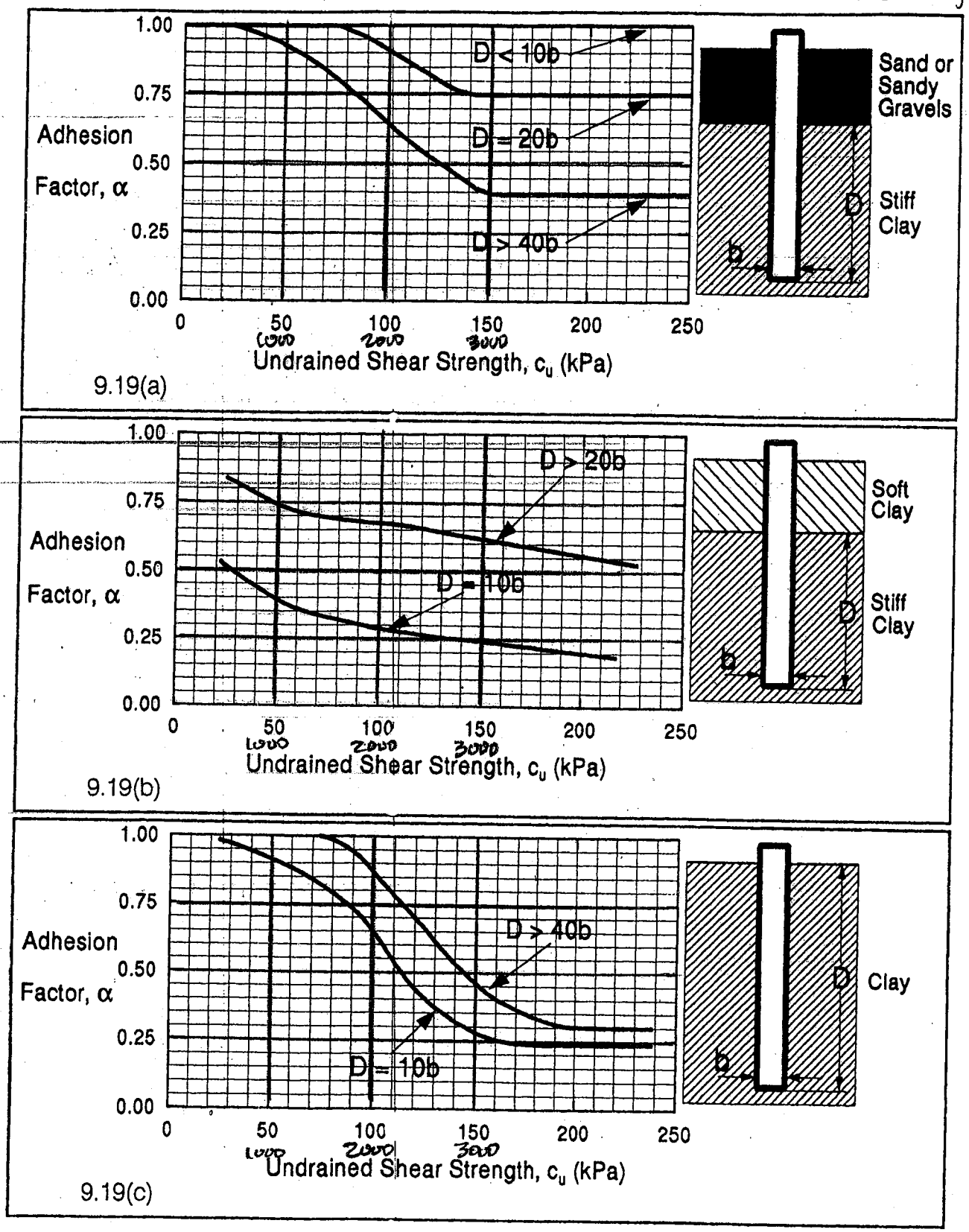
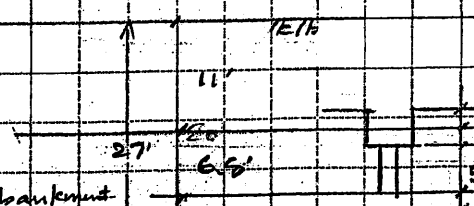


Figure 9.19 Adhesion Factors for Driven Piles in Clay (after Tomlinson, 1980)

(Inside Widening)

Lateral Pile Capacity Analysis - Swain Road UC (Boring A-09-001) (Hwy 5) Abutment

14" □ precast concrete pile - 159 tons service load



Poorly Graded Sand / Silty Sand
K = 225 lb/in³ Avg SPT N = 80 Elev. 11.5' γ' = 120 pcf

Poorly Graded Sand / Silty Sand Avg SPT N = 23
K = 90 lb/in³ γ' = 120 pcf

Lean Clay Avg SPT N = 11
c = 1250 psf γ' = 115 pcf
ε50 = 0.007 γsat = 130 pcf

K = 60 lb/in³ φ' = 33° Poorly Graded Sand SPT N = 23 (Medium dense) γsat = 130 pcf

c = 2500 psf Lean Clay Avg SPT N = 22
ε50 = 0.005 γsat = 130 pcf

c = 4000 psf Lean Clay Avg SPT N = 45
ε50 = 0.005 γsat = 130 pcf

Lean Clay SPT N = 28
c = 3250 psf γsat = 130 pcf

Lean Clay Avg SPT N > 50
γsat = 130 pcf

Poorly Graded Sand with Silt Avg SPT N > 50
γsat = 135 pcf

```
LPILEP5
I-5 Stockton-Swain-Abut-1 (median)
1       1       0       0       0       0
100     2       -36     480     0
0       14      3112    194     3000000
480     14      3112    194     3000000
7       14      14      0       2
4       -36     1       225     225
4       1       60      90      90
3       60      174     0       0
3       174     294     0       0
4       294     354     60      60
3       354     492     0       0
3       492     642     0       0
-36     0.069
1       0.069
1       0.069
60      0.069
60      0.067
174     0.067
174     0.039
294     0.039
294     0.039
354     0.039
354     0.039
492     0.039
492     0.039
642     0.039
-36     0       36      0       0
1       0       36      0       0
1       0       36      0       0
60      0       36      0       0
60      8.68    0       0.007   0
174     8.68    0       0.007   0
174     8.68    0       0.007   0
294     8.68    0       0.007   0
294     0       33      0       0
354     0       33      0       0
354     17.36   0       0.005   0
492     17.36   0       0.005   0
492     27.77   0       0.005   0
642     27.77   0       0.005   0
0       0.6     1
1248    0.6     1
0       1       1
1
2       30000   0       143000
0
1       1       0
100     1E-5    100
```

Depths -36 - 1 = Reese Sand

Depths 1 - 60 = Reese Sand

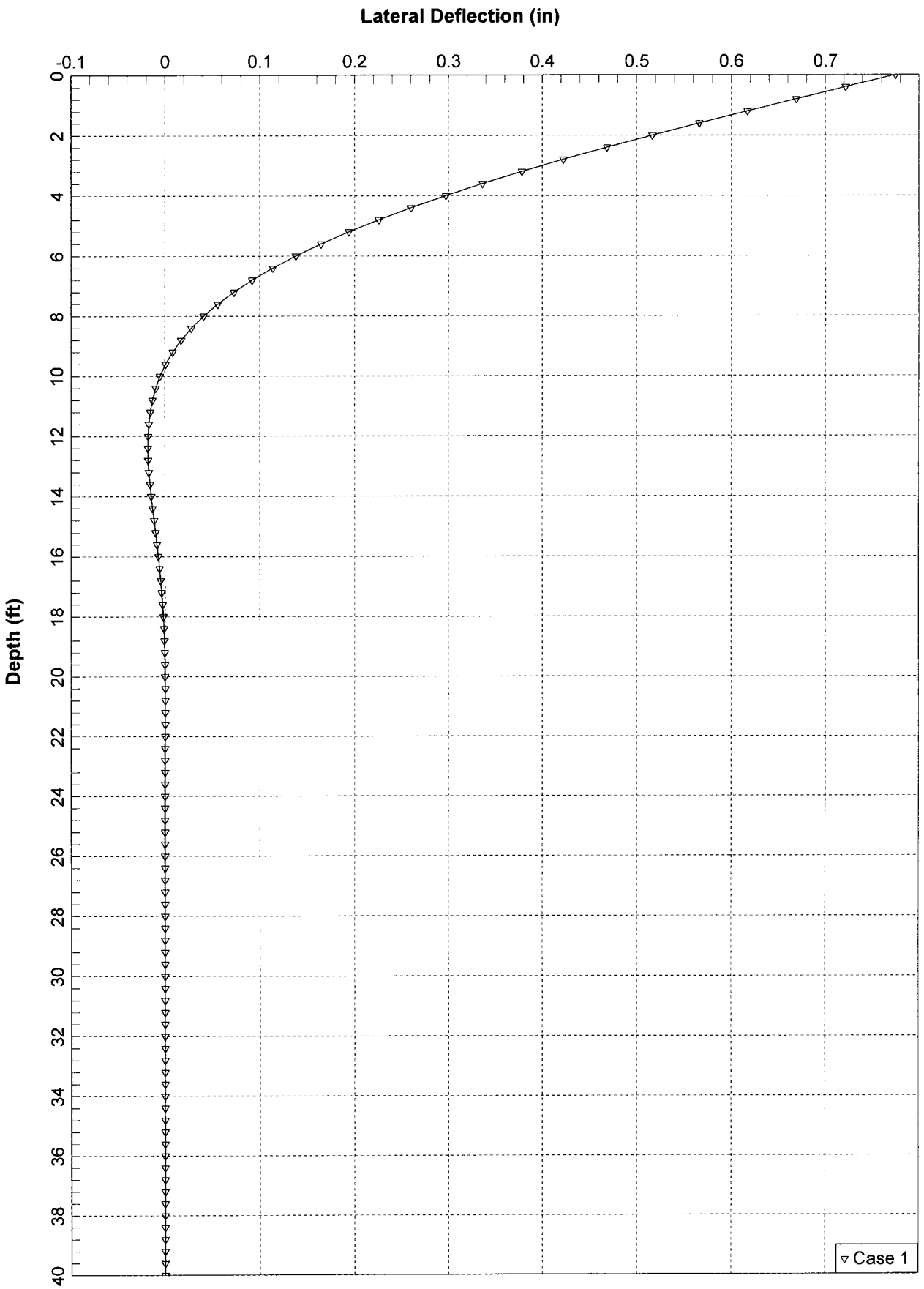
Depths 60 - 174 = Stiff Clay w/o free water

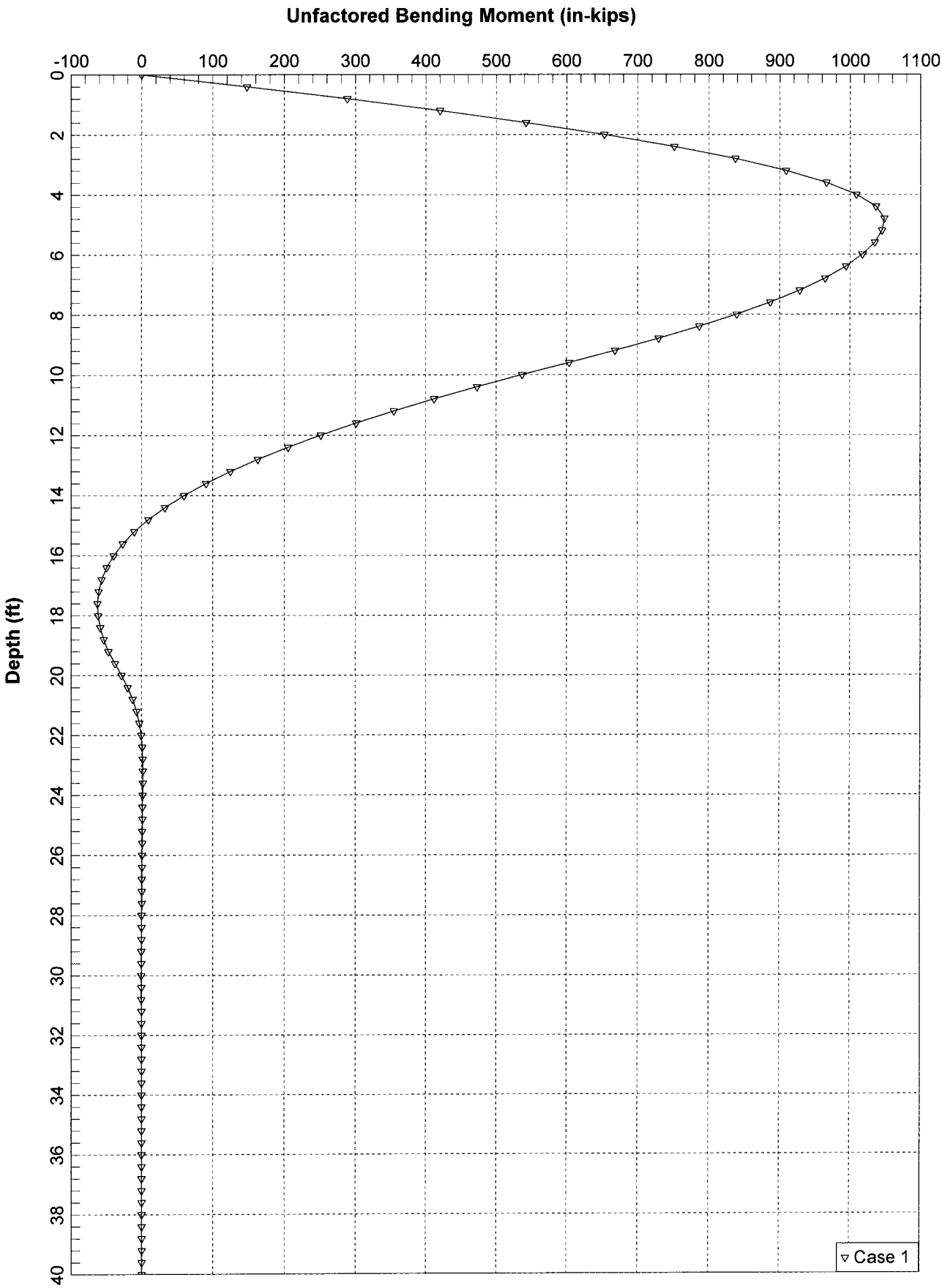
Depths 174 - 294 = Stiff Clay w/o free water

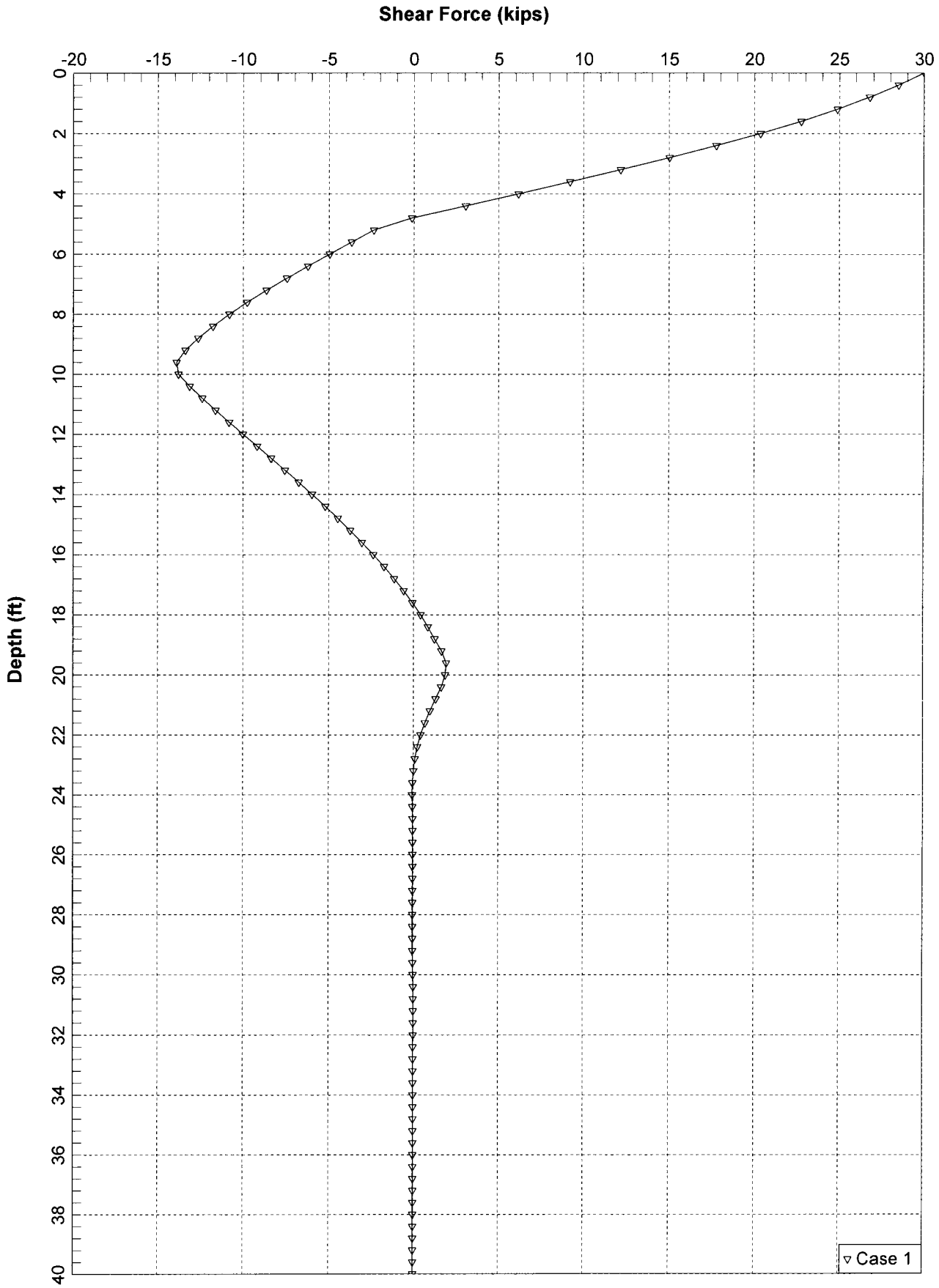
Depths 294 - 354 = Reese Sand

Depths 354 - 492 = Stiff Clay w/o free water

Depths 492 - 642 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

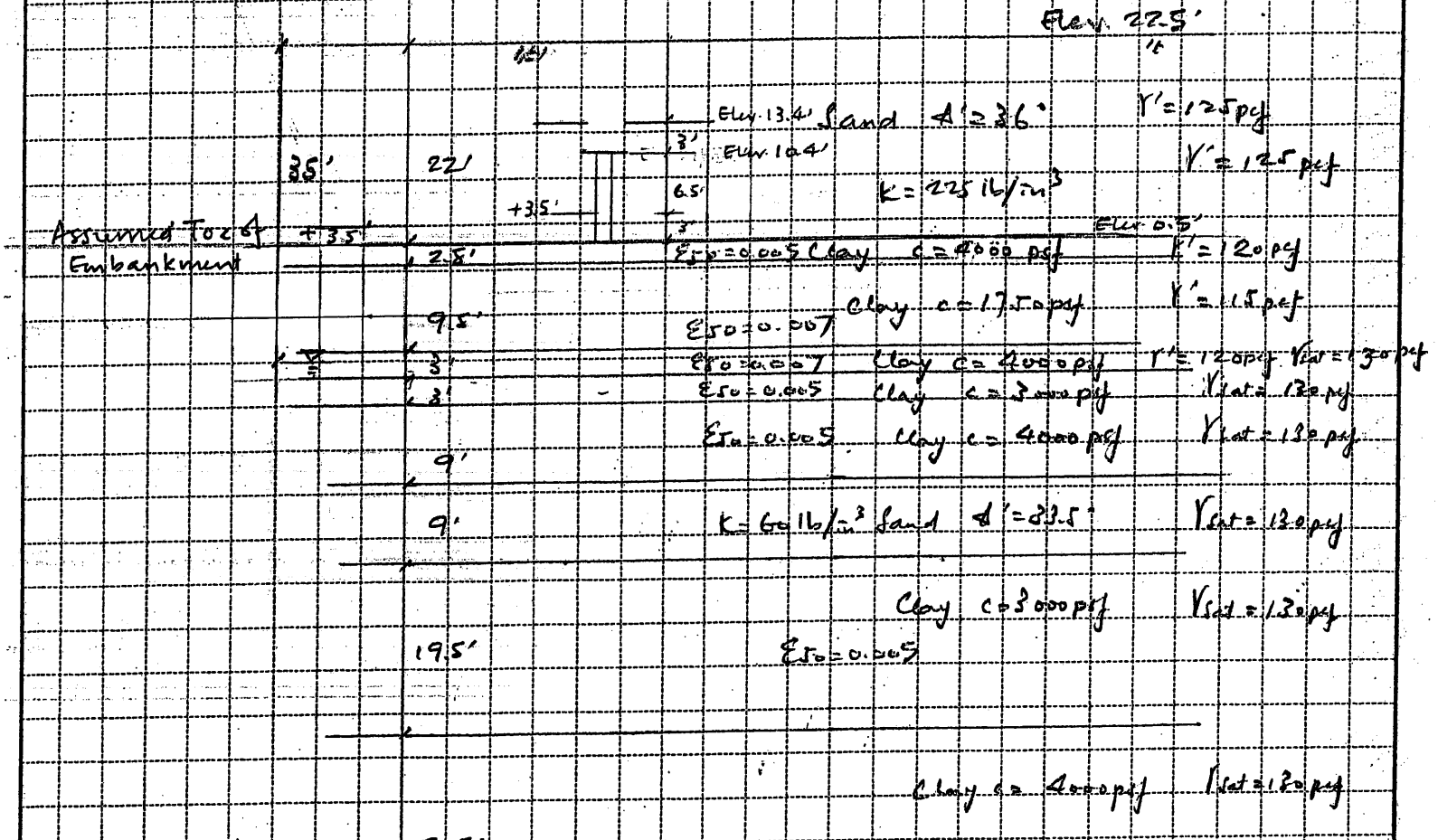

Unfactored Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

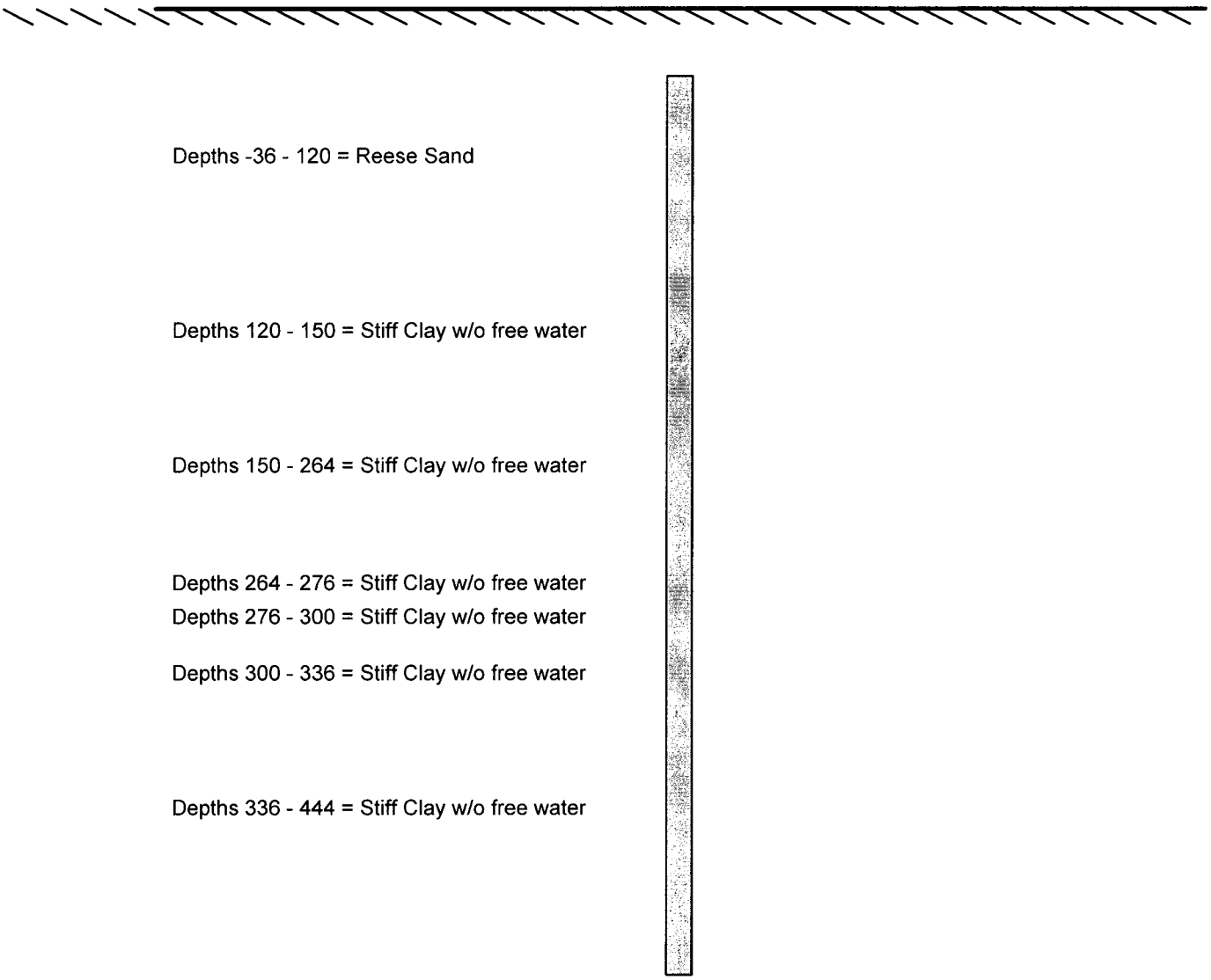
PARIKH
Practicing in the Geosciences

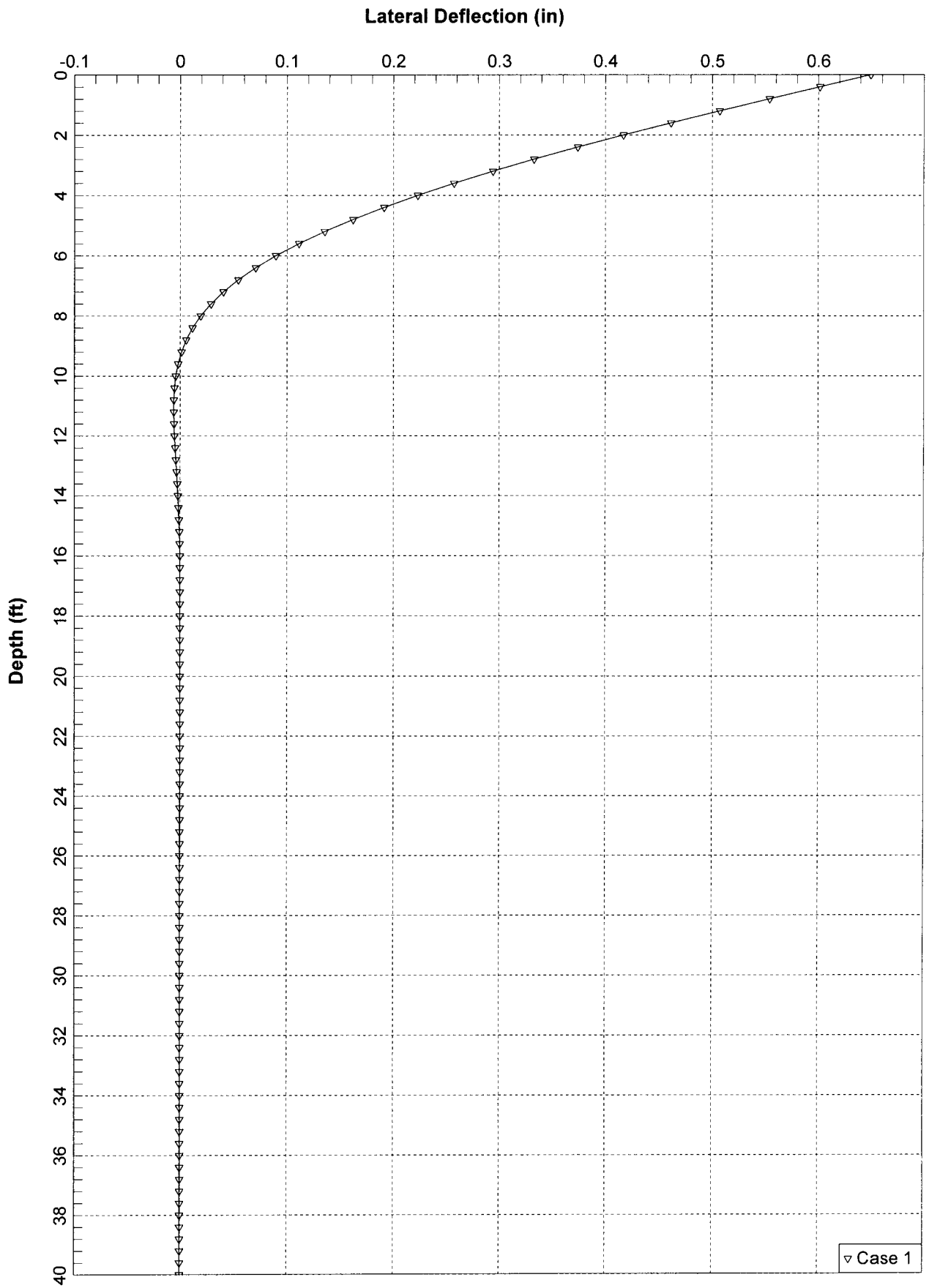
JOB I-5 North Stockton Corridor Improvement (Widening)
SHEET NO _____ OF _____
CALCULATED BY A Lam DATE 4/2010
CHECKED BY _____ DATE _____
SCALE _____

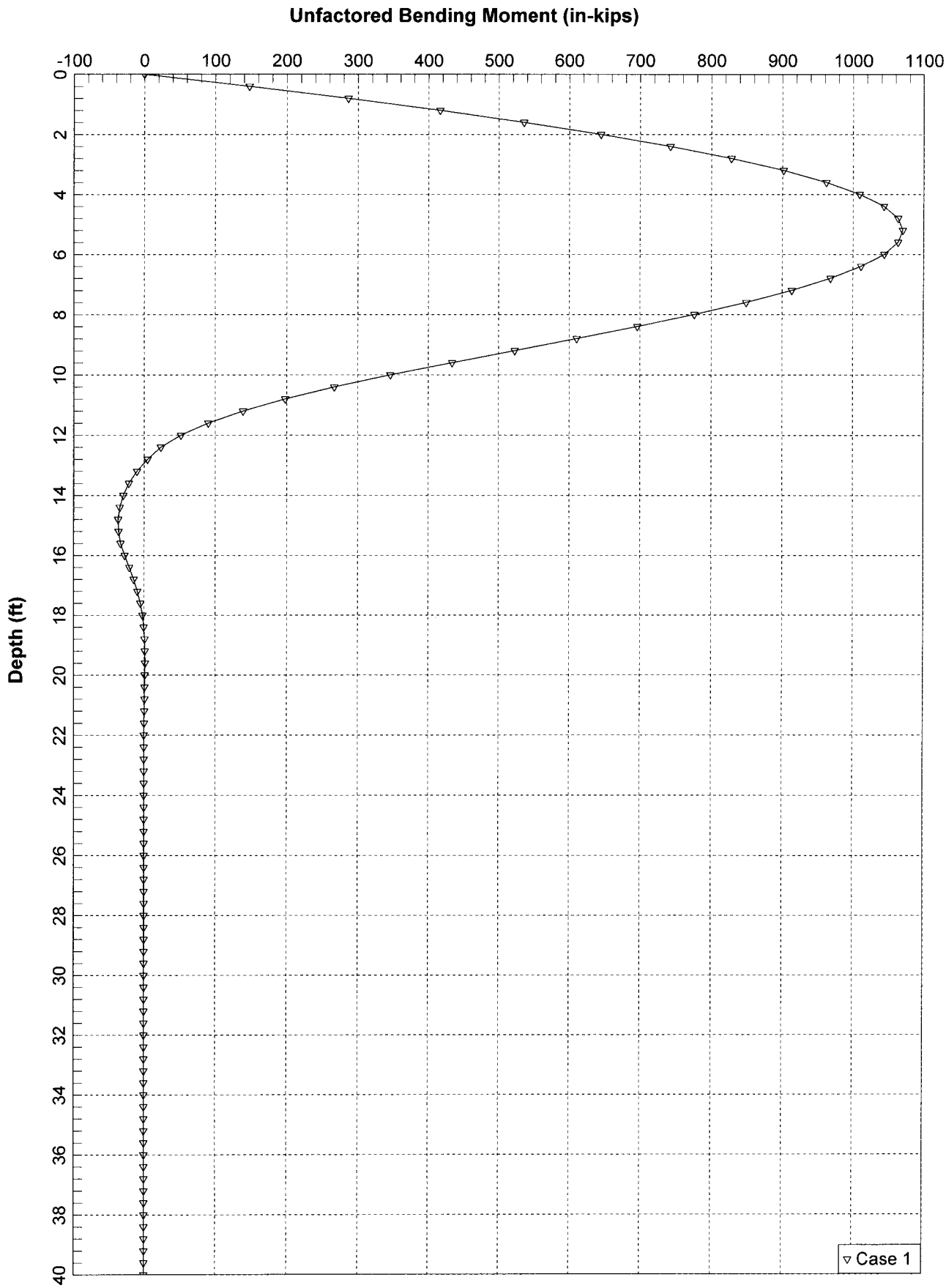
Lateral Pile Capacity Analysis Sinclair Road UC (Widen) - CPT-09-001 Abutment 2 Inside Widening
14" □ precast concrete pile 159 tons service load

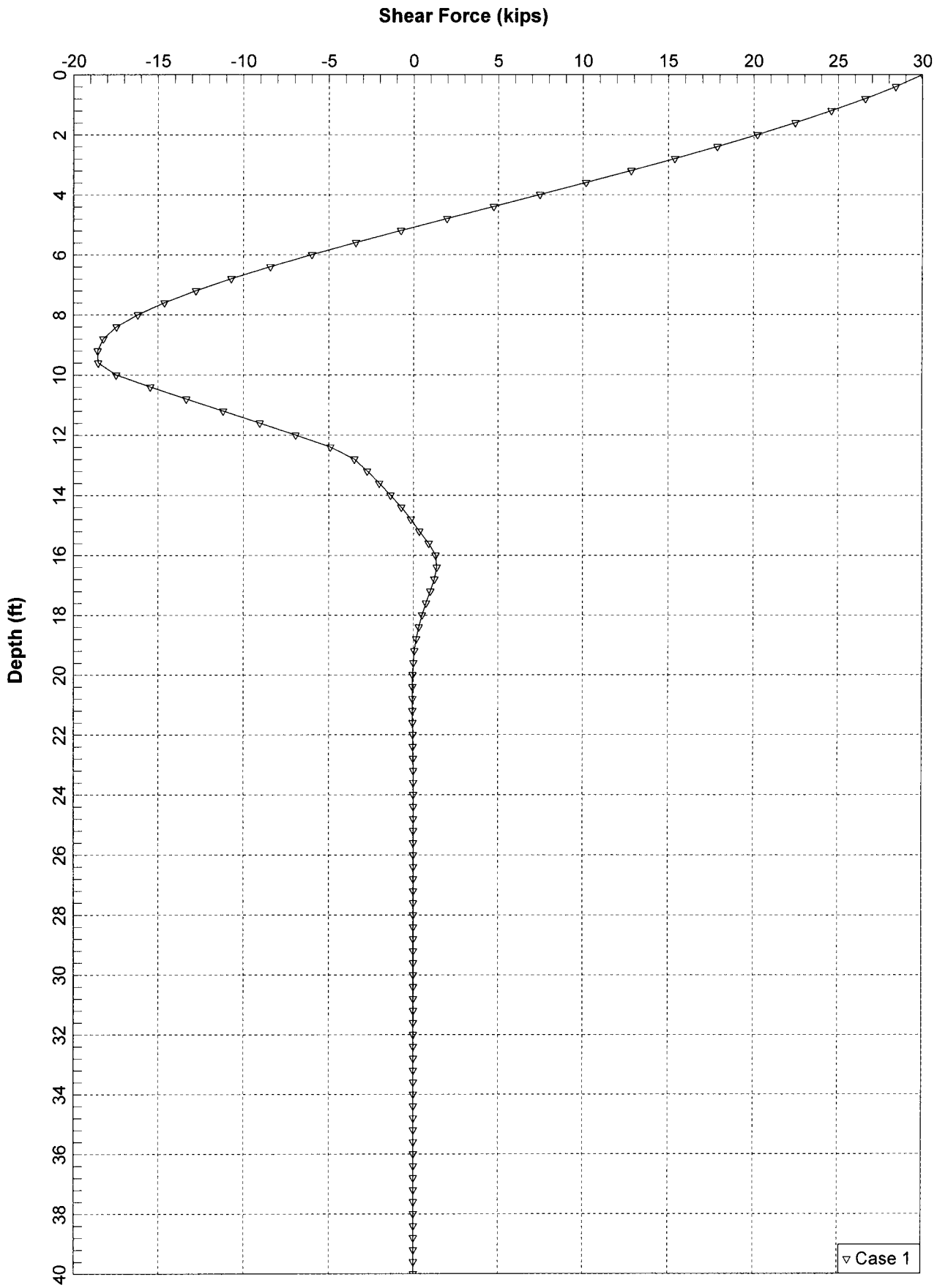


```
LPILEP5
I-5 Stockton-Swain-Abut-2 (Median)
1       1       0       0       0       0
100     2       -36     480     0
0       14      3112    194     3000000
480     14      3112    194     3000000
9       18      18      0       2
4       -36     120     225     225
3       120     150     0       0
3       150     264     0       0
3       264     276     0       0
3       276     300     0       0
3       300     336     0       0
3       336     444     0       0
4       444     552     60      60
3       552     786     0       0
-36     0.072
120     0.072
120     0.069
150     0.069
150     0.067
264     0.067
264     0.069
276     0.069
276     0.039
300     0.039
300     0.039
336     0.039
336     0.039
444     0.039
444     0.039
552     0.039
552     0.039
786     0.039
-36     0       36      0       0
120     0       36      0       0
120     27.77   0       0.005   0
150     27.77   0       0.005   0
150     12.15   0       0.007   0
264     12.15   0       0.007   0
264     27.77   0       0.005   0
276     27.77   0       0.005   0
276     27.77   0       0.005   0
300     27.77   0       0.005   0
300     20.83   0       0.005   0
336     20.83   0       0.005   0
336     27.77   0       0.005   0
444     27.77   0       0.005   0
444     0       33.5    0       0
552     0       33.5    0       0
552     20.83   0       0.005   0
786     20.83   0       0.005   0
0       0.6     1
1206    0.6     1
0       1       1
1
1       30000   0       143000
0
1       1       0
100     1E-5    100
```


Depths -36 - 120 = Reese Sand
Depths 120 - 150 = Stiff Clay w/o free water
Depths 150 - 264 = Stiff Clay w/o free water
Depths 264 - 276 = Stiff Clay w/o free water
Depths 276 - 300 = Stiff Clay w/o free water
Depths 300 - 336 = Stiff Clay w/o free water
Depths 336 - 444 = Stiff Clay w/o free water
Depths 444 - 552 = Reese Sand
Depths 552 - 786 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Unfactored Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

Lateral Pile Capacity Analysis - Swain Road UC (Widen) (Median Widening)

Abutment 1

Depth of Second Point of Contraflexure = 19'

Design pile tip elevation = 10.2' - 19'

= -8.8' Say - 9.0 feet

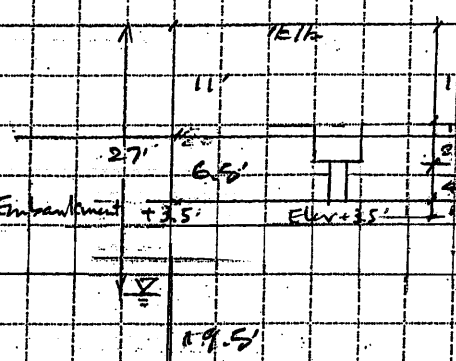
Abutment 2

Depth of Second Point of Contraflexure = 16.5'

Design pile tip elevation = 10.4' - 16.5'

= -6.1' Say - 6.5 feet

Lateral Pile Capacity Analysis - Swain Road UC (Boring A-09-010) (Hwy 5) (Outside widening Abutment) 14" ø precast concrete pile · 154 tons service load



Elev +22.5'

Poorly Graded Sand (Silty sand) $\phi' = 35.5°$
Avg SPT N = 30 $\gamma' = 120$ pcf

Poorly Graded Sand (Silty sand) Avg SPT N = 23
$k = 96$ lb/in^3 $\gamma' = 120$ pcf

Lean Clay Avg SPT N = 11
$\varepsilon_{50} = 0.007$ $\gamma' = 115$ pcf
$c = 1250$ psf $\gamma_{sat} = 130$ pcf

19.5'
$k = 60$ lb/in^3 $\phi' = 33°$ Poorly Graded Sand SPT N = 23 (Medium dense) $\gamma_{sat} = 130$ pcf

5'

11.5'
$c = 2500$ psf Lean Clay Avg SPT N = 22
$\varepsilon_{50} = 0.005$ $\gamma_{sat} = 130$ pcf

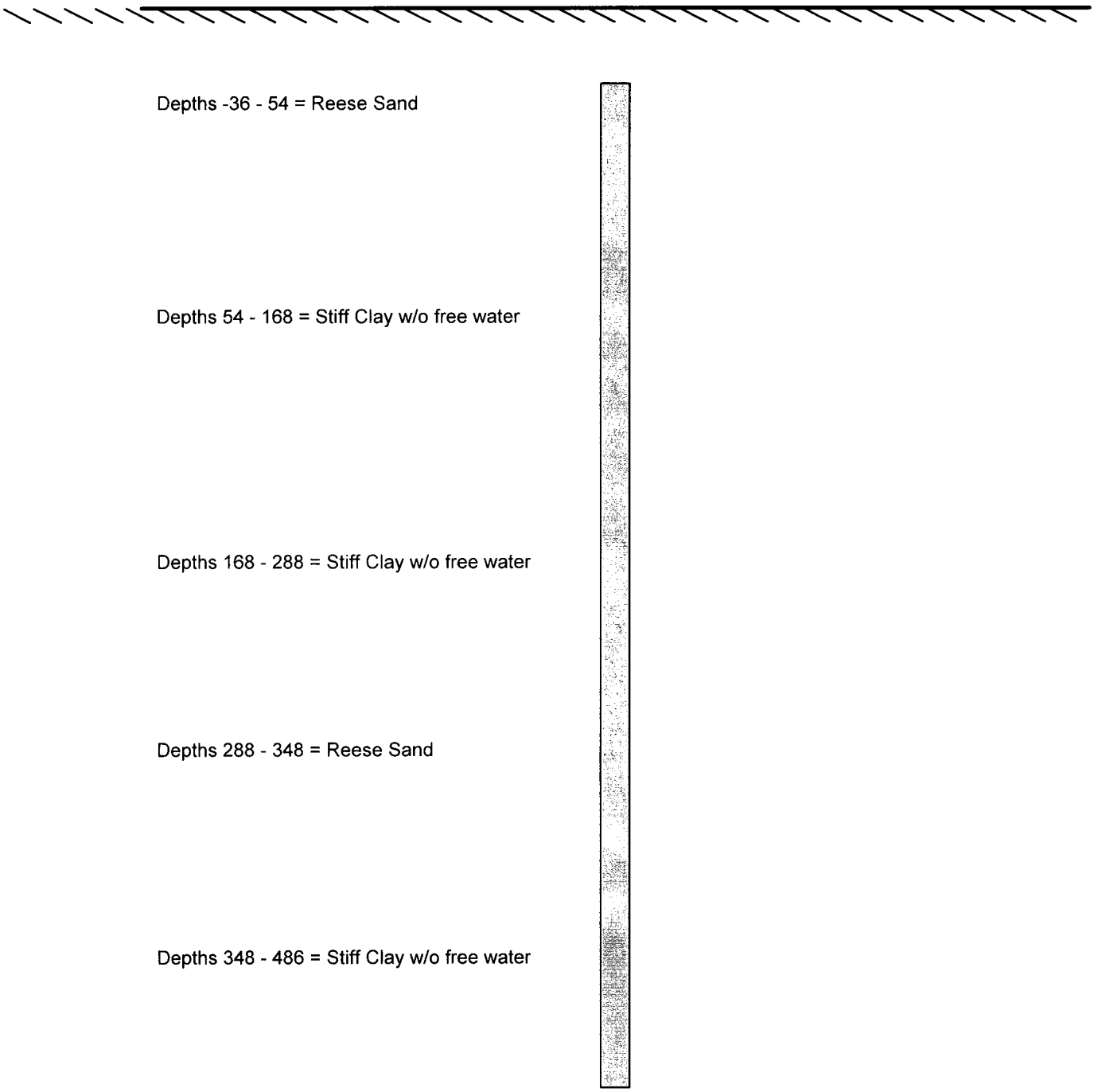
12.5'
$c = 4000$ psf Lean Clay Avg SPT N = 45
$\varepsilon_{50} = 0.005$ $\gamma_{sat} = 130$ pcf

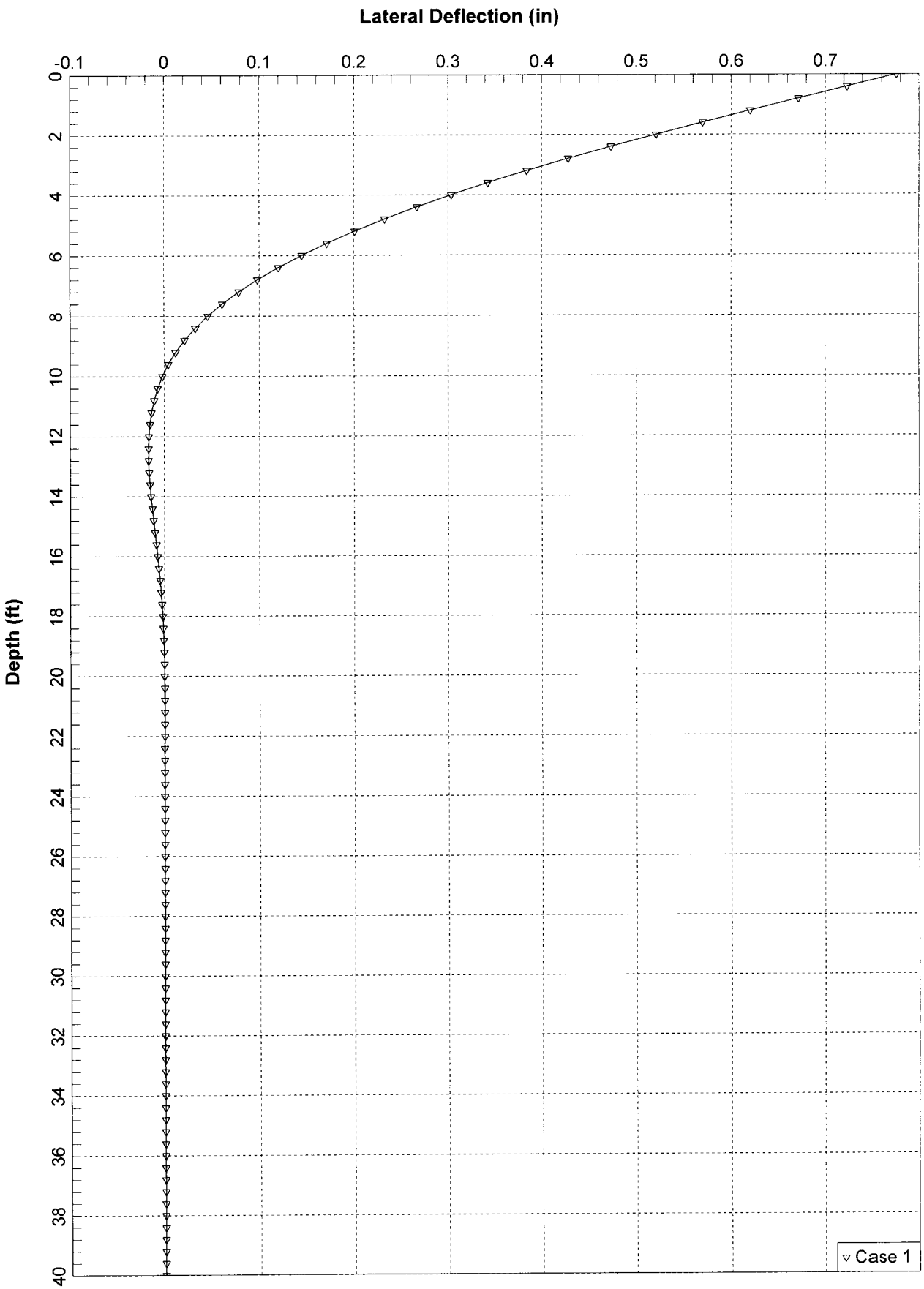
14'
Lean Clay SPT N = 28
$c = 3250$ psf $\gamma_{sat} = 130$ pcf

