GEOTECHNICAL DESIGN & MATERIALS REPORT
RICHMOND-SAN RAFAEL BRIDGE ACCESS
IMPROVEMENT PROJECT
04-CC-580 PM 5.0/7.8, 04-MRN-580 PM 0.0/3.3, EA 04-2J6800

For

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April 29, 2016 Job No. 2015-125-GDR

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April 29, 2016

Attn: Mr. Chadi Chazbek

Sub: Geotechnical Design & Materials Report
Richmond- San Rafael Bridge Access Improvement Project
Contra Costa County and Marin County, California
EA 04-2J6800

Dear