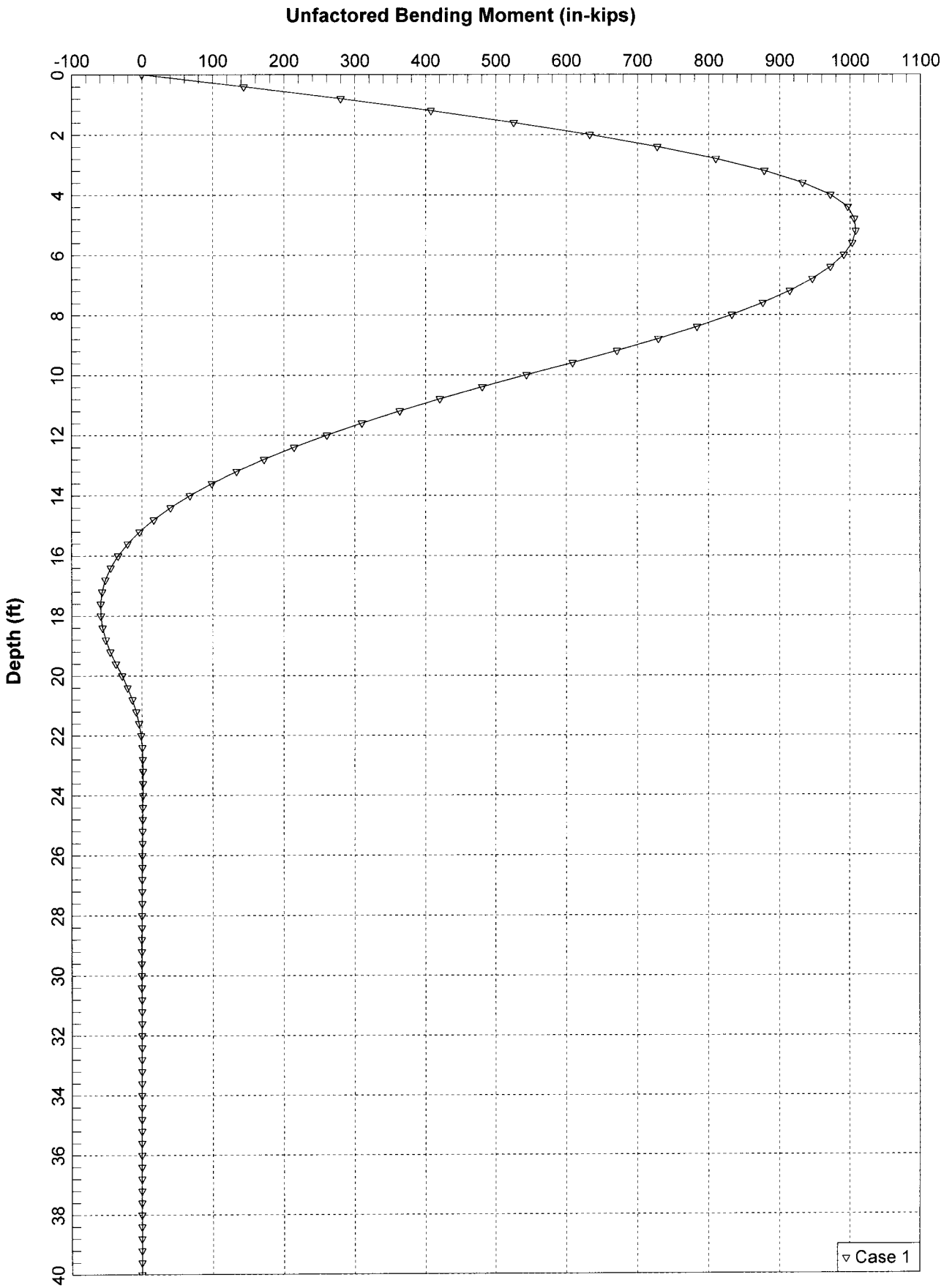
20'
Lean Clay Avg SPT N > 50
$\gamma_{sat} = 130$ pcf

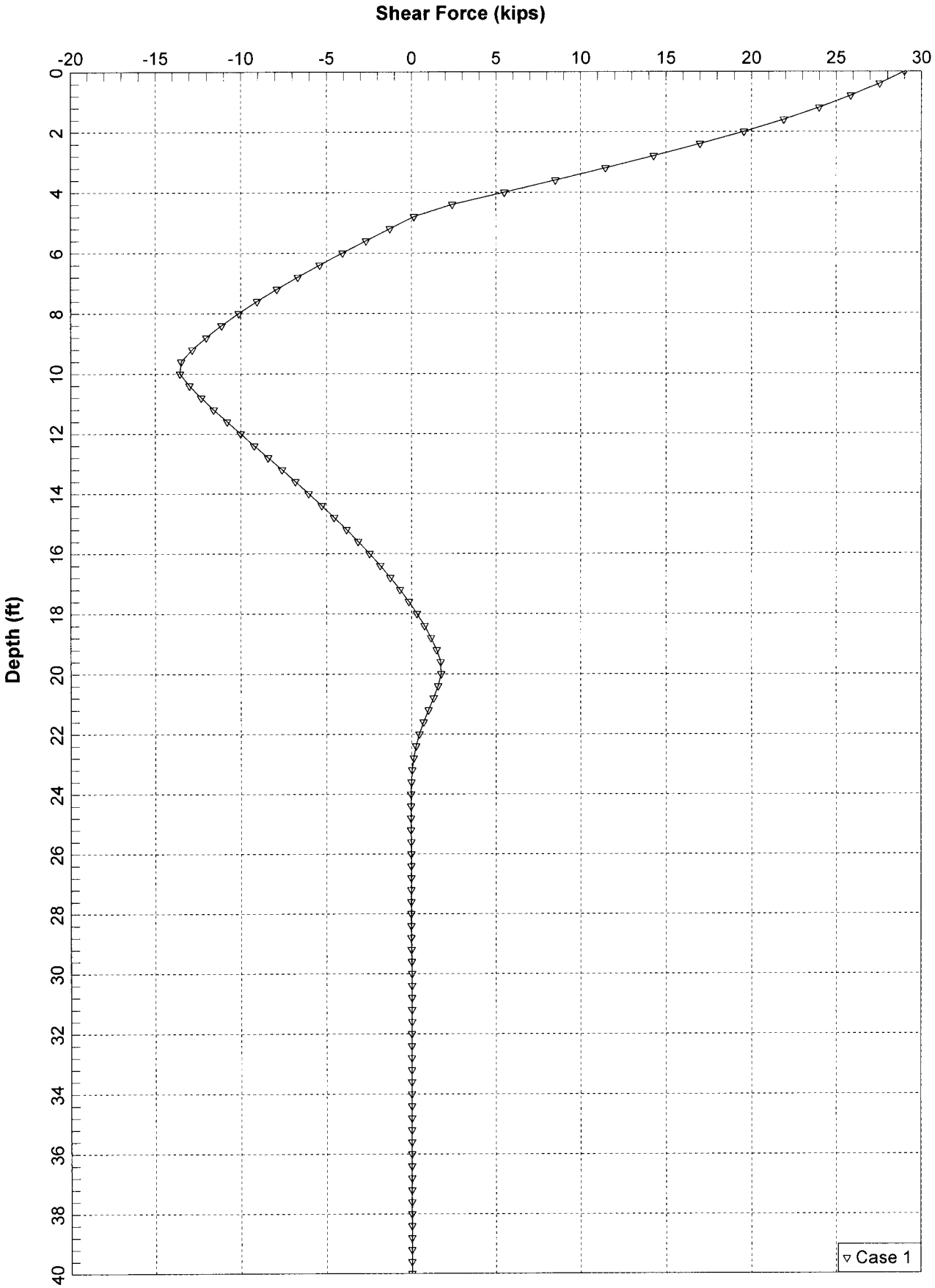
11.5'
Poorly Graded Sand with Silt Avg SPT N > 50
$\gamma_{sat} = 135$ pcf

```
LPILEP5
I-5 Stockton-Swain-Abut-1(outside)
1       1       0       0       0       0
100     2       -36     480     0
0       14      3112    194     3000000
480     14      3112    194     3000000
6       12      12      0       2
4       -36     54      90      90
3       54      168     0       0
3       168     288     0       0
4       288     348     60      60
3       348     486     0       0
3       486     636     0       0
-36     0.069
54      0.069
54      0.067
168     0.067
168     0.039
288     0.039
288     0.039
348     0.039
348     0.039
486     0.039
486     0.039
636     0.039
-36     0       35.5    0       0
54      0       35.5    0       0
54      8.68    0       0.007   0
168     8.68    0       0.007   0
168     8.68    0       0.007   0
288     8.68    0       0.007   0
288     0       33      0       0
348     0       33      0       0
348     17.36   0       0.005   0
486     17.36   0       0.005   0
486     27.77   0       0.005   0
636     27.77   0       0.005   0
0       0.6     1
1248    0.6     1
0       1       1
1
1       29000   0       150000
0
1       1       0
100     1E-5    100
```

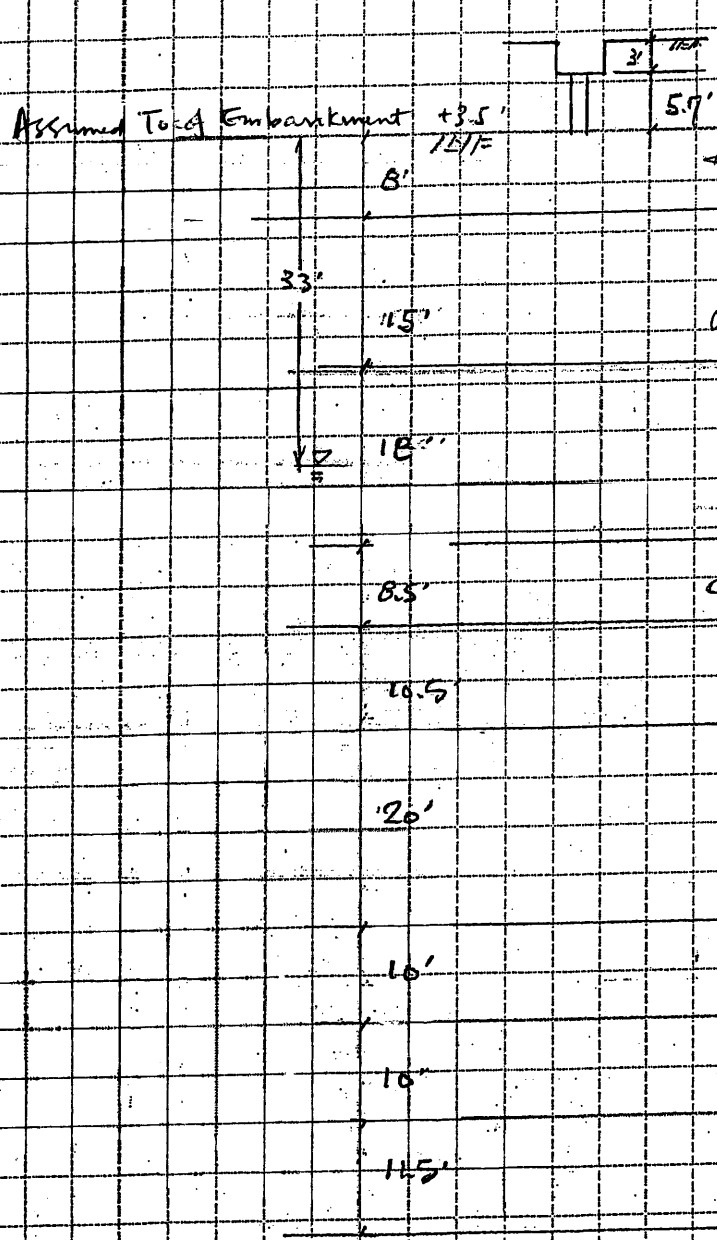

Depths -36 - 54 = Reese Sand
Depths 54 - 168 = Stiff Clay w/o free water
Depths 168 - 288 = Stiff Clay w/o free water
Depths 288 - 348 = Reese Sand
Depths 348 - 486 = Stiff Clay w/o free water
Depths 486 - 636 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Unfactored Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

Lateral Pile Capacity Analysis - Swain Road UC (Boring R-09-001) Outside Widening
Abut 2 14" □ precast concrete pile 154 tons service load



$\phi = 36°$ Silty Sand SPT N = 19 $\gamma' = 115$ pcf
(Medium dense) $k = 90$ lb/in³

Lean Clay Avg SPT N = 10
$\alpha = 0.85$ $c = 1250$ psf $\gamma' = 115$ pcf
$\varepsilon_{50} = 0.007$

Poorly graded Sand Avg SPT N = 22
(Medium dense) $\gamma' = 115$ pcf $\gamma_{sat} = 130$ pcf
$k = 90$ lb/in³ $\phi' = 35°$ $k = 60$ lb/in³ (below water)

Lean Clay Avg SPT N = 9 $\gamma_{sat} = 125$ pcf
$c = 1125$ psf $\alpha = 1.00$ $\varepsilon_{50} = 0.007$

Lean Clay Avg SPT N = 13
$\gamma_{sat} = 130$ pcf
$c = 1625$ psf $\alpha = 1.0$ $\varepsilon_{50} = 0.007$

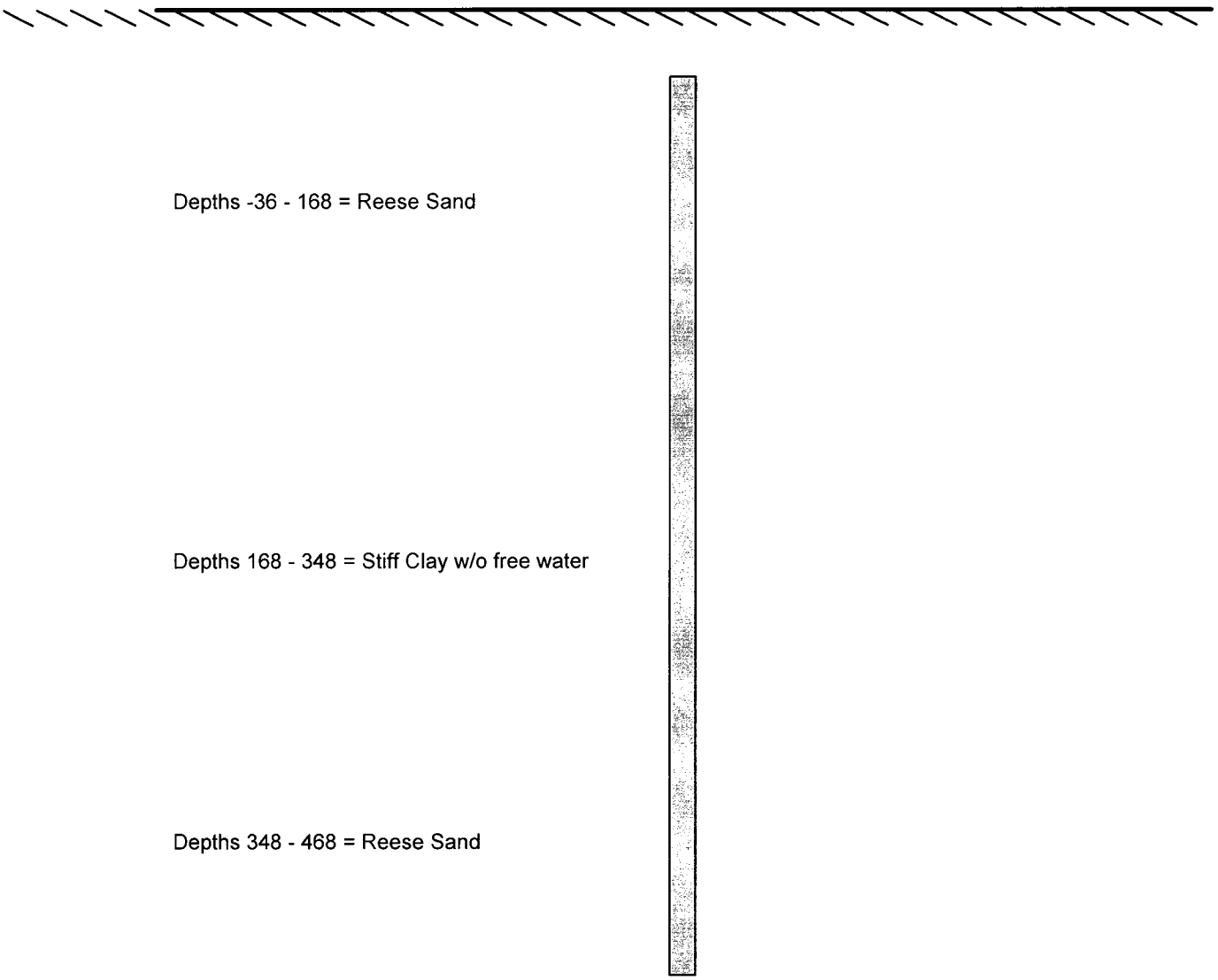
Lean Clay Avg SPT N = 20
$\gamma_{sat} = 130$ pcf
$c = 2500$ psf $\alpha = 0.65, 0.50$

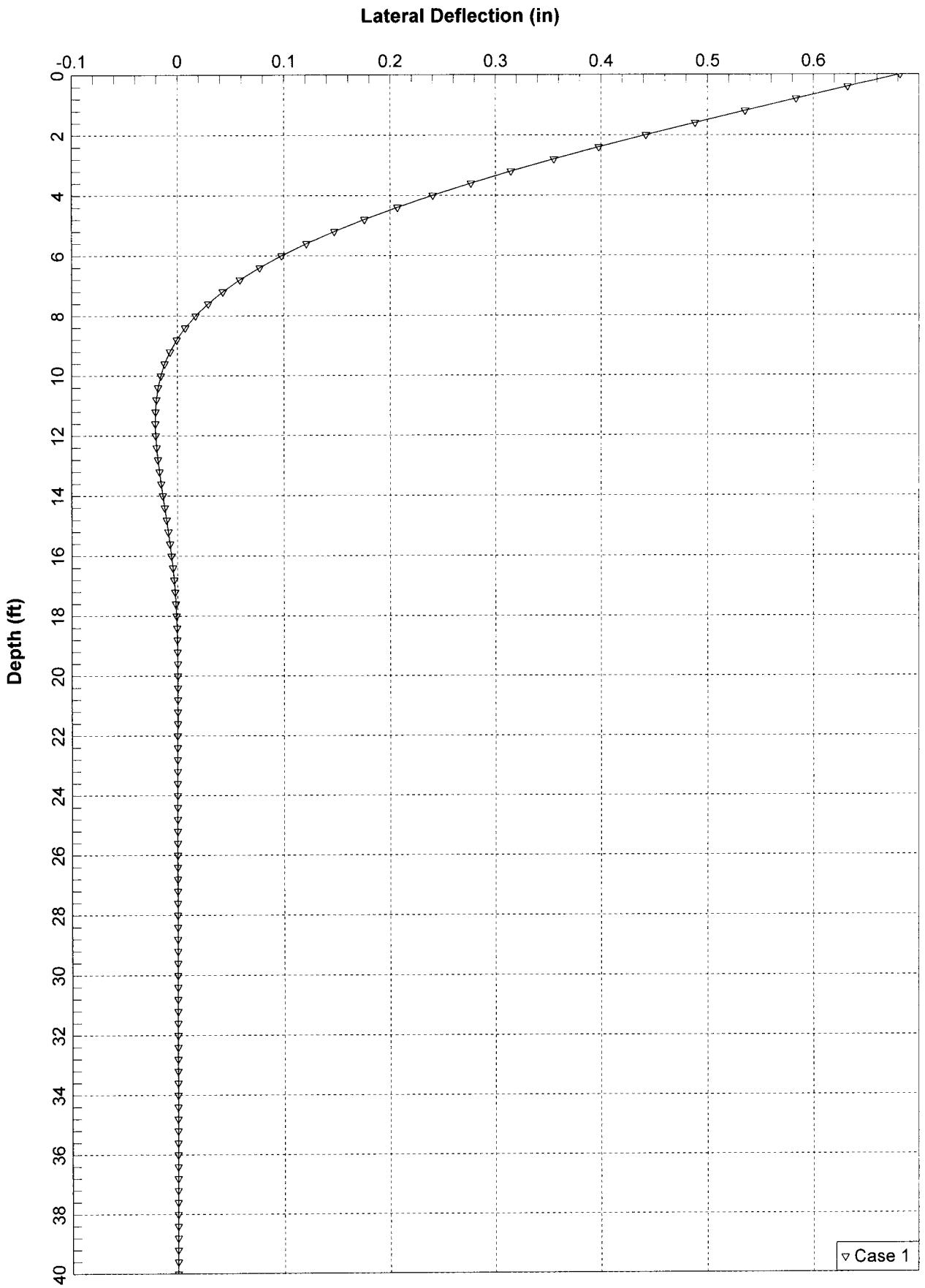
Lean Clay SPT N = 40
$c = 4000$ psf $\alpha = 0.25$ $\gamma_{sat} = 130$ pcf

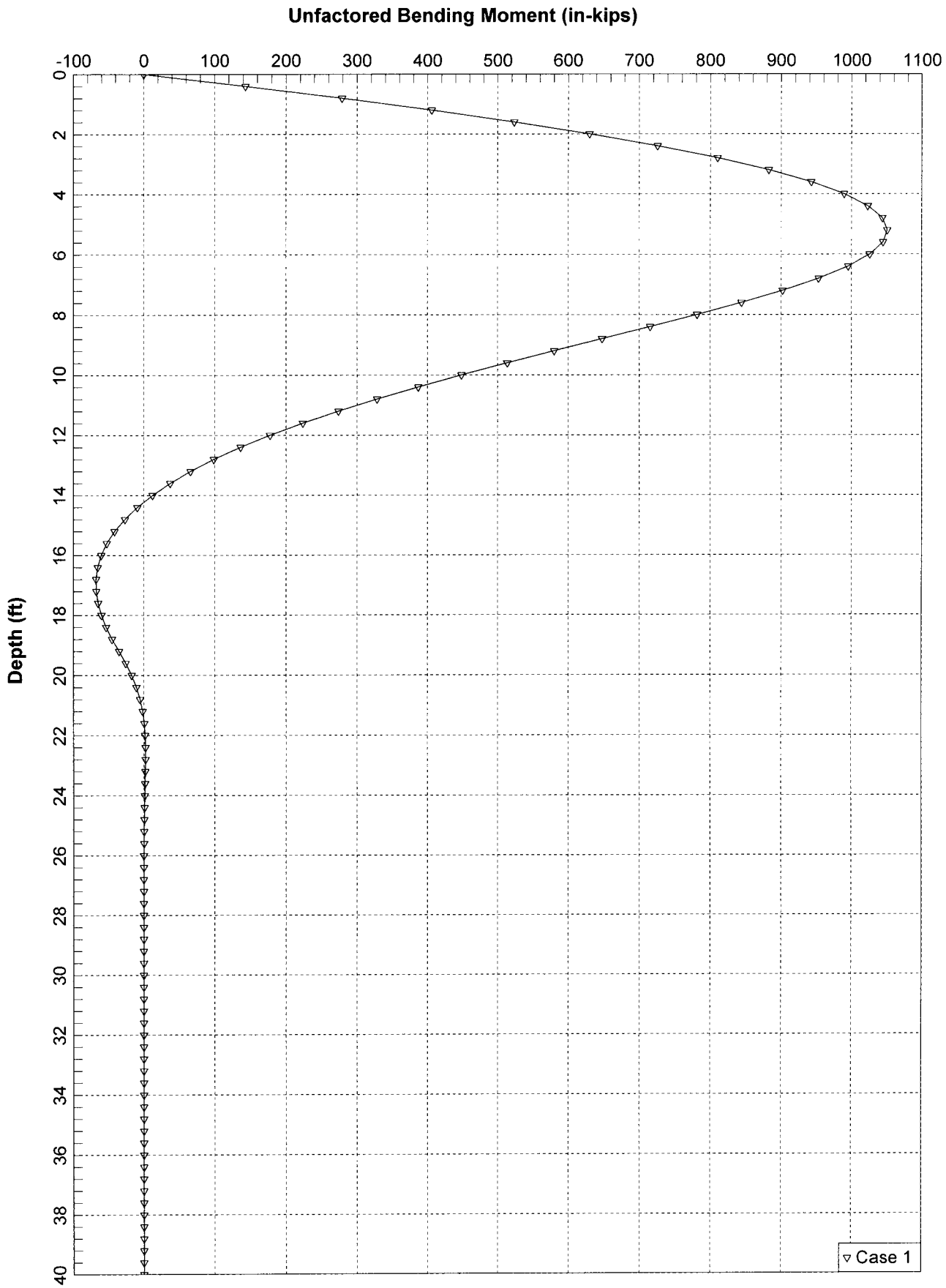
Lean Clay SPT N = 24
$c = 3000$ psf $\alpha = 0.25$ $\gamma_{sat} = 130$ pcf

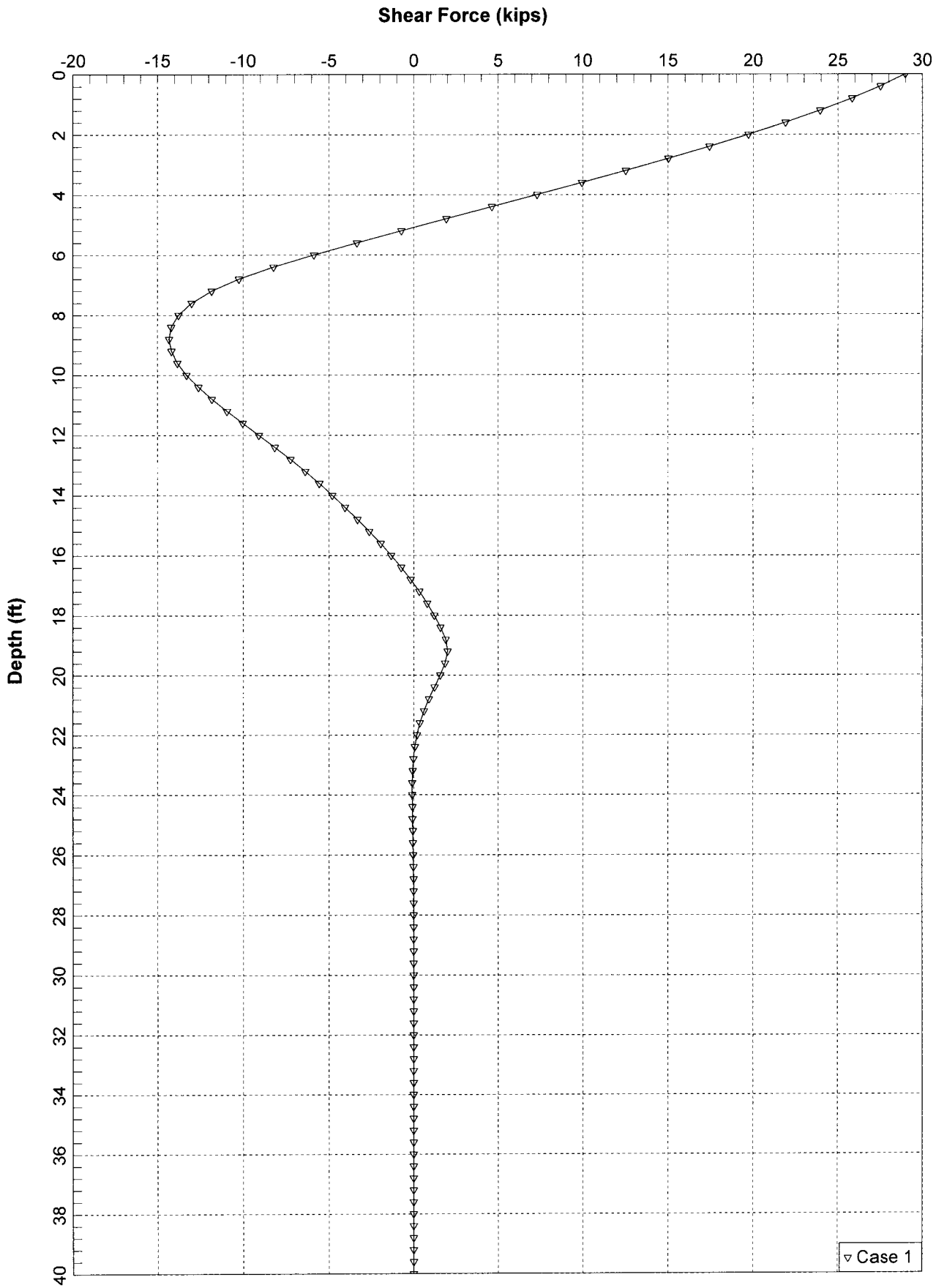
Lean Clay Avg SPT N = 40
$c = 4000$ psf $\alpha = 0.25$ $\gamma_{sat} = 130$ pcf

```
LPILEP5
I-5 Stockton-Swain-Abut-2(Outside)
1       1       0       0       0       0
100     2       -36     480     0
0       14      3112    194     3000000
480     14      3112    194     3000000
6       12      12      0       2
4       -36     168     90      90
3       168     348     0       0
4       348     468     90      90
4       468     564     60      60
3       564     666     0       0
3       666     792     0       0
-36     0.067
168     0.067
168     0.067
348     0.067
348     0.067
468     0.067
468     0.039
564     0.039
564     0.036
666     0.036
666     0.039
792     0.039
-36     0       36      0       0
168     0       36      0       0
168     8.7     0       0.007   0
348     8.7     0       0.007   0
348     0       35      0       0
468     0       35      0       0
468     0       35      0       0
564     0       35      0       0
564     7.8     0       0.007   0
666     7.8     0       0.007   0
666     11.3    0       0.007   0
792     11.3    0       0.007   0
0       0.6     1
1206    0.6     1
0       1       1
1
1       29000   0       150000
0
1       1       0
100     1E-5    100
```


Depths -36 - 168 = Reese Sand
Depths 168 - 348 = Stiff Clay w/o free water
Depths 348 - 468 = Reese Sand
Depths 468 - 564 = Reese Sand
Depths 564 - 666 = Stiff Clay w/o free water
Depths 666 - 792 = Stiff Clay w/o free water


Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Unfactored Bending Moment (in-kips)
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

Lateral Pile Capacity Analyses - Swain Road UC (Widening) (Outside Widening)

Abutment 1

Depth of Second Point of Counterflexure = 20'

Design pile tip elevation = 9.0' - 20.0'

= -11.0 feet

Abutment 2

Depth of Second Point of Counterflexure = 19.2

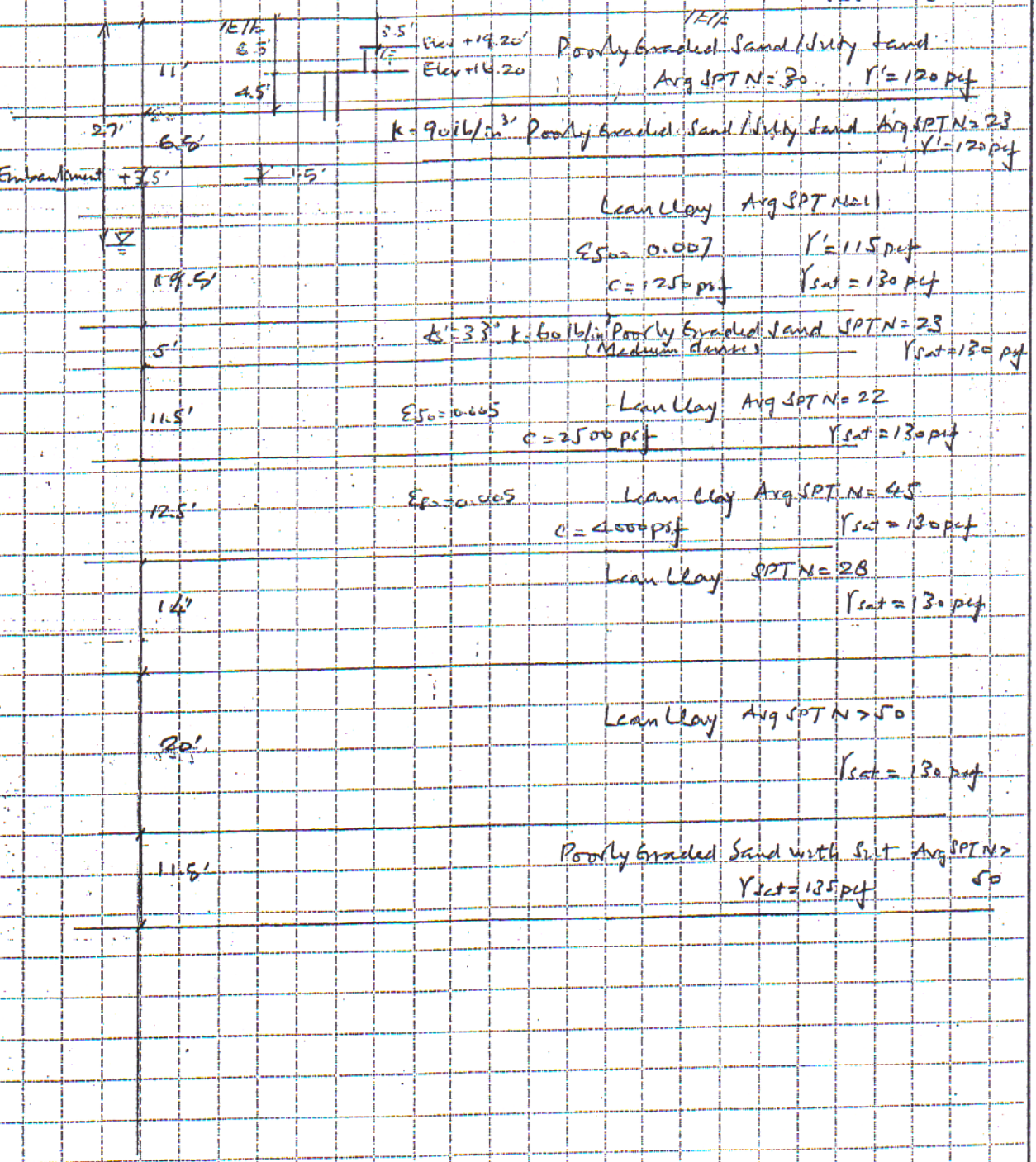
Design pile tip elevation = 9.2 - 19.2

= -10.0 feet

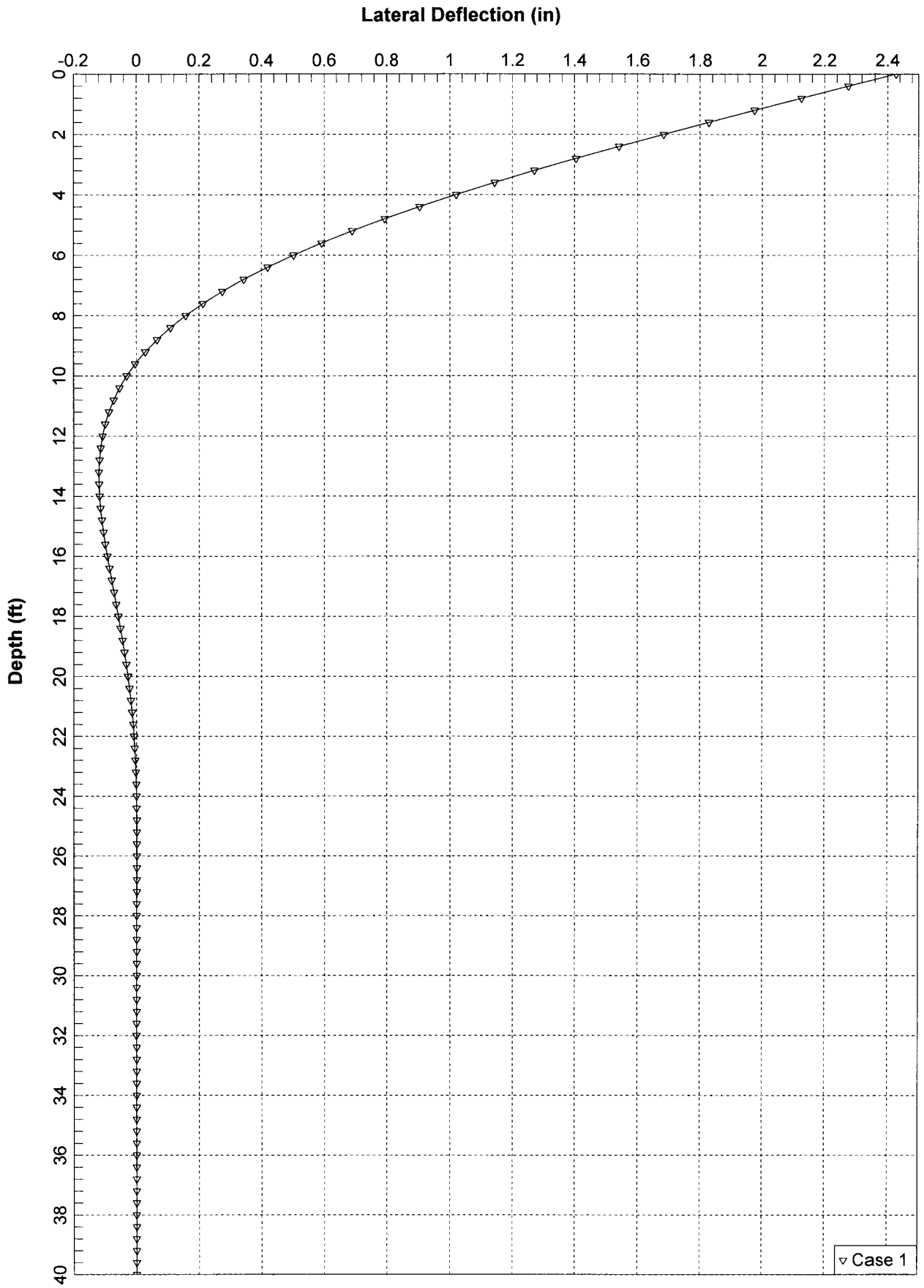
PARIKH
Practicing in the Geosciences

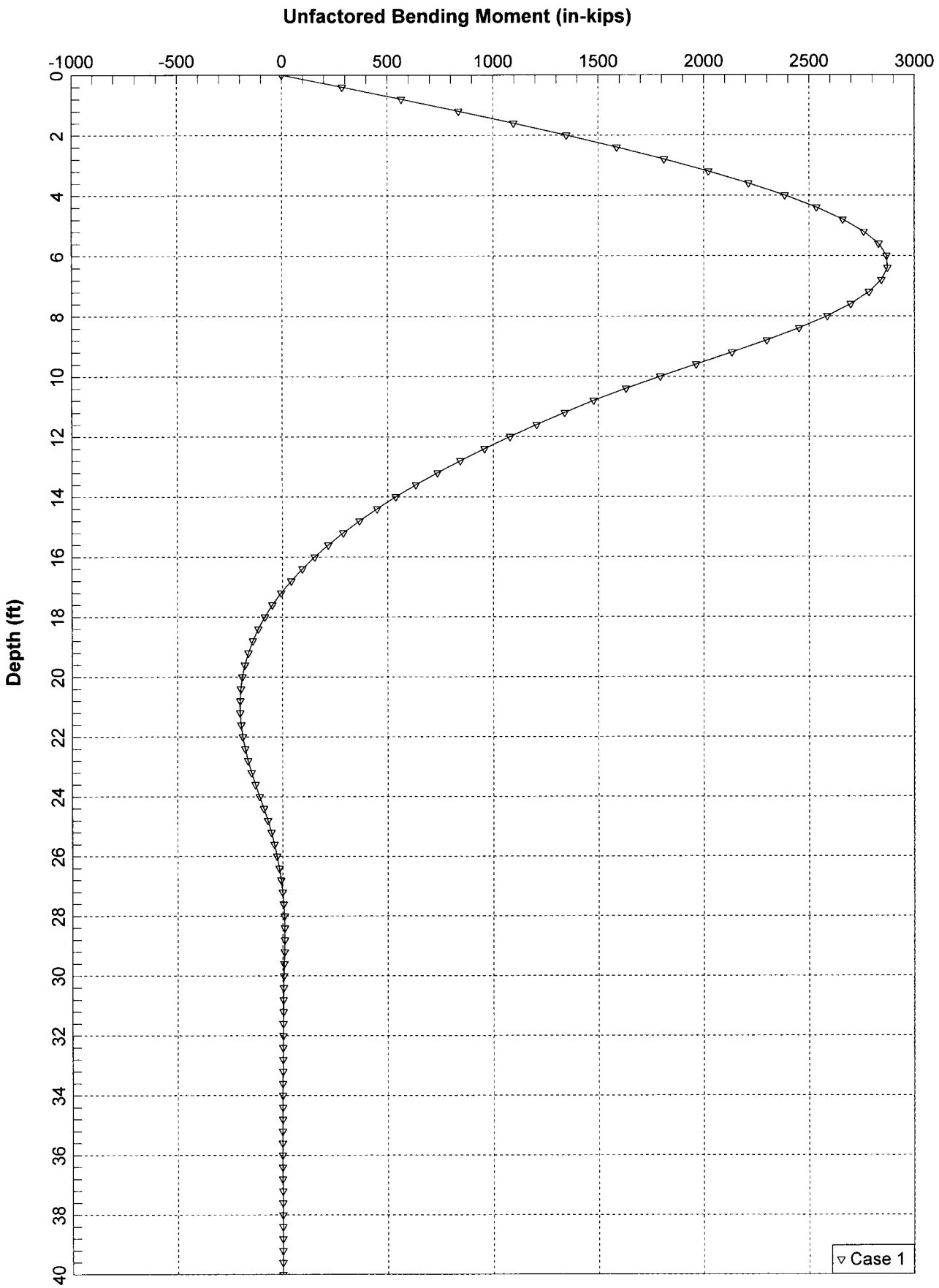
JOB I-5 North Stockton Corridor Improvement
SHEET NO OF
CALCULATED BY A Lam DATE 4/2010
CHECKED BY DATE
SCALE

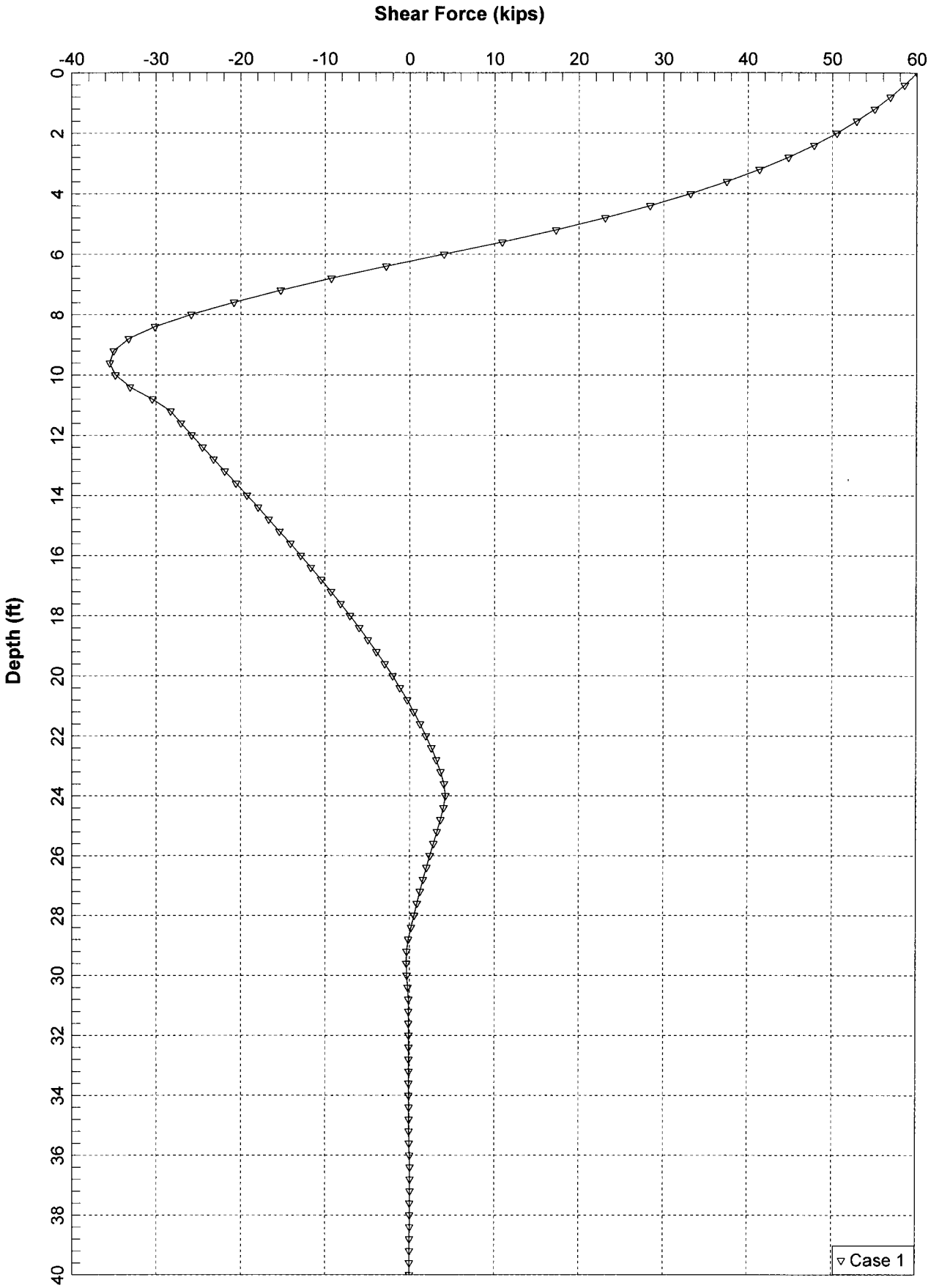
Lateral Pile Capacity Analysis - Swain Road UC (Bowing A-09-010) (Hwy 5) (Outside widening Abutment 1 WW1) 14" ⌀ precast concrete pile · 154 tons service load



```
LPILEP5
I-5 Stockton-Swain-Abut-1(outside) WW1
1       1       0       0       0       0
100     2       -36     480     0
0       14      3112    194     3000000
480     14      3112    194     3000000
6       12      12      0       2
4       -36     132     90      90
3       132     246     0       0
3       246     366     0       0
4       366     426     60      60
3       426     564     0       0
3       564     714     0       0
-36     0.069
132     0.069
132     0.067
246     0.067
246     0.039
366     0.039
366     0.039
426     0.039
426     0.039
564     0.039
564     0.039
714     0.039
-36     0       35.5    0       0
132     0       35.5    0       0
132     8.68    0       0.007   0
246     8.68    0       0.007   0
246     8.68    0       0.007   0
366     8.68    0       0.007   0
366     0       33      0       0
426     0       33      0       0
426     17.36   0       0.005   0
564     17.36   0       0.005   0
564     27.77   0       0.005   0
714     27.77   0       0.005   0
0       0.6     1
1248    0.6     1
0       1       1
1
1       60000   0       14000
0
1       1       0
100     1E-5    100
```


Lateral Deflection (in)
-0.2
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
2.4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Unfactored Bending Moment (in-kips)
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-40
-30
-20
-10
0
10
20
30
40
50
60
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

Lateral Pile capacity Analysis - Swain Road UC (Boring R-09-001) Outside Widening

Abut 2 w/ 14" □ precast concrete pile 154 tons service load

Flow 9.42'

Flow 6.42'

~3'

Assumed Top of Embankment +3.5'

Flow 3.5'

12/1/=

8' — $k = 90\ lb/in^3$ — Silty Sand (Medium dense) SPT N = 19 — $\phi = 36°$ — $\gamma' = 115$ pcf

33'

15' — Lean Clay Avg SPT N = 10 — $\alpha = 0.85$ — $c = 1250$ psf — $\varepsilon_{50} = 0.007$ — $\gamma' = 115$ pcf

18' — $\phi' = 35°$ — $k = 90\ lb/in^3$ — Poorly Graded Sand (Medium dense) Avg SPT N = 22 — $\gamma' = 115$ pcf — $\gamma_{sat} = 130$ pcf

$k = 60\ lb/in^3$

8.5' — Lean Clay Avg SPT N = 9 — $c = 1125$ psf — $\alpha = 1.00$ — $\varepsilon_{50} = 0.007$ — $\gamma_{sat} = 125$ pcf

10.5' — Lean Clay Avg SPT N = 13 — $c = 1625$ psf — $\alpha = 1.0$ — $\varepsilon_{50} = 0.007$ — $\gamma_{sat} = 130$ pcf

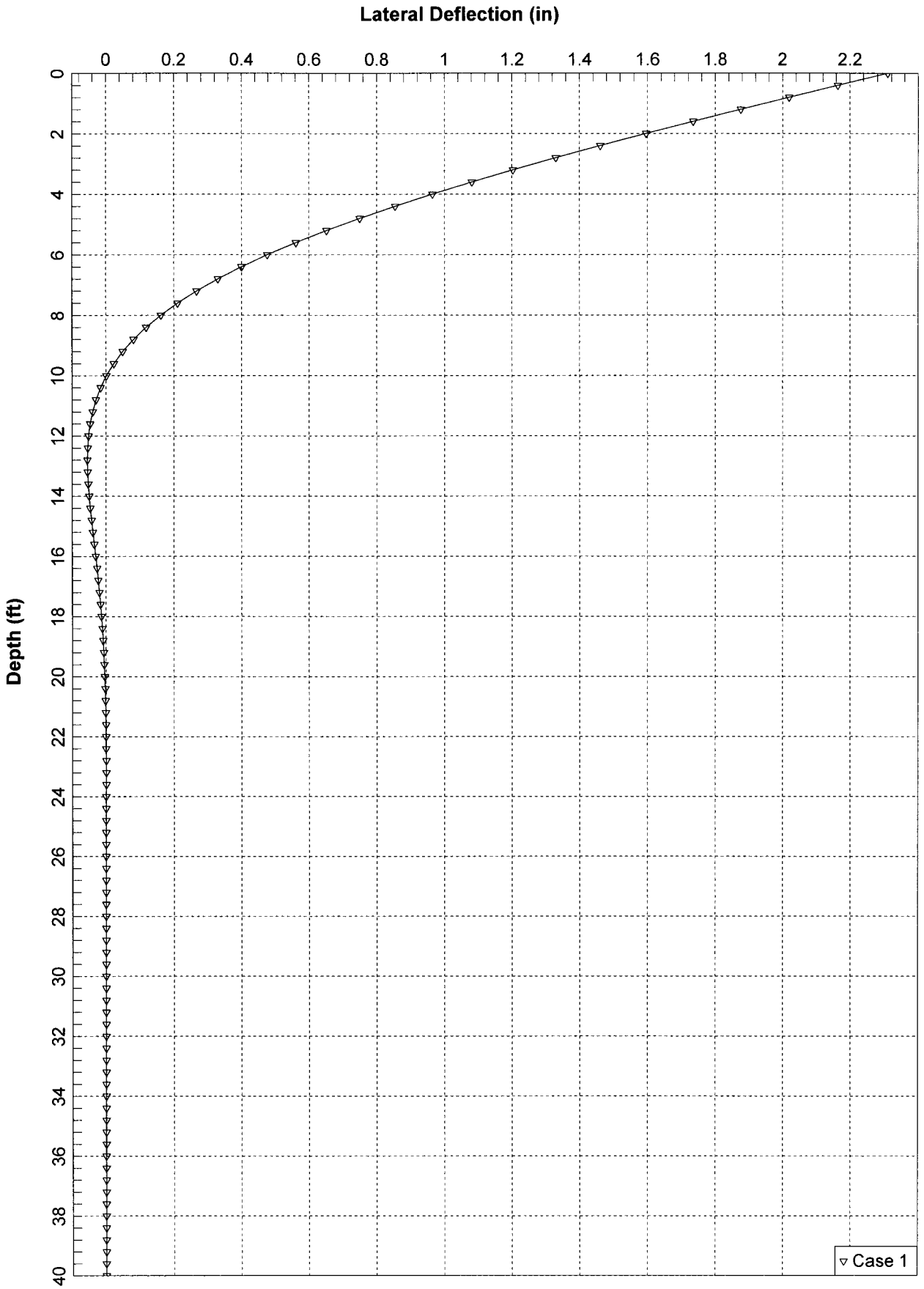
20' — Lean Clay Avg SPT N = 20 — $c = 2500$ psf — $\alpha = 0.65, 0.50$ — $\gamma_{sat} = 130$ pcf

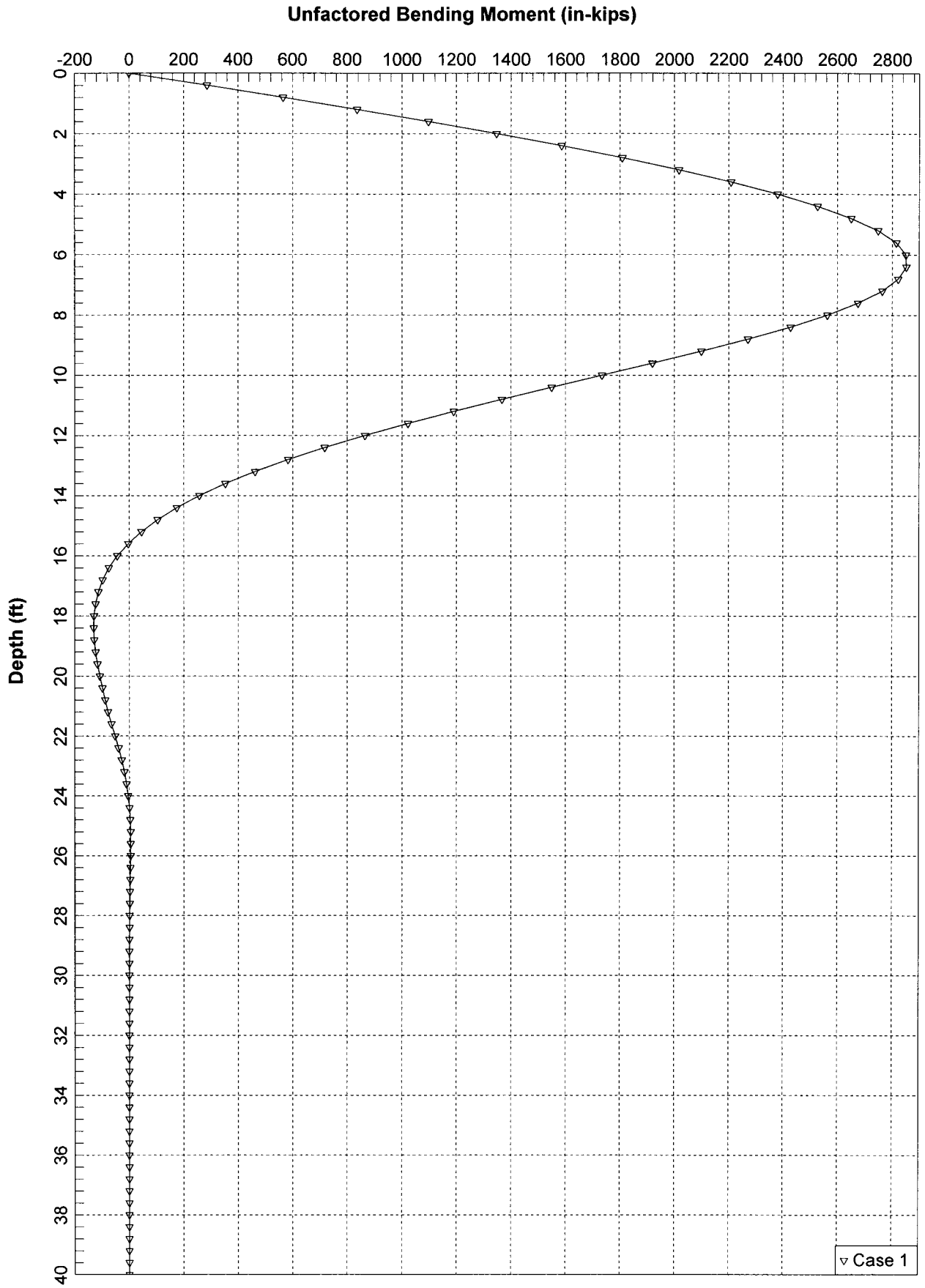
10' — Lean Clay SPT N = 40 — $c = 4000$ psf — $\alpha = 0.25$ — $\gamma_{sat} = 130$ pcf

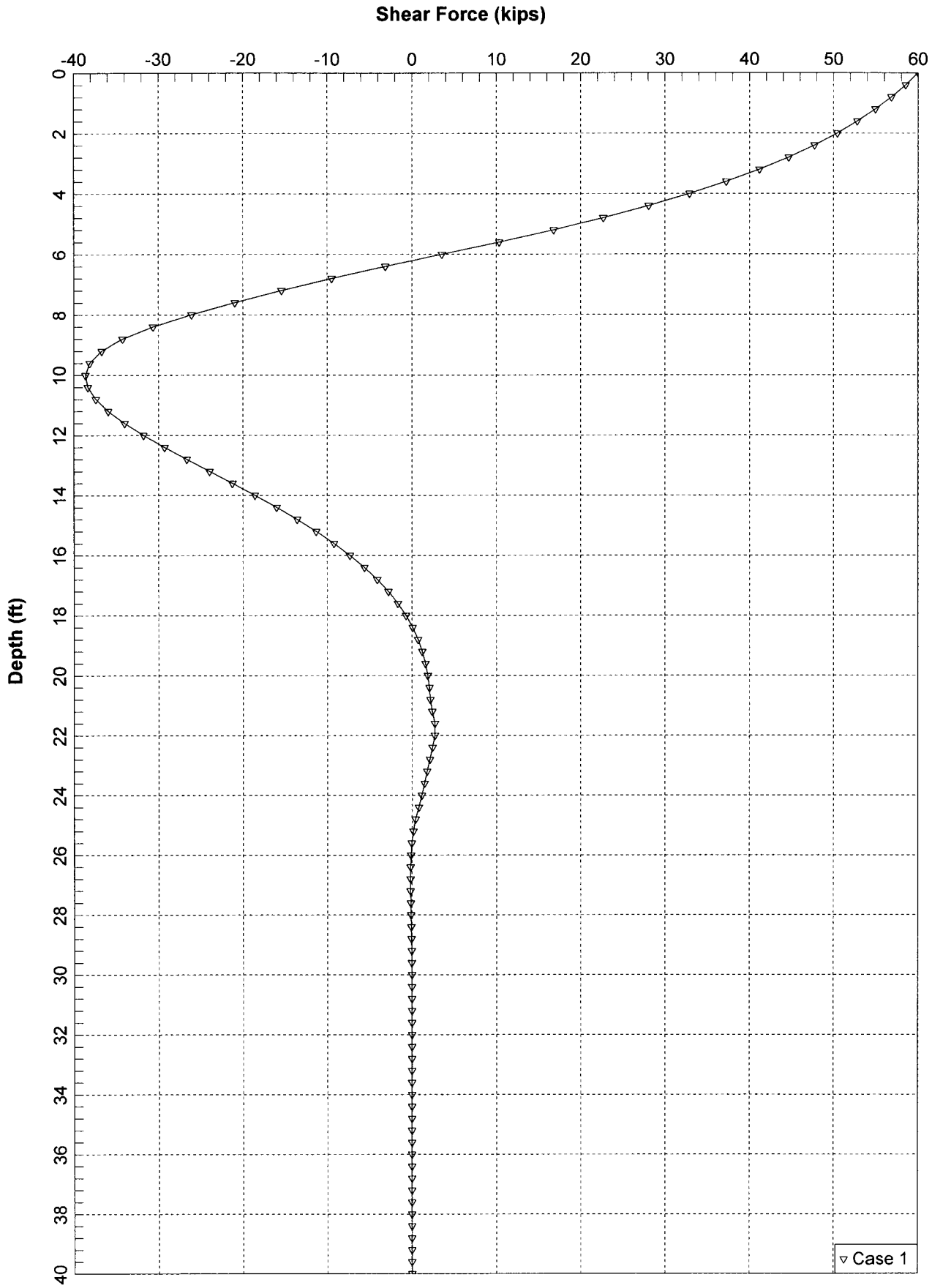
10' — Lean Clay SPT N = 24 — $c = 3000$ psf — $\alpha = 0.25$ — $\gamma_{sat} = 130$ pcf

11.5' — Lean Clay Avg SPT N = 40 — $c = 4000$ psf — $\alpha = 0.25$ — $\gamma_{sat} = 130$ pcf

```
LPILEP5
I-5 Stockton-Swain Road (UC)-Abut-2-WW1-R-09-001 (Outside)
1       1       0       0       0       0
100     2       -36     480     0
0       14      3112    194     3000000
480     14      3112    194     3000000
6       12      12      0       2
4       -36     252     90      90
3       252     432     0       0
4       432     552     90      90
4       552     648     60      60
3       648     750     0       0
3       750     876     0       0
-36     0.067
252     0.067
252     0.067
432     0.067
432     0.067
552     0.067
552     0.039
648     0.039
648     0.036
750     0.036
750     0.039
876     0.039
-36     0       36      0       0
252     0       36      0       0
252     8.7     0       0.007   0
432     8.7     0       0.007   0
432     0       35      0       0
552     0       35      0       0
552     0       35      0       0
648     0       35      0       0
648     7.8     0       0.007   0
750     7.8     0       0.007   0
750     11.3    0       0.007   0
876     11.3    0       0.007   0
0       0.6     1
1206    0.6     1
0       1       1
1
1       60000   0       14000
0
1       1       0
100     1E-5    100
```


Lateral Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Unfactored Bending Moment (in-kips)
-200
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1


Shear Force (kips)
-40
-30
-20
-10
0
10
20
30
40
50
60
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
Case 1

Lateral Pile Capacity Analysis - Swain Road UC (Widen) (Outside Widening)

Abutment 1 Wing Wall

Depth of Second Point of Counterflexure = 24.0'

Design pile tip elevation = 16.2' - 24.0'

= -7.8' say -8.0 ft

Abutment 2 Wing Wall

Depth of Second Point of Counterflexure = 22.0'

Design pile tip elevation = 16.42' - 22.0'

= -5.58' say -6.0 ft

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Swain Road UC (Median Widen)

Boring A-09-SR-001 (Abutment 1) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
22.5* to 11.5	0 to 11	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.069	-
11.5 to 5.0	11 to 17.5	Sand (medium dense)	$\phi = 35.5°$; c= 0 psf	4, Sand	90	-	0.069	-
5.0 to -4.5	17.5 to 27	Clay (stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.067	-
-4.5 to -14.5	27 to 37	Clay (stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.039	Assumed GWL at 27 feet deep below existing ground surface for design.
-14.5 to -19.5	37 to 42	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.039	-
-19.5 to -31.0	42 to 53.5	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-31.0 to -43.5	53.5 to 66	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-43.5 to -57.5	66 to 80	Clay (very stiff)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-57.5 to -77.5	80 to 100	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-77.5 to -89.0	100 to 111.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.042	-

* Finish ground elevation of +13.20 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Swain Road UC (Median Widen)

CPT-09-SR-001 (Abutment 2) (14-inch square Alt "X" pile) **10-14-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
22.5* to 0.5	0 to 22	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	225	-	0.072	-
0.5 to -2.0	22 to 24.5	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.069	-
-2.0 to -11.5	24.5 to 34	Clay (very stiff)	ϕ = 0°; c=1750 psf	3, Clay	500	0.007	0.067	-
-11.5 to -12.5	34 to 35	Clay (very stiff to hard)	ϕ =0°; c= 4000 psf	3, Clay	1000	0.005	0.069	-
-12.5 to -14.5	35 to 37	Clay (very stiff to hard)	ϕ = 0°; c= 4000 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 35 feet deep below existing ground surface for design.
-14.5 to -17.5	37 to 40	Clay (very stiff)	ϕ = 0°; c=3000 psf	3, Clay	1000	0.005	0.039	
-17.5 to -26.5	40 to 49	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-26.5 to -35.5	49 to 58	Sand (medium dense)	ϕ = 33.5°; c= 0 psf	4, Sand	60	-	0.039	-
-35.5 to -55.0	58 to 77.5	Clay (very stiff)	ϕ = 0°; c=3000 psf	3, Clay	1000	0.005	0.039	-
-55.0 to -77.5	77.5 to 100	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +13.40 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Swain Road UC (Outside Widen)

Boring R-09-SR-001 (Abutment 1) (14-inch square Alt "X" pile) **10-15-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
22.5* to 11.5	0 to 11	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	225	-	0.069	-
11.5 to 5.0	11 to 17.5	Sand (medium dense)	$\phi = 35.5°$; c= 0 psf	4, Sand	90	-	0.069	-
5.0 to -4.5	17.5 to 27	Clay (stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.067	-
-4.5 to -14.5	27 to 37	Clay (stiff)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.039	Assumed GWL at 27 feet deep below existing ground surface for design.
-14.5 to -19.5	37 to 42	Sand (medium dense)	$\phi = 33°$; c= 0 psf	4, Sand	60	-	0.039	-
-19.5 to -31.0	42 to 53.5	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-31.0 to -43.5	53.5 to 66	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-43.5 to -57.5	66 to 80	Clay (very stiff)	$\phi = 0°$; c=3250 psf	3, Clay	1000	0.005	0.039	-
-57.5 to -77.5	80 to 100	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-77.5 to -89.0	100 to 111.5	Sand (dense)	$\phi = 36°$; c= 0 psf	4, Sand	125	-	0.042	-

* Finish ground elevation of +13.20 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Swain Road UC (Outside Widen)

Boring R-09-SR-001 (Abutment 2) (14-inch square Alt "X" pile)

10-15-09

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
3.5* to -4.5	0 to 8	Sand (medium dense)	$\phi = 36°$; c= 0 psf	4, Sand	90	-	0.067	-
-4.5 to -19.5	8 to 23	Clay (firm)	$\phi = 0°$; c=1250 psf	3, Clay	500	0.007	0.067	-
-19.5 to -29.5	23 to 33	Sand (medium dense)	$\phi = 35°$; c= 0 psf	4, Sand	90	-	0.067	-
-29.5 to -37.5	33 to 41	Sand (medium dense)	$\phi = 35°$; c= 0 psf	4, Sand	60	-	0.039	Assumed GWL at 33 feet deep below existing ground surface for design.
-37.5 to -46.0	41 to 49.5	Clay (firm)	$\phi = 0°$; c=1125 psf	3, Clay	500	0.007	0.036	-
-46.0 to -56.5	49.5 to 60	Clay (stiff)	$\phi = 0°$; c=1625 psf	3, Clay	500	0.007	0.039	-
-56.5 to -76.5	60 to 80	Clay (very stiff)	$\phi = 0°$; c=2500 psf	3, Clay	1000	0.005	0.039	-
-76.5 to -86.5	80 to 90	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-
-86.5 to -96.5	90 to 100	Clay (very stiff)	$\phi = 0°$; c=3000 psf	3, Clay	1000	0.005	0.039	-
-96.5 to -108.0	100 to 111.5	Clay (very stiff to hard)	$\phi = 0°$; c=4000 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +12.35 feet is used in the vertical pile capacity analyses. The soil profile is extrapolated from +3.50 feet to +12.35 feet.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection __ 65% PS&E Unchecked Details _X PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 7/8/2010 **Structure Name*:** Swain Road UC_ **Br No*:** 29-0215 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	_____	_____	_____	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 4th review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated June 11, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** Swain Road UC_ **Br No*:** 29-0215 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	6/07/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	9	Please provide the Nominal Driving Resistance in Table 5.	Comment is incorporated. "Nominal Driving Resistance" is provided in Tables 5A and 5B.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection _X 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Swain Road UC **Br No*:** 29-0215 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection X 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 12/16/2009 **Structure Name*:** Swain Road UC **Br No*:** 29-0215 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
3	FR	8	Tables 2 and 3, Although the pile tip appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	
4	FR	9	Table 4A: Pile data table shows two different design tip elevations for the two abutments. It is recommended to keep the same pile length for ease of pile fabrication.	Comment is incorporated. Same pile length is used in the design of specified pile tip elevation.	
5	FR	LOTB	As is stated in the Soil Legend, the sample number or	Comment is incorporated.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

			unconfined compressive strength is generally not located to the left of the graphic on the LOTB.		
6	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #3 above.	
7	FR	LOTB	Boring A-09-SR-001 shows siltstone at an elevation of about –58-feet and Poorly graded SAND with SILT at an elevation of –78-feet. Typically alluvial sequences do not appear below rock, please clarify.	The Poorly graded SAND with SILT at an elevation of -78 feet is to be changed to "siltstone" in the LOTB after further review of the field boring log.	
8	FR	LOTB	The 1999 As-Built LOTB is upside down, please correct.	Comment is incorporated.	
9	FR	NA	C2 – Approved subject to OSFP Verification	NA	

FOUNDATION REPORT
BENJAMIN HOLT DRIVE UC (WIDEN)
(BRIDGE NO. 29-0194)
STOCKTON, CALIFORNIA
10-SJ-05, PM 31.45 EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

PARIKH CONSULTANTS, INC.

2360 Qume Drive, Suite A, San Jose, CA 95131
(408) 452-9000

June 11, 2010

Job No. 206127.BR

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FOUNDATION REPORT
BENJAMIN HOLT DRIVE UC (WIDEN)
(BRIDGE NO. 29-0194)
STOCKTON, CALIFORNIA
10-SJ-05 PM 31.45 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Benjamin Holt Drive Undercrossing (Widen) (Bridge No. 29-0194)" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site including available as-built Log of Test Borings (LOTB) for the existing structure; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Benjamin Holt Drive Undercrossing (UC) (Widen) (Bridge No. 29-0194).

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter

unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING BRIDGE AND PROPOSED CONSTRUCTION

Existing Bridge

The existing left and right bridges are single-span structures. Based on the as-built general plan and foundation plan of the Benjamin Holt Drive UC, the structures were constructed in 1970. The existing left bridge and right bridge (Bridge No. 29-0194 L/R) are approximately 127 feet long and 53 feet wide. The left and right bridges have not been widened since their construction in 1970.

The abutments of the existing left and right bridge are supported on "Reinforced Concrete Raymond Step-Tapered Piles" (Class I 45 tons) based on the as-built data. The specified tip elevation is -45 to -50 feet (NAVD 29).

Proposed Construction

The widening of the Benjamin Holt Drive UC is part of the proposed "I-5 North Stockton Corridor Improvement" project. The deck of the existing bridge will be connected to the inside widening by a closure pour. There will be no outside widening for the Benjamin Holt Drive UC.

According to the designer, the proposed Benjamin Holt Drive UC (Widen) will match the existing span with supports at Abutments 1 and 2. The widen structure will be a cast-in-place prestressed concrete box girder with approximate length of 127 feet measured along the "I-5" Line. The width of the inside widening will be 53 feet 7½ inches. The abutments will incorporate diaphragm type abutments.

Based on our discussions with the designer, the proposed bridge abutments are to be supported on Caltrans Standard Class 140 Alternate "W" (14-inch diameter) steel pipe piles.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The Benjamin Holt Drive UC is located at the intersection of Interstate 5 (I-5) and West Benjamin Holt Drive. It is approximately 0.35 miles northwest of West Swain Road UC. The Project Location Map, Plate 1, shows the location of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

The as-built Log of Test Borings (LOTB) of the Benjamin Holt Drive UC (March and May 1964) (Ref. 1) and Ben Holt/I-5 Retaining Walls (March 1997) (Ref. 2) have been used as a reference. Based on the as-built LOTB, Borings B-122, B-123, B-2 and B-4 were drilled by Caltrans in March and May 1964 and Cone Penetration Test (CPT) at B-1 and B-3 were performed by Caltrans in March 1964. The soil borings were drilled to approximate depths between 25 feet and 71.5 feet and to the bottom elevation approximately -25 feet and -70 feet. The CPTs were pushed to the approximate depths of 58 feet and 64 feet and to the bottom elevation approximately -57 feet and -62.5 feet. The soil borings (Borings B-7, B-8 and B-10) in 1997 were drilled to the approximate depths between 24.5 feet and 26 feet and to the approximate bottom elevations between -2 feet and -25 feet.

Field Exploration in 2009

Additional field exploration was conducted in April 2009. Boring R-09-BH-001 was drilled on the south side and CPT-09-BH-001 was performed on the north side of Benjamin Holt Drive UC. The soil boring was drilled to the depth of 111 feet and the CPT was pushed to the depth of approximately 109.5 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit weight and Plasticity Index of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on the as-built borings/CPTs, the subsurface soils generally consist of soft to very soft peaty clay and loose to medium dense silty sand, underlain by medium dense to dense sands, underlain by stiff to very stiff clays, underlain by medium dense to very dense sands .

Based on Boring R-09-BH-001 and CPT-09-BH-001, the subsurface soils generally consist of medium dense silty sand (embankment fill), underlain by stiff to hard lean clays with intermittent layers of medium dense to dense silty sand.

Groundwater was recorded at elevations between -7 feet and -8 feet in the soil borings in May 1964 and at Elevation -7 feet in the CPTs in March 1964. Groundwater was recorded at elevations between -7.5 feet and -10.5 feet in March 1997. Groundwater was measured at Elevation -5 feet (the depth of 30 feet below existing ground surface) during drilling for R-09-BH-001 in April 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 2).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 3) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip). This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

In the early design phase, it was agreed upon with Caltrans to use the 1996 maps since there was no significant difference between the old and new maps and the project area is in a low seismicity zone. Since the design has progressed using these maps, we have maintained the same understanding for the ARS design curve.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 4), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 5). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

Based on the available boring information, the subsurface soil conditions at the project site consist of medium dense sand, underlain by stiff to hard lean clays with intermittent layers of medium dense to dense silty sand. These types of soil generally match the criteria for Soil Type D, as per Caltrans Seismic Design Criteria (Version 1.4, June 2006) (Ref. 6). The seismic design criteria for the project are as follows:

1. Soil Profile Type D
2. Peak Bedrock Acceleration = 0.2 g
3. ARS Design Curve = Design curve per Caltrans Seismic Design Criteria: Figures B.8, with no modifications. The peak bedrock acceleration has been selected to account for the reverse fault effect.

An "ARS Design Curve" (Plate No. 5), which is based on Caltrans Seismic Design Criteria, (Version 1.4) is attached.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for

fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring R-09-BH-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-BH-001 included in this report, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the pile capacity analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Bridge Foundations

For the proposed widen bridge structure, different foundation systems have been evaluated with respect to their design capacity and construction considerations. Caltrans standard Class 140 Alternative "W" Piles are recommended to be the foundation system for the abutments of the proposed bridge structure. These piles may be designed for service load of 70 tons at the abutments to the indicated pile tip elevations. Pertinent foundation design information provided by Biggs Cardosa Associates (Structural designer), including Foundation Design Data and Foundation Loads, are presented in the following tables.

TABLE 2: FOUNDATION DESIGN DATA

Support No	Design Method	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elev. (ft)	Pile Cap Size(ft)		Permissible Settlement (in)	No. of Piles per Support
					B	L		
Abut 1	WSD	Class 140 Alt. W	12.50	9.50	2.50	54.60	1.00	14
Abut 2	WSD	Class 140 Alt. W	12.50	9.50	2.50	54.60	1.00	14

TABLE 3: FOUNDATION DESIGN LOADS (per Structural Designer)

Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.	Per Support	Max. Per Pile.
Abut 1	1,926	138	1,376	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1,926	138	1,376	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Consistent with Caltrans requirements, Working Stress Design (WSD) is used for the abutment foundations. The abutment foundations are evaluated for the foundation design data and loading condition using Caltrans November 2003 Bridge Design Specifications for foundations, using Working Stress Design (WSD) methods with "LRFD Service-I Loads", based upon WSD is presented in Table 4 below.

The evaluation of Load Demands on the piles, based upon WSD is presented in Table 4 below. The estimated specified tip elevations for the anticipated design loading of the piles are shown in Table 4 below. The pile cut-off elevations are shown in Table 2.

TABLE 4: FOUDNATION RECOMMENDATIONS FOR ABUTMENTS

Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Total Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elev. (ft)	Specified Tip Elev. (ft)
			Total	Permanent				
Abut 1	Class140 Alt W	9.50	1,926	1,376	140	280	-53.5 (a)	-59.5
Abut 2	Class140 Alt W	9.50	1,926	1,376	140	280	-59.5 (a)	-59.5

Notes: (i) Design tip elevations are controlled by (a) Compression, (b) Lateral Load, respectively.
(ii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.

TABLE 5: PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev.(1) (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 140 Alt W	280	0	-53.5 (a)	-59.5	280
Abut 2	Class 140 Alt W	280	0	-59.5 (a)	-59.5	280

Notes

(i) Design tip elevations for the ***Abutments*** are controlled by (a) Compression (b) Lateral Load.
(ii) The design tip elevation due to the lateral loads are estimated by Biggs Cardosa Associates and not included in the pile data table above.
(iii) The specified tip elevation shall not be raised above the design tip elevations for lateral load.
(iv) Based on Caltrans review comment, it was recommended to keep the same pile length (specified tip elevation) for "ease of pile fabrication".

The estimation of the driven pile capacity in clay is based on the adhesion factor per FHWA manual (Tomlinson 1980). The pile capacity will be derived primarily from frictional resistance along the pile shafts. A factor of safety of 2 was used to estimate the allowable pile capacity. Based on the Pile Data Table 5, the pile tip elevation is controlled by the demand in compression on the pile. The pile capacity calculations are provided in Appendix C.

The design tip elevations and specified tip elevations are based on the "pile cut-off" elevation provided by the designer. In the event that the "pile cut-off" elevations are changed, the design pile tip elevations may have to be revised accordingly.

The minimum horizontal distance between the "near face" of the footing of the pile cap (measured at the top of the footing) and the face of the finished slope should be in accordance with Caltrans "Bridge Design Specifications" November 2003. The piles should not be spaced closer than 3 times the pile diameter measured center-to-center. For piles spaced at center-to-center distance greater than or equal to 3 times the pile diameter,

there is no group effect for vertical pile capacity.

It is anticipated that the pile capacity will develop after driving as a result of soil "freeze" and dissipation of excess pore water pressures. The final pile capacity can be verified and the gain of pile capacity after initial driving may be evaluated based on "re-driving" after 24-hour (min.) set up. Caltrans Standard Specifications (Section 49-1.08), using the energy formula, should be used as the pile driving acceptance criteria. As an option, the contractor can consider to use Pile Driving Analyzer (PDA) to evaluate the pile capacity of the driven piles. Typical applications of the PDA include capacity evaluation (for both during driving and re-driving) and integrity testing for piles that have experienced hard driving.

Based on the available boring/CPT information, dense sands were encountered in the embankment fill. It is recommended to follow Caltrans standard specifications (Section 49-1.06) (Ref. 7) for oversize predrill through the existing embankment.

Due to the hard driving condition that may be encountered during pile driving, pile driving should be allowed to terminate short of the specified tip elevation provided the following conditions are satisfied:

- Other requirements including tension and lateral demands are met;
- Pile attaining its capacity and refusal within 5 feet above the specified tip elevation.

9.3 Lateral Design for Piles

Under seismic loading conditions, lateral load analyses should be performed for the foundation piles. It is our understanding that the structural designer will perform lateral load analyses for the piles using LPILE Program (v. 5). "Generalized Vertical Soil Profile and Strength Parameters" based on R-09-BH-001 and CPT-09-BH-001 are included in Appendix C. To account for group effect, p-multipliers of 0.6 and 0.8 are recommended for the pile spacing of 3D and 4D, respectively.

Generally, abutment piles are designed for service loads only, and batter piles are used for resisting static lateral earth pressures. In common engineering practice, a maximum pile head deflection of ¼ inch is recommended for service condition for vertical piles.

9.4 Lateral Earth Pressures

Abutment retaining walls and wing walls should be designed to resist the following applied lateral earth pressures and live loads. These values assume no hydrostatic pore pressure build-up behind the wall and are based on well-drained backfill behind the walls.

	Applied Lateral Earth Pressures
Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
At-Rest Condition	55 pcf Equivalent Fluid Pressure (EFP) for engineered fill.
Passive Resistance	5.0 ksf (ultimate) for seismic design of the abutment wall (5.5 feet or greater); for activated height less than 5.5 feet, modify proportionally i.e. 5.0 x (H/5.5) ksf per Caltrans SDC v.1.4. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.
Traffic Load Surcharge	The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. An additional height with equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf is added to the ground profile to account for the additional earth pressure resulting from the surcharge. Surcharge load due to the traffic has to be included in the design if the traffic is within a horizontal distance of one half of the wall height. A coefficient of 0.4 may be used to determine the additional lateral earth pressures resulting from the surcharge.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.5 Embankment Settlement

It is our understanding that there will be no new approach embankment required for this project. The existing embankment will be used for new structures. Settlement is not a geotechnical concern for this project.

9.6 Corrosion

Chemical tests were performed on soil samples from Boring R-09-BH-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 6: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
R-09-BH-001	31.0	910	7.4	42.4	35.3

Based on the corrosion test results, the native subsurface soils in Boring R-09-BH-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans Bridge Design Specifications – September 2003.

9.7 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.8 Construction Observation

To a degree, the performance of any structure is dependent upon construction procedures and quality. Hence, geotechnical observation of foundation excavation, and pile installations should be carried out by regulating agencies. If the subsurface conditions different from those forming the basis of our recommendations are encountered, this

office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

10. INVESTIGATION LIMITATIONS

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

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

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

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

Respectfully submitted,

PARIKH CONSULTANTS, INC.

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GEOTECHNICAL
STATE OF CALIFORNIA

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Benjamin Holt Drive. UC (Widen) Foundation Report (6-2010).rpt

REFERENCES

1. Benjamin Holt Drive UC As-built Log of Test Borings (March and May 1964).

2. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

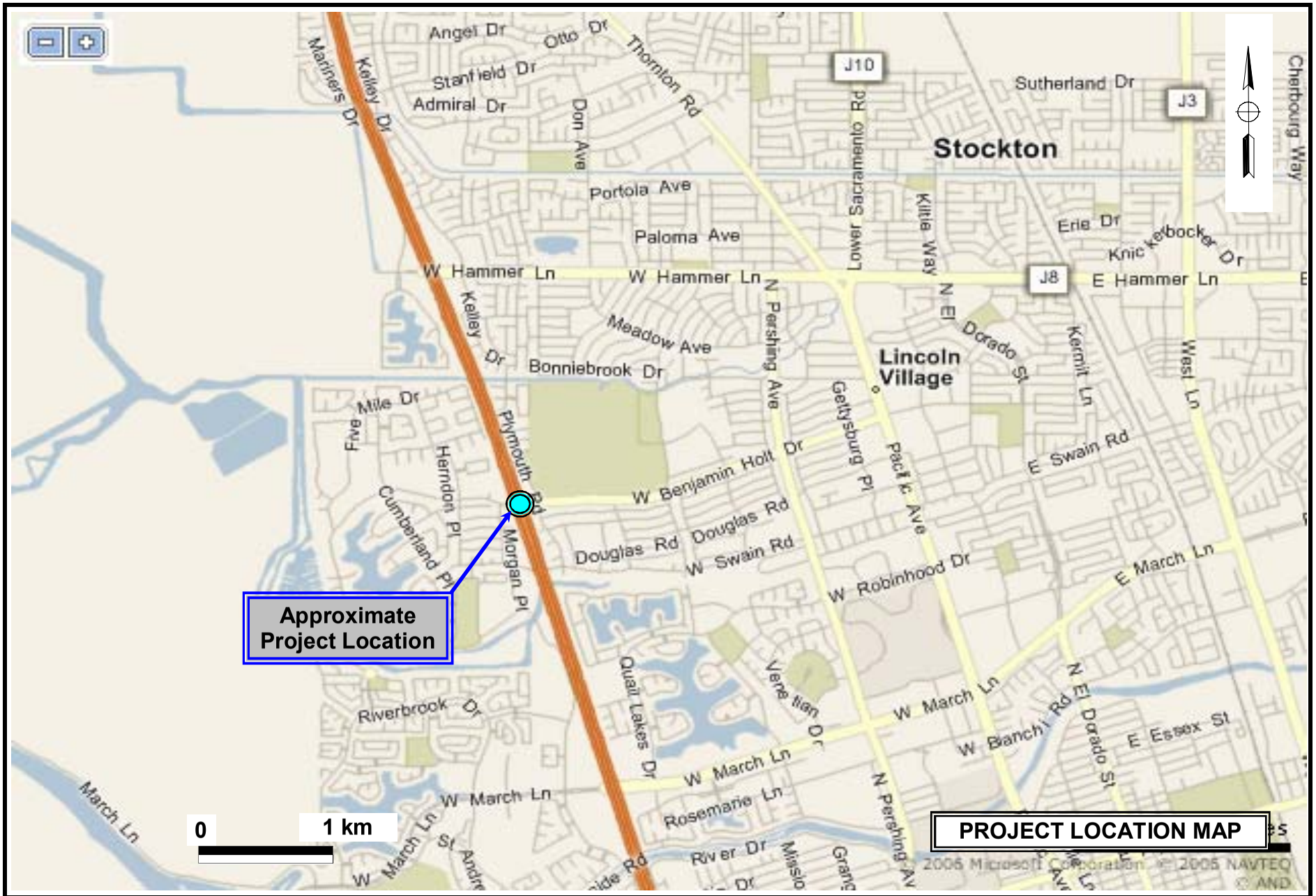
3. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

4. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

5. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.

6. Caltrans Department of Transportation, "Seismic Design Criteria", Version 1.4, June 2006.

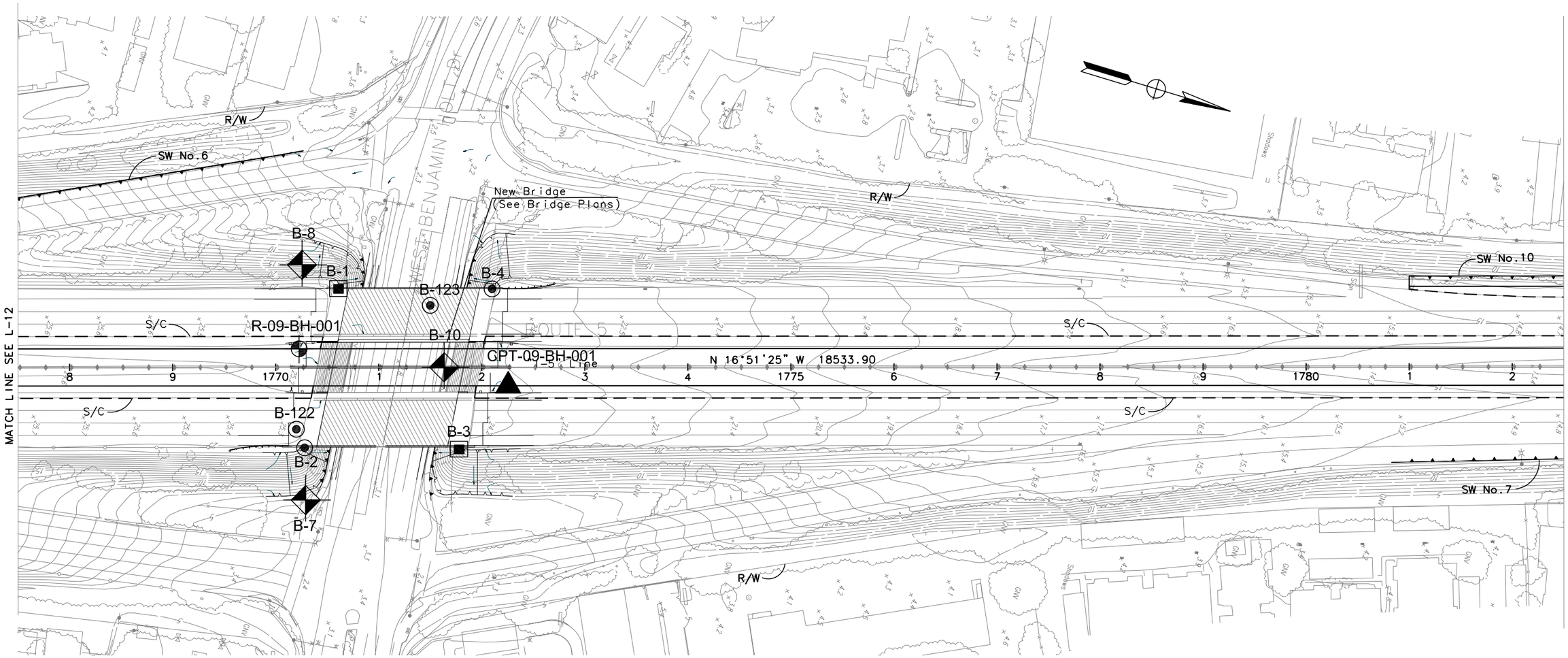
7. State of California Department of Transportation Standard Specifications May 2006.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

BENJAMIN HOLT DRIVE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0194)	PLATE NO.: 1



LEGEND

A-09-BH-001
Approximate Boring Location

B-3
Approximate CPT location performed in 1964

B-123
Approximate Boring location drilled in 1964

B-8
Approximate Boring location drilled in 1997

CPT-09-BH-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

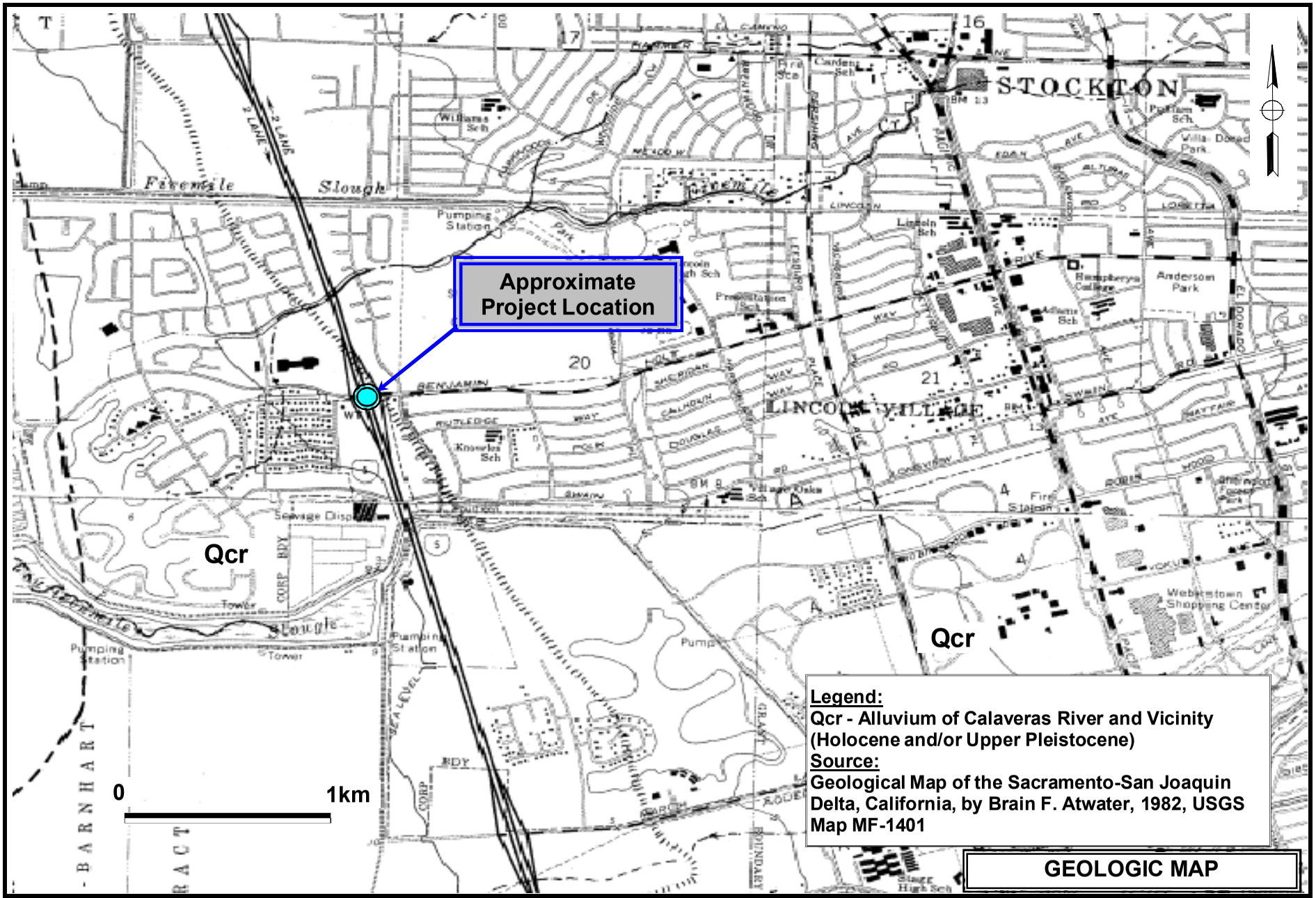
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

BENJAMIN HOLT DRIVE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

PLATE NO: 2

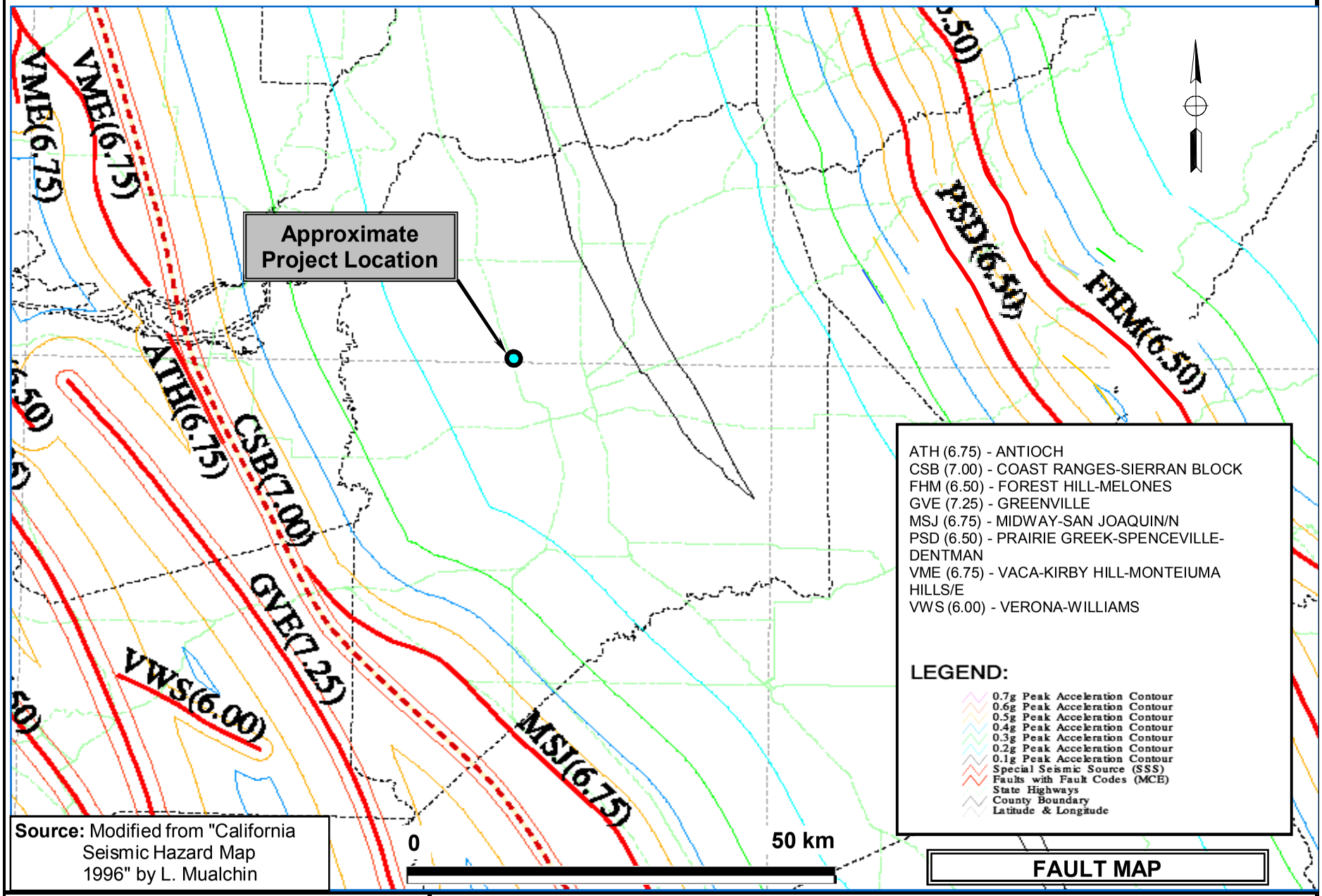


GEOLOGIC MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

BENJAMIN HOLT DRIVE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0194) | PLATE NO.: 3



Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

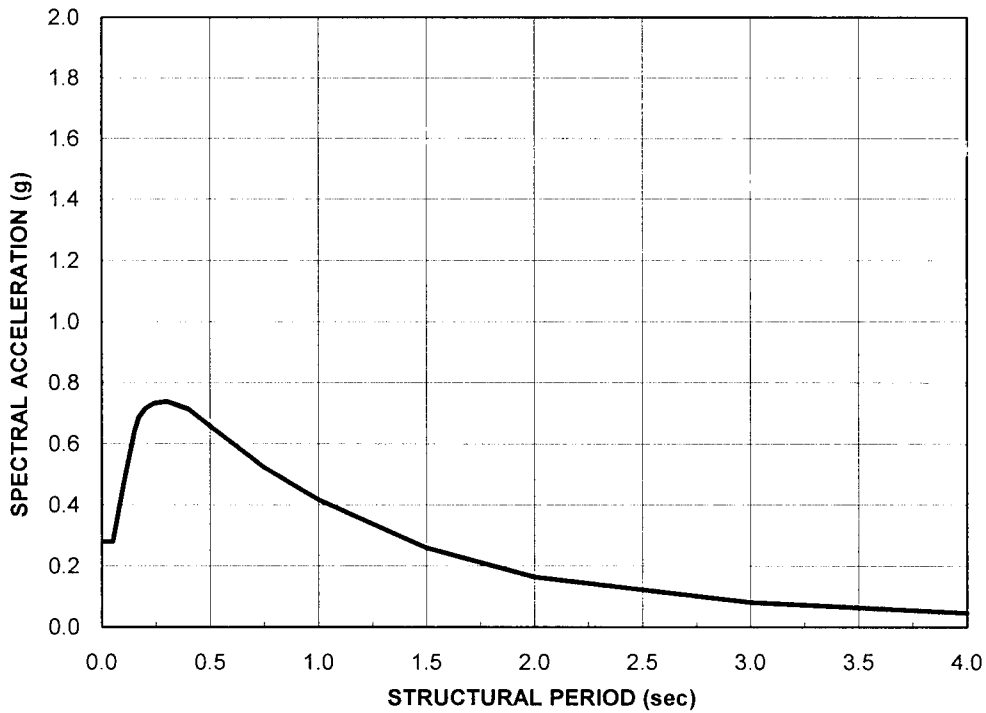
FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

BENJAMIN HOLT DRIVE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0194) | **PLATE NO.: 4**

ARS DESIGN CURVE
BENJAMIN HOLT DRIVE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA



Spectral Data

Period (sec)	CSB (7.00) Spectral Accel. (g)
0.010	0.280
0.020	0.280
0.030	0.280
0.050	0.280
0.075	0.370
0.100	0.466
0.120	0.537
0.150	0.640
0.170	0.686
0.200	0.716
0.240	0.733
0.300	0.739
0.400	0.714
0.500	0.658
0.750	0.523
1.000	0.417
1.500	0.260
2.000	0.164
3.000	0.081
4.000	0.046

1. Caltrans SDC (v 1.4, June 2006), Figure B.8,
   Governing Fault: COAST RANGES-SIERRAN BLOCK
   (Mw = 7.00, Soil Profile Type D, PBA = 0.2g)
   with no modifications.

   The peak bedrock acceleration has been selected to
   account for the reverse fault effect.

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

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SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

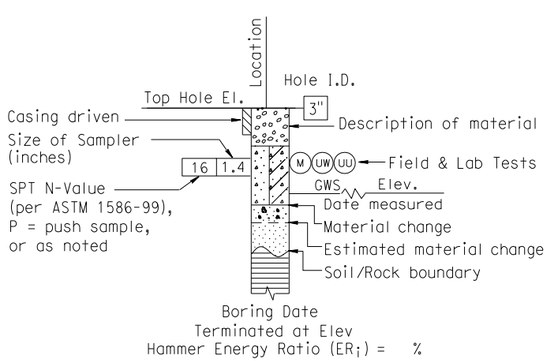
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

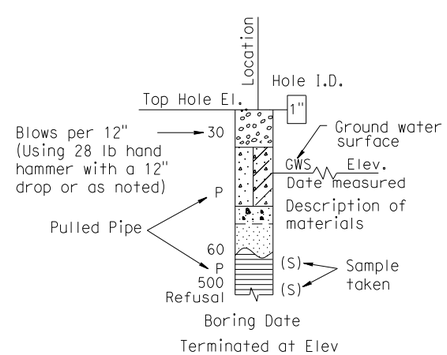
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

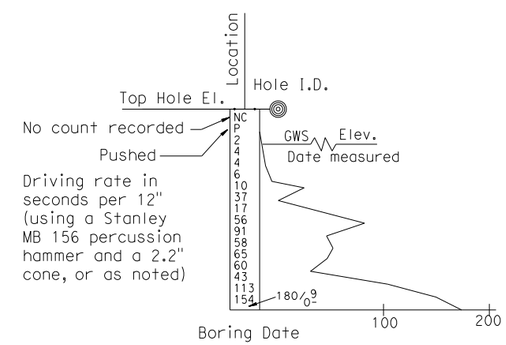
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



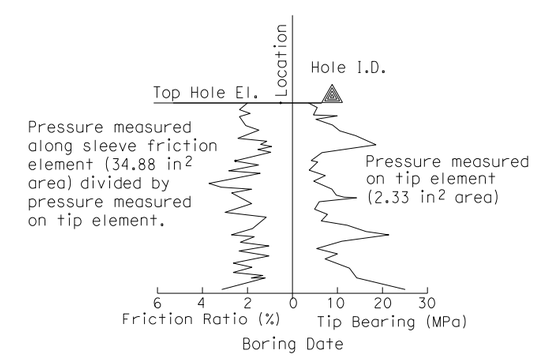
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER
SIGN OFF DATE	CHECKED BY	D. WANG	DATE: MARCH 2009 - APRIL 2009		

BRIDGE NO. 29-0194 | POST MILES 31.45

WEST BENJAMIN HOLT DRIVE UC (WIDEN)

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES → 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

REVISION DATES

SHEET 22 OF 26

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REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 5333)
CP	Compaction Curve (CTM 216)
CR	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content-% (ASTM D 2974)
P	Permeability (CTM 220)
PA	Particle Size Analysis (ASTM D 422)
PI	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (CTM 217)
SG	Specific Gravity (AASHTO T 100)
SL	Shrinkage Limit (ASTM D 427)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 4767)
VS	Vane Shear (AASHTO T 223)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL 6/11/10 DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

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SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. 29-0194
POST MILES 31.45

WEST BENJAMIN HOLT DRIVE UC (WIDEN)

SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/18/10 04/23/10 06/11/10

SHEET 23 OF 26

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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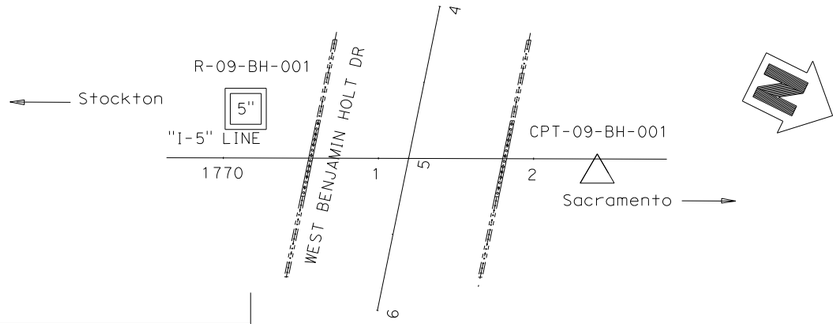
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131



PLAN
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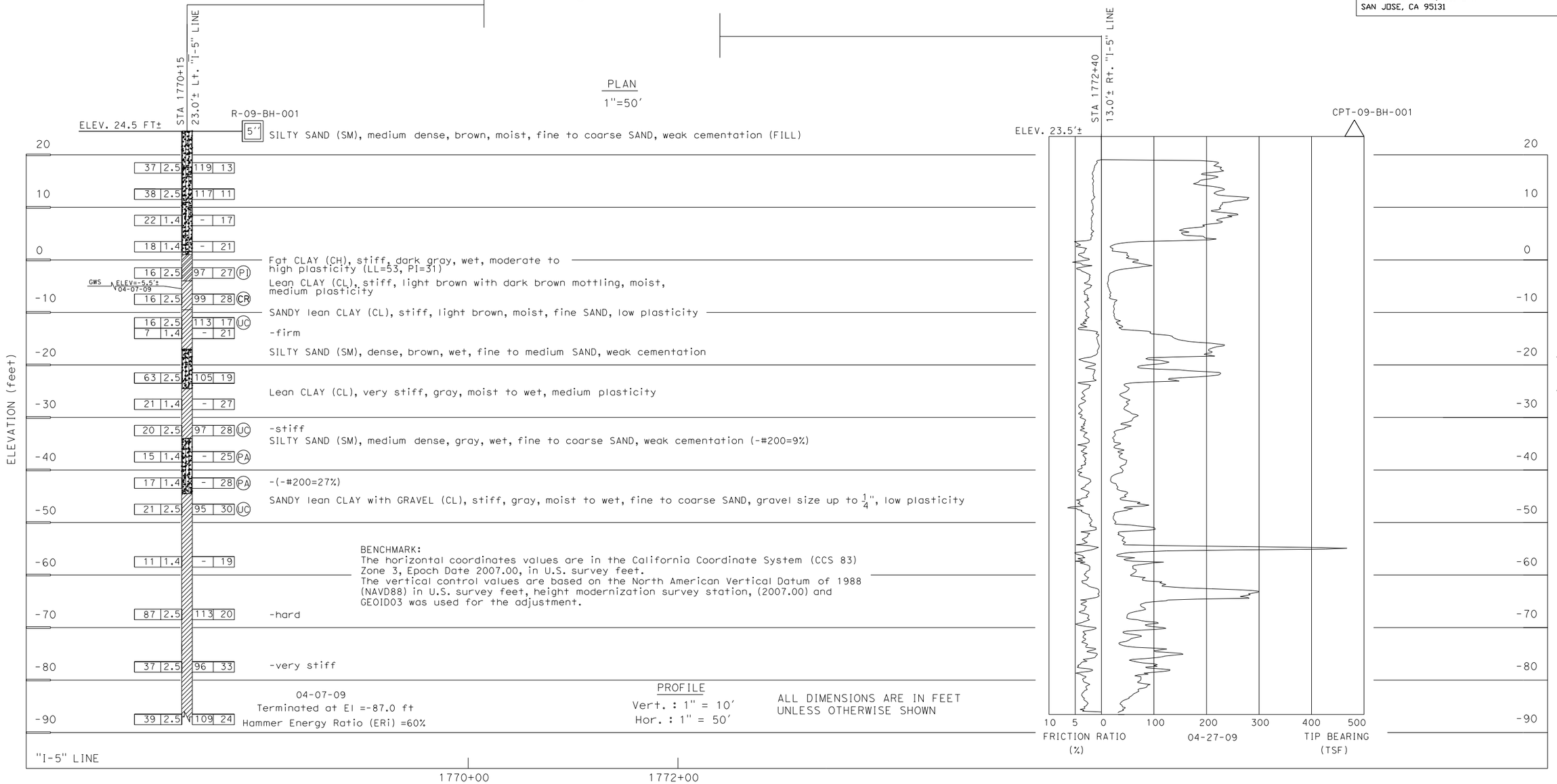

DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009		PROJECT ENGINEER

BRIDGE NO. 29-0194 — POST MILES 31.45

WEST BENJAMIN HOLT DRIVE UC (WIDEN)

LOG OF TEST BORINGS 1 OF 3

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES → REVISION DATES 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 24 OF 26

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DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5	———		

REGISTERED CIVIL ENGINEER — DATE

BENJAMIN HOLT DRIVE UNDERCROSSING (WIDEN)

LOG OF TEST BORINGS 2 OF 3

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240 — EA: 0G4701 — BRIDGE No. 29-0215 — Sheet 25 of 26

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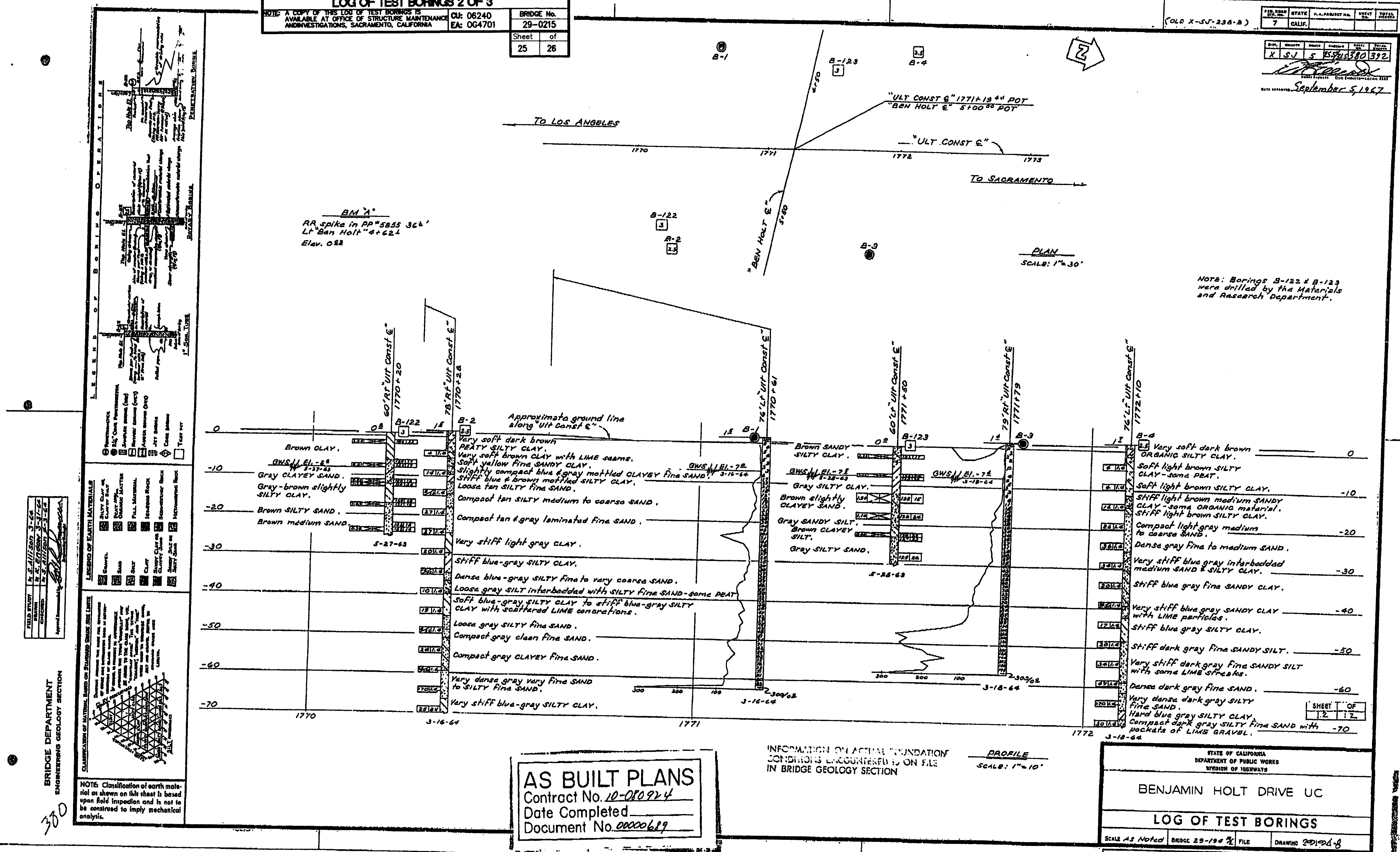

AS BUILT PLANS
Contract No. 10-0R0924
Date Completed ______
Document No. 00000689

63-10710H0803.4

I HEREBY CERTIFY THAT THIS IS A TRUE AND ACCURATE COPY OF THE ABOVE DOCUMENT TAKEN UNDER MY DIRECTION AND CONTROL ON THIS DATE IN SACRAMENTO, CALIFORNIA PURSUANT TO AUTHORIZATION BY THE DIRECTOR OF PUBLIC WORKS.
DATE 4/6/71 SIGNATURE

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BENCHMARK

CALTRANS 57.2mm BRASS DISK "632", EL. 0.116m, 6.062 Lt "BL2" STATION 16+70.237, NORTHING 667368.588, EASTING 1925376.705 AND CALTRANS 19mm ROD IN MONUMENT BOX "526", EL. 0.032m, 0.518 Lt "BR1" STATION 13+88.280, NORTHING 667346.548, EASTING 1925095.081. ELEVATIONS ARE BASED ON THE CITY STOCKTON BENCHMARK LIST, DATED OCTOBER, 1992, WHICH REFERS TO NGVD LEVEL DATUM OF 1929.

NO CORRECTIONS THIS SHEET

AS BUILT

CORRECTIONS BY D.R. SMALL RHT

CONTRACT NO. 10-448344

DATE 11-8-00 7-18-01

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
10	SJ	5	49.9/52.9	272	299

REGISTERED PROFESSIONAL ENGINEER

2-1-99

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER CARLOS ESPAÑA No. 298 Exp. 9/30/01 GEOTECHNICAL STATE OF CALIFORNIA

ESPAÑA Geotechnical Consulting
502 Ginseppe Ct, Ste 11, Roseville, CA 95678 (916) 773-2680 FAX (916) 782-4846

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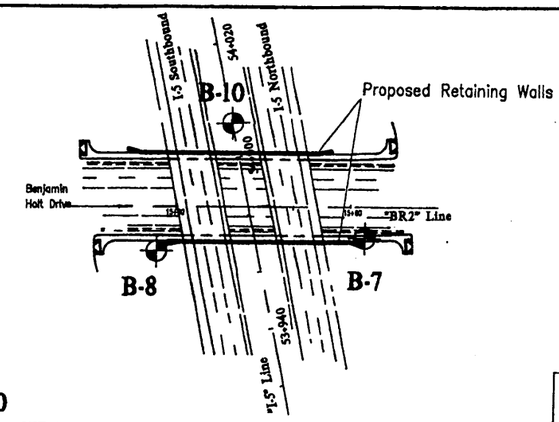
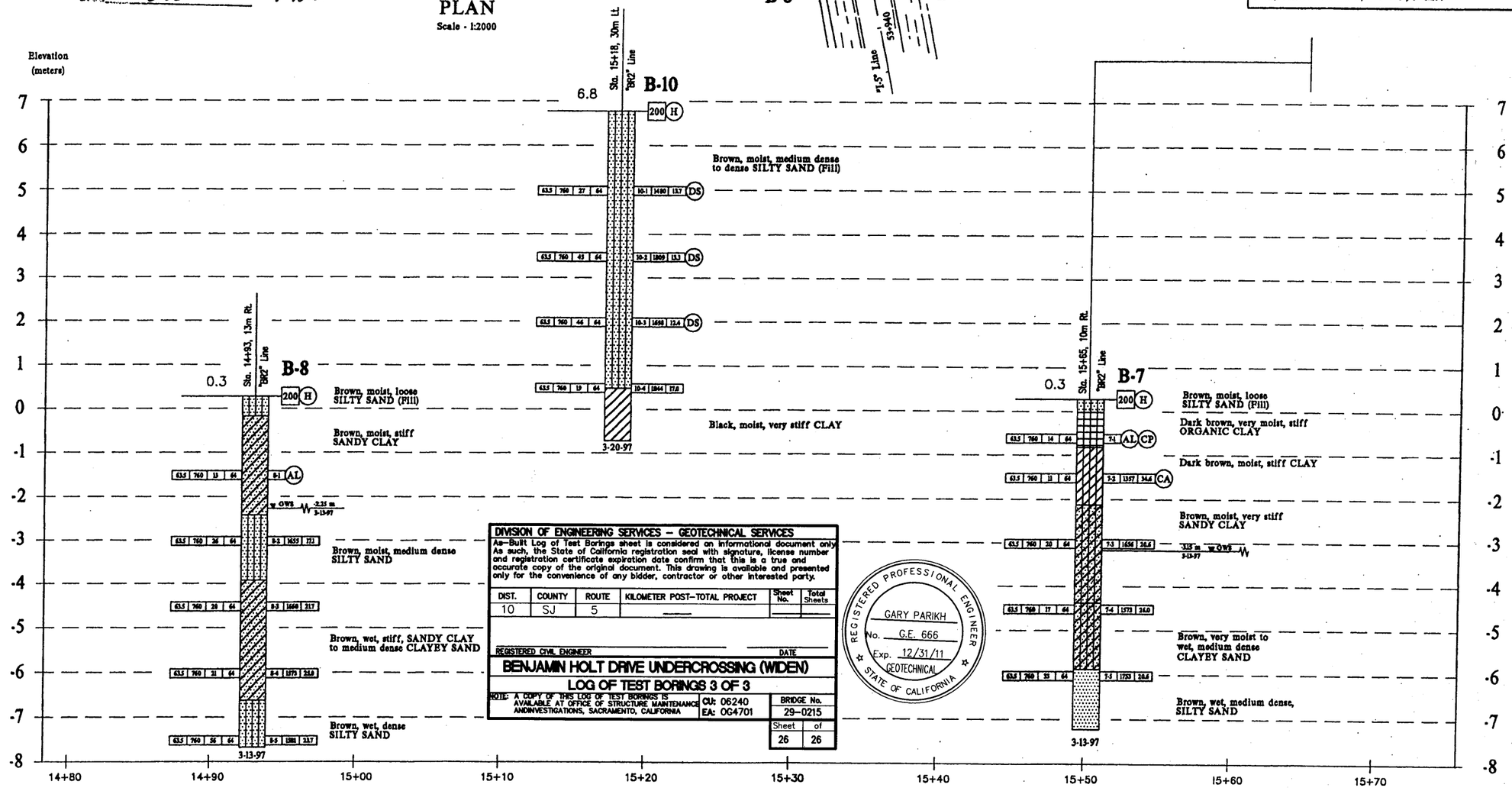
Caltrans Metric



PLAN

Scale - 1:2000



PROFILE

Stationing: "BR2" Line, meters
1:150 (Horizontal)
1:50 (Vertical)

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DIST.	COUNTY	ROUTE	KILOMETER POST–TOTAL PROJECT	Sheet No.	Total Sheets
10	SJ	5			

REGISTERED CIVIL ENGINEER DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

BENJAMIN HOLT DRIVE UNDERCROSSING (WIDEN)

LOG OF TEST BORINGS 3 OF 3

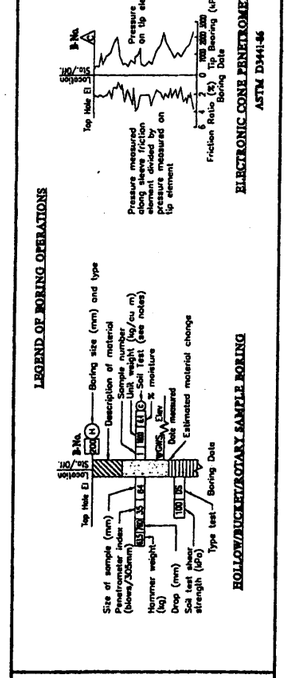
NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA

CU: 06240
EA: 0G4701

BRIDGE No. 29-0215

Sheet 26 of 26

LEGEND OF BORING OPERATIONS



NOTES:

1. TYPES OF BORINGS: Bucket Auger; Rotary Wash; Cone Penetrometer; Hollow Stem Auger; Test Pit
2. SOIL TEST DESIGNATIONS: Atterberg limits; Chemical analysis; Max. dry density; R-value; Direct shear; Expansion index; Consolidation; Sieve analysis; Pocket penetrometer; Unconfined compression; Unconsolidated undrained triaxial

LEGEND OF EARTH MATERIALS

CL, Lean Clay; ML, Silt; OL, Organic Clay or Silt; CH, Fat Clay; MH, Elastic Silt; OH, Organic Clay or Silt; GC, Clayey Gravel; SW, Well-Graded Sand; SP, Poorly Graded Sand; SM, Silty Sand; SC, Clayey Sand; GW, Well-Graded Gravel; GP, Poorly Graded Gravel; GM, Silty Gravel; Sandstone; Shale; Claystone; Siltstone

RELATIVE DENSITY AND CONSISTENCY CLASSIFICATION ASTM D1586

Sand and Gravels Consistency	Blows/305mm
Very loose	0 – 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very dense	Over 50

Clays and Silts Consistency	Blows/305mm
Very soft	0 – 2
Soft	2 – 4
Firm	4 – 8
Stiff	8 – 15
Very stiff	15 – 30
Hard	Over 30

CALTRANS DESIGN OVERSIGHT — DATE 10-7-02

DRAWN BY A. WORCESTER

CHECKED BY C. ESPAÑA

S. HARVEY — FIELD INVESTIGATOR — DATE 3/97

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Carlos España — PROJECT ENGINEER

BRIDGE NO. 29-212 R/L

POST KILOMETER 49.9

BEN HOLT/I-5 RETAINING WALLS
LOG OF TEST BORINGS

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 20 40 60 80

CU 10115
EA 4483U1

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 5/15/97 11/3/97 6/12/98 8/14/98

SHEET 4 OF 4

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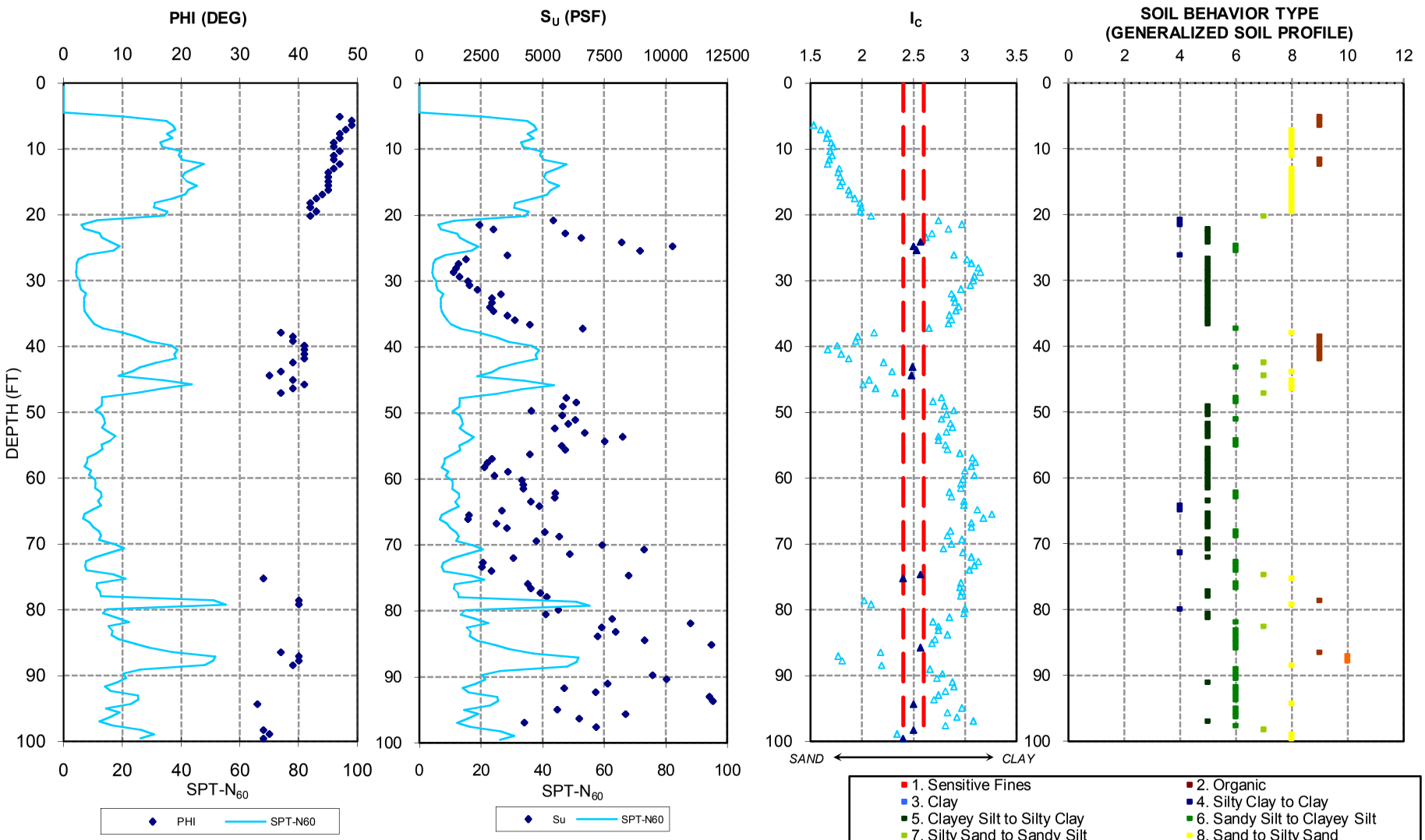
CPT DATA CORRELATION

CPT-09-BH-001



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

BENJAMIN HOLT DRIVE UNDERCROSSING (WIDEN)
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR (BR NO. 29 0194)

PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

PROJECT NAME	***I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-BENJAMIN HOLT DRIVE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0194)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-BH-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*$CRR_{7.5}$*						*F.S.=($CRR_{7.5}$/CSR)*MSF*K_σ*K_α*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	$K\sigma$	$K\alpha$	F.S.
5.085	9	Sand	124.528	1.032	0.316		0.316	1.44	1.45119	0.99	0.129	1.440	1.45	180.71	1.000	180.713		1.00	1	
5.741	9	Sand	219.073	2.207	0.356		0.356	1.4	1.41388	0.99	0.129	1.400	1.41	309.74	1.000	309.743		1.00	1	
6.398	9	Sand	223.368	3.022	0.397		0.397	1.53	1.37758	0.99	0.128	1.530	1.38	307.71	1.000	307.708		1.00	1	
7.054	8	Sand to Silty Sand	223.17	3.429	0.437		0.437	1.6	1.34392	0.99	0.128	1.600	1.34	299.92	1.000	299.923		1.00	1	
7.71	8	Sand to Silty Sand	201.275	3.331	0.476		0.476	1.67	1.31265	0.98	0.128	1.670	1.31	264.20	1.017	268.794		1.00	1	
8.366	8	Sand to Silty Sand	212.411	3.416	0.516		0.516	1.66	1.28205	0.98	0.128	1.660	1.28	272.32	1.010	275.155		1.00	1	
9.022	8	Sand to Silty Sand	187.373	2.886	0.556		0.556	1.7	1.25285	0.98	0.128	1.700	1.25	234.75	1.038	243.637		1.00	1	
9.678	8	Sand to Silty Sand	189.511	3.043	0.595		0.595	1.72	1.22563	0.98	0.127	1.720	1.23	232.27	1.051	244.192		1.00	1	
10.335	8	Sand to Silty Sand	225.979	3.523	0.635		0.635	1.69	1.19891	0.98	0.127	1.690	1.20	270.93	1.031	279.350		1.00	1	
10.991	8	Sand to Silty Sand	222.101	3.457	0.675		0.675	1.71	1.17333	0.98	0.127	1.710	1.17	260.60	1.045	272.221		1.00	1	
11.647	9	Sand	231.043	3.315	0.716		0.716	1.68	1.14823	0.98	0.127	1.680	1.15	265.29	1.024	271.726		1.00	1	
12.303	9	Sand	273.907	4.079	0.756		0.756	1.67	1.12474	0.97	0.127	1.670	1.12	308.08	1.017	313.428		1.00	1	
12.959	8	Sand to Silty Sand	245.211	4.481	0.796		0.796	1.78	1.1022	0.97	0.126	1.780	1.10	270.27	1.092	295.231		1.00	1	
13.615	8	Sand to Silty Sand	228.145	3.714	0.836		0.836	1.77	1.08055	0.97	0.126	1.770	1.08	246.52	1.085	267.562		1.00	1	
14.272	8	Sand to Silty Sand	221.857	3.666	0.875		0.875	1.79	1.06024	0.97	0.126	1.790	1.06	235.22	1.099	258.613		1.00	1	
14.928	8	Sand to Silty Sand	229.415	3.905	0.915		0.915	1.81	1.04019	0.97	0.126	1.810	1.04	238.63	1.114	265.842		1.00	1	
15.584	8	Sand to Silty Sand	249.487	4.199	0.955		0.955	1.79	1.02088	0.97	0.126	1.790	1.02	254.70	1.099	280.024		1.00	1	
16.24	8	Sand to Silty Sand	226.909	4.285	0.994		0.994	1.87	1.00273	0.97	0.126	1.870	1.00	227.53	1.162	264.352		1.00	1	
16.896	8	Sand to Silty Sand	220.394	4.052	1.034		1.034	1.88	0.98478	0.96	0.125	1.880	0.98	217.04	1.171	254.061		0.99	1	
17.552	8	Sand to Silty Sand	191.591	3.627	1.074		1.074	1.93	0.96746	0.96	0.125	1.930	0.97	185.36	1.218	225.850		0.97	1	
18.209	8	Sand to Silty Sand	158.626	2.86	1.113		1.113	1.98	0.95115	0.96	0.125	1.980	0.95	150.88	1.275	192.327		0.97	1	
18.865	8	Sand to Silty Sand	157.249	2.764	1.153		1.153	1.99	0.93498	0.96	0.125	1.990	0.93	147.02	1.287	189.243		0.96	1	
19.521	8	Sand to Silty Sand	180.972	3.458	1.193		1.193	1.98	0.91935	0.96	0.125	1.980	0.92	166.38	1.275	212.086		0.94	1	
20.177	7	Silty Sand to Sandy Silt	168.107	4.092	1.231		1.231	2.09	0.90498	0.96	0.124	2.090	0.90	152.13	1.437	218.603		0.94	1	
20.833	4	Silty Clay to Clay	42.003	1.987	1.269		1.269	2.74	0.89105	0.95	0.124							0.95	1	
21.49	4	Silty Clay to Clay	19.578	0.766	1.307		1.307	2.97	0.87754	0.95	0.124							0.95	1	
22.146	5	Clayey Silt to Silty Clay	23.751	0.739	1.344		1.344	2.84	0.86478	0.95	0.124							0.94	1	
22.802	5	Clayey Silt to Silty Clay	45.762	1.806	1.382		1.382	2.68	0.85205	0.95	0.123							0.94	1	
23.458	5	Clayey Silt to Silty Clay	50.564	1.749	1.419		1.419	2.62	0.84002	0.95	0.123							0.93	1	
24.114	5	Clayey Silt to Silty Clay	62.941	2.264	1.457		1.457	2.57	0.828	0.94	0.123	2.570	0.83	52.12	3.148	164.057		0.93	1	
24.77	6	Sandy Silt to Clayey Silt	78.539	2.791	1.495		1.495	2.5	0.81633	0.94	0.123	2.500	0.82	64.11	2.768	177.494		0.92	1	
25.427	6	Sandy Silt to Clayey Silt	68.679	2.294	1.532	0.013	1.519	2.53	0.80912	0.94	0.123	2.530	0.81	55.57	2.925	162.528		0.92	1	
26.083	4	Silty Clay to Clay	28.242	1.111	1.57	0.034	1.536	2.89	0.80409	0.94	0.125							0.92	1	
26.739	5	Clayey Silt to Silty Clay	15.745	0.4	1.607	0.054	1.553	3.02	0.79913	0.94	0.126							0.92	1	
27.395	5	Clayey Silt to Silty Clay	13.404	0.304	1.645	0.075	1.57	3.06	0.79422	0.93	0.127							0.91	1	
28.051	5	Clayey Silt to Silty Clay	12.712	0.334	1.683	0.095	1.588	3.13	0.7891	0.93	0.128							0.91	1	
28.707	5	Clayey Silt to Silty Clay	11.799	0.298	1.72	0.115	1.605	3.15	0.78431	0.93	0.129							0.91	1	
29.364	5	Clayey Silt to Silty Clay	13.347	0.343	1.758	0.136	1.622	3.09	0.77959	0.92	0.130							0.91	1	
30.02	5	Clayey Silt to Silty Clay	16.012	0.523	1.795	0.156	1.639	3.08	0.77492	0.92	0.131							0.91	1	
30.676	5	Clayey Silt to Silty Clay	16.408	0.489	1.833	0.177	1.656	3.05	0.77031	0.92	0.132							0.90	1	
31.332	5	Clayey Silt to Silty Clay	19.017	0.507	1.871	0.197	1.673	2.96	0.76575	0.91	0.133							0.90	1	
31.988	5	Clayey Silt to Silty Clay	26.013	0.769	1.908	0.218	1.69	2.87	0.76125	0.91	0.134							0.90	1	
32.644	5	Clayey Silt to Silty Clay	23.258	0.621	1.946	0.238	1.707	2.89	0.75679	0.91	0.134							0.90	1	
33.301	5	Clayey Silt to Silty Clay	23.241	0.638	1.983	0.259	1.725	2.9	0.75214	0.90	0.135							0.90	1	
33.957	5	Clayey Silt to Silty Clay	22.861	0.681	2.021	0.279	1.742	2.94	0.74779	0.90	0.135							0.89	1	
34.613	5	Clayey Silt to Silty Clay	23.848	0.671	2.059	0.3	1.759	2.91	0.74349	0.89	0.136							0.89	1	
35.269	5	Clayey Silt to Silty Clay	27.794	0.796	2.096	0.32	1.776	2.85	0.73925	0.89	0.136							0.89	1	
35.925	5	Clayey Silt to Silty Clay	30.674	1.03	2.134	0.341	1.793	2.87	0.73505	0.88	0.137							0.89	1	
36.581	5	Clayey Silt to Silty Clay	35.545	1.27	2.171	0.361	1.81	2.84	0.7309	0.88	0.137							0.89	1	
37.238	6	Sandy Silt to Clayey Silt	51.715	1.472	2.209	0.382	1.827	2.65	0.72679	0.87	0.137							0.89	1	
37.894	8	Sand to Silty Sand	95.985	0.833	2.249	0.402	1.847	2.12	0.72202	0.87	0.138	2.120	0.72	69.30	1.492	103.411	0.18	0.88	1	1.59
38.55	9	Sand	129.778	0.916	2.289	0.423	1.867	1.96	0.71731	0.86	0.138	1.960	0.72	93.09	1.251	116.467	0.23	0.86	1	1.97
39.206	9	Sand	150.246	1.199	2.33	0.443	1.887	1.94	0.71267	0.86	0.138	1.940	0.71	107.08	1.229	131.594	0.29	0.85	1	2.53
39.862	9	Sand	203.282	1.215	2.371	0.464	1.907	1.76	0.70808	0.85	0.138	1.760	0.71	143.94	1.078	155.230	0.43	0.82	1	3.70
40.518	9	Sand	222.447	1.013	2.411	0.484	1.928	1.67	0.70332	0.85	0.138	1.670	0.70	156.45	1.017	159.171	0.46	0.81	1	3.94
41.175	9	Sand	206.684	1.428	2.452	0.504	1.948	1.8	0.69886	0.84	0.138	1.800	0.70	144.44	1.107	159.849	0.46	0.81	1	3.99
41.831	9	Sand	204.133	1.764	2.493	0.525	1.968	1.87	0.69444	0.83	0.137	1.870	0.69	141.76	1.162	164.701		0.81	1	
42.487	7	Silty Sand to Sandy Silt	146.652	2.883	2.532	0.545	1.986	2.21	0.69052	0.83	0.137	2.210	0.69	101.27	1.692	171.304		0.84	1	
43.143	6	Sandy Silt to Clayey Silt	107.325	3.72	2.569	0.566	2.003	2.49	0.68686	0.82	0.137	2.490	0.69	73.72	2.718	200.391		0.87	1	
43.799	8	Sand to Silty Sand	105.783	1.772	2.609	0.586	2.023	2.29	0.68259	0.82	0.137	2.290	0.68	72.21	1.917	138.424	0.33	0.86	1	2.85
44.455	7	Silty Sand to Sandy Silt	77.695	1.743	2.647	0.607	2.041	2.48	0.6788	0.81	0.136	2.480	0.68	52.74	2.669	140.780	0.34	0.87	1	2.97
45.112	8	Sand to Silty Sand	168.413	2.278	2.687	0.627	2.06	2.07	0.67485	0.80	0.136	2.070	0.67	113.65	1.403	159.446	0.46	0.82	1	4.01
45.768	8	Sand to Silty Sand	221.216	3.289	2.727	0.648	2.079	2.01	0.67094	0.80	0.136	2.010	0.67	148.42	1.313	194.920		0.80	1	
46.424	8	Sand to Silty Sand	160.243	2.412	2.767	0.668	2.098	2.13	0.66707	0.79	0.135	2.130	0.67	106.89	1.512	161.596		0.82	1	
47.08	7	Silty Sand to Sandy Silt	112.723	2.148	2.805	0.689	2.116	2.32	0.66345	0.78	0.135	2.320	0.66	74.79	2.015	150.662	0.40	0.85	1	3.52
47.736	6	Sandy Silt to Clayey Silt	46.431	1.493	2.843	0.709	2.134	2.77	0.65987	0.78	0.134							0.86	1	
48.392	6	Sandy Silt to Clayey Silt	48.568	1.284	2.88	0.73	2.151	2.69	0.65652	0.77	0.134							0.86	1	
49.049	5	Clayey Silt to Silty Clay	45.342	1.581	2.918	0.75	2.168	2.8	0.65321	0.76	0.133							0.86	1	
49.705	5	Clayey Silt to Silty Clay	36.026	1.237	2.956	0.771	2.185	2.89	0.64993	0.76	0.133							0.86	1	
50.361	5	Clayey Silt to Silty Clay	45.336	1.641	2.993	0.791	2.202	2.82	0.64668	0.75	0.132							0.85	1	
51.017	6	Sandy Silt to Clayey Silt	49.317	1.661	3.031	0.812	2.219	2.77	0.64346	0.74	0.132							0.85	1	
51.673	5	Clayey Silt to Silty Clay	47.911	2.018	3.068	0.832	2.236	2.86	0.64028	0.74	0.131							0.85	1	
52.329	5	Clayey Silt to Silty Clay	43.652	1.745	3.106	0.852	2.253	2.88	0.63713	0.73	0.131							0.85	1	
52.986	5	Clayey Silt to Silty Clay	52.673	2.15	3.144	0.873	2.271	2.82	0.63382	0.72	0.130							0.85	1	
53.642	5	Clayey Silt to Silty Clay	63.871	2.549	3.181	0.893	2.288	2.74	0.63073	0.72	0.129							0.85	1	
54.298	6	Sandy Silt to Clayey Silt	57.794	2.053	3.219	0.914	2.305	2.74	0.62767	0.71	0.129							0.85	1	
54.954	6	Sandy Silt to Clayey Silt	45.184	1.491	3.256	0.934	2.322	2.81	0.62465	0.70	0.128							0.84	1	
55.61	5	Clayey Silt to Silty Clay	45.796	1.67	3.294	0.955	2.339	2.83	0.62164	0.70	0.128							0.84	1	
56.266	5	Clayey Silt to Silty Clay	35.596	1.331	3.331	0.975	2.356	2.95	0.61867	0.69	0.127							0.84	1	
56.923	5	Clayey Silt to Silty Clay	24.613	0.799	3.369	0.996	2.373	3.07	0.61573	0.69	0.127							0.84	1	
57.579	5	Clayey Silt to Silty Clay	23.337	0.779	3.407	1.016	2.39	3.1	0.61281	0.68	0.126							0.84	1	
58.235	5	Clayey Silt to Silty Clay	21.784	0.575	3.444	1.037	2.408	3.06	0.60976	0.67	0.125							0.84	1	
58.891	5	Clayey Silt to Silty Clay	28.888	0.955	3.482	1.057	2.425	3	0.6069	0.67	0.125							0.84	1	
59.547	5	Clayey Silt to Silty Clay	25.532	0.9	3.519	1.078	2.442	3.09	0.60406	0.66	0.124							0.84	1	
60.203	5	Clayey Silt to Silty Clay	34.042	1.21	3.557	1.098	2.459	2.98	0.60126	0.66	0.124							0.84	1	
60.86	5	Clayey Silt to Silty Clay	34.433	1.165	3.595	1.119	2.476	2.96	0.59848	0.65	0.123							0.83	1	
61.516	5	Clayey Silt to Silty Clay	34.184	1.138	3.632	1.139	2.493	2.96	0.59572	0.65	0.123							0.83	1	
62.172	6	Sandy Silt to Clayey Silt	43.647	1.389	3.67	1.16	2.51	2.85	0.59299	0.64	0.122							0.83	1	
62.828	6	Sandy Silt to Clayey Silt	43.828	1.45	3.707	1.18	2.527	2.87	0.59029	0.64	0.122							0.83	1	
63.484	5	Clayey Silt to Silty Clay	36.99	1.437	3.745	1.201	2.545	2.99	0.58745	0.63	0.121							0.83	1	
64.14	4	Silty Clay to Clay	39.888	1.704	3.783	1.221	2.562	2.99	0.5848	0.63	0.121							0.83	1	
64.797	4	Silty Clay to Clay	28.792	1.181	3.82	1.241	2.579	3.12	0.58216	0.62	0.120							0.83	1	
65.453	5	Clayey Silt to Silty Clay	18.75	0.59	3.858	1.262	2.596	3.26	0.57956	0.62	0.120							0.83	1	
66.109	5	Clayey Silt to Silty Clay	18.359	0.396	3.895	1.282	2.613	3.18	0.57697	0.61	0.119							0.83	1	
66.765	5	Clayey Silt to Silty Clay	26.734	0.756	3.933	1.303	2.63	3.06	0.57441	0.61	0.119							0.82	1	
67.421	5	Clayey Silt to Silty Clay	29.982	1.047	3.971	1.323	2.647	3.06	0.57187	0.61	0.118							0.82	1	
68.077	6	Sandy Silt to Clayey Silt	40.596	1.12	4.008	1.344	2.664	2.86	0.56936	0.60	0.118							0.82	1	
68.734	6	Sandy Silt to Clayey Silt	44.577	1.24	4.046	1.364	2.681	2.83	0.56686	0.60	0.118							0.82	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

	I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT-BENJAMIN		
PROJECT NAME	***HOLT DRIVE UC (WIDEN)***	*FAULT INFO*	
PROJECT NO.	***JOB NO.: 206127.BR (BR NO. 29 0194)***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-BH-001***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

			CPT RESULT INPUT							*CSR*		*CRR 7.5*						*F.S.=(CRR 7.5/(CSR)*MSF*Kσ*Kα*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	Kσ	Kα	F.S.
69.39	5	Clayey Silt to Silty Clay	38.374	1.392	4.083	1.385	2.699	2.97	0.56425	0.60	0.117							0.82	1	
70.046	5	Clayey Silt to Silty Clay	59.443	2.575	4.121	1.405	2.716	2.87	0.5618	0.59	0.117							0.82	1	
70.702	5	Clayey Silt to Silty Clay	72.444	3.06	4.159	1.426	2.733	2.79	0.55937	0.59	0.116							0.82	1	
71.358	4	Silty Clay to Clay	49.351	2.478	4.196	1.446	2.75	2.98	0.55696	0.58	0.116							0.82	1	
72.014	5	Clayey Silt to Silty Clay	32.358	1.157	4.234	1.467	2.767	3.06	0.55458	0.58	0.116							0.82	1	
72.671	6	Sandy Silt to Clayey Silt	22.169	0.569	4.271	1.487	2.784	3.13	0.55221	0.58	0.115							0.81	1	
73.327	6	Sandy Silt to Clayey Silt	21.489	0.461	4.309	1.508	2.801	3.09	0.54986	0.58	0.115							0.81	1	
73.983	6	Sandy Silt to Clayey Silt	24.284	0.533	4.347	1.528	2.818	3.04	0.54754	0.57	0.115							0.81	1	
74.639	7	Silty Sand to Sandy Silt	67.06	1.114	4.385	1.549	2.837	2.57	0.54496	0.57	0.114	2.570	0.54	36.54	3.148	115.042	0.22	0.81	1	2.31
75.295	8	Sand to Silty Sand	90.526	1.146	4.425	1.569	2.856	2.4	0.54241	0.57	0.114	2.400	0.54	49.10	2.312	113.542	0.22	0.81	1	2.26
75.951	6	Sandy Silt to Clayey Silt	36.457	1.037	4.462	1.589	2.873	2.96	0.54014	0.56	0.114							0.81	1	
76.608	6	Sandy Silt to Clayey Silt	36.27	1.047	4.5	1.61	2.89	2.95	0.5379	0.56	0.114							0.81	1	
77.264	5	Clayey Silt to Silty Clay	39.451	1.376	4.538	1.63	2.907	2.97	0.53567	0.56	0.113							0.81	1	
77.92	5	Clayey Silt to Silty Clay	40.863	1.503	4.575	1.651	2.924	2.96	0.53346	0.56	0.113							0.81	1	
78.576	9	Sand	256.767	3.12	4.616	1.671	2.945	2.02	0.53076	0.55	0.113	2.020	0.53	136.28	1.327	180.847		0.72	1	
79.232	8	Sand to Silty Sand	270.851	4.443	4.656	1.692	2.964	2.09	0.52834	0.55	0.113	2.090	0.53	143.10	1.437	205.625		0.72	1	
79.888	4	Silty Clay to Clay	46.674	2.065	4.693	1.712	2.981	3	0.52619	0.55	0.112							0.80	1	
80.545	5	Clayey Silt to Silty Clay	42.258	1.588	4.731	1.733	2.998	2.99	0.52406	0.55	0.112							0.80	1	
81.201	5	Clayey Silt to Silty Clay	61.365	2.437	4.768	1.753	3.015	2.85	0.52195	0.54	0.112							0.80	1	
81.857	6	Sandy Silt to Clayey Silt	83.313	2.834	4.806	1.774	3.032	2.69	0.51985	0.54	0.112							0.80	1	
82.513	7	Silty Sand to Sandy Silt	55.854	1.367	4.845	1.794	3.05	2.74	0.51765	0.54	0.111							0.80	1	
83.169	6	Sandy Silt to Clayey Silt	60.827	1.643	4.882	1.815	3.068	2.74	0.51546	0.54	0.111							0.80	1	
83.825	6	Sandy Silt to Clayey Silt	56.557	1.816	4.92	1.835	3.085	2.83	0.51342	0.54	0.111							0.80	1	
84.482	6	Sandy Silt to Clayey Silt	69.252	1.978	4.957	1.856	3.102	2.71	0.51139	0.53	0.111							0.80	1	
85.138	6	Sandy Silt to Clayey Silt	90.259	3.166	4.995	1.876	3.119	2.68	0.50938	0.53	0.111							0.80	1	
85.794	6	Sandy Silt to Clayey Silt	115.723	3.407	5.033	1.897	3.136	2.57	0.50738	0.53	0.111	2.570	0.51	58.72	3.148	184.834		0.80	1	
86.45	9	Sand	175.069	2.107	5.073	1.917	3.156	2.18	0.50505	0.53	0.110	2.180	0.51	88.42	1.619	143.164	0.35	0.76	1	3.81
87.106	10	Gravelly Sand to Sand	285.706	1.346	5.115	1.938	3.178	1.77	0.50251	0.53	0.110	1.770	0.50	143.57	1.085	155.824	0.43	0.69	1	4.68
87.762	10	Gravelly Sand to Sand	277.637	1.491	5.157	1.958	3.199	1.81	0.50011	0.52	0.110	1.810	0.50	138.85	1.114	154.680	0.42	0.69	1	4.60
88.419	8	Sand to Silty Sand	226.563	3.798	5.197	1.978	3.218	2.19	0.49796	0.52	0.110	2.190	0.50	112.82	1.643	185.321		0.73	1	
89.075	6	Sandy Silt to Clayey Silt	99.525	3.428	5.234	1.999	3.235	2.66	0.49605	0.52	0.110							0.79	1	
89.731	6	Sandy Silt to Clayey Silt	72.65	2.578	5.272	2.019	3.252	2.78	0.49416	0.52	0.110							0.79	1	
90.387	6	Sandy Silt to Clayey Silt	77.541	2.504	5.309	2.04	3.269	2.73	0.49228	0.52	0.109							0.79	1	
91.043	5	Clayey Silt to Silty Clay	60.832	2.304	5.347	2.06	3.287	2.88	0.49031	0.52	0.109							0.79	1	
91.699	6	Sandy Silt to Clayey Silt	47.128	1.287	5.385	2.081	3.304	2.89	0.48845	0.52	0.109							0.79	1	
92.356	6	Sandy Silt to Clayey Silt	55.973	1.498	5.422	2.101	3.321	2.81	0.48662	0.51	0.109							0.79	1	
93.012	6	Sandy Silt to Clayey Silt	91.796	3.559	5.46	2.122	3.338	2.74	0.4848	0.51	0.109							0.79	1	
93.668	6	Sandy Silt to Clayey Silt	94.778	3.078	5.497	2.142	3.355	2.7	0.48299	0.51	0.109							0.78	1	
94.324	8	Sand to Silty Sand	94.149	1.428	5.537	2.163	3.374	2.5	0.48098	0.51	0.109	2.500	0.48	45.28	2.768	125.365	0.26	0.78	1	2.89
94.98	6	Sandy Silt to Clayey Silt	46.192	1.524	5.575	2.183	3.391	2.97	0.4792	0.51	0.109							0.78	1	
95.636	6	Sandy Silt to Clayey Silt	65.674	2.288	5.612	2.204	3.409	2.83	0.47733	0.51	0.108							0.78	1	
96.293	6	Sandy Silt to Clayey Silt	52.208	1.768	5.65	2.224	3.426	2.92	0.47557	0.51	0.108							0.78	1	
96.949	5	Clayey Silt to Silty Clay	36.276	1.202	5.687	2.245	3.443	3.08	0.47383	0.50	0.108							0.78	1	
97.605	6	Sandy Silt to Clayey Silt	57.748	1.417	5.725	2.265	3.46	2.81	0.4721	0.50	0.108							0.78	1	
98.261	7	Silty Sand to Sandy Silt	108.335	1.934	5.764	2.286	3.478	2.5	0.47029	0.50	0.108	2.500	0.47	50.95	2.768	141.048	0.34	0.78	1	3.76
98.917	8	Sand to Silty Sand	135.596	1.829	5.803	2.306	3.497	2.34	0.46838	0.50	0.108	2.340	0.47	63.51	2.084	132.342	0.30	0.78	1	3.26
99.573	8	Sand to Silty Sand	112.7	1.424	5.843	2.326	3.516	2.4	0.4665	0.50	0.108	2.400	0.47	52.57	2.312	121.571	0.25	0.78	1	2.73
100.23	6	Sandy Silt to Clayey Silt	63.231	1.958	5.881	2.347	3.534	2.84	0.46472	0.50	0.108							0.78	2	
100.886	5	Clayey Silt to Silty Clay	88.422	3.679	5.918	2.367	3.551	2.8	0.46306	0.50	0.108							0.78	3	
101.542	5	Clayey Silt to Silty Clay	97.726	3.821	5.956	2.388	3.568	2.76	0.46141	0.50	0.108							0.78	4	
102.198	7	Silty Sand to Sandy Silt	117.945	2.749	5.994	2.408	3.586	2.55	0.45967	0.49	0.107	2.550	0.46	54.22	3.034	164.505		0.77	5	
102.854	5	Clayey Silt to Silty Clay	80.155	3.263	6.032	2.429	3.603	2.85	0.45805	0.49	0.107							0.77	6	
103.511	6	Sandy Silt to Clayey Silt	70.423	2.381	6.07	2.449	3.62	2.83	0.45643	0.49	0.107							0.77	7	
104.167	6	Sandy Silt to Clayey Silt	69.326	2.453	6.107	2.47	3.637	2.84	0.45483	0.49	0.107							0.77	8	
104.823	6	Sandy Silt to Clayey Silt	78.93	2.177	6.145	2.49	3.655	2.72	0.45314	0.49	0.107							0.77	9	
105.479	6	Sandy Silt to Clayey Silt	75.608	2.271	6.182	2.511	3.672	2.76	0.45156	0.49	0.107							0.77	10	
106.135	6	Sandy Silt to Clayey Silt	65.924	2.217	6.22	2.531	3.689	2.85	0.44999	0.49	0.107							0.77	11	
106.791	5	Clayey Silt to Silty Clay	58.503	2.209	6.258	2.552	3.706	2.95	0.44843	0.49	0.107							0.77	12	
107.448	5	Clayey Silt to Silty Clay	49.453	1.728	6.295	2.572	3.723	3	0.44688	0.49	0.107							0.77	13	
108.104	6	Sandy Silt to Clayey Silt	38.55	1.146	6.333	2.593	3.74	3.08	0.44534	0.49	0.107							0.77	14	
108.76	6	Sandy Silt to Clayey Silt	36.877	1.083	6.37	2.613	3.757	3.09	0.44382	0.48	0.107							0.77	15	
109.416	5	Clayey Silt to Silty Clay	41.747	1.438	6.408	2.634	3.774	3.09	0.4423	0.48	0.107							0.77	16	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

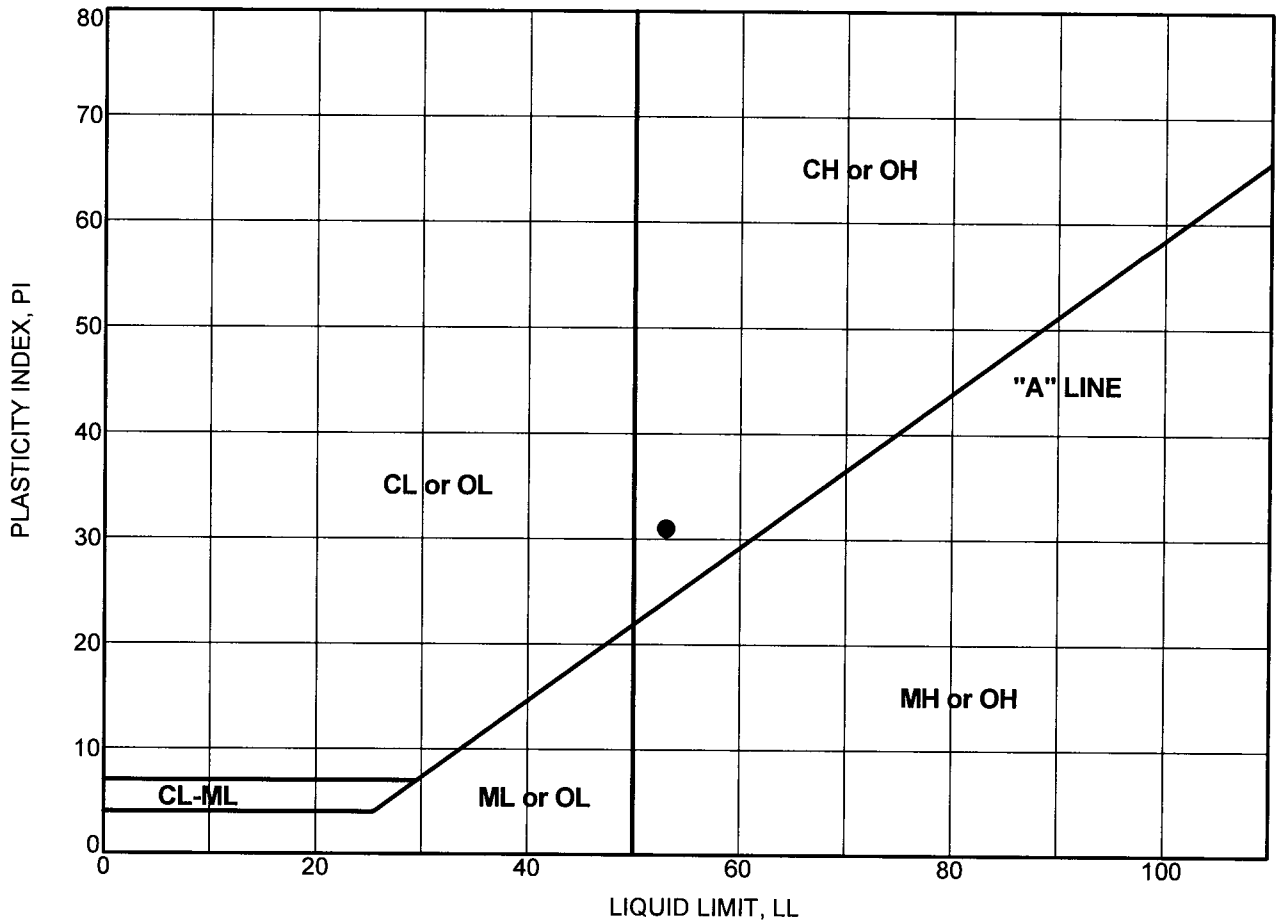
The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The results of the test are presented on Plate B-2, "Plasticity Chart".

Unconfined Compression Tests

Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-4A through B-4C.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-3.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
R-09-001	MC-5	26.0	●	27	53	22	31	FAT CLAY (CH)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Benjamin Holt Drive UC (Widen)
Stockton, California

JOB NO: 206127.BR (BHB)

PLATE NO: B-2

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/06/2009
Date Submitted 05/01/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR I-5 NORTH Site ID : R-09-BH-001#6@31.0'
Thank you for your business.

* For future reference to this analysis please use SUN # 55616-112071.

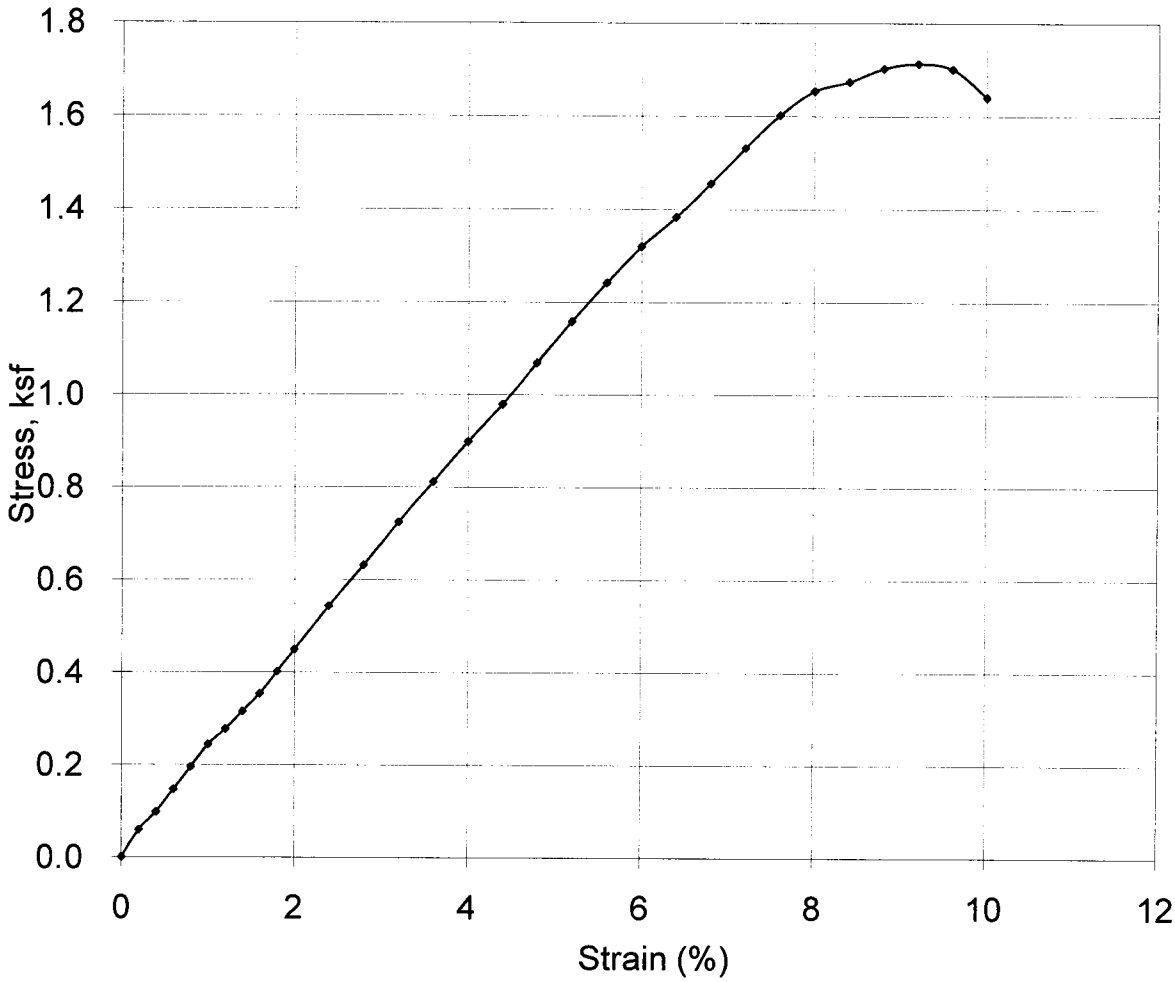
EVALUATION FOR SOIL CORROSION

Soil pH	7.41		
Minimum Resistivity	0.91 ohm-cm (x1000)		
Chloride	42.4 ppm	00.00424	%
Sulfate	35.3 ppm	00.00353	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 8 2009
By

UNCONFINED COMPRESSION TEST



Boring No.: R-09-BH-001

Sample No. : MC-7

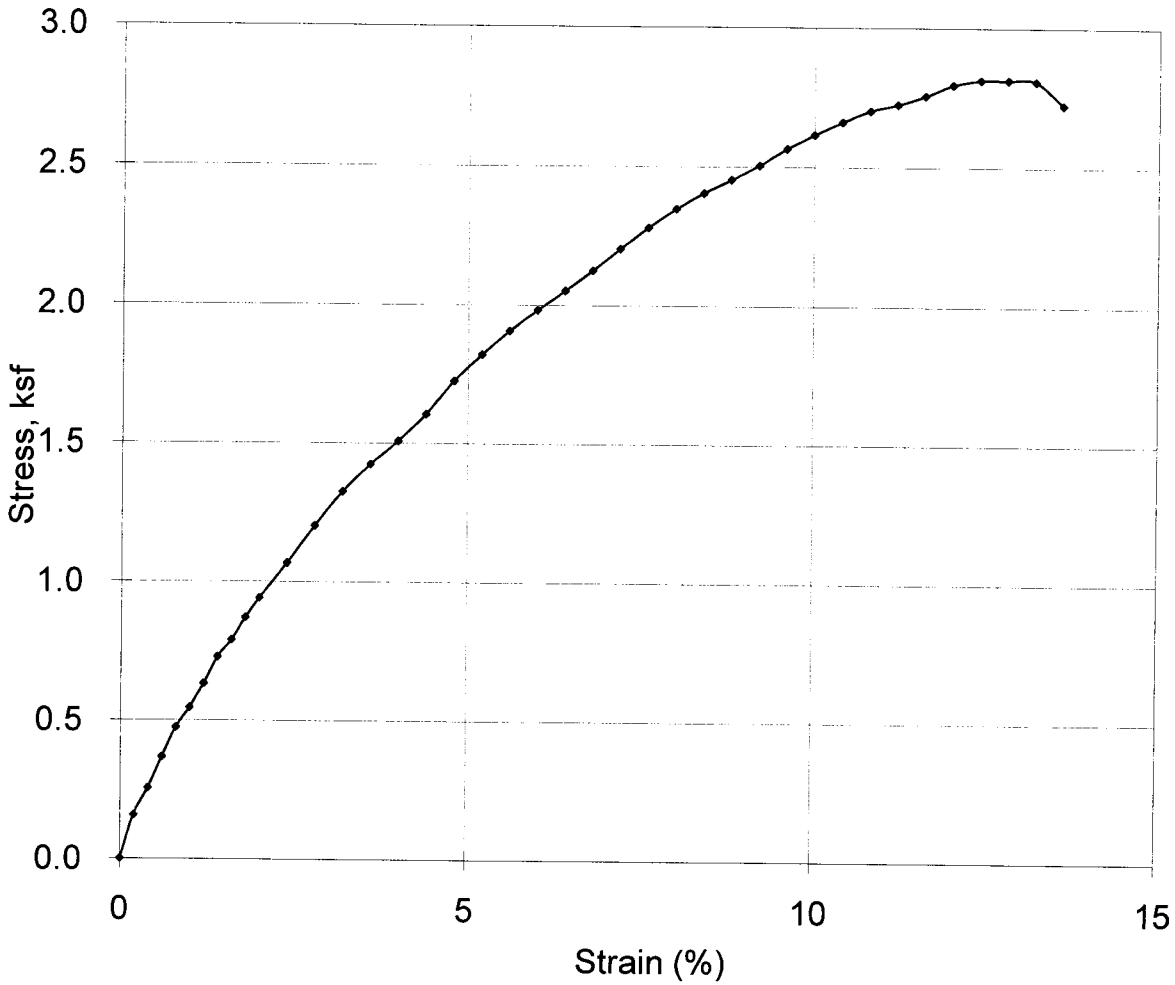
Depth (feet): 36

Material Description:
Lean Clay

Unconfined Compressive Strength: 1.7 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**BENJAMIN HOLT DRIVE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	**PLATE NO.: B-4A**

UNCONFINED COMPRESSION TEST



Boring No.: R-09-BH-001

Sample No. : MC-11

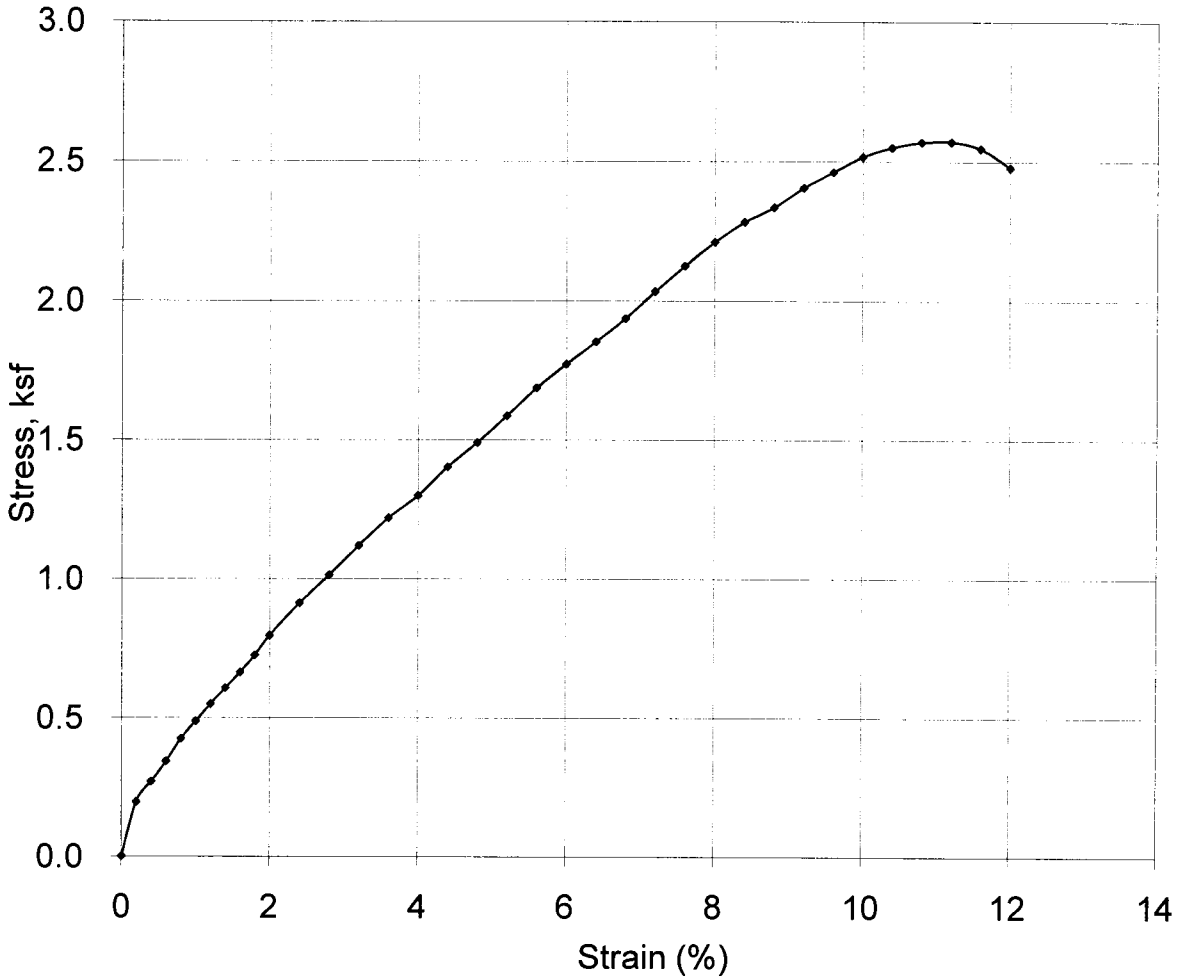
Depth (feet): 56

Material Description:
Lean Clay

Unconfined Compressive Strength: 2.8 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**BENJAMIN HOLT DRIVE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-4B

UNCONFINED COMPRESSION TEST



Boring No.: R-09-BH-001

Sample No. : MC-14

Depth (feet): 71

Material Description:
Lean Clay

Unconfined Compressive Strength: 2.6 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**BENJAMIN HOLT DRIVE. UC (WIDEN)** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-4C

APPENDIX C

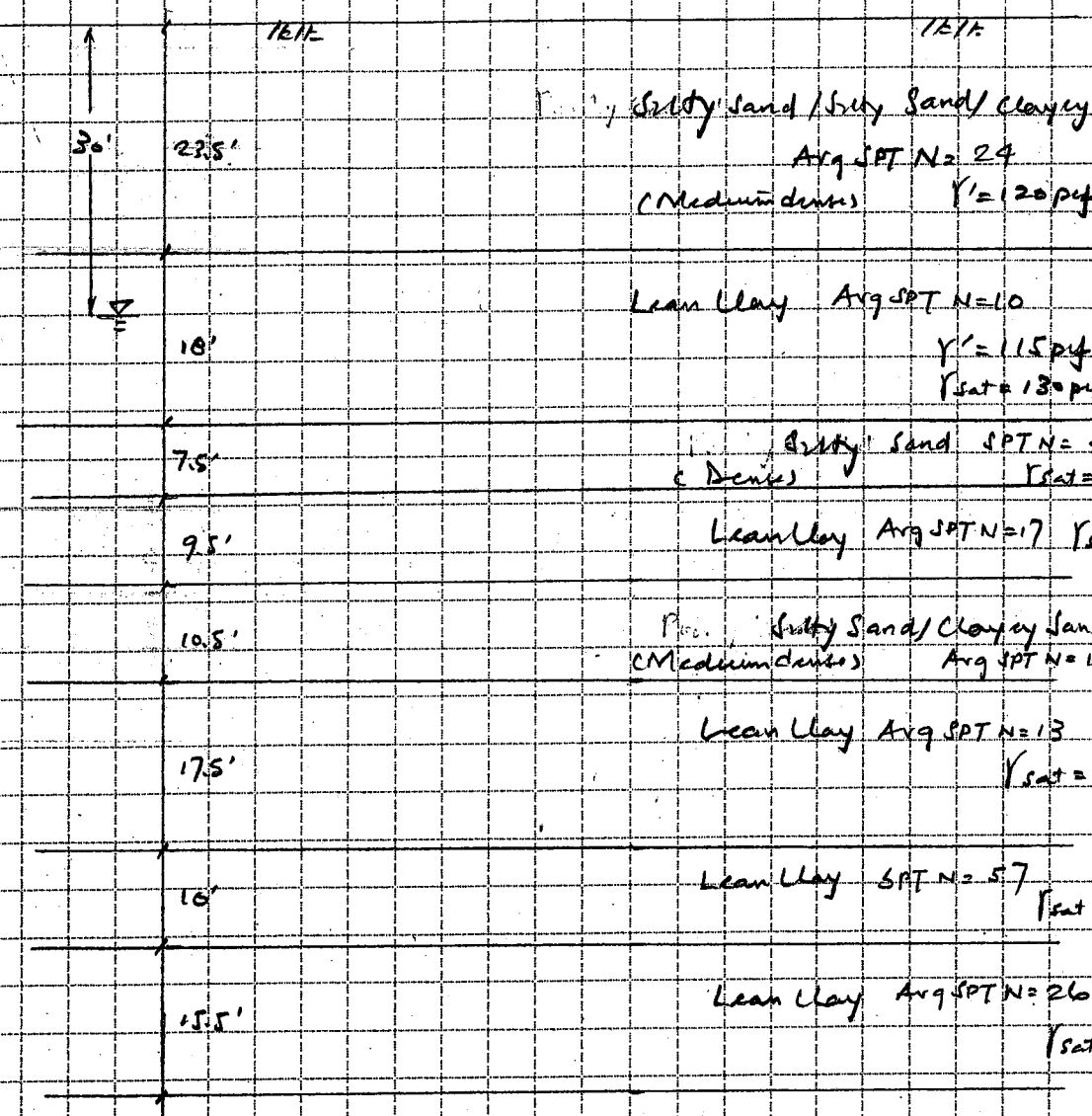
Evaluation of Liquefaction Potential

PARIKH

Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement 206127.001
SHEET NO OF
CALCULATED BY A. Lam DATE 5/2009
CHECKED BY DATE
SCALE

Evaluation of Liquefaction Potential - Benjamin Holt Drive UC (Boring A-09-BH-001)



LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-BH-001 (Benjamin Holt Drive UC)(Widen)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 30

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
*F.S.=(CRR $_{7.0}$/CSR)*MSF*K σ*K α*

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE ($CRR_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	23.5	1	24	SPT	2820.0	2820.0	0.89	0.12	28	0.84	1	0.95	1.2	1.15	30.57	15%	34.54		0.90	1	
2	41.5	2	10	SPT	5062.5	4343.8	0.74		12	0.68	1	0.80	1.2	1.15	8.64	55%			0.79	1	
3	49.0	1	41	SPT	6075.0	4887.5	0.66	0.11	47	0.64	1	0.80	1.2	1.15	33.41	15%	37.51		0.76	1	
4	58.5	2	17	SPT	7310.0	5481.3	0.59		20	0.60	1	0.80	1.2	1.15	13.08	55%			0.74	1	
5	69.0	1	16	SPT	8675.0	6190.0	0.53	0.10	18	0.57	1	0.80	1.2	1.15	11.59	15%	14.64	0.16	0.71	1	1.37
6	86.5	2	13	SPT	10950.0	7371.3	0.49		15	0.52	1	0.80	1.2	1.15	8.63	55%			0.68	1	
7	96.5	2	57	SPT	12250.0	8046.3	0.47		66	0.50	1	0.80	1.2	1.15	36.20	55%			0.66	1	
8	111.5	2	26	SPT	14200.0	9058.8	0.46		30	0.47	1	0.80	1.2	1.15	15.56	55%			0.64	1	

Vertical Pile Capacity Calculations

FOUNDATION DESIGN INFORMATION FROM BRIDGE DESIGNER TO GEOTECHNICAL ENGINEER

BENJAMIN HOLT DRIVE UC (WIDEN)

Table 1. Foundation Design Data Sheet

Location	Design Method	Pile Type	Finish Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	No. of Piles Per Support
					B	L		
Abut 1	WSD	Class 140	12.50	9.50	2.50	54.60	1.00	14
Abut 2	WSD	Class 140	12.50	9.50	2.50	54.60	1.00	14

Note: Finish Grade is at the front face of abutment diaphragm; Pile Cut-off Elevations are the Bottom of Footing Elevations

Table 2. Foundation Design Loads

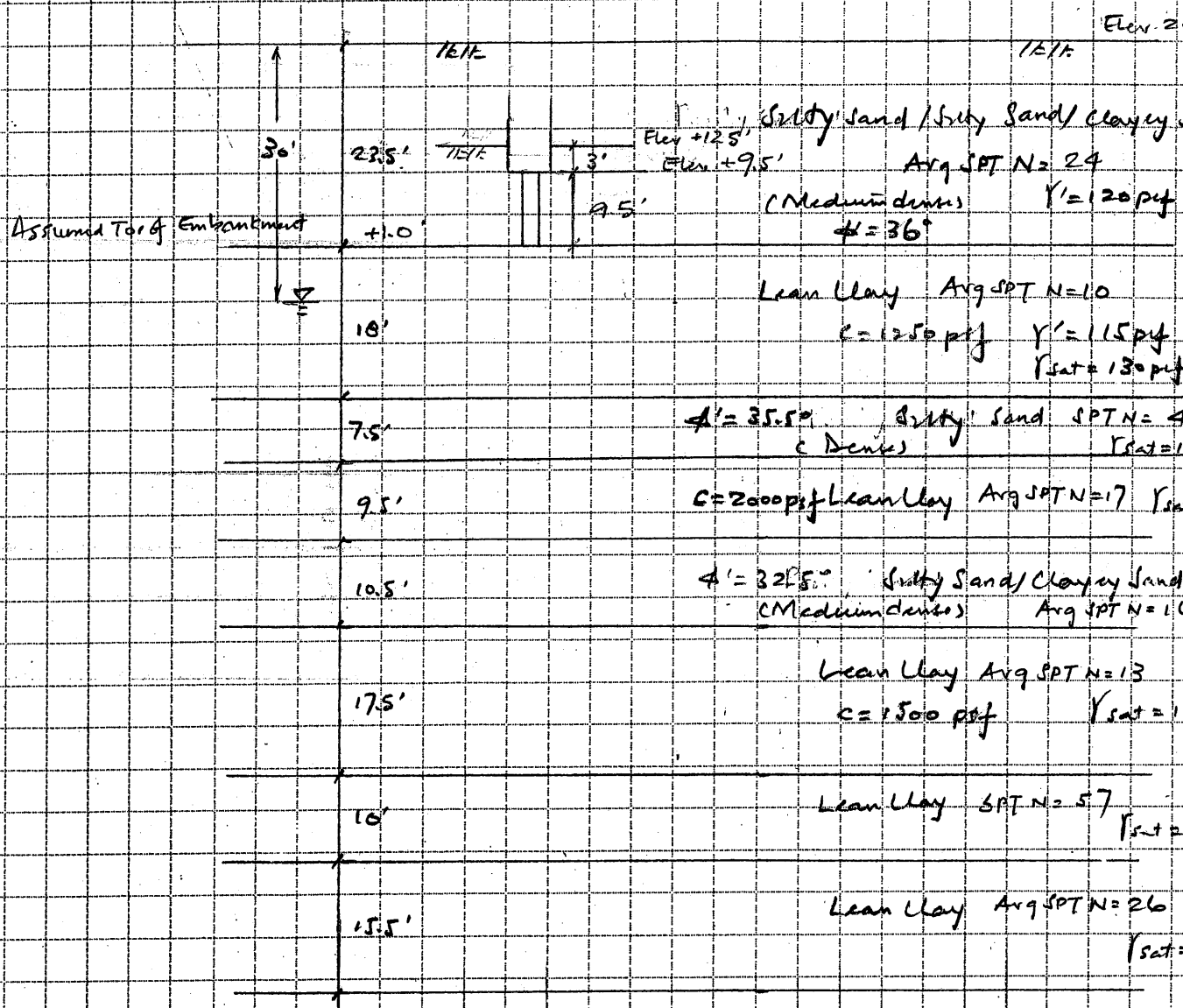
Location	Service Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent	Compression		Tension		Compression		Tension	
	Per Support	Max per Pile	Per Support	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile
Abut 1	1,926	**138**	1,376	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	1,926	**138**	1,376	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 3. Lateral Load on Piles

Location	Service Limit (kips / pile)	Extreme Limit (kips / pile)
Abut 1	Negligible	27
Abut 2	Negligible	27

About 1

Pile Capacity Analyses - Benjamin Holt Drive UC (Boring R-09-0013) 14" ⌀ AH "W" pile - Service load 69 tons



(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Benjamin Holt Drive UC (Widen)*** ***Abut 1 - 70 tons service load (140 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/16/10

206127.BR

Design per FHWA Manual

PILE DIA, B (ft) = 1.17

PILE SHAPE = 1

(1=CIRCLE, 2=SQ., 3=OCT.)

PILE MATERIAL = 2

(1=CONCRETE ,2=STEEL)

DRIVEN/DRILLED = 1

(1=DRIVEN ,2=DRILLED)

GROUPS INCREAMENTS

1- GRAVELS, SANDS (FT)= 5

AND NONPLASTIC SILTS

2- CLAYS AND PLASTIC SILTS

SOIL TYPES

GRAVEL, SAND, NSILT Ref Boring : R-09-001 (PARIKH 2009)

CLAY, PSILT

F.S. FOR COMPRESSION = 1

F.S. FOR UPLIFT = 1

PERIMETER OF THE PILE (ft) : 3.67 SQ. FT.

AREA OF THE PILE TIP 1.07 SQ. FT.

Class 140 Alt. "W" - 14" diameter steel pipe pile

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	120	360	180	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	12.5		1	120	1140	930	SAND	V. DENSE	0	39	0.00	17.95	17.95	6.01	6.01
12.5	19.0		2	115	747.5	1874	CLAY	STIFF	1250	0	0.85	12.67	30.63	12.67	18.69
19.0	24.5		2	68	374	2435	CLAY	STIFF	1250	0	0.85	10.72	41.35	10.72	29.41
24.5	30.5		2	68	408	2826	CLAY	STIFF	1250	0	0.85	11.70	53.05	11.70	41.11
30.5	38.0		1	73	547.5	3303	SAND	V. DENSE	0	39	0.00	37.47	90.52	0.00	41.11
38.0	43.0		2	68	340	3200	CLAY	STIFF	2000	0	0.60	11.01	101.53	11.01	52.12
43.0	47.5		2	68	306	4070	CLAY	STIFF	2000	0	0.60	9.91	111.44	9.91	62.03
47.5	57.0		1	68	646	4546	SAND	MED DEN	0	34	0.00	31.39	142.83	10.51	72.54
57.0	58.0		1	68	68	4903	SAND	MED DEN	0	34	0.00	3.29	146.11	0.00	72.54
58.0	62.0		2	68	272	5005	CLAY	STIFF	1500	0	0.90	9.91	156.02	9.91	82.45
62.0	66.0		2	68	272	5345	CLAY	STIFF	1500	0	0.95	10.46	166.48 ←	10.46	92.91
66.0	71.0		2	68	340	5651	CLAY	STIFF	1500	0	1.00	13.76	180.24	11.01	103.92
71.0	75.5		2	68	306	5974	CLAY	STIFF	1500	0	1.00	12.39	192.63	9.91	113.83
75.5	80.5		2	68	340	6297	CLAY	V.STIFF	4000	0	0.30	11.01	203.64	0.00	113.83
80.5	85.5		2	68	340	5651	CLAY	V.STIFF	4000	0	0.30	11.01	214.65	11.01	124.84
85.5	91.0		2	68	374	6008	CLAY	V.STIFF	3000	0	0.40	12.11	226.76	12.11	136.95
91.0	96.0		2	68	340	6977	CLAY	V.STIFF	3000	0	0.40	11.01	237.77	11.01	147.96
96.0	101.0		2	68	340	7351	CLAY	V.STIFF	3000	0	0.40	11.01	248.78	0.00	147.96

Abutment 1 --> Pile cut-off elevation is +9.5 feet. Finished grade elevation is +12.5 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +1.0 feet

Pile Length @ Abutment 1 = 66.0' - 3.0' = 63.0'

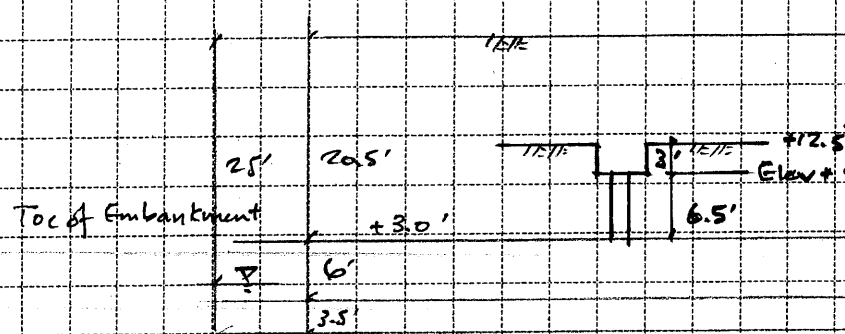
Pile tip at approxiamtely 63.0' below pile cut-off(pile tip @Elev. -53.5', pile cut-off @Elev. +9.5'), available Ult. Pile Capacity=166.5T (Friction)-18.0T(Predrilled)=148.5T > Nominal Resistance (280k/140T), OK

I-5 Benjamin Holt Dr. UC Abut 1 Class 140 Alt W (4-2010)

Sheet 2 of 5

About 2

Pile Capacity Analysis - Benjamin Holt Drive UC (Widen) CPT-09-001 14" ⌀ Alt "W" pile
Service Load 69 tons



Clay c = 2500 psf γ' = 120 pcf γsat = 130 pcf
Clay c = 1500 psf γsat = 130 pcf
Clay c = 2000 psf γsat = 130 pcf
Clay c = 3000 psf γsat = 130 pcf
Sand ⌀' = 36° γsat = 135 pcf
Clay c = 4000 psf γsat = 130 pcf
Clay c = 2750 psf γsat = 130 pcf
Clay c = 4000 psf γsat = 130 pcf
Clay c = 2500 psf γsat = 130 pcf
Clay c = 4000 psf γsat = 130 pcf
Clay c = 2500 psf γsat = 130 pcf
Clay c = 4000 psf γsat = 130 pcf
Sand ⌀' = 36° γsat = 135 pcf
Clay c = 4000 psf γsat = 130 pcf

(TEMPLATE)

DRIVEN PILE CAPACITY - ***I-5 Benjamin Holt Drive UC (Widen)*** ***Abut 2 - 70 tons service load (140 tons)*** **ALLOWABLE FRICTIONAL PILE CAPACTY** 04/16/10

Design per FHWA Manual — GROUPS — INCREAMENTS — 206127.BR

PILE DIA, B (ft) = 1.16 — 1- GRAVELS, SANDS AND NONPLASTIC SILTS (FT)= 5 — F.S. FOR COMPRESSION = 1

PILE SHAPE = 1 (1=CIRCLE, 2=SQ., 3=OCT.) — 2- CLAYS AND PLASTIC SILTS — F.S. FOR UPLIFT = 1

PILE MATERIAL = 2 (1=CONCRETE ,2=STEEL) — SOIL TYPES — GRAVEL, SAND, NSILT Ref Boring : CPT-09-001 (PARIKH 2009) — PERIMETER OF THE PILE (ft) : 3.67 SQ. FT.

DRIVEN/DRILLED = 1 (1=DRIVEN ,2=DRILLED) — CLAY, PSILT — AREA OF THE PILE TIP 1.07 SQ. FT.

Class 140 Alt. "W" - 14" diameter steel pipe pile

DEPTH FROM	TO		SOIL GROUP	EFFECTIVE UNIT WEIGHT(pcf)	WEIGHT OF LAYER (psf)	EFFECTIVE OVERBURDEN PRESSURE (psf)	SOIL TYPE	CONSISTENCY	SHEAR STRENGTH PARAMETERS c(psf)	ϕ (°)	FHWA Manual (Tomlinson, 1980) α	PER SECTION COMP. (Ton)	ULTIMATE COMPRESSION CAPACITY (Ton)	PER SECTION UPLIFT (Ton)	ULTIMATE UPLIFT CAPACITY (Ton)
0.0	3.0	BT2 FTG	1	125	375	188	SAND		0		0.00	0.00	0.00	0.00	0.00
3.0	9.5		1	125	812.5	781	SAND	DENSE	0	36	0.00	9.60	9.60	3.21	3.21
9.5	14.0		2	120	540	1458	CLAY	V.STIFF	2500	0	0.40	8.26	17.85	8.26	11.47
14.0	15.5		2	68	102	1779	CLAY	V.STIFF	2500	0	0.40	2.75	20.61	2.75	14.22
15.5	19.0		2	68	238	1949	CLAY	STIFF	1500	0	0.85	8.19	28.80	8.19	22.41
19.0	21.5		2	68	170	2153	CLAY	STIFF	2000	0	0.70	6.42	35.22	4.82	27.23
21.5	26.0		2	68	306	2221	CLAY	V.STIFF	3000	0	0.35	8.67	43.89	8.67	35.90
26.0	31.0		1	73	365	2726	SAND	DENSE	0	36	0.00	16.67	60.56	5.59	41.49
31.0	36.0		1	73	365	3091	SAND	DENSE	0	36	0.00	16.54	77.10	5.54	47.03
36.0	41.0		2	68	340	3444	CLAY	V.STIFF	4000	0	0.25	9.18	86.28	9.18	56.20
41.0	45.5		2	68	306	3427	CLAY	V.STIFF	4000	0	0.25	8.26	94.54	8.26	64.46
45.5	49.0		2	68	238	4039	CLAY	V.STIFF	2750	0	0.45	7.95	102.48	7.95	72.41
49.0	53.0		2	68	272	4294	CLAY	V.STIFF	4000	0	0.30	8.81	111.29	11.93	84.34
53.0	56.5		2	68	238	4549	CLAY	V.STIFF	2500	0	0.60	9.63	120.92	7.71	92.04
56.5	60.5		2	68	272	4804	CLAY	V.STIFF	4000	0	0.30	8.81	129.73	8.81	100.85
60.5	63.0		2	68	170	4243	CLAY	V.STIFF	2500	0	0.65	7.45	137.19	7.45	108.31
63.0	72.0		2	68	612	4736	CLAY	V.STIFF	4000	0	0.30	19.82	157.01 ←	19.82	128.12
72.0	75.0		2	68	204	5212	CLAY	V.STIFF	4000	0	0.30	6.61	163.61	6.61	134.73
75.0	78.0		1	73	219	5831	SAND	DENSE	0	36	0.00	9.88	173.49	0.00	134.73
78.0	89.0		2	68	748	6300	CLAY	V.STIFF	4000	0	0.30	24.22	197.71	0.00	134.73

Abutment 2 --> Pile cut-off elevation is +9.5 feet. Finished grade elevation is +12.5 feet

Pre-drill through the exsiting embankment to toe of existing embankment. Toe of existing embankment assumed to be at Elev. +3.0 feet

Pile Length @ Abutment 1 = 72.0' - 3.0' = 69.0'

Pile tip at approxiamtely 69.0' below pile cut-off(pile tip @Elev. -59.5', pile cut-off @Elev. +9.5'), available Ult. Pile Capacity =157.0T (Friction)-9.6T (Predrilled)=147.4T > Nominal Resistance (280k/140T), OK

I-5 Benjamin Holt Dr. UC Abut 2 Class 140 Alt W (4-2010)

Sheet 4 of 5

BENJAMIN HOLT DRIVE UC (WIDEN)
Sheet 5 of 5

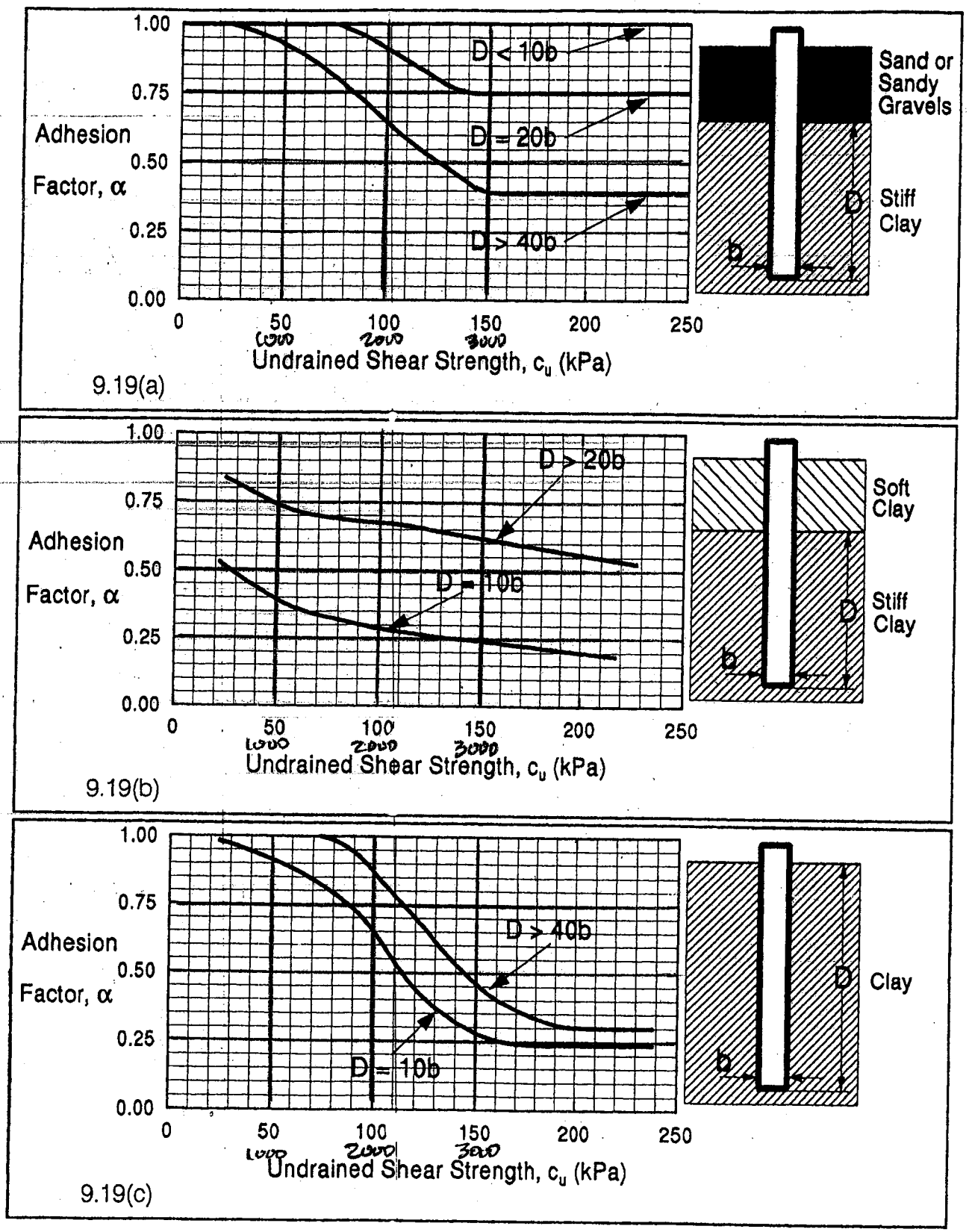


Figure 9.19 Adhesion Factors for Driven Piles in Clay (after Tomlinson, 1980)

Geotechnical Parameters for LPILE (v.5) Analyses

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Benjamin Holt Drive UC (Widen)

Boring R-09-BH-001 (Abutment 1) (16-inch dia. Alt "W" pile) **10-15-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
24.5* to 1.0	0 to 23.5	Sand (medium dense)	ϕ = 35.5°; c= 0 psf	4, Sand	90	-	0.069	-
1.0 to -5.5	23.5 to 30	Clay (firm to stiff)	ϕ = 0°; c=1250 psf	3, Clay	500	0.007	0.067	-
-5.5 to -17.0	30 to 41.5	Clay (firm to stiff)	ϕ = 0°; c=1250 psf	3, Clay	500	0.007	0.039	Assumed GWL at 30 feet deep below existing ground surface for design.
-17.0 to -24.5	41.5 to 49	Sand (dense)	ϕ = 35.5°; c= 0 psf	4, Sand	125	-	0.042	-
-24.5 to -34.0	49 to 58.5	Clay (stiff to very stiff)	ϕ = 0°; c=2000 psf	3, Clay	500	0.007	0.039	-
-34.0 to -44.5	58.5 to 69	Sand (medium dense)	ϕ = 32°; c= 0 psf	4, Sand	60	-	0.039	-
-44.5 to -62.0	69 to 86.5	Clay (stiff)	ϕ = 0°; c=1500 psf	3, Clay	500	0.007	0.039	-
-62.0 to -72.0	86.5 to 96.5	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-72.0 to -87.5	96.5 to 112	Clay (very stiff)	ϕ = 0°; c=3000 psf	3, Clay	1000	0.005	0.039	-

*Finish ground elevation of +12.50 feet is used in the vertical pile 5apacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT
GENERALIZED VERTICAL SOIL PROFILE AND STRENGTH PARAMETERS
Benjamin Holt Drive UC (Widen)

CPT-09-BH-001 (Abutment 2) (16-inch dia. Alt "W" pile) **10-15-09**

Elevation (ft)	Approx. Depth. (ft.)	Generalized Vertical Soil Profile	Strength Parameters	Soil Type	Soil Modulus Parameter k (pci)	Soil Strain Parameter ε_{50}	Effective Unit Weight γ' (pci)	Remark
23.5* to 3.0	0 to 20.5	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	225	-	0.072	-
3.0 to -1.5	20.5 to 25	Clay (very stiff)	ϕ = 0°; c=2500 psf	3, Clay	1000	0.005	0.069	-
-1.5 to -3.0	25 to 26.5	Clay (very stiff)	ϕ = 0°; c=2500 psf	3, Clay	1000	0.005	0.039	Assumed GWL at 25 feet deep below existing ground surface for design.
-3.0 to -6.5	26.5 to 30	Clay (very stiff)	ϕ =0°; c= 1500 psf	3, Clay	500	0.007	0.039	-
-6.5 to -9.0	30 to 32.5	Clay (very stiff)	ϕ = 0°; c= 2000 psf	3, Clay	500	0.007	0.039	
-9.0 to -13.5	32.5 to 37	Clay (very stiff)	ϕ = 0°; c= 3000 psf	3, Clay	1000	0.005	0.039	
-13.5 to -23.5	37 to 47	Sand (dense)	ϕ = 35°; c= 0 psf	4, Sand	125	-	0.042	-
-23.5 to -33.0	47 to 56.5	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-33.0 to -36.5	56.5 to 60	Clay (very stiff)	ϕ = 0°; c=2750 psf	3, Clay	1000	0.005	0.039	-
-36.5 to -40.5	60 to 64	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-40.5 to -44.0	64 to 67.5	Clay (very stiff)	ϕ = 0°; c=2500 psf	3, Clay	1000	0.005	0.039	-
-44.0 to -48.0	67.5 to 71.5	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-48.0 to -50.5	71.5 to 74	Clay (very stiff)	ϕ = 0°; c=2500 psf	3, Clay	1000	0.005	0.039	-
-50.5 to -62.5	74 to 86	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-
-62.5 to -65.5	86 to 89	Sand (dense)	ϕ = 36°; c= 0 psf	4, Sand	125	-	0.042	-
-65.5 to -76.5	89 to 100	Clay (very stiff to hard)	ϕ = 0°; c=4000 psf	3, Clay	1000	0.005	0.039	-

* Finish ground elevation of +12.50 feet is used in the vertical pile capacity analyses.

Note: (1) A "P-Y Curve Modification Factor" of 0.6 should be adopted in the lateral pile analysis for pile spacing of 3 times the pile diameter.

APPENDIX D

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection __ 65% PS&E Unchecked Details _X PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 7/8/2010 **Structure Name*:** Benjamin Holt Drive UC_ **Br No*:** 29-0194 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	_____	_____	_____	_____

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 4th review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated June 11, 2010.	NA	
2	FR	NA	C1 – Approved as Submitted		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details _X_ PS&E (Review No. 91%) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 5/20/2010 **Structure Name*:** Benjamin Holt Drive UC **Br No*:** 29-0194 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	6/08/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 3rd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated April 20, 2010.	NA	
2	FR	9	Please provide the Nominal Driving Resistance in Table 5.	Comment is incorporated. "Nominal Driving Resistance" is provided in Table 5.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Benjamin Holt Drive UC **Br No*:** 29-0194 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	9	Need to provide the correct Pile Data Table based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7	Comment will be incorporated. Pile Data Table will be based on Memo to Designers 3-1, July 2008, Attachment 1, Table 7.	
3	FR	NA	C2 – Approved subject to OSFP Verification		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 12/16/2009 **Structure Name*:** Benjamin Holt Drive UC **Br No*:** 29-0194 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	
3	FR	8	Tables 2 and 3, Although the pile tip appeared sufficient, the unconfined compressive strength was not included in the laboratory data and may affect our future analysis of the piles.	Comment is incorporated. A plot of unconfined compressive strength versus strain is included in Appendix B, Laboratory Test Results.	
4	FR	9	Table 4: Pile data table shows two different design tip elevations for the two abutments. It is recommended to keep the same pile length for ease of pile fabrication.	Comment is incorporated. Same pile length is used in the design of specified pile tip elevation.	
5	FR	LOTB	As is stated in the Soil Legend, the sample number or	Comment is incorporated.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

			unconfined compressive strength is generally not located to the left of the graphic on the LOTB.		
6	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Unconfined compression test data for each soil sample is included in Appendix B as described in #3 above.	
7	FR	LOTB	Boring R-09-BH-001 shows lithological breaks at elevations of –63-feet, and –73-feet. It appears that this is the same lithology, please correct or clarify.	Comment is incorporated. The lithological breaks at Elevation -63 feet and Elevation -73 feet are removed.	
8	FR	NA	C2 – Approved subject to OSFP Verification	NA	

FOUNDATION REPORT
FIVE MILE SLOUGH CULVERT MODIFICATION
STOCKTON, CALIFORNIA
10-SJ-05, EA 10-0G4701

For

RAJAPPAN & MEYER CONSULTING ENGINEERS, INC.
1038 Leigh Avenue, Suite 100
San Jose, CA 95126

PARIKH CONSULTANTS, INC.

2360 Qume Drive, Suite A, San Jose, CA 95131
(408) 452-9000

April 20, 2010 Job No. 206127.BR

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FOUNDATION REPORT
FIVE MILE SLOUGH CULVERT MODIFICATION
STOCKTON, CALIFORNIA
10-SJ-05 EA 10-0G04701

1. INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the proposed "Five Mile Slough Culvert Modification" in Stockton, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Rajappan & Meyer Consulting Engineers, Inc. (Designer). The general location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or directly for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2. PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to evaluate their engineering properties and to provide geotechnical recommendations for the foundation design of the proposed project.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the site; obtaining representative soil samples and logging soil materials encountered in one soil boring and one cone penetration test (CPT); laboratory testing of the collected soil samples; performing engineering analyses based on the field and laboratory data, and preparation of this foundation report for the Five Mile Slough Culvert Modification.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering

services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

3. EXISTING CULVERT AND PROPOSED CONSTRUCTION

Existing Culvert

The "Five Mile Slough Culvert Modification" is located at the intersection of Interstate 5 (I-5) and Five Mile Slough. According to the Caltrans as-built plans dated September 1967, the existing culvert is cast-in-place reinforced concrete double box culvert with span of 12 feet and height of 6 feet in each box. The invert elevation is 1.70 feet (NAVD 88) at the inlet and 0.83 feet (NAVD 88) at the outlet. The length of the culvert is 190 feet. The toe of the existing embankment slope of I-5 (above the culvert) is located at the end of the culvert.

Proposed Construction

The modification of the Five Mile Slough culvert is part of the proposed "I-5 North Stockton Corridor Improvement" project. Based on the general plan of "Five Mile Slough Culvert Modification", the modification at both ends of the culvert generally consists of the following:

1. Portion (7 feet 3 inches) of the culvert at both ends and existing wingwalls will be removed.
2. A concrete barrier [Type 736A (Mod.)] with soundwall (No. SW8 in the northbound I-5 and No. SW9 in the southbound I-5) will be constructed with its footing replacing the top part of the culvert that has been removed. The bottom part of the double-cell culvert will be extended from 7 feet 3 inches to 10 feet 9 inches.
3. A new retaining wall supporting the concrete barrier [Type 736 (Mod.)] with soundwall (No. SW8 in the northbound I-5 and No. SW9 in the southbound I-5) will be constructed on both sides of the modified culvert at both ends (total number of 4). The length of each retaining wall is estimated to be between 10 feet and 15 feet.
4. The gap between the existing slope (above the culvert) and the concrete barriers below the soundwall will be backfilled with sliver fill for the widening of I-5.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4. SITE CONDITIONS

The Five Mile Slough Culvert Modification is located at the intersection of Interstate 5 (I-5) and Five Slough Culvert (Post Mile 32.10). It is approximately 0.6 miles southeast of W. Hammer Lane. The Project Location Map, Plate 1, shows the location of the site. According to the designer, there are existing levees on both sides of the existing box culvert.

5. FIELD EXPLORATION AND LABORATORY TESTING

As-Built LOTB

There is no as-built boring information available for the Five Mile Slough Culvert Modification.

Field Exploration in 2009

Field exploration was conducted in March and April 2009. Boring A-09-FS-001 was drilled on the south side and CPT-09-FS-001 was performed on the north side of Five Mile Slough. The soil boring was drilled and the CPT was pushed to the approximate depth of 80.0 feet.

The approximate locations of the borings are shown on the Site Plan, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration and relevant boring information are presented in the LOTB in Appendix A.

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, dry unit weight and Plasticity Index of the soil samples are presented on the LOTB in Appendix A.

The descriptions and related information presented on the LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum

changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

6. SUBSURFACE CONDITIONS

Based on Boring A-09-FS-001 and CPT-09-FS-001, the subsurface soils generally consist of loose silty sand, underlain by firm to stiff clay/silt, underlain by loose to very dense sands, underlain by very stiff lean clay.

Groundwater was measured at Elevation -4.0 feet (the depth of 18 feet below existing ground surface) during drilling for A-09-FS-001 in March 2009. The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other factors that may not be present at the time of the investigation.

7. GEOLOGY

General geologic features pertaining to the site were evaluated with reference to the "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1:24000" (Ref. 1).

Based on the geologic map, the project site is mapped as mainly Alluvial-floodplain Deposits, Undivided (Qfp). A geologic map of the general project area is shown on Plate 3.

8. EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

Based on the Caltrans California Seismic Hazard map (Mualchin, L., 1996) (Ref. 2) and readily available geological data, the governing fault for the proposed structure is the Coast Ranges-Sierran Block fault, (Mw=7.0), style of faulting: right lateral strike-slip).

This fault is located at about 20 miles from the project site. The other fault in the vicinity of the project site is Midway-San Joaquin/N Fault. The "Fault Map" (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum credible earthquake magnitudes for some of the major faults in the area determined by Mualchin (California Seismic Hazard Map 1996) and peak rock accelerations are summarized in Table 1 below. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

TABLE 1: EARTHQUAKE DATA

Fault	Estimated Closest Distance to Fault from Project Area (mile)	Maximum Credible Earthquake	Peak Bedrock Acceleration (g)
Coast Ranges – Sierran Block (Reverse)(CSB)	20.0	7.00	0.2
Midway – San Joaquin/N* (Not known/published)(MSJ)	20.0	6.75	0.2

Based on the seismic hazard map prepared by Mualchin (1996), the PBA at the project site is generally 0.2 g. According to Caltrans Guidelines for Structures Foundation Report (March 2006) (Ref. 3), the value of PBA (for a specific project site) from the seismic hazard map should be verified with that calculated using the attenuation relation by Sadigh et al. (1997) (Ref. 4). Based on attenuation relation by Sadigh, the maximum PBA anticipated at the project site is 0.2 g.

8.2 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered low to moderate.

Soil Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a

temporary but essentially total loss of shear strength due to build-up of excess pore water pressure under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Generally, cohesionless soils below a depth of 50 feet and cohesionless soils above the groundwater level were considered to have relatively low liquefaction potential. Clays are generally not susceptible to liquefaction.

Liquefaction potential of the sand layers was analyzed based on Boring A-09-FS-001. The analysis was performed using the procedure developed by Seed and Idriss (1982) for a magnitude 7.0 Earthquake, on the nearby Coast Ranges-Sierra Block fault, resulting in a peak ground acceleration of about 0.2 g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. It is estimated that the post-liquefaction settlement is unlikely to occur based on the result of the evaluation of liquefaction potential. Based on the CPT liquefaction analysis data for CPT-09-FS-001 attached, it appears that there is no significant stratum of sand layer that may be subject to liquefaction. Therefore the potential post-liquefaction settlement should not be a geotechnical concern in the foundation analyses.

9. FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Box Culvert

The proposed reinforced concrete culvert may be designed in accordance with Caltrans Standard Plans D80. The proposed concrete culvert should be supported on a pad, which should be constructed as follows:

(a) Excavate 2 feet of existing soil below the bottom of the concrete culvert. The excavation should extend 2 feet beyond the culvert footprint.

(b) Place a layer of subgrade enhancement engineering fabric on the subgrade. The engineering fabric may be either woven or non-woven type fabric and should conform to Caltrans Standard Specification (Section 88-1.04, Type B).

(c) Place 2 feet of Caltrans Class 3 Aggregate Base over the engineering fabric. The Class 3 Aggregate Base should be compacted to 95-percent relative compaction per Caltrans Test 216.

(d) The engineering fabric should be wrapped on top of the compacted Class 3 Aggregate Base for a minimum overlap of two feet.

The Class 3 Aggregate Base can serve as a "local distribution bridge" for reducing load imposed on the underlying materials and provide a working platform.

For the proposed box culvert, the design vertical and lateral earth pressures should conform to Section 6.2.1 of Caltrans Bridge Design Specifications (BDS). Per Caltrans BDS, the recommended unit weight of soil (vertical pressure) is 140 pcf. The recommended lateral earth pressures for design should include two design conditions, 42 pcf and 140 pcf.

9.2.1 Foundation Design

a) Recommended allowable bearing capacity for the structure (include the invert slab) is 2.25 kip per square foot.

9.3 Scour

Based on the hydraulic study, the combined scour depth is 1.99 feet and the scour elevation is -1.7 feet at Five Mile Slough. Cutoff wall is proposed at both ends of the box culvert to mitigate the potential scour to the foundation subgrade of the box culvert.

9.4 Retaining Wall Foundation Design

For the retaining wall supporting the concrete barrier and soundwall (Item 3 of the “Proposed Construction”) of the box culvert modifications, different foundation systems

have been evaluated with respect to their design capacity and construction considerations. In our opinion, Caltrans standard "Retaining Wall Type 1SWB" (File no. xs14-220e) supported on spread footing can be used. An ultimate bearing capacity of 8.6 kips per square foot (ksf) and an allowable bearing capacity of 2.75 ksf (with a factor of safety of 3.0) is available for design. This ultimate bearing capacity is adequate for "Retaining Wall Type 1SWB" with the maximum design height of 14 feet. Based on the information provided by MGE Engineering, Inc., the approximate retaining wall foundations elevations are 0.1' +/- (at east end) and -0.8' +/- (at west end). The scour elevation (-1.7 feet) is below the approximate bottom of the foundation subgrade elevation and mitigation measure will be required. Construction of foundation pad as described in Section 9.2 "Box Culvert" is recommended to mitigate the problem of scour below the footing of the retaining wall. As an option, the aggregate base can be replaced with Lean Concrete Base (LCB).

9.5 Lateral Earth Pressures

The retaining wall and wingwalls should be designed to resist the following applied lateral earth pressures (Equivalent Fluid Pressures) and design parameters:

(a) The recommended active pressure is 36 pcf EFP for the engineered fill.

(b) The recommended at-rest pressure is 55 pcf EFP for the engineered fill.

(c) The recommended passive resistance against the side of the wall footing is 460 pcf EFP.

(d) A coefficient of friction of 0.3 may be used to estimate the frictional resistance along the bottom of the footing. Only dead loads should be used to estimate the frictional resistance at the bottom of footings.

(e) No more than 50% of the available passive lateral earth pressures and 100% friction between the footing and the foundation soil shall be considered in determining the factor of safety against sliding.

(f) The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. Use equivalent earth pressure of not less than 2 feet of uniform soil weight at 125 pcf if the traffic (HS20 loading) is within a horizontal distance of one half of the wall height.

Cantilever walls which are free to rotate at least 0.005 radii may be assumed flexible for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 and 0.5 may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls and rigid walls, respectively.

9.6 Settlement Evaluation

Based on the toe pressure of the retaining wall footing and the subsurface soil conditions, typical soil profiles of Boring A-09-FS-001 was selected for settlement analyses. Based on Boring A-09-FS-001 and CPT-09-FS-001, the subsurface soil conditions at the project site generally consists of firm to very stiff lean clays with pockets/lenses of loose to dense silty sands.

The evaluation of the consolidation settlements under the retaining wall was based on the understanding of the consolidation behavior of the clay and empirical correlations in the literatures.

Based on the toe pressure of the maximum height of the retaining wall, the consolidation settlement is anticipated to be on the order of 0.6 inches and in the over-consolidated range. It is our opinion that the consolidation settlement due to the construction of the retaining wall appears to be not a geotechnical concern.

The calculation for the estimation of the consolidation settlement is contained in Appendix C of this report.

9.7 Corrosion

Chemical tests were performed on soil samples from Boring A-09-FS-001 to evaluate the corrosion potential of the subsoil. The test results are as follows:

TABLE 2: SUMMARY OF CORROSION TEST RESULT

Boring	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
A-09-FS-001	24.5	1370	7.2	41.1	4.5

Based on the corrosion test results, the native subsurface soils in Boring A-09-FS-001 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure. The minimum cement factor and cover thickness should be per Section 8.22 of Caltrans BDS – September 2003.

9.8 Potential Impact on Existing Levees and Construction Considerations

The proposed modifications only a) extend the bottom portion of the box culvert; b) construct a new retaining wall supporting the concrete barrier [Type 736 (Mod.)] with soundwall to replace the existing retaining wall and c) fill above the existing slope between I-5 and the proposed concrete barrier. In our opinion, the permanent impact of the proposed box culvert modification on the existing levees should be negligible.

Temporary slope will be required for the construction of the modified box culvert and retaining walls. Some localized settlement may result from construction equipments on the levees during construction. The temporary impact due to construction should be minimized with the implementation of precautionary measures recommended below.

Construction Precautionary Measures and Construction Monitoring

Construction on the levees shall abide by Army Corps of Engineers and/or Department of Water Resources construction and backfill requirements.

The following precautionary measures and construction monitoring can be considered during construction to minimize the potential impact due to construction on the existing levees:

a) The construction activities related to excavation and lateral earth support must conform to safety requirements of OSHA and other applicable municipal and State regulatory agencies.

b) The temporary slope height, inclination, and excavation depths should not exceed those specified in local, state, or federal safety regulations. The design of the temporary slopes by the contractor or his specialty subcontractor should conform to the OSHA's "Guidelines for Excavations and Temporary Sloping". The contractor or

responsible subcontractor should develop or modify their design based on the subsurface conditions exposed at the time of construction.

For excavations up to 20 feet deep in homogenous soils, OSHA guidelines state that the maximum allowable slope should be 3/4H: 1V, 1H:1V and 1-1/2H:1V for Type A, B and C soils, respectively. In general, Type A soils are stronger; Type B soils are intermediate, and Type C soils are weaker. Based on the available boring information, the clay and sand should be considered as OSHA Type C. It should be noted that the slope ratios recommended by OSHA are for temporary, un-surcharged slopes.

c) Pockets/lenses of loose to dense silty sands were encountered at various depths in Borings A-09-FS-001 and CPT-09-FS-001. Surficial drying of this granular soil exposed along the excavation may result in sloughing and caving. To minimize sloughing and erosion of these soils, construction should be planned in the dry season. Adequate surface protection should be provided to protect the slope surface from excessive drying and/or saturation during construction and the exposed slopes should be kept moist (but not saturated) by occasional light spraying of water during construction.

d) Vibration in the vicinity of the proposed temporary excavation should be controlled to the minimum level during the temporary slope construction.

e) The maximum allowable load on the levee from the construction equipment should be in conformance with the regulatory agency's requirements. Normally a 200 to 250 pound per square foot (psf) traffic surcharge loading condition is used in the slope stability analyses, which generally represents maintenance equipment loads. This transient loading should not be exceeded by the total (dead + live) load of the construction equipment during construction. Parking and leaving equipment for a long term should be avoided. Also materials should not be stored at the levees.

f) No equipment shall be allowed within 10 feet from the edge of the levee regardless of its weight. In order to further reduce the loading on the levees, timber planks or steel plates may also be considered to reduce point loads.

g) Contractor should be required to set horizontal and vertical monuments in levee section, which will be periodically checked for horizontal or vertical movements.

h) Construction monitoring may be considered to evaluate the settlement and movement in the levees. However, the level of sophisticated instrumentation may have to be further discussed, if it is deemed necessary.

In summary, it is our opinion that the impact on the existing levees due to the proposed project should be insignificant. However, precautionary measures should be taken and construction monitoring should be implemented during construction to minimize any potential impact on the integrity of the existing levees.

The above opinions are based on our understanding of the project and limited to the portion of the levee to be impacted by the proposed structure. Evaluation of the existing levee corridor has not been performed since it is beyond the scope of our work for this project.

9.9 Plan Review

We recommend that final foundation plans for the proposed project be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

9.10 Construction Observation

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

10. INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with

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

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

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

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

Respectfully submitted,

PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605
Project Engineer

Gary Parikh, P.E., G.E. 666
Project Manager

AL/GP: S:\Ongoing Projects\2006\206127\I-5 Five Mile Slough Culvert Mod Foundation Report (4-2010).rpt

REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-11
GEOTECHNICAL
STATE OF CALIFORNIA

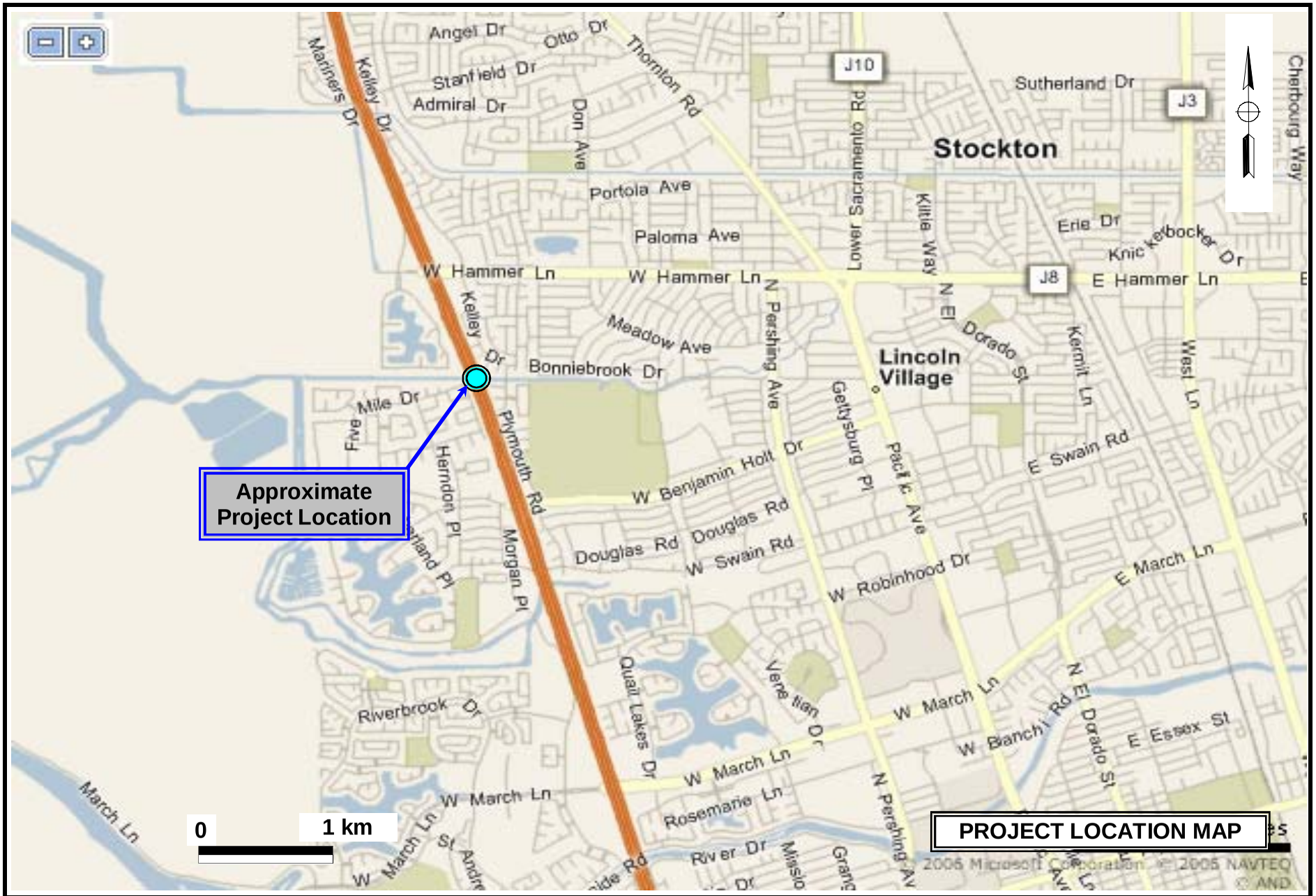
REFERENCES

1. "Geologic Map of the Sacramento-San Joaquin Delta, California, by Brian F. Atwater, 1982, USGS Map MF-1401, Scale: 1: 24000"

2. Mualchin, L., 1996, A Technical Report to accompany the Caltrans California Seismic Hazard map 1996 (Based on Maximum Credible Earthquakes (MCE), California Department of Transportation, Office of Earthquake Engineering, 65 p. 1 Plate.

3. Caltrans Department of Transportation, Division of Engineering Services Geotechnical Services "Guidelines for Structures Foundation Report", Version 2, March 2006.

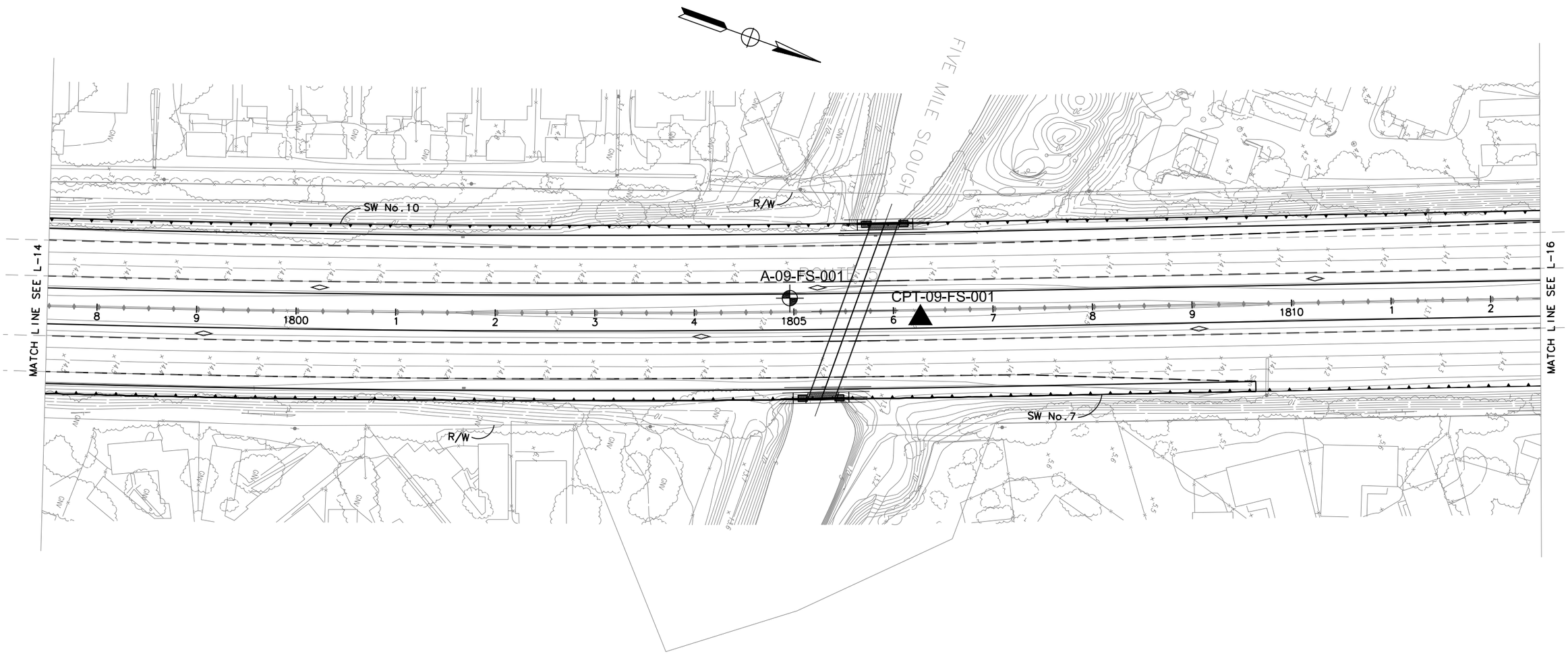
4. Sadigh, K., Chang, C. Y., Egan, J. A., Makdisi, F., and Youngs, R. R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letter, Vol. 68, No. 1, January, pp. 180-189.



PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

FIVE MILE SLOUGH CULVERT MODIFICATION
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR	PLATE NO.: 1



LEGEND

A-09-FS-001
Approximate Boring Location

B-3
Approximate CPT location performed in 1964

B-123
Approximate Boring location drilled in 1964

B-8
Approximate Boring location drilled in 1997

CPT-09-FS-001
Approximate Cone Penetration Test Location

Existing Structure

SCALE 1 inch = 50 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by Rajappan & Meyer Consulting Engineers, Inc.

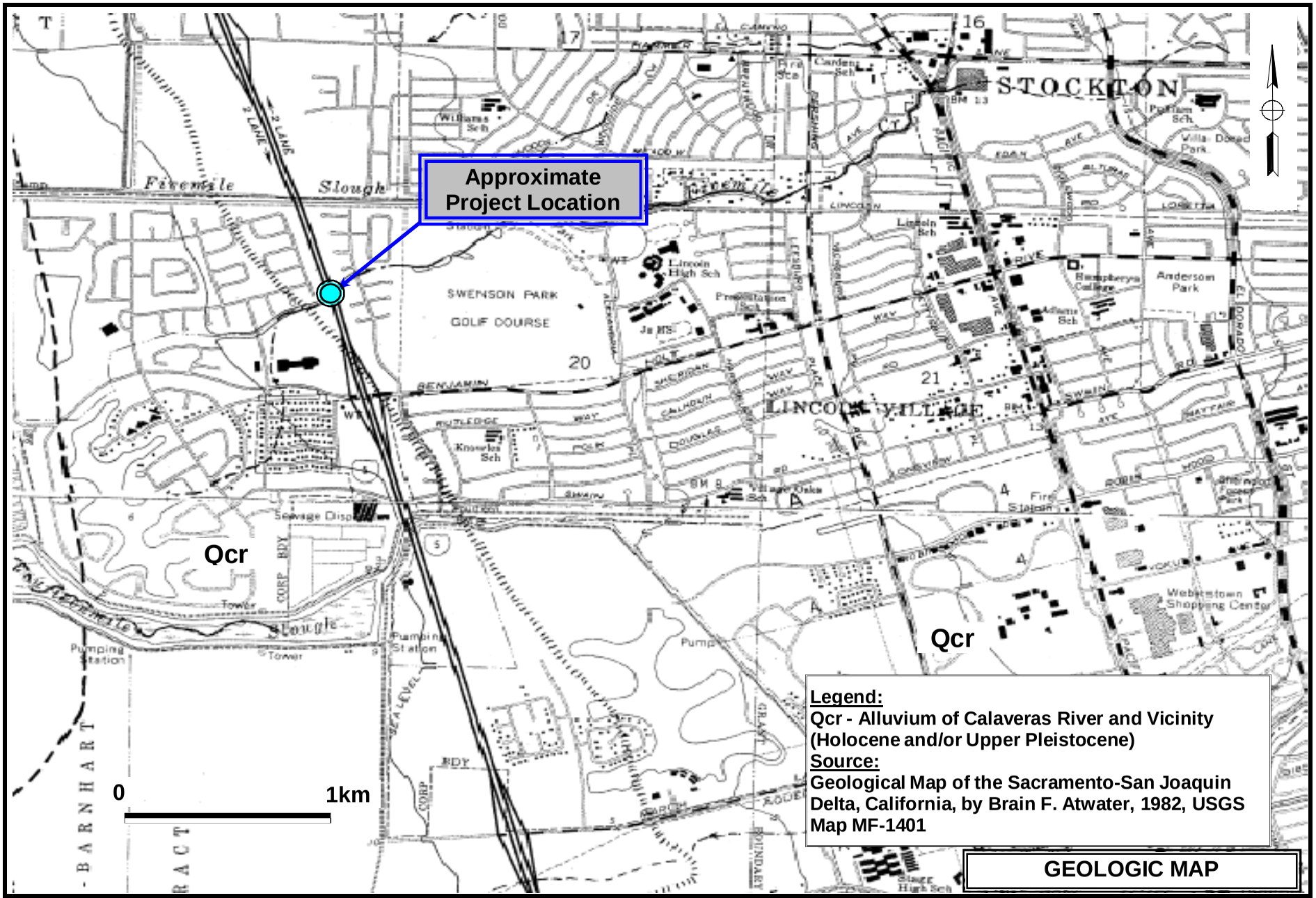
SITE PLAN

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

FIVE MILE SLOUGH CULVERT MODIFICATION
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR

PLATE NO: 2



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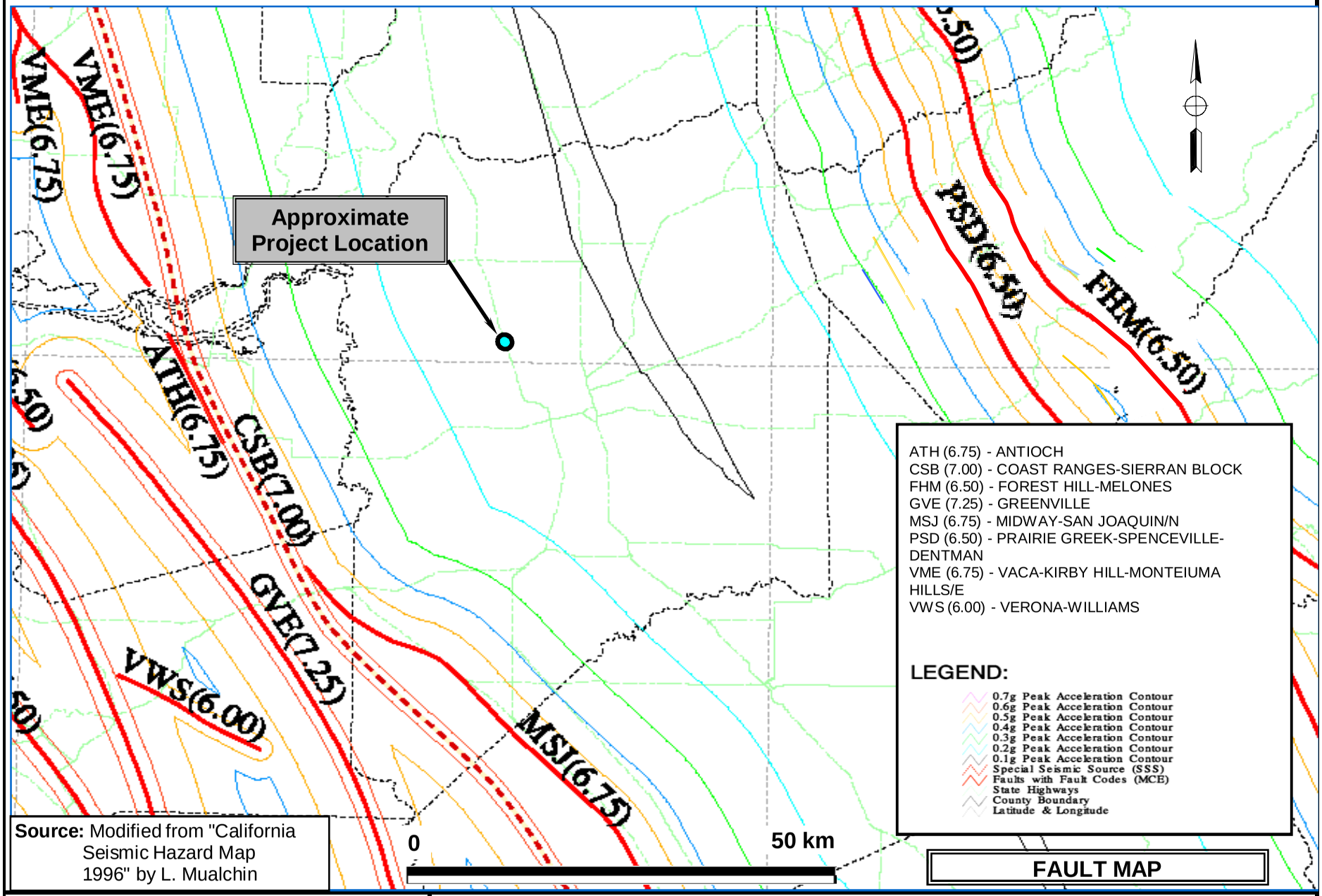
MATERIALS TESTING

FIVE MILE SLOUGH CULVERT MODIFICATION
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR	PLATE NO.: 2



Source: Modified from "California Seismic Hazard Map 1996" by L. Mualchin

FAULT MAP

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

FIVE MILE SLOUGH CULVERT MODIFICATION
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR | **PLATE NO.: 3**

APPENDIX A

APPENDIX A
FIELD EXPLORATION

The test borings were advanced with truck-mounted drill rig with 8-inch diameter hollow-stem auger drilling method. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5-inch Inside Diameter (I. D.) Modified California Sampler or a 1.375-inch I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings (LOTB), Appendix A. When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils. Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the drillhole was backfilled with cement grout.

The borings were drilled under the technical supervision of our engineer, who visually classified in the field (according to the Unified Soil Classification System) and continuously logged the soils encountered during drilling. The engineer supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the boring logs in the LOTB in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these LOTB depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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SAN JOAQUIN COUNCIL OF GOVERNMENTS
555 EAST WEBER AVENUE
STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

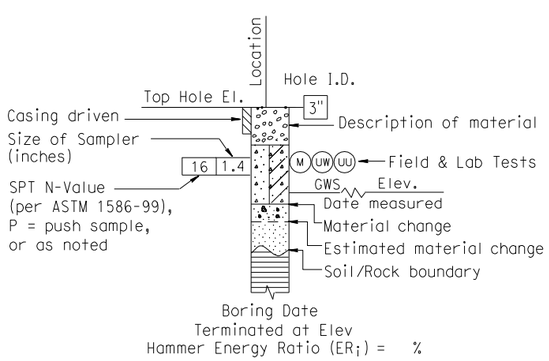
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778-95)
	O	Other

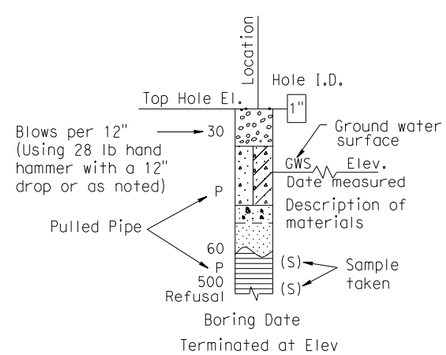
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

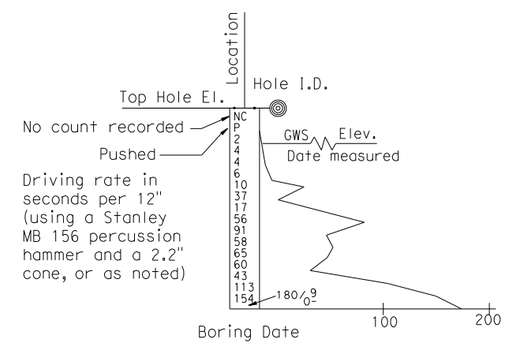
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



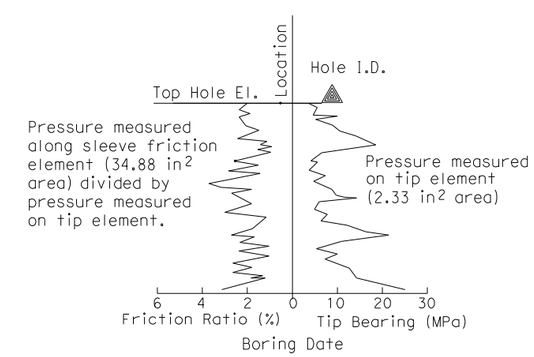
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

DESIGN OVERSIGHT	DRAWN BY: O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	A. LAM PROJECT ENGINEER	BRIDGE NO. N/A POST MILES 32.10
SIGN OFF DATE	CHECKED BY: D. WANG				

FIVE MILE SLOUGH CULVERT MODIFICATION

SOIL LEGEND 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 11 OF 13

...\29-z-ltb01.dgn 6/8/2010 9:38:24 AM

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER GARY PARIKH No. G.E. 666 Exp. 12/31/11 GEOTECHNICAL STATE OF CALIFORNIA

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STOCKTON, CALIFORNIA 95202

PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 5333)
CP	Compaction Curve (CTM 216)
CR	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content-% (ASTM D 2974)
P	Permeability (CTM 220)
PA	Particle Size Analysis (ASTM D 422)
PI	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (CTM 217)
SG	Specific Gravity (AASHTO T 100)
SL	Shrinkage Limit (ASTM D 427)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 4767)
VS	Vane Shear (AASHTO T 223)

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY O. GOUTHIER

CHECKED BY D. WANG

S. DUDDU - A. R. BHARADWAJ
FIELD INVESTIGATION BY:
DATE: MARCH 2009 - APRIL 2009

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

A. LAM
PROJECT ENGINEER

BRIDGE NO. N/A
POST MILES 32.10

FIVE MILE SLOUGH CULVERT MODIFICATION

SOIL LEGEND 2 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 12 OF 13

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5			

GEOTECHNICAL PROFESSIONAL DATE 6/11/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. G.E. 666 — Exp. 12/31/11 — GEOTECHNICAL — STATE OF CALIFORNIA

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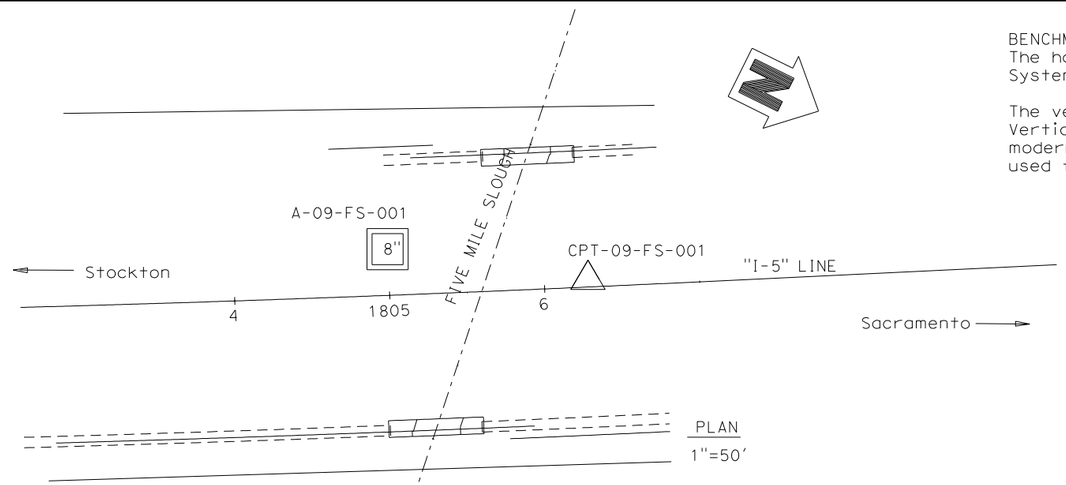
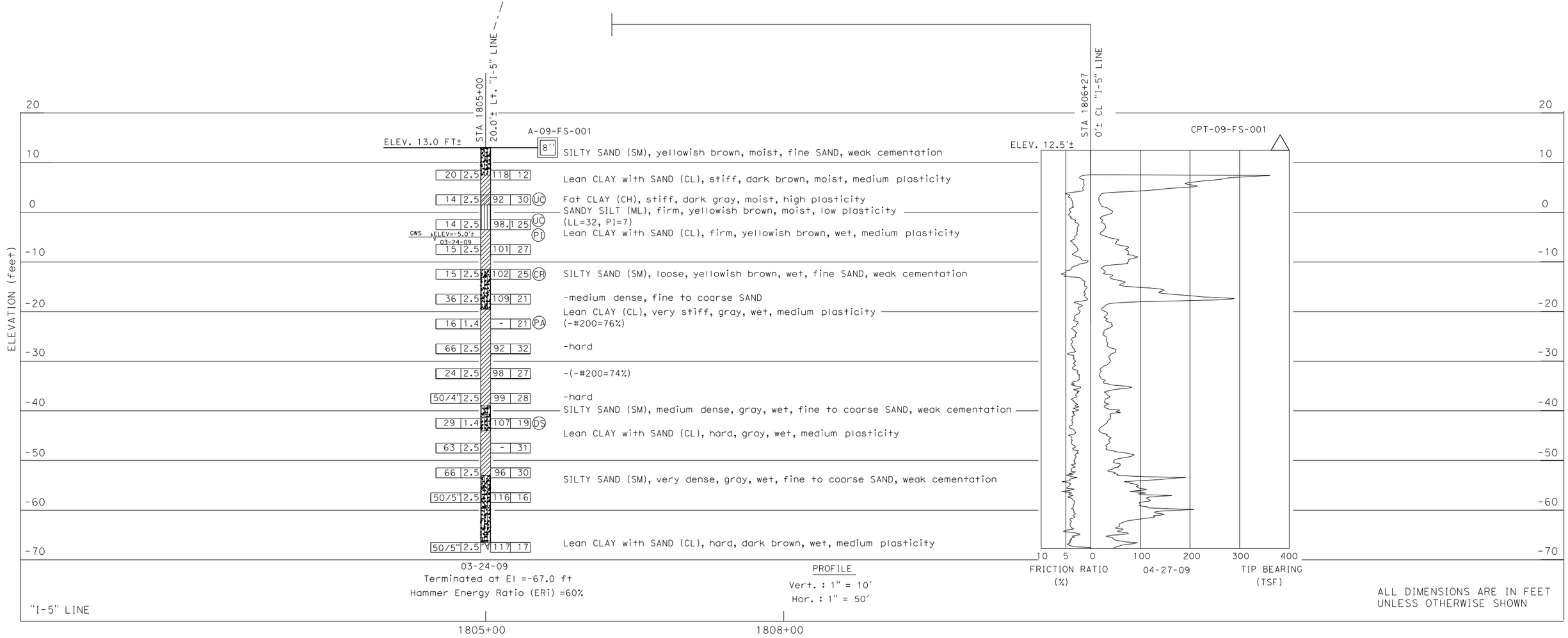
PARIKH CONSULTANTS, INC.
2360 QUME DRIVE, SUITE A
SAN JOSE, CA 95131

BENCHMARK:
The horizontal coordinates values are in the California Coordinate System (CCS 83) Zone 3, Epoch Date 2007.00, in U.S. survey feet.

The vertical control values are based on the North American Vertical Datum of 1988 (NAVD88) in U.S. survey feet, height modernization survey station, (2007.00) and GEOID03 was used for the adjustment.





DESIGN OVERSIGHT	DRAWN BY	O. GOUTHIER	S. DUDDU - A. R. BHARADWAJ	A. LAM	BRIDGE NO. N/A
SIGN OFF DATE	CHECKED BY	D. WANG	FIELD INVESTIGATION BY: DATE: MARCH 2009 - APRIL 2009	PROJECT ENGINEER	POST MILES 32.10

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

FIVE MILE SLOUGH CULVERT MODIFICATION

LOG OF TEST BORINGS

OGS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 10/16/09 12/04/09 01/15/10 04/23/10 06/11/10

SHEET 13 OF 13

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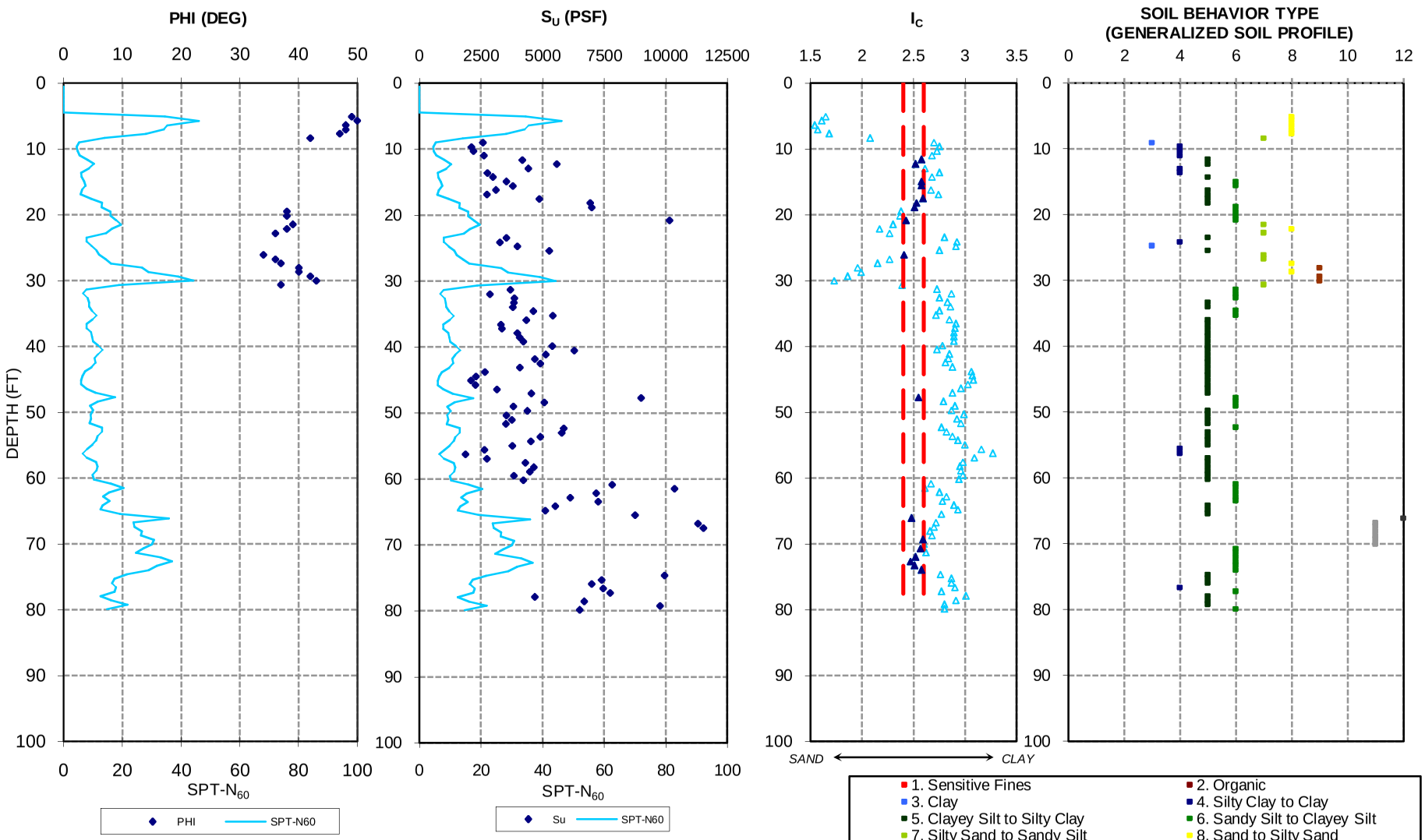
CPT DATA CORRELATION

CPT-09-011



Reference: CPT data and correlation provided by Gregg Drilling and Testing, Inc.

PARIKH

Practicing in the Geosciences

FIVE MILE SLOUGH CULVERT MODIFICATION
STOCKTON, CALIFORNIA

JOB NO.: 206127.BR	PLATE NO.:

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

	I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT- FIVE MILE		
PROJECT NAME	***SLOUGH CULVERT MODIFICATION***	*FAULT INFO*	
PROJECT NO.	***206127.FMS***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-011***	*a_{max} (g)=* 0.2	*FAULT M_w =* 7.0
			MSF = 1.19

Depth (ft)		Soil Behavior Type	*CPT RESULT INPUT* qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	*CSR* γ_d	CSR	*$CRR_{7.5}$* Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	*F.S.=($CRR_{7.5}$/CSR)*MSF*Kσ*Kα* Kσ	Kα	F.S.
5.085	8	Sand to Silty Sand	198.35	3.86	0.308		0.308	1.65	1.45889	0.99	0.129	1.650	1.46	289.37	1.003	290.335		1.00	1	
5.741	8	Sand to Silty Sand	269.564	5.127	0.347		0.347	1.61	1.42211	0.99	0.129	1.610	1.42	383.35	1.000	383.349		1.00	1	
6.398	8	Sand to Silty Sand	211.657	2.914	0.387		0.387	1.54	1.38626	0.99	0.128	1.540	1.39	293.41	1.000	293.412		1.00	1	
7.054	8	Sand to Silty Sand	202.778	2.769	0.427		0.427	1.57	1.35218	0.99	0.128	1.570	1.35	274.19	1.000	274.193		1.00	1	
7.71	8	Sand to Silty Sand	160.197	2.419	0.466		0.466	1.68	1.32053	0.98	0.128	1.680	1.32	211.54	1.024	216.677		1.00	1	
8.366	7	Silty Sand to Sandy Silt	69.048	1.614	0.505		0.505	2.08	1.29032	0.98	0.128	2.080	1.29	89.09	1.420	126.482	0.27	1.00	1	
9.022	3	Clay	19.867	0.922	0.541		0.541	2.7	1.26364	0.98	0.128							1.00	1	
9.678	4	Silty Clay to Clay	16.426	0.682	0.579		0.579	2.75	1.23665	0.98	0.127							1.00	1	
10.335	4	Silty Clay to Clay	17.032	0.64	0.617		0.617	2.73	1.21079	0.98	0.127							1.00	1	
10.991	4	Silty Clay to Clay	20.247	0.743	0.654		0.654	2.68	1.18662	0.98	0.127							1.00	1	
11.647	5	Clayey Silt to Silty Clay	31.989	1.277	0.692		0.692	2.58	1.16279	0.98	0.127	2.580	1.16	37.20	3.206	119.269	0.24	1.00	1	
12.303	5	Clayey Silt to Silty Clay	42.467	1.763	0.729		0.729	2.52	1.14049	0.97	0.127	2.520	1.14	48.43	2.872	139.081	0.33	1.00	1	
12.959	4	Silty Clay to Clay	33.787	1.46	0.767		0.767	2.61	1.11845	0.97	0.126							1.00	1	
13.615	4	Silty Clay to Clay	21.432	0.833	0.805		0.805	2.75	1.09726	0.97	0.126							1.00	1	
14.272	5	Clayey Silt to Silty Clay	23.008	0.724	0.842		0.842	2.68	1.07738	0.97	0.126							1.00	1	
14.928	6	Sandy Silt to Clayey Silt	27.096	0.696	0.88		0.88	2.58	1.05769	0.97	0.126	2.580	1.06	28.66	3.206	91.894	0.15	1.00	1	
15.584	6	Sandy Silt to Clayey Silt	29.086	0.776	0.917		0.917	2.58	1.03921	0.97	0.126	2.580	1.04	30.23	3.206	96.919	0.16	1.00	1	
16.24	5	Clayey Silt to Silty Clay	23.632	0.657	0.955		0.955	2.67	1.02088	0.97	0.126							1.00	1	
16.896	5	Clayey Silt to Silty Clay	20.888	0.615	0.993		0.993	2.74	1.00319	0.96	0.125							1.00	1	
17.552	5	Clayey Silt to Silty Clay	36.724	1.192	1.03		1.03	2.59	0.98655	0.96	0.125	2.590	0.99	36.23	3.266	118.328	0.23	0.99	1	
18.209	5	Clayey Silt to Silty Clay	52.458	1.96	1.068		1.068	2.53	0.97002	0.96	0.125	2.530	0.97	50.89	2.925	148.827	0.39	0.99	1	
18.865	6	Sandy Silt to Clayey Silt	52.821	1.822	1.105		1.105	2.51	0.95445	0.96	0.125	2.510	0.95	50.41	2.820	142.145	0.35	0.98	1	
19.521	6	Sandy Silt to Clayey Silt	68.827	2.04	1.143		1.143	2.38	0.93897	0.96	0.125	2.380	0.94	64.63	2.233	144.283	0.36	0.97	1	
20.177	6	Sandy Silt to Clayey Silt	69.433	1.982	1.181		1.181	2.37	0.92398	0.96	0.124	2.370	0.92	64.15	2.194	140.759	0.34	0.97	1	
20.833	6	Sandy Silt to Clayey Silt	75.619	2.624	1.218		1.218	[illegible]	0.90984	[illegible]	0.124	2.430	[illegible]	68.80	2.439	167.808		[illegible]	1	
21.49	7	Silty Sand to Sandy Silt	88.172	2.229	1.257		1.257	2.3	0.8954	0.95	0.124	2.300	0.90	78.95	1.949	153.851	0.42	0.95	1	
22.146	8	Sand to Silty Sand	77.91	1.046	1.296		1.296	2.17	0.88141	0.95	0.124	2.170	0.88	68.67	1.596	109.625	0.20	0.95	1	
22.802	7	Silty Sand to Sandy Silt	65.697	0.958	1.335		1.335	2.27	0.86785	0.95	0.123	2.270	0.87	57.02	1.856	105.825	0.19	0.94	1	
23.458	5	Clayey Silt to Silty Clay	27.686	0.891	1.373		1.373	2.8	0.85503	0.95	0.123							0.94	1	
24.114	4	Silty Clay to Clay	25.764	1.14	1.41		1.41	2.92	0.84291	0.94	0.123							0.93	1	
24.77	3	Clay	31.173	1.632	1.447		1.447	2.91	0.83113	0.94	0.123							0.93	1	
25.427	5	Clayey Silt to Silty Clay	40.976	1.662	1.484	0.013	1.471	2.75	0.82366	0.94	0.123							0.93	1	
26.083	7	Silty Sand to Sandy Silt	51.692	0.817	1.523	0.034	1.489	2.41	0.81815	0.94	0.125	2.410	0.82	42.29	2.354	99.540	0.17	0.92	1	1.64
26.739	7	Silty Sand to Sandy Silt	64.58	0.781	1.562	0.054	1.508	2.27	0.81241	0.94	0.126	2.270	0.81	52.47	1.856	97.380	0.17	0.92	1	1.57
27.395	8	Sand to Silty Sand	77.507	0.75	1.601	0.075	1.527	2.15	0.80675	0.93	0.127	2.150	0.81	62.53	1.553	97.094	0.17	0.92	1	1.55
28.051	9	Sand	137.137	1.399	1.642	0.095	1.547	1.96	0.80087	0.93	0.128	1.960	0.80	109.83	1.251	137.408	0.32	0.89	1	2.99
28.707	8	Sand to Silty Sand	146.85	1.752	1.682	0.115	1.566	1.99	0.79537	0.93	0.129	1.990	0.80	116.80	1.287	150.340	0.40	0.88	1	3.65
29.364	9	Sand	209.644	2.349	1.723	0.136	1.587	1.86	0.78938	0.92	0.130	1.860	0.79	165.49	1.153	190.865		0.86	1	
30.02	9	Sand	247.814	2.139	1.763	0.156	1.607	1.73	0.78375	0.92	0.131	1.730	0.78	194.23	1.058	205.502		0.84	1	
30.676	7	Silty Sand to Sandy Silt	81.102	1.898	1.802	0.177	1.625	2.39	0.77876	0.92	0.132	2.390	0.78	63.16	2.272	143.499	0.35	0.91	1	3.20
31.332	6	Sandy Silt to Clayey Silt	28.905	0.609	1.839	0.197	1.642	2.73	0.7741	0.91	0.133							0.91	1	
31.988	6	Sandy Silt to Clayey Silt	22.515	0.56	1.877	0.218	1.659	2.87	0.7695	0.91	0.134							0.90	1	
32.644	6	Sandy Silt to Clayey Silt	29.693	0.695	1.915	0.238	1.676	2.75	0.76495	0.91	0.135							0.90	1	
33.301	5	Clayey Silt to Silty Clay	29.846	0.912	1.952	0.259	1.693	2.83	0.76046	0.90	0.135							0.90	1	
33.957	5	Clayey Silt to Silty Clay	29.631	1	1.99	0.279	1.711	2.86	0.75575	0.90	0.136							0.90	1	
34.613	6	Sandy Silt to Clayey Silt	35.233	1.021	2.027	0.3	1.728	2.75	0.75137	0.89	0.136							0.90	1	
35.269	6	Sandy Silt to Clayey Silt	41.294	1.291	2.065	0.32	1.745	2.72	0.74703	0.89	0.137							0.89	1	
35.925	5	Clayey Silt to Silty Clay	33.724	1.256	2.103	0.341	1.762	2.85	0.74274	0.88	0.137							0.89	1	
36.581	5	Clayey Silt to Silty Clay	25.866	0.823	2.14	0.361	1.779	2.91	0.7385	0.88	0.137							0.89	1	
37.238	5	Clayey Silt to Silty Clay	25.928	0.816	2.178	0.382	1.796	2.9	0.73431	0.87	0.138							0.89	1	
37.894	5	Clayey Silt to Silty Clay	31.411	1.162	2.215	0.402	1.813	2.89	0.73017	0.87	0.138							0.89	1	
38.55	5	Clayey Silt to Silty Clay	32.392	1.223	2.253	0.423	1.83	2.89	0.72607	0.86	0.138							0.89	1	
39.206	5	Clayey Silt to Silty Clay	33.401	1.303	2.291	0.443	1.848	2.89	0.72178	0.86	0.138							0.88	1	
39.862	5	Clayey Silt to Silty Clay	41.98	1.481	2.328	0.464	1.865	2.78	0.71778	0.85	0.138							0.88	1	
40.518	5	Clayey Silt to Silty Clay	48.205	1.699	2.366	0.484	1.882	2.73	0.71382	0.85	0.138							0.88	1	
41.175	5	Clayey Silt to Silty Clay	39.865	1.669	2.403	0.504	1.899	2.85	0.70991	0.84	0.138							0.88	1	
41.831	5	Clayey Silt to Silty Clay	36.418	1.279	2.441	0.525	1.916	2.84	0.70603	0.83	0.138							0.88	1	
42.487	5	Clayey Silt to Silty Clay	38.011	1.267	2.479	0.545	1.933	2.81	0.7022	0.83	0.138							0.88	1	
43.143	5	Clayey Silt to Silty Clay	32.29	1.087	2.516	0.566	1.95	2.88	0.69841	0.82	0.138							0.87	1	
43.799	5	Clayey Silt to Silty Clay	22.028	0.784	2.554	0.586	1.967	3.06	0.69466	0.82	0.138							0.87	1	
44.455	5	Clayey Silt to Silty Clay	19.312	0.566	2.591	0.607	1.985	3.07	0.69074	0.81	0.137							0.87	1	
45.112	5	Clayey Silt to Silty Clay	17.713	0.467	2.629	0.627	2.002	3.08	0.68707	0.80	0.137							0.87	1	
45.768	5	Clayey Silt to Silty Clay	18.603	0.453	2.667	0.648	2.019	3.03	0.68344	0.80	0.137							0.87	1	
46.424	5	Clayey Silt to Silty Clay	25.168	0.749	2.704	0.668	2.036	2.96	0.67985	0.79	0.136							0.87	1	
47.08	5	Clayey Silt to Silty Clay	36.134	1.306	2.742	0.689	2.053	2.88	0.6763	0.78	0.136							0.87	1	
47.736	6	Sandy Silt to Clayey Silt	70.063	1.742	2.779	0.709	2.07	2.55	0.67278	0.78	0.135	2.550	0.67	47.14	3.034	143.025	0.35	0.86	1	3.10
48.392	6	Sandy Silt to Clayey Silt	40.454	1.163	2.817	0.73	2.087	2.79	0.6693	0.77	0.135							0.86	1	
49.049	6	Sandy Silt to Clayey Silt	29.965	0.9	2.855	0.75	2.104	2.9	0.66586	0.76	0.134							0.86	1	
49.705	5	Clayey Silt to Silty Clay	34.24	1.112	2.892	0.771	2.122	2.87	0.66225	0.76	0.134							0.86	1	
50.361	5	Clayey Silt to Silty Clay	28.536	1.047	2.93	0.791	2.139	2.99	0.65888	0.75	0.133							0.86	1	
51.017	5	Clayey Silt to Silty Clay	30.402	0.931	2.967	0.812	2.156	2.92	0.65554	0.74	0.133							0.86	1	
51.673	5	Clayey Silt to Silty Clay	28.378	0.887	3.005	0.832	2.173	2.96	0.65224	0.74	0.132							0.86	1	
52.329	6	Sandy Silt to Clayey Silt	46.357	1.415	3.042	0.852	2.19	2.77	0.64897	0.73	0.132							0.85	1	
52.986	5	Clayey Silt to Silty Clay	45.767	1.611	3.08	0.873	2.207	2.82	0.64573	0.72	0.131							0.85	1	
53.642	5	Clayey Silt to Silty Clay	38.884	1.401	3.118	0.893	2.224	2.88	0.64252	0.72	0.131							0.85	1	
54.298	5	Clayey Silt to Silty Clay	36.168	1.366	3.155	0.914	2.241	2.93	0.63935	0.71	0.130							0.85	1	
54.954	5	Clayey Silt to Silty Clay	30.946	1.167	3.193	0.934	2.259	3	0.63602	0.70	0.129							0.85	1	
55.61	4	Silty Clay to Clay	22.521	0.934	3.23	0.955	2.276	3.16	0.63291	0.70	0.129							0.85	1	
56.266	4	Silty Clay to Clay	16.964	0.624	3.268	0.975	2.293	3.27	0.62983	0.69	0.128							0.85	1	
56.923	5	Clayey Silt to Silty Clay	23.275	0.763	3.306	0.996	2.31	3.09	0.62678	0.69	0.128							0.85	1	
57.579	5	Clayey Silt to Silty Clay	35.227	1.395	3.343	1.016	2.327	2.98	0.62376	0.68	0.127							0.84	1	
58.235	5	Clayey Silt to Silty Clay	37.495	1.473	3.381	1.037	2.344	2.95	0.62077	0.67	0.126							0.84	1	
58.891	5	Clayey Silt to Silty Clay	36.197	1.412	3.418	1.057	2.361	2.96	0.6178	0.67	0.126							0.84	1	
59.547	5	Clayey Silt to Silty Clay	31.332	1.069	3.456	1.078	2.378	2.98	0.61487	0.66	0.125							0.84	1	
60.203	5	Clayey Silt to Silty Clay	33.877	1.103	3.494	1.098	2.395	2.94	0.61196	0.66	0.125							0.84	1	
60.86	6	Sandy Silt to Clayey Silt	59.897	1.672	3.531	1.119	2.413	2.67	0.60891	0.65	0.124							0.84	1	
61.516	6	Sandy Silt to Clayey Silt	79.067	2.476	3.569	1.139	2.43	2.61	0.60606	0.65	0.124							0.84	1	
62.172	6	Sandy Silt to Clayey Silt	55.797	1.799	3.606	1.16	2.447	2.75	0.60324	0.64	0.123							0.84	1	
62.828	6	Sandy Silt to Clayey Silt	47.128	1.608	3.644	1.18	2.464	2.82	0.60044	0.64	0.123							0.83	1	
63.484	6	Sandy Silt to Clayey Silt	55.931	1.988	3.682	1.201	2.481	2.78	0.59766	0.63	0.122							0.83	1	
64.14	5	Clayey Silt to Silty Clay	44.055	1.646	3.719	1.221	2.498	2.89	0.59492	0.63	0.122							0.83	1	
64.797	5	Clayey Silt to Silty Clay	40.806	1.569	3.757	1.241	2.515	2.93	0.59219	0.62	0.121							0.83	1	
65.453	5	Clayey Silt to Silty Clay	68.532	2.864	3.794	1.262	2.532	2.77	0.5895	0.62	0.121							0.83	1	
66.109	12	Sand to Clayey Sand	148.681	5.457	3.834	1.282	2.552	2.48	0.58635	0.61	0.120	2.480	0.59	87.18	2.669	232.713		0.83	1	
66.765	11	Sitff Fine-Grained	87.265	4.026	3.877	1.303	2.574	2.72	0.58294	0.61	0.120							0.83	1	
67.421	11	Sitff Fine-Grained	89.664	3.868	3.92	1.323	2.596	2.7	0.57956	0.61	0.119							0.83	1	
68.077	11	Stiff Fine-Grained	100.938	4.283	3.963	1.344	2.619	2.66	0.57607	0.60	0.119							0.82	1	
68.734	11	Sitff Fine-Grained	98.435	4.352	4.005	1.364	2.641	2.68	0.57277	0.60	0.118							0.82	1	

LIQUEFACTION POTENTIAL ANALYSIS (per Youd & Idriss, 2001)

	I-5 NORTH STOCKTON CORRIDOR IMPROVEMENT- FIVE MILE		
PROJECT NAME	***SLOUGH CULVERT MODIFICATION***	*FAULT INFO*	
PROJECT NO.	***206127.FMS***	*COAST-RANGES SIERRAN BLOCK*	
CPT NO.	***CPT-09-011***	a_{max} *(g)=* 0.2	*FAULT* M_w = 7.0
			MSF = 1.19

		CPT RESULT INPUT							*CSR*		*CRR 7.5*						*F.S.=(CRR 7.5/(CSR)*MSF*Kσ*Kα*		
Depth (ft)		Soil Behavior Type	qc (tsf)	fs (tsf)	σ_v (tsf)	u (tsf)	σ_v' (tsf)	Ic*	C_N*	γ_d	CSR	Ic	C_Q	q_{c1N}	K_C	$(q_{c1N})_{cs}$	$CRR_{7.5}$	Kσ	Kα	F.S.
69.39	11	Sitff Fine-Grained	120.598	4.992	4.048	1.385	2.663	2.59	0.56951	0.60	0.118	2.590	0.57	68.68	3.266	224.315		0.82	1	
70.046	11	Sitff Fine-Grained	117.123	4.913	4.091	1.405	2.686	2.6	0.56613	0.59	0.117							0.82	1	
70.702	6	Sandy Silt to Clayey Silt	107.195	3.777	4.129	1.426	2.703	2.57	0.56367	0.59	0.117	2.570	0.56	60.42	3.148	190.208		0.82	1	
71.358	6	Sandy Silt to Clayey Silt	95.294	3.391	4.166	1.446	2.72	2.62	0.56122	0.58	0.116							0.82	1	
72.014	6	Sandy Silt to Clayey Silt	133.912	4.937	4.204	1.467	2.737	2.52	0.5588	0.58	0.116	2.520	0.56	74.83	2.872	214.884		0.82	1	
72.671	6	Sandy Silt to Clayey Silt	153.353	5.399	4.241	1.487	2.754	2.47	0.5564	0.58	0.116	2.470	0.56	85.33	2.621	223.664		0.82	1	
73.327	6	Sandy Silt to Clayey Silt	129.597	4.551	4.279	1.508	2.771	2.51	0.55402	0.58	0.115	2.510	0.55	71.80	2.820	202.438		0.82	1	
73.983	6	Sandy Silt to Clayey Silt	113.823	4.193	4.317	1.528	2.788	2.58	0.55165	0.57	0.115	2.580	0.55	62.79	3.206	201.336		0.81	1	
74.639	5	Clayey Silt to Silty Clay	77.745	3.202	4.354	1.549	2.806	2.76	0.54918	0.57	0.115							0.81	1	
75.295	5	Clayey Silt to Silty Clay	58.627	2.46	4.392	1.569	2.823	2.87	0.54686	0.57	0.115							0.81	1	
75.951	5	Clayey Silt to Silty Clay	55.669	2.174	4.429	1.589	2.84	2.87	0.54455	0.56	0.114							0.81	1	
76.608	4	Silty Clay to Clay	59.796	2.727	4.467	1.61	2.857	2.9	0.54227	0.56	0.114							0.81	1	
77.264	6	Sandy Silt to Clayey Silt	62.431	1.836	4.504	1.63	2.874	2.77	0.54001	0.56	0.114							0.81	1	
77.92	5	Clayey Silt to Silty Clay	39.031	1.423	4.542	1.651	2.891	3.01	0.53777	0.56	0.114							0.81	1	
78.576	5	Clayey Silt to Silty Clay	53.195	2.236	4.58	1.671	2.908	2.91	0.53554	0.55	0.113							0.81	1	
79.232	5	Clayey Silt to Silty Clay	76.583	3.41	4.617	1.692	2.925	2.8	0.53333	0.55	0.113							0.81	1	
79.888	6	Sandy Silt to Clayey Silt	51.89	1.367	4.655	1.712	2.943	2.8	0.53102	0.55	0.113							0.81	1	

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-98. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Test Borings", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. The result was used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-00. The result of the test is presented on Plate B-2, "Plasticity Chart".

Unconfined Compression Tests

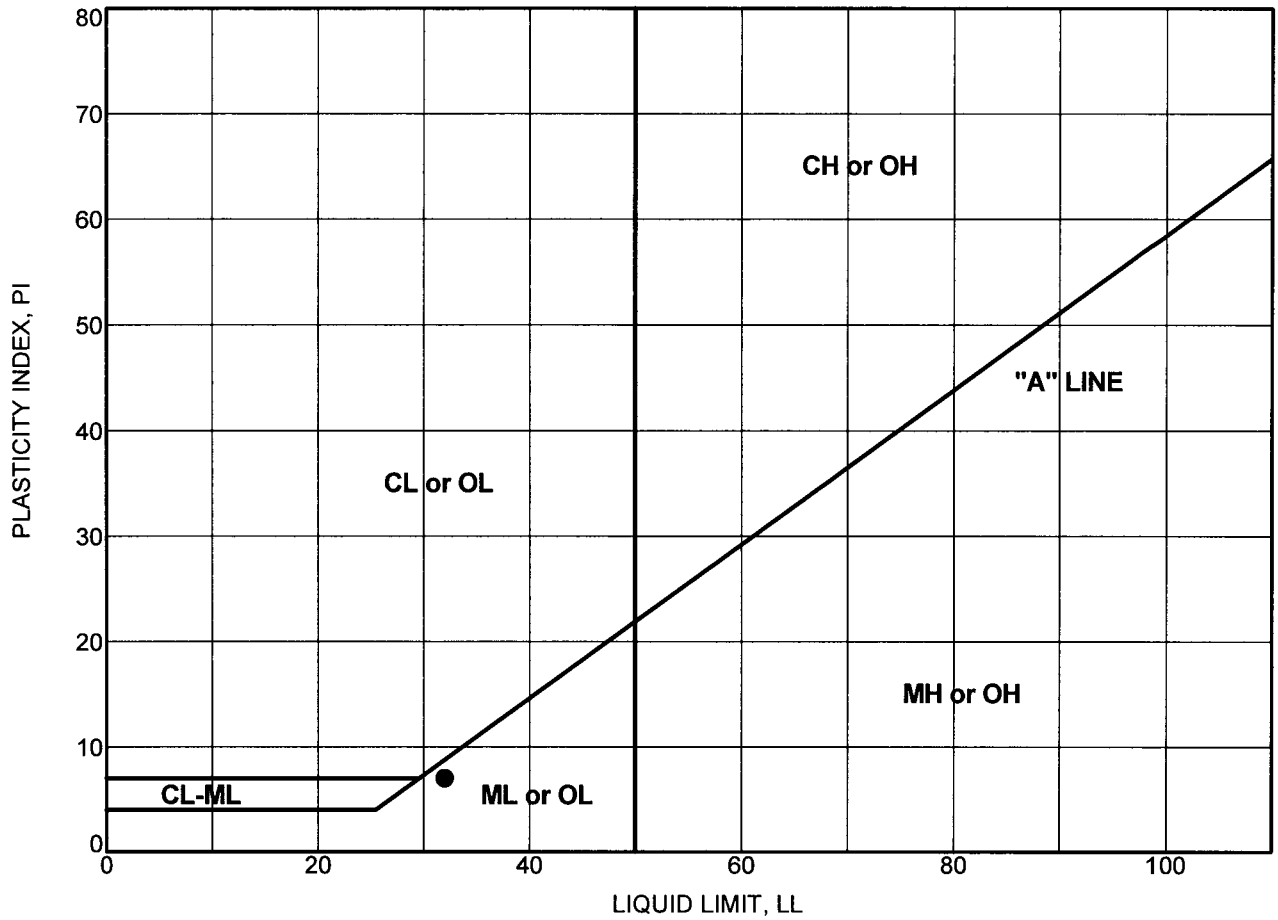
Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression tests were performed in general accordance with ASTM Test Method D 2166-00. The results are presented on Plates B-5A and B-5B.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by Sunland Analytical. The test results are presented on Plate B-3.

Direct Shear Tests

Direct Shear test was performed on selected relatively undisturbed sample to determine the shear strength of a soil material in direct shear. The test was performed according to ASTM Test Method D 3080 by Cooper . The test result is presented on Plate B-4.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
A-09-001	MC-4	19.5	●	27	32	25	7	SANDY SILT (ML)

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Five Mile Slough Culvert Modification
Stockton, California

JOB NO: 206127.BR (FMS)

PLATE NO: B-2

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/06/2009
Date Submitted 05/01/2009

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

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

The reported analysis was requested for the following location:
Location : 206127.GDR I-5 NORTH Site ID : A-09-FS-001#5@24.5'
Thank you for your business.

* For future reference to this analysis please use SUN # 55616-112070.

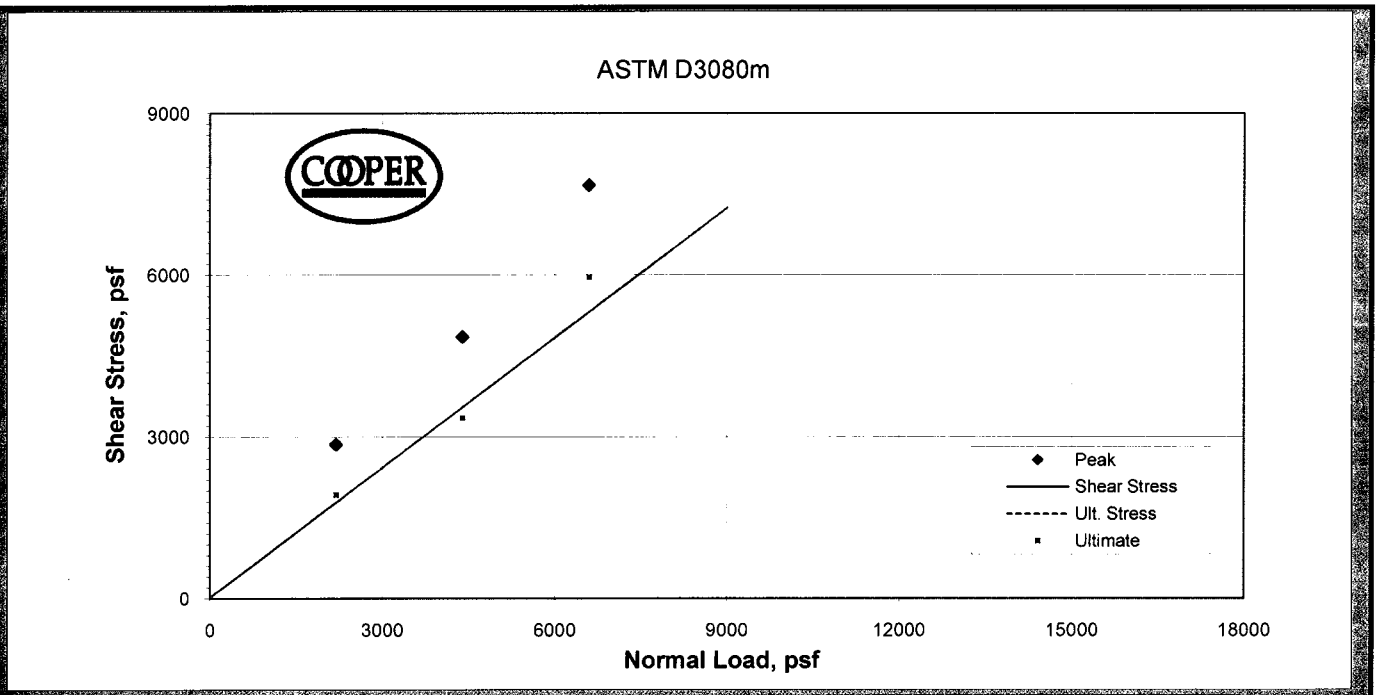
EVALUATION FOR SOIL CORROSION

Soil pH	7.24		
Minimum Resistivity	1.37 ohm-cm (x1000)		
Chloride	41.1 ppm	00.00411	%
Sulfate	4.5 ppm	00.00045	%

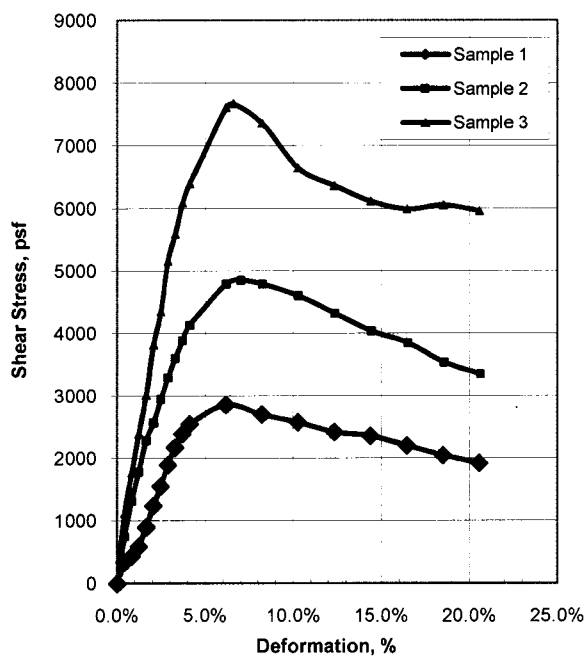
METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
MAY 8 2009
By

Direct Shear



P. Phi (degrees)	37	Ult. Phi (degrees)	37
P. Cohesion(psf)	0	Ult. Cohesion (psf)	0



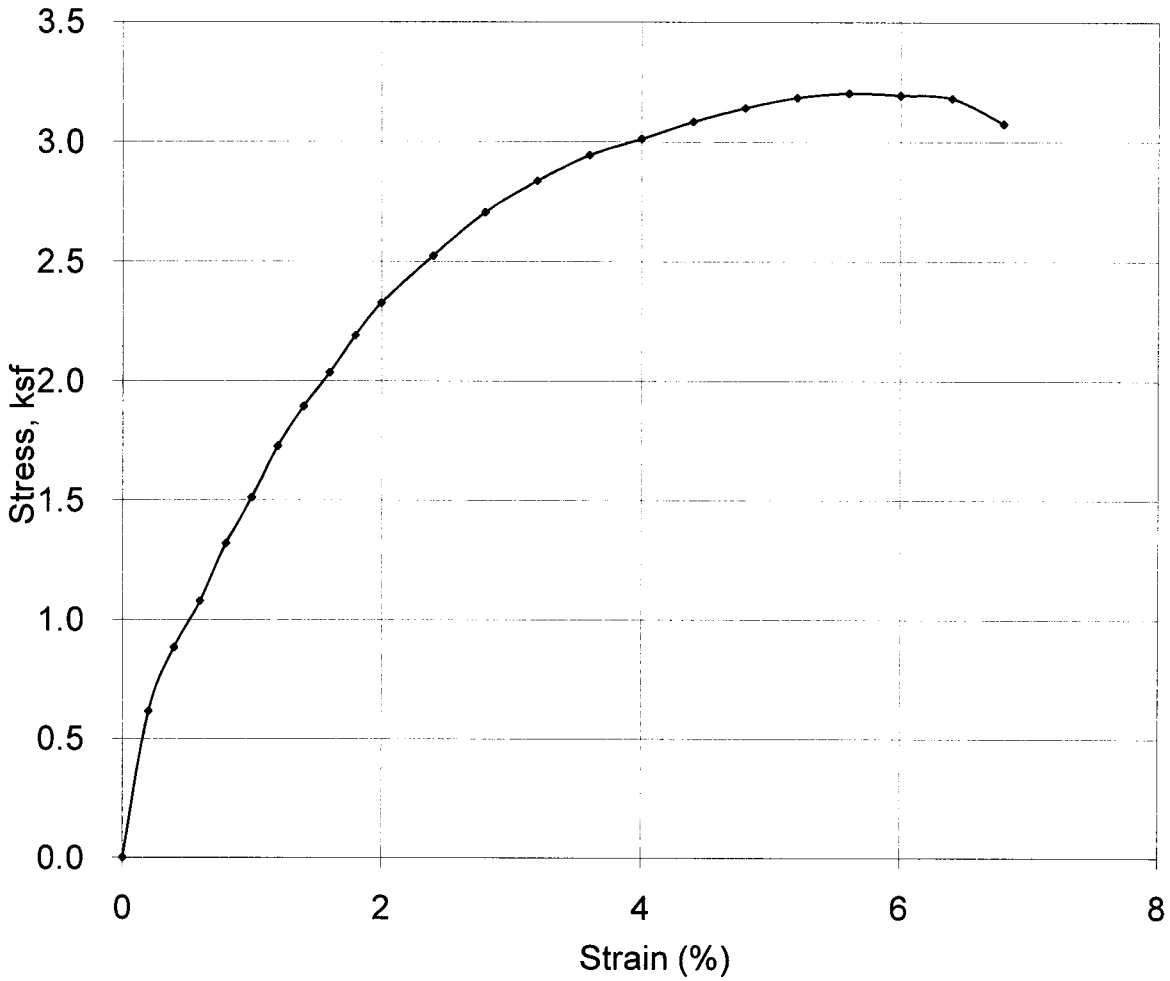
	Sample Data: Initial			
	1	2	3	4
Moisture %	20.6	21.7	16.0	
Dry Dens., pcf	105.3	103.4	113.3	
Void Ratio	0.600	0.630	0.488	
Saturation %	92.5	93.1	88.6	
Diameter	2.43	2.42	2.43	
Height	1.02	1.01	1.02	
	Sample Data: At Test			
Moisture %	20.4	21.1	15.6	
Dry Dens., pcf	108.1	107.4	117.7	
Void Ratio	0.559	0.570	0.432	
Saturation %	98.7	100.0	97.3	
Diameter	2.43	2.42	2.43	
Height	0.99	0.97	0.98	
Normal Stress, psf	2200	4400	6600	
Shear Stress, psf	2857	4853	7669	
Strengths picked at	6.2%	7.0%	6.6%	
Ult. Stress, psf	1925	3350	5962	
Strain Rate, in/min	0.020	0.020	0.020	

CTL #	157-272
Client:	Parikh Consultants, Inc.
Project	206127.GDR
Tested By:	MD
Reduced By:	RU
Date:	4/23/2009

Specimen #	Boring:	Sample:	Depth, ft:	Visual Soil Classification:
1	A-09-FS-001	#11	54.5	Greenish Gray Silty SAND
2	A-09-FS-001	#11	54.5	Greenish Gray Silty SAND
3	A-09-FS-001	#11	54.5	Greenish Gray Silty SAND
4				

Remarks: *DS-CU* A fully undrained condition may not be attained in this test.

UNCONFINED COMPRESSION TEST



Boring No.: A-09-FS-001

Sample No. : MC-2

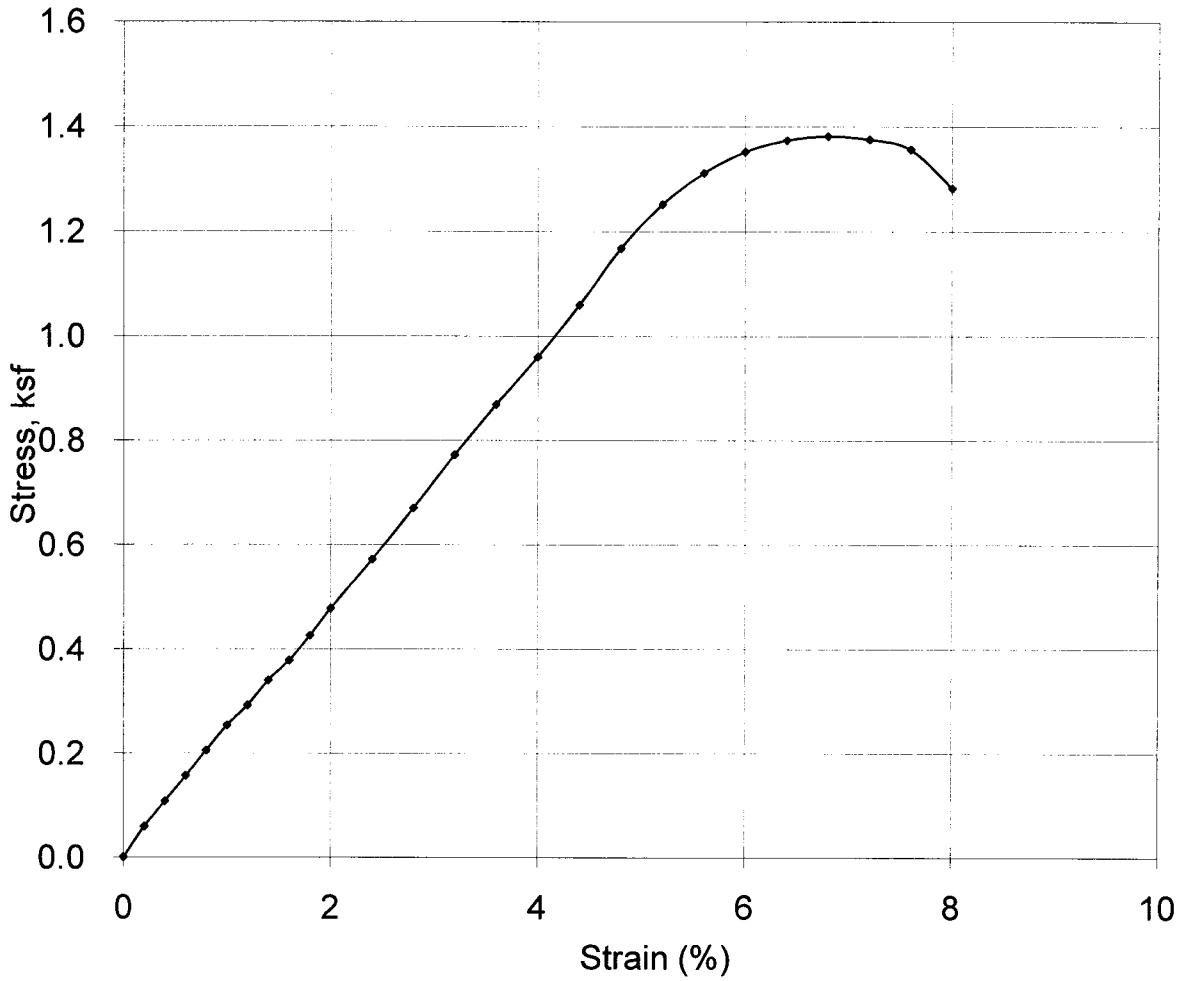
Depth (feet): 9.5

Material Description:
Fat Clay

Unconfined Compressive Strength: 3.2 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**FIVE MILE SLOUGH CULVERT MODIFICATION** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5A

UNCONFINED COMPRESSION TEST



Boring No.: A-09-FS-001

Sample No. : MC-3

Depth (feet): 14.5

Material Description:
Lean Clay, Stiff

Unconfined Compressive Strength: 1.4 ksf

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	**FIVE MILE SLOUGH CULVERT MODIFICATION** **STOCKTON, CALIFORNIA**	
	JOB NO.: 206127.BR	PLATE NO.: B-5B

APPENDIX C

Evaluation of Liquefaction Potential

PARIKH

Practicing in the Geosciences

JOB I-5 North Stockton Corridor Improvement 206127.GBR

SHEET NO _____ OF _____

CALCULATED BY A Lam DATE 5/2009

CHECKED BY _____ DATE _____

SCALE _____

Evaluation of Liquefaction Potential - Five Mile Slough RBC Modification (Boring A-09-BS-0901)

5.5'	Silty Sand (Medium dense) SPT N = 13	$\gamma' = 115$ pcf
18'		
19'	Silt/Lean Clay Avg SPT N = 10	$\gamma' = 115$ pcf, $\gamma_{sat} = 120$ pcf
8'	Silty Sand (Medium dense) SPT N = 24	$\gamma_{sat} = 130$ pcf
4.5'	Lean Clay/Clayey Sand SPT N = 16	$\gamma_{sat} = 120$ pcf
5.5'	Lean Clay SPT N = 42	$\gamma_{sat} = 130$ pcf
4.5'	Lean Clay SPT N = 16	$\gamma_{sat} = 120$ pcf
5'	Lean Clay SPT N = 50/4"	$\gamma_{sat} = 120$ pcf
5'	Silty Sand (Medium dense) SPT N = 29	$\gamma_{sat} = 130$ pcf
9'	Lean Clay Avg SPT N = 42	$\gamma_{sat} = 120$ pcf
13.5'	Silty Sand (Dense) Avg SPT N > 50	$\gamma_{sat} = 135$ pcf

LIQUEFACTION POTENTIAL ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement***
PROJECT NO. ***206127.GDR***
BORING NO. ***A-09-FS-001 (Five Mile Slough Culvert Modification)***

BOREHOLE DIA (in)= 8
GW DEPTH (ft)= 18

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

HAMMER TYPE (1/2) = 2
(1. ROPE AND PULLEY; 2. AUTOMATIC)

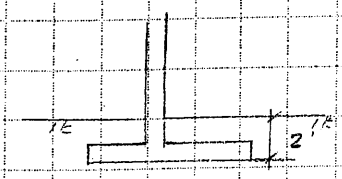
FAULT INFO
COAST RANGES-SIERRAN BLOCK
a_{max} *(g)=* 0.2
FAULT M_w = 7.0

MSF = 1.19
(Min. FS = 1.1 based on SPT)
F.S.=(CRR$_{7.0}$*/CSR)*MSF*K*σ**K*α

					CYCLIC STRESS RATIO (CSR)				*LIQUEFACTION RESISTANCE (CRR$_{7.0}$)*													
Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	σ_v (psf)	σ_v' (psf)	γ_d	CSR	SPT-N_{60}	C_N	C_E	C_R	C_S	C_B	$(N_1)_{60}$	F.C.	$(N_1)_{60,cs}$	$CRR_{7.0}$	Kσ	Kα	F.S.
1	5.5	1	13	SPT	632.5	632.5	0.98	0.13	15	1.78	1	0.75	1.2	1.15	27.61	15%	31.43		1.00	1	
2	24.5	2	10	SPT	3102.5	2508.8	0.88		12	0.89	1	0.80	1.2	1.15	11.37	55%			0.93	1	
3	32.5	1	24	SPT	4142.5	3048.8	0.82	0.14	28	0.81	1	0.80	1.2	1.15	24.76	15%	28.45	0.39	0.88	1	2.80
4	37.0	2	16	SPT	4727.5	3352.5	0.78		18	0.77	1	0.80	1.2	1.15	15.74	55%			0.86	1	
5	42.5	2	42	SPT	5442.5	3723.8	0.73		48	0.73	1	0.80	1.2	1.15	39.21	55%			0.83	1	
6	47.0	2	16	SPT	6027.5	4027.5	0.68		18	0.70	1	0.80	1.2	1.15	14.36	15%			0.81	1	
7	52.0	2	50	SPT	6677.5	4365.0	0.64		58	0.68	1	0.80	1.2	1.15	43.11	55%			0.79	1	
8	57.0	1	29	SPT	7327.5	4702.5	0.60	0.12	33	0.65	1	0.80	1.2	1.15	24.09	15%	27.75	0.36	0.77	1	2.75
9	66.0	2	42	SPT	8497.5	5310.0	0.55		48	0.61	1	0.80	1.2	1.15	32.83	55%			0.75	1	
10	79.5	1	50	SPT	10320.0	6288.8	0.50	0.11	58	0.56	1	0.80	1.2	1.15	35.92	15%	40.14		0.71	1	

Bearing Capacity Analyses

Bearing Capacity Analysis - Five Mile Slough RCB Modification - Retaining Wall



According to the designer. For channel soil c = 1250 psf N_c = 5.7, D = 2'
L = 14', Caltrans RW Type 1SWB, (14' high wall)

$$Q_{ult} = cN_c(1+0.3\frac{B}{L}) + \gamma D N_q + 0.4\gamma B N_\gamma$$

$$= 5.7 \times 1250 \times (1+0.3\times\frac{8}{14}) + 2\times 115\times 1$$

$$= 8576 \text{ psf}$$

Q_{ult} = 8576 psf Say 8.6 ksf × 7.6 ksf (14' high wall)

$$Q_{all} = \frac{8576 \text{ psf}}{3} = 2859 \text{ psf}$$ Say 2.75 ksf

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
10	SJ	5	25.0/32.7		

REGISTERED CIVIL ENGINEER DATE

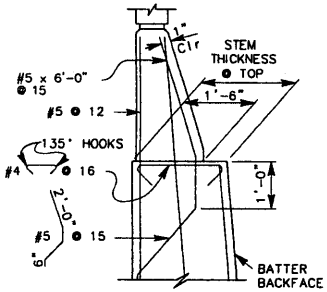
PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER No. Exp. CIVIL STATE OF CALIFORNIA

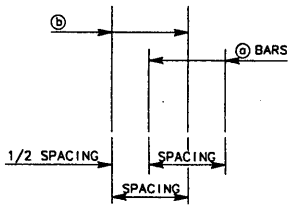
RAJAPPAN & MEYER CONSULTING ENGINEERS INC. 1038 LEIGH AVE., SUITE 100 SAN JOSE, CA 95126

SJCOG 555 EAST WEBER AVENUE STOCKTON, CALIFORNIA 95202



DETAIL A WITHOUT HAUNCH

NO SCALE

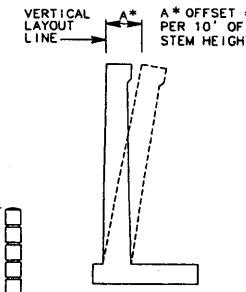


DETAIL B

NO SCALE

GENERAL NOTES

1. FOR SOUND WALL AND RETAINING WALL ARCHITECTURAL FINISH OR TEXTURE, SEE DETAILS ELSEWHERE IN PROJECT PLANS.
2. FOR DETAILS NOT SHOWN AND DRAINAGE NOTES, SEE B3-8
3. FOOTING COVER, 1'-6" MINIMUM.
4. LIMIT OF NO SPLICING REBARS = 3 TIMES THE BOTTOM THICKNESS OF THE STEM.
5. PLACEMENT OF REINFORCEMENTS :
   ⓑ & ⓒ BARS ARE SPLICED TOGETHER.
   * ⓐ & ⓑ BARS ARE BUNDLED TOGETHER.
   ** ALTERNATE ⓐ & ⓑ BARS ARE SHOWN IN "DETAIL B".
   Cont = CONTINUOUS.
6. FOR SOUND WALL & BARRIER REINFORCEMENT, SEE "SOUND WALL MASONRY BLOCK ON BARRIERS" SHEETS IN STANDARD PLANS.



WALL OFFSET

NO SCALE

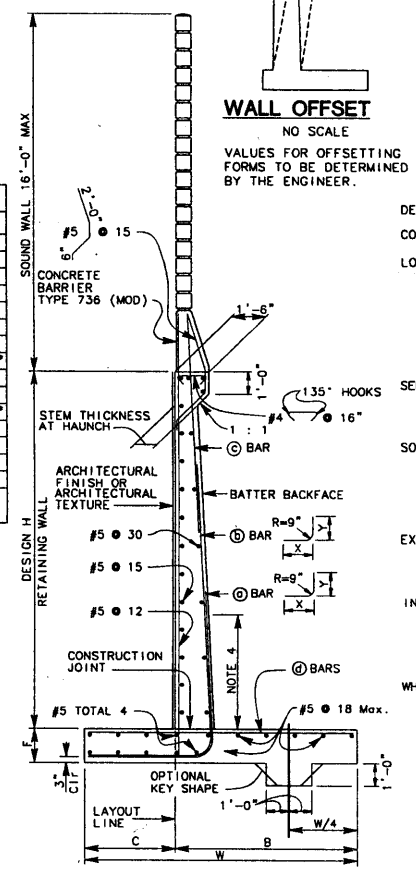
VALUES FOR OFFSETTING FORMS TO BE DETERMINED BY THE ENGINEER.

TABLE 1: TABLE OF REINFORCING STEEL DIMENSIONS AND DATA

	STEM WITH HAUNCH												
DESIGN H	6'	8'	10'	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'
W	7'-9"	8'-0"	8'-9"	9'-9"	10'-9"	12'-0"	13'-0"	14'-3"	15'-3"	16'-6"	18'-6"	19'-9"	21'-3"	22'-6"
C	2'-9"	2'-9"	3'-0"	3'-3"	3'-6"	4'-0"	4'-3"	4'-9"	5'-0"	5'-6"	6'-3"	6'-9"	7'-3"	7'-9"
B	5'-0"	5'-3"	5'-9"	6'-6"	7'-3"	8'-0"	8'-9"	9'-6"	10'-3"	11'-0"	12'-3"	13'-0"	14'-0"	14'-9"
F SPREAD FOOTING	1'-3"	1'-3"	1'-3"	1'-3"	1'-3"	1'-6"	1'-6"	1'-9"	2'-0"	2'-3"	2'-9"	3'-0"	3'-3"	3'-9"
BATTER	0	1/2:12	1/2:12	1/2:12	1/2: 12	1/2: 12	1/2:12	1/2:12	1/2:12	5/8:12	3/4:12	7/8: 12	1:12	1:12
STEM THICKN. @ HAUNCH	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"
ⓐ BARS				#6 @18**	#7 @18**	#8 @18**	#9 @18**	#10 @18**	#8 @9*	#8 @9*	#7 @6*	#10 @12**	#9 @9*	#11 @12**
X				Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont
Y				8'-0"	6'-6"	7'-6"	8'-6"	9'-6"	9'-6"	11'-0"	11'-6"	13'-0"	14'-0"	15'-6"
ⓑ BARS	#5 @12	#5 @12	#5 @9	#6 @18**	#7 @18**	#8 @18**	#9 @18**	#10 @18**	#8 @9*	#8 @9*	#7 @6*	#10 @12**	#9 @9*	#11 @12**
X	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	5'-6"	6'-6"	7'-0"	8'-0"	8'-6"
Y	Cont	Cont	Cont	Cont	10'-6"	13'-0"	15'-0"	17'-6"	19'-6"	21'-0"	18'-6"	19'-0"	25'-6"	23'-6"
ⓒ BARS					#6 @18	#6 @18	#6 @18	#6 @18	#6 @18	#6 @18	#6 @12	#7 @12	#7 @18	#7 @12
ⓓ BARS	#5 @12	#5 @12	#5 @12	#5 @9	#6 @9	#6 @9	#6 @6	#8 @9	#8 @9	#8 @9	#7 @6	#7 @6	#9 @9	#9 @9
ULTIMATE BEARING CAPACITY REQ'D K/SF	4.6	5.5	6.1	6.6	7.3	8.1	9.0	9.9	11.3	12.2	12.5	13.4	14.1	15.4

TABLE 2: TABLE OF REINFORCING STEEL DIMENSIONS AND DATA

	STEM WITH HAUNCH						STEM WITHOUT HAUNCH						
DESIGN H	8'	10'	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'
W	8'-0"	8'-6"	9'-3"	10'-3"	11'-3"	13'-0"	13'-9"	16'-0"	16'-6"	18'-6"	20'-0"	20'-6"	21'-9"
C	2'-9"	2'-9"	3'-0"	3'-6"	3'-9"	4'-3"	4'-6"	5'-3"	5'-6"	6'-3"	6'-9"	6'-9"	7'-6"
B	5'-3"	5'-9"	6'-3"	6'-9"	7'-6"	8'-9"	9'-3"	10'-9"	11'-0"	12'-3"	13'-3"	13'-9"	14'-3"
F SPREAD FOOTING	1'-3"	1'-3"	1'-3"	1'-3"	1'-6"	1'-6"	1'-9"	2'-0"	2'-3"	2'-9"	3'-0"	3'-3"	3'-6"
BATTER	0	0	0	0	0	0	0	0	0	?:12	ø:12	ø:12	ø:12
STEM THICKN. @ HAUNCH	1'-0"	1'-0"	1'-0"	1'-0"	1'-3"	1'-3"							
STEM THICKN. @ TOP							1'-6"	2'-0"	2'-0"	2'-0"	2'-0"	2'-0"	2'-0"
ⓐ BARS			#6 @12**	#5 @6*	#9 @18**	#9 @12**	#9 @12**	#9 @12**	#7 @6*	#7 @6*	#7 @6*	#9 @9*	#11 @12**
X			Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont
Y			5'-6"	5'-6"	8'-6"	8'-6"	8'-6"	8'-6"	9'-6"	11'-0"	12'-6"	13'-6"	14'-6"
ⓑ BARS	#5 @9	#6 @9	#6 @12**	#5 @6*	#9 @18**	#9 @12**	#9 @12**	#9 @12**	#7 @6*	#7 @6*	#7 @6*	#9 @9*	#11 @12**
X	Cont	Cont	Cont	Cont	Cont	Cont	Cont	Cont	5'-6"	6'-0"	7'-6"	7'-6"	8'-0"
Y	Cont	Cont	Cont	Cont	11'-6"	11'-6"	12'-0"	12'-0"	17'-6"	20'-0"	21'-6"	24'-0"	24'-0"
ⓒ BARS					#7 @18	#7 @12	#7 @12	#7 @12	#6 @12	#6 @12	#6 @12	#7 @18	#7 @12
ⓓ BARS	#5 @12	#5 @9	#5 @9	#6 @9	#6 @9	#9 @12	#9 @12	#7 @7	6 @6	#7 @6	#7 @6	#9 @9	#9 @9
ULTIMATE BEARING CAPACITY REQ'D k/sf	5.5	6.5	7.3	7.6	9.0	9.0	10.6	10.5	12.2	12.6	13.3	15.4	16.0



SPREAD FOOTING SECTION

NO SCALE

DESIGN DATA

DESIGN: LOAD FACTOR DESIGN (LFD)

CONCRETE: REINFORCED CONCRETE, f'c = 3600 psi
fy = 60,000 psi

LOADING CASE:
LEVEL GROUND WITH 240 psf SURCHARGE AND 16' SOUNDWALL

SEISMIC LOAD = 0.3 DEAD LOAD
WIND LOAD = 30 psf
DEAD LOAD OF SOUNDWALL = 1414 lb/ft
DEAD LOAD OF BARRIER = 372 lb/ft

SEISMIC LOAD: SOIL
Kh = 0.3g
Kv = 0.0
Kae : MONONOBE-OKABE METHOD

SOIL: Ø = 34° γ = 120 pcf
EQUIVALENT FLUID PRESSURE:
= 36 pcf FOR DETERMINATION OF TOE PRESSURE
= 27 pcf FOR DETERMINATION OF HEEL PRESSURE

EXTERNAL STABILITY:
GROUP 1 : D + E + SC
GROUP 2 : D + E + SC + W
GROUP 3 : D + PYM

INTERNAL STABILITY (LFD):
GROUP A : βD + 1.7E + 1.7SC
GROUP B : βD + 1.7E + 1.3W
GROUP C (STEM) : 1.0D + 1.0E + 1.0EQD + 1.0EQE
GROUP C (FOOTING) : D + PYM

WHERE : β = 1.0 OR 1.3 WHICHEVER CONTROLS DESIGN
D = DEAD LOAD
E = LATERAL EARTH PRESSURE
SC = SURCHARGE
W = WIND LOAD
EQD = SEISMIC DEAD LOAD
EQE = SEISMIC LATERAL EARTH PRESSURE
PYM = PROBABLE YIELD MOMENT (1.3 X NOMINAL YIELD MOMENT OF STEM)

PROGRESS SUBMITTAL
OCTOBER 2009

SOUND WALL DETAILS

RETAINING WALL TYPE 1SWB

NO SCALE

SW-53

REVISED BY | DATE REVISED | CALCULATED-DESIGNED BY | CHECKED BY | CONSULTANT FUNCTIONAL SUPERVISOR

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

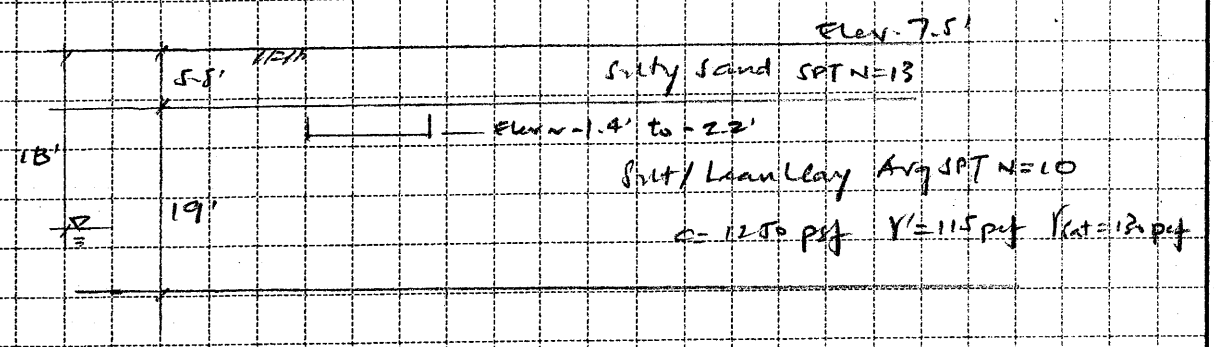
LAST REVISION 06-08-09 | DATE PLOTTED => $DATE | TIME PLOTTED => $TIME

BORDER LAST REVISED 4/11/2008 | RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3 | USERNAME => $USER | DGN FILE => $REQUEST | CU 06240 | EA 10-0G4701

PARIKH
Practicing in the Geosciences

JOB: I-5 North Stockton Corridor Improvement
SHEET NO: ____ OF: ____
CALCULATED BY: A Lam DATE: 4/20/2010
CHECKED BY: ____ DATE: ____
SCALE: ____

Bearing Capacity Analysis - Five Mile Slough Box Culvert



Assume strip footing, D = 2 feet

$$Q_{ult} = cN_c + \gamma D N_q$$

$$= 5.7 \times 1250 + 115 \times 2 \times 1$$

$$= 7355 \text{ psf}$$

$$Q_{all} = \frac{7355}{3.0}$$ Assume F.O.S. = 3.0

$$= 2452 \text{ psf}$$ Say **2250 psf**

Lateral Earth Pressure and Coefficient of Friction

Lateral Earth Pressure for Spread Footing of Retaining Wall

Using Coulomb to calculate passive equivalent fluid pressure

Assume $\phi' = 26°$ $\beta = 0$ $\delta/\phi' = 0.6$

Based on Figure 5.5.4-2 (Assuming wall with friction)

$\phi' = 26°$, $-\delta/\phi = -0.6$ Reduction Factor $= 0.86 - (0.86 - 0.81) \times 1/3$

$= 0.85$

$k_p = 4.7 \times 0.85$

$= 4.0$

Passive Equivalent Fluid Pressure $= 4.0 \times 115$

$= \underline{460 \text{ pcf}}$

Coefficient of Friction

$\beta = \tan \frac{2}{3} \phi'$

$= \tan \frac{2}{3} (26°)$

$= 0.31$ Say $\underline{0.30}$

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

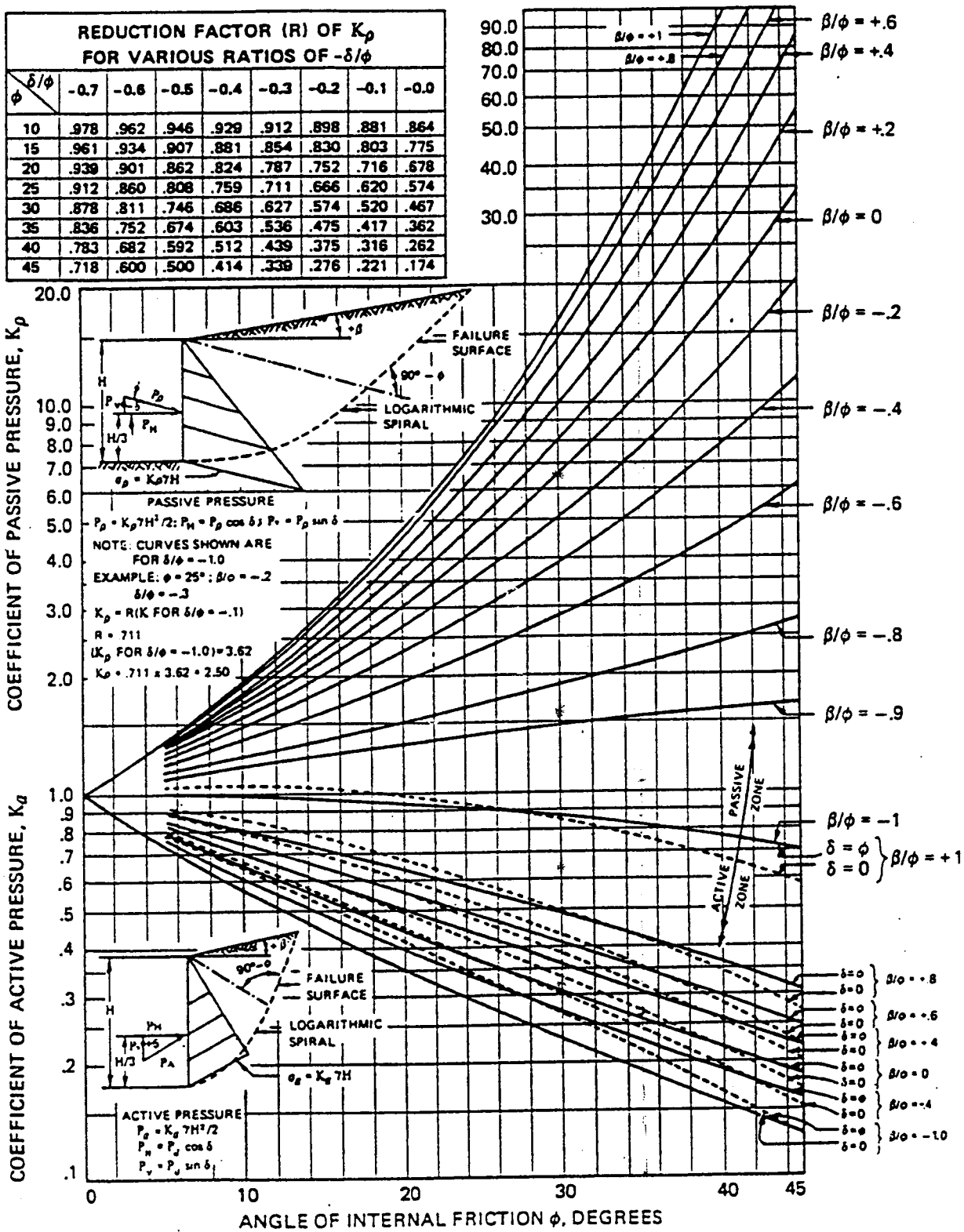
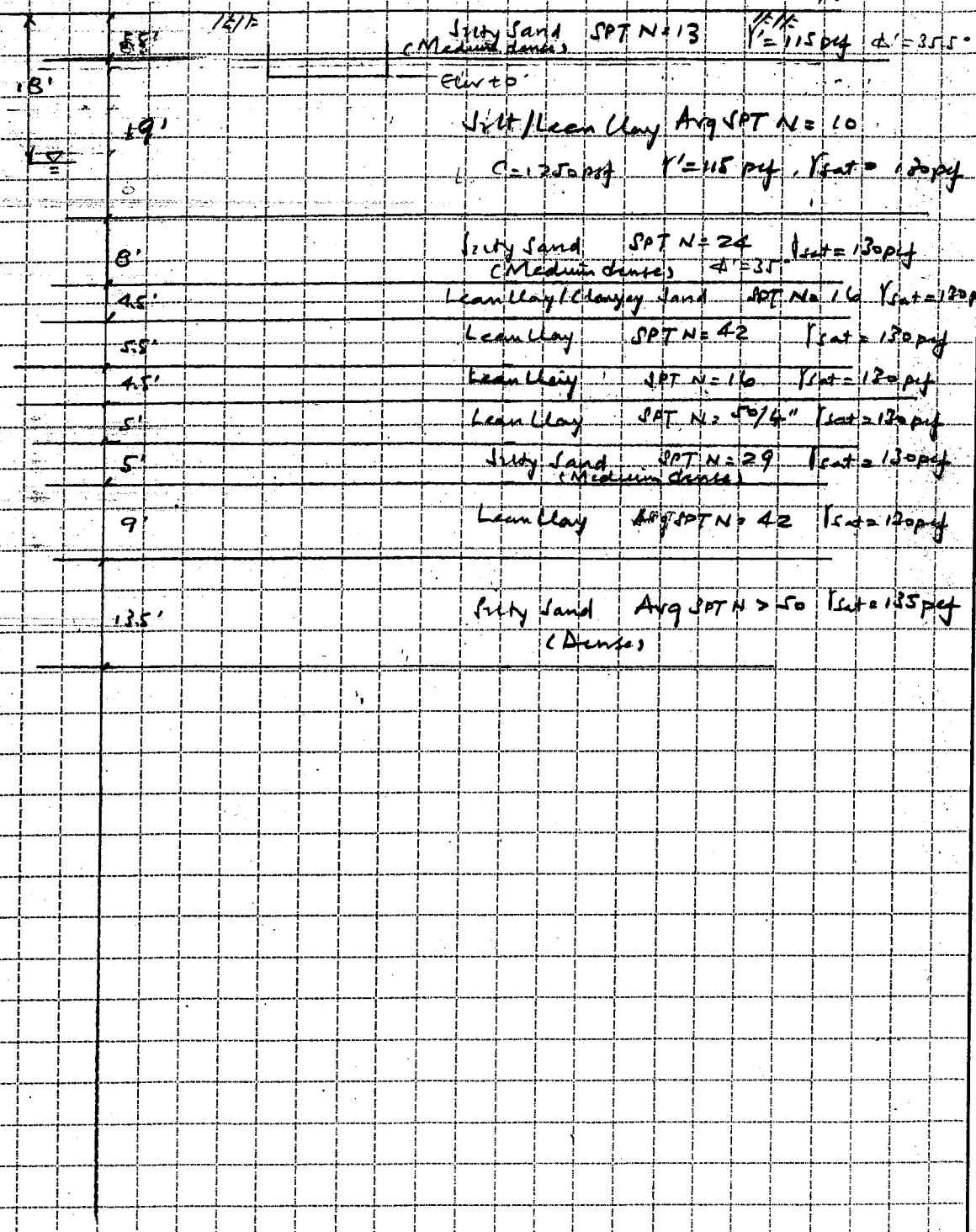


Figure 1
Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel)

Settlement Analyses

Settlement Analyses - Five Mile Slough (Boring A-09-FS-001)



SETTLEMENT ANALYSIS

PROJECT NAME ***I-5 North Stockton Corridor Improvement Project - Five Mile Slough Culvert Modification***
PROJECT NO. ***206127.BR***
BORING NO. ***A-09-FS-001 (A-09-013)***

Embankment H (ft)= 20
Unit Weight (pcf)= 125
Contact Pressure= 2500

GW Level (ft)= 18 *(ft)*

Contact Area, B (ft)= 8
Contact Area, L (ft)= 14
Plain Strain? (Y/N)= N

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

Estimated Total Settlement (in)= 0.56

Soil Type	Depth From	Depth To	BLOW COUNT	SAMPLER TYPE	AVG SPT-N	ω	γ_d' (pcf)	γ_T (pcf)	γ' (pcf)	σ_v' (psf)	$\Delta\sigma_v'$ (psf)	Pp (psf)	$Cr/1+e_0$	$Cc/1+e_0$	Settlements (in) OC	Settlements (in) NC	Settlements (in) Sum
1	0	5.5	20	MC	13	12.0%	118.0	132.2	132.2	363	1555.0						
2	5.5	11	14	MC	9	30.0%	91.8	119.3	119.3	1055	774.4	4550	0.0188	0.1500	0.296		0.296
2	11	16	14	MC	9	25.0%	98.1	122.6	122.6	1690	473.6	4550	0.0172	0.1376	0.111		0.111
2	16	18	15	MC	10	27.0%	100.5	127.6	127.6	2124	361.3	4875	0.0178	0.1426	0.029		0.029
2	18	24.5	15	MC	10	27.0%	100.5	127.6	65.2	2464	271.6	4875	0.0178	0.1426	0.063		0.063
1	24.5	32.5	36	MC	23	23.0%	105.0	129.2	66.8	2943	180.5						
2	32.5	37	16	SPT	16	21.0%	95.0	115.0	52.6	3328	134.4	8000	0.0160	0.1277	0.015		0.015
2	37	42.5	66	MC	43	32.0%	91.5	120.8	58.4	3607	109.1	21450	0.0194	0.1550	0.017		0.017
2	42.5	47	24	MC	16	27.0%	98.0	124.5	62.1	3907	90.3	7800	0.0178	0.1426	0.010		0.010
2	47	52	50	MC	33	28.0%	98.9	126.6	64.2	4207	76.7	16250	0.0181	0.1451	0.009		0.009
1	52	57	29	SPT	29	19.0%	107.3	127.7	65.3	4531	65.4						
2	57	66	65	MC	42	31.0%	96.3	126.2	63.8	4981	53.4	21125	0.0191	0.1525	0.010		0.010
1	66	79.5	50	MC	33	17.0%	116.5	136.3	73.9	5767	40.0						
																	0.558

APPENDIX D

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	25.0/32.7		

REGISTERED CIVIL ENGINEER — DATE 1/15/01

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — Robert Sennett — No. 3976 — Exp. 12/31/10 — STRUCTURAL — STATE OF CALIFORNIA

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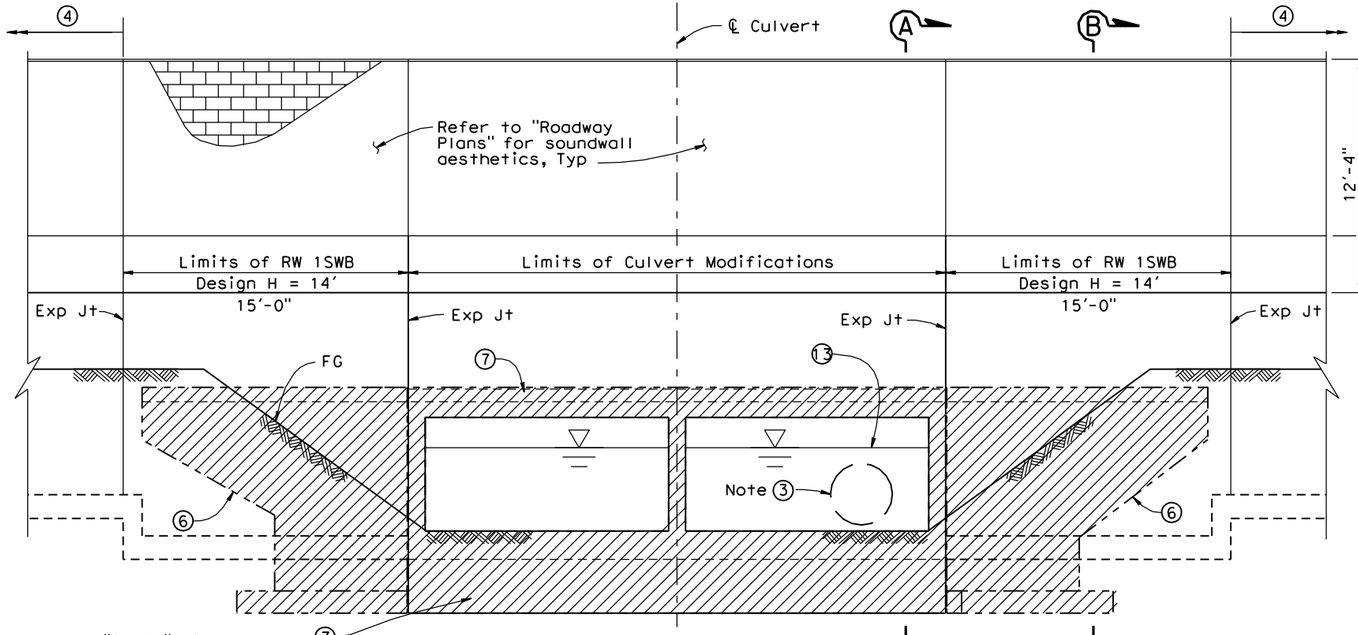
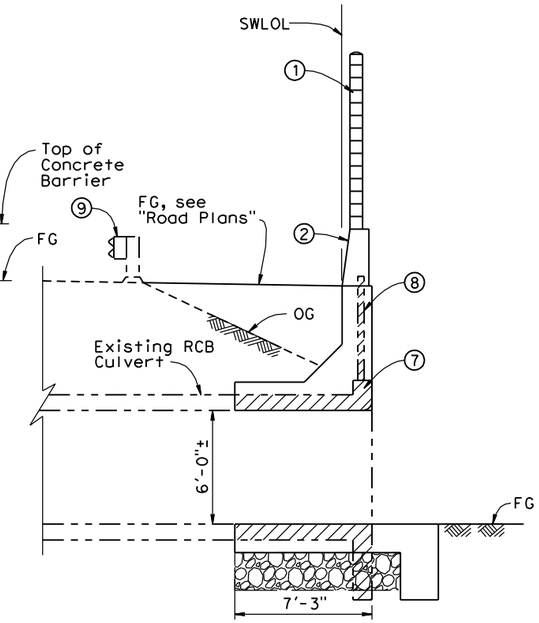
7415 GREENHAVEN DRIVE, SUITE 100
SACRAMENTO, CA 95831

SAN JOAQUIN COUNCIL OF GOVERNMENT
555 E. WEBER AVENUE
STOCKTON, CA 95202



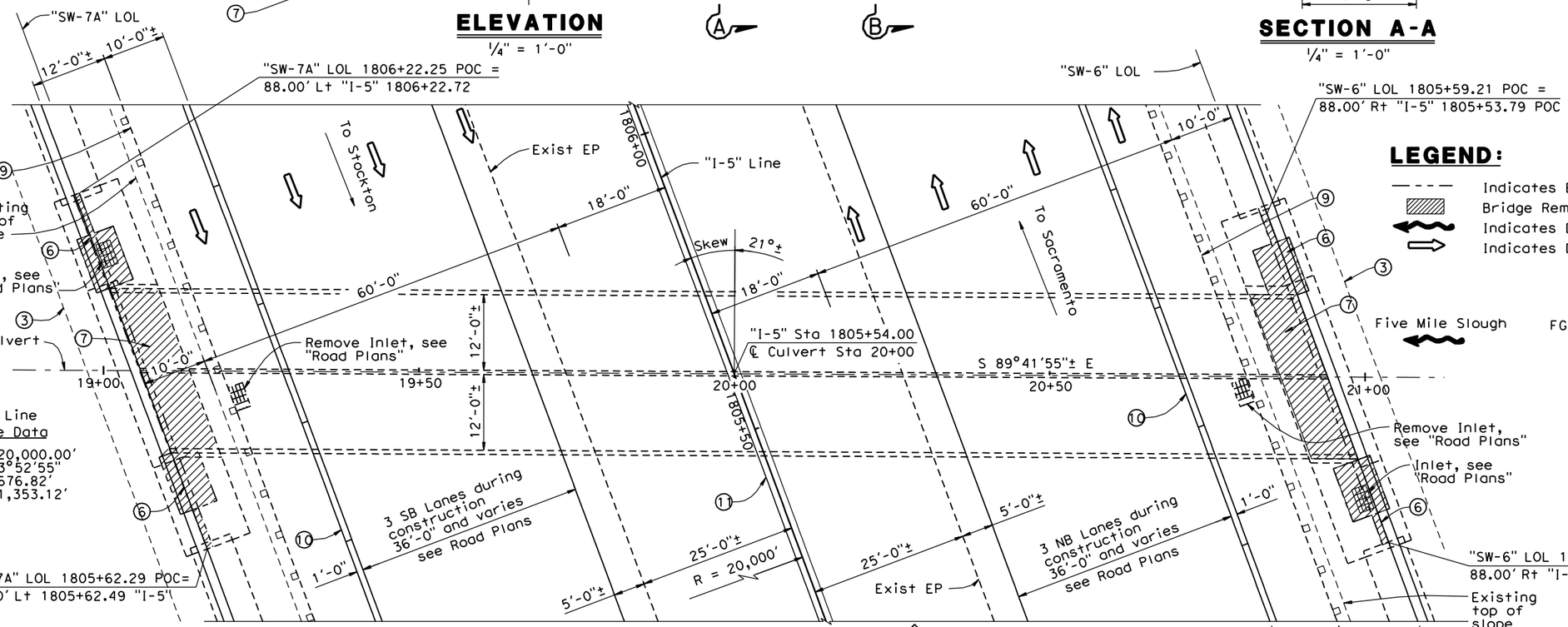
ELEVATION

1/4" = 1'-0"



SECTION A-A

1/4" = 1'-0"



PLAN

1" = 10'

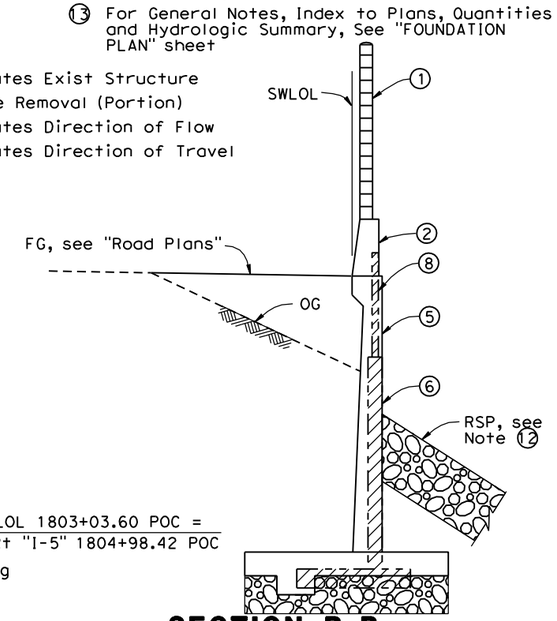
N

NOTES:

1. Soundwall No. 6 & No. 7A, see "Road Plans"
2. Concrete Barrier, Type 736 (Mod)
3. Temporary gravel cofferdam and Culvert as required, see project permits and specifications.
4. For adjacent Soundwall not shown, see "Road Plans"
5. Retaining Wall Type 1SWB
6. Remove Exist Wingwall
7. Remove Portion of RCB Culvert
8. Remove Exist Chain Link Fence
9. Remove Exist MBGR, see "Road Plans"
10. Temporary Railing Type K for stage construction, see "Road Plans"
11. Concrete Barrier Type 60, see "Road Plans"
12. Remove, Stockpile and Replace Exist RSP as necessary
13. For General Notes, Index to Plans, Quantities and Hydrologic Summary, See "FOUNDATION PLAN" sheet

LEGEND:

- Indicates Exist Structure
- Bridge Removal (Portion)
- Indicates Direction of Flow
- Indicates Direction of Travel



SECTION B-B

1/4" = 1'-0"

NOTE:

The Contractor shall verify all controlling field dimensions before ordering and fabricating any material.

PLAN CHECK SET/NOT FOR CONSTRUCTION (1/15/10)

DESIGN OVERSIGHT	DESIGN	BY P. Zhao	CHECKED G. Dizon	LOAD FACTOR DESIGN	
	DETAILS	BY P. Zhao	CHECKED G. Dizon	LAYOUT	BY R. Sennett — CHECKED G. Dizon
SIGN OFF DATE	QUANTITIES	BY R. Huang	CHECKED G. Dizon	SPECIFICATIONS	BY R. Sennett — PLANS AND SPECS COMPARED

DESIGN GENERAL PLAN SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Robert Sennett
PROJECT ENGINEER

CU 06240
EA 10-0G4701

BRIDGE NO. N/A
POST MILES 32.1

FIVE MILE SLOUGH CULVERT MODIFICATION

GENERAL PLAN

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY): 06/08/09 10/16/09 12/04/09 01/15/10

SHEET 1 OF 12

FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
10	SJ	5	25.0/32.7		

REGISTERED CIVIL ENGINEER — DATE 1/15/10

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — Robert Sennett — No. 3976 — Exp. 12/31/10 — STRUCTURAL — STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

MGE ENGINEERING, INC.
7415 GREENHAVEN DRIVE, SUITE 100
SACRAMENTO, CA 95831

SAN JOAQUIN COUNCIL OF GOVERNMENT
555 E. WEBER AVENUE
STOCKTON, CA 95202

INDEX TO PLANS

SHEET NO.	TITLE
1	GENERAL PLAN
2	FOUNDATION PLAN
3	DEMOLITION DETAILS
4	CULVERT LAYOUT
5	MODIFICATION DETAILS NO. 1
6	MODIFICATION DETAILS NO. 2
7	RETAINING WALL TYPE 1SWB
8	SOUNDWALL DETAILS NO. 1
9	SOUNDWALL DETAILS NO. 2
10	SOIL LEGEND 1 OF 2
11	SOIL LEGEND 2 OF 2
12	LOG OF TEST BORINGS

LEGEND

Indicates Direction of Flow

X.x — Indicates Bottom of Footing Elevation

13.7x — Indicates spot elevation, Typ

GENERAL NOTES
LOAD FACTOR DESIGN

DESIGN: BRIDGE DESIGN SPECIFICATIONS
2000 AASHTO with Interims and Revisions by CALTRANS

REINFORCED CONCRETE: f_y = 60,000 psi

f'_c = 3,600 psi

CULVERT

LOAD FACTORS: 1.5D + 1.5E + 2.5(L+I)

Where D=Dead Load
E=Earth Load
L=Live Load
I=Impact
Capacity reduction factor is included.

LOADING:
LIVE LOAD: HS20-44 truck
Surcharge on walls due to live load.

EARTH LOAD: Earth pressures for two conditions:
140 LB/CF vertical, 42 LB/CF horizontal.
140 LB/CF vertical, 140 LB/CF horizontal.

RETAINING WALLS

For Design Notes, see "RETAINING WALL TYPE 1SWB" sheet

SOUND WALLS

For Design Notes, see "SOUNDWALL DETAILS NO.2" sheet

STANDARD PLANS

Dated May 2006

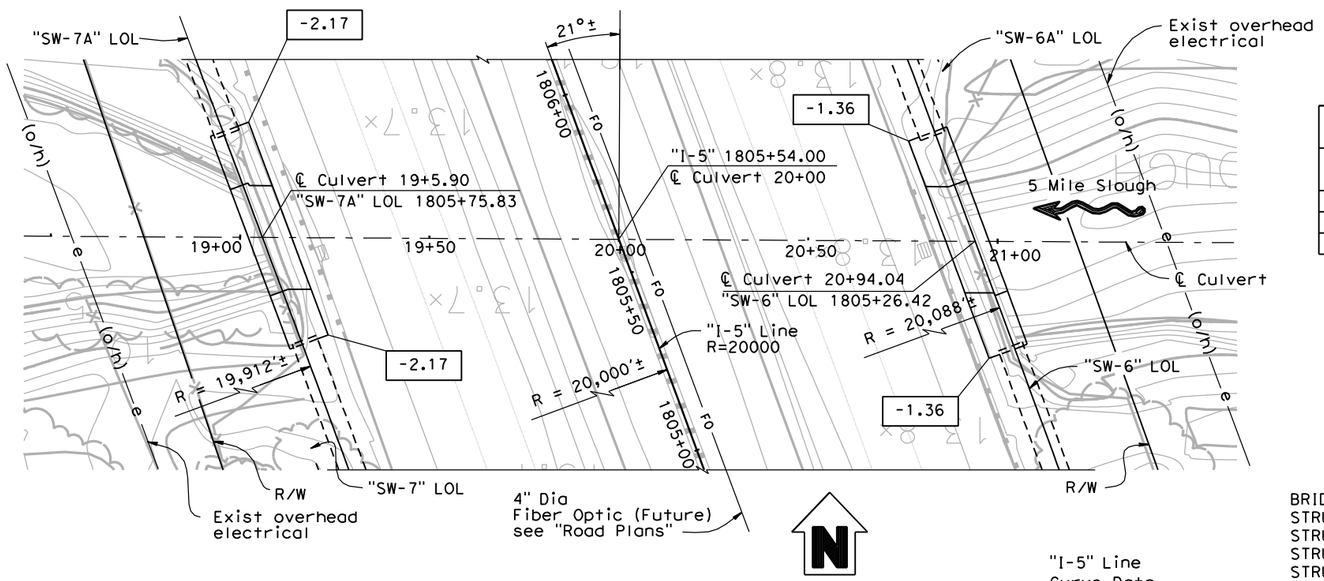
A10A ACRONYMS AND ABBREVIATIONS (SHEET 1 OF 2)
A10B ACRONYMS AND ABBREVIATIONS (SHEET 2 OF 2)
A10C SYMBOLS (SHEET 1 OF 2)
A10D SYMBOLS (SHEET 2 OF 2)
A62A EXCAVATION AND BACKFILL MISCELLANEOUS DETAILS
A62B LIMITS OF PAYMENT FOR EXCAVATION AND BACKFILL BRIDGE SURCHARGE AND WALL
A62E EXCAVATION AND BACKFILL CAST-IN-PLACE REINFORCED CONCRETE BAX AND ARCH CULVERTS
D81 CAST-IN-PLACE REINFORCED CONCRETE DOUBLE BOX CULVERT
D82 CAST-IN-PLACE REINFORCED CONCRETE BOX CULVERT MISCELLANEOUSE DETAILS
B3-8 RETAINING WALL DETAILS NO. 1
B15-6 SOUND WALL MASONRY BLOCK ON TYPE 736S/SV BARRIER DETAILS (1)
B15-7 SOUND WALL MASONRY BLOCK ON TYPE 736S/SV BARRIER DETAILS (2)
B15-9 SOUND WALL MASONRY BLOCK MISCELLANEOUS DETAILS

STANDARD PLAN SHEET NO.
DETAIL NO.

HYDROLOGIC SUMMARY

	Design Flood	Design Flood
Frequency (years)	50	100
Discharge (cubic feet per second)	136	141
Water surface (elevation at bridge)	9.4	9.8
Velocity (feet per second)	0.94	0.98

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



FOUNDATION PLAN

1" = 20'

"I-5" Line
Curve Data

R = 20,000.00'
Δ = 3°52'55"
T = 676.82'
L = 1,353.12'

BENCH MARK AND DATUM

Monument	Coordinates Northing	Coordinates Easting	Elevation	Description/Location
46	2192675.55	6315520.45	13.62	PK Nail
604	2179257.74	6267471.36	13.65	Iorn Pipe w/ "MVE CONTROL" Cap
102M	2192105.52	6315533.46	13.43	Iorn Pipe w/ "MVE CONTROL" Cap

NOTE:

There is a vertical datum difference between the project datum (NAVD 88) and the as-built datum (NGVD 29). The following conversion was used in design: Datum Conversion: Elev (NAVD 88) = Elev (NGVD 29) + 2.37'

QUANTITIES

BRIDGE REMOVAL (PORTION), LOCATION K	1 LS
STRUCTURE EXCAVATION (RETAINING WALL)	288 CY
STRUCTURE BACKFILL (RETAINING WALL)	268 CY
STRUCTURAL CONCRETE, RETAINING WALL	66 CY
STRUCTURAL CONCRETE (BOX CULVERT)	103 CY
SOUND WALL (BARRIER) (MASONRY BLOCK)	1010 SF
BAR REINFORCING STEEL (RETAINING WALL)	7213 LB
BAR REINFORCING STEEL (BOX CULVERT)	23608 LB
CONCRETE BARRIER (TYPE 736 MODIFIED)	110 LF

GEOTECHNICAL PROFESSIONAL — APPROVAL DATE 1/15/10

NOTE:

The Contractor shall verify all controlling field dimensions before ordering and fabricating any material.

PLAN CHECK SET/NOT FOR CONSTRUCTION (1/15/10)

DESIGN OVERSIGHT — SIGN OFF DATE

SCALE: 1" = 20' | VERT.DATUM NAVD88 | HORZ.DATUM CCS83, (2007.00), ZONE 3
PHOTOGRAMMETRY AS OF: (N/A) FIELD SURVEYS | ALIGNMENT TIES APPROVED GPS REPORT DATED 7-6-09
SURVEYED BY RYAN VANCE | DRAFTED BY MIKE KORDAZAKIS
FIELD CHECKED BY JEFF CISSELL | CHECKED BY KEVIN LANG

DESIGN BY P. Zhao CHECKED G. Dizon
DETAILS BY P. Zhao CHECKED G. Dizon
QUANTITIES BY R. Huang CHECKED G. Dizon

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Robert Sennett
PROJECT ENGINEER

BRIDGE NO. N/A
POST MILE 32.1

FIVE MILE SLOUGH CULVERT MODIFICATION
FOUNDATION PLAN

FOUNDATION PLAN SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 06240
EA 10-0G4701

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY) 06/08/09 10/16/09 12/04/09 01/15/10

SHEET 2 OF 12

FILE => $REQUEST

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

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APPENDIX E

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** Five Mile Slough Culvert Modification **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** johnfujimoto @dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection __ 65% PS&E Unchecked Details __ PS&E (Review No. 1) __ Construction __ Other:_____	**Reviewer Name:** Huy Tran **Functional Unit:** 59 **Cost Center:** 198 **Phone Number:** 916-227-9527 **e-mail:** Huy D Tran@dot.ca.gov **Date of Review:** 02/26/10 **Structure Name*:** Five Mile Slough Culvert Mod **Br No*:** N/A (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	18, 19, 20	DC	The bar size does not match with what shown on plan sheet No 6		
2	7, 10	DC	What's the conclusion for checking bearing pressure at base (culvert and ret wall)?	Comment will be incorporated. Allowable bearing capacity for the box culvert structure will be recommended in the report.	
3					
4					
5					
6					
7					
8					
9					
10					
11					

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 2/23/2010 **Structure Name*:** Five Mile Slough Culvert Mod. **Br No*:** (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	4/20/2010

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 2nd review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 1/14/2010.	NA	
2	FR	NA	C1 – Approved as Submitted	NA	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓ = Comment Resolved (for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 10 **EA:** 0G4701 **Project Name:** I-5 North Stockton Widening **OSFP Liaison:** John Fujimoto **Phone:** 916-227-8757 **e-mail:** john_fujimoto@dot.ca.gov	__ PSR/PDS (Review No.___ __ APS/PSR (Review No.___) __ APS/PR (Review No.___) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.___) __ Construction __ Other:	**Reviewer Name:** Brandon Badeker **Functional Unit:** Geotechnical Services **Cost Center:** 59-323 **Phone Number:** 916-227-1046 **e-mail:** brandon_badeker@dot.ca.gov **Date of Review:** 12/16/2009 **Structure Name*:** Five Mile Slough Culvert Mod. **Br No*:** (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Keith Meyer	Rajappan & Meyer Consulting Engineers, Inc.	(408) 280 2772	keith@rmengineers.com	1/14/10

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	NA	This is the 1st review of the following documents by the Caltrans Office of Geotechnical Design – North. -FR prepared by Parikh, dated 10/15/2009.	NA	
2	FR	p.3, S.4	Please provide a reference PM or Stationing.	Comment is incorporated. The post mile is referred in this section.	
3	FR	p.3, S.5	Field exploration states that the soil boring was drilled to a depth of 111-feet bgs. The LOTB shows the boring terminated at 80-feet bgs. Please clarify.	Comment is incorporated. The boring depth is changed to 80 feet in Section 5.	
4	FR	p.4, S.6	The subsurface condition states loose to medium dense sands underlie firm to stiff clay/silt although the LOTB states the sands vary from loose to very dense, please clarify.	Comment is incorporated. The statement in Section 6 is changed to "….underlain by loose to very dense sands,…".	
5	FR	p.6	Table 1, According to the seismic map provided as Plate 4, the maximum distance to both faults is 20 miles.	Comment is incorporated. The maximum distance to both faults is changed to 20 miles.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

Submittal Data (Reviewer to complete)

Dist-EA: 10-0G4701 Reviewer: BHB Str Name*: Five Mile Slough

Date of Review: 12/16/09 Functional Unit: 59-323 Br No*. ____ *=if applicable

6	FR	p.10, S.9.8	A construction consideration may be to abide by Army Corps of Engineers and/or Department of Water Resources construction and backfill requirements, please respond.	This statement is correct. This will be included in the construction consideration. The details provided in this section were discussed with the design team and the agency. Some of these will be incorporated in the specifications.	
7	FR	LOTB	As is stated in the Soil Legend, the sample number or unconfined compressive strength is generally not located to the left of the graphic on the LOTB.	Comment is incorporated.	
8	FR	LOTB	No unconfined compression data was contained in the appendices although lab testing shown on LOTB's. Please submit UC data with next submittal.	Comment is incorporated. Unconfined compression test data for each soil sample is included in Appendix B.	
9	FR	NA	C2 – Approved subject to OSFP Verification	NA	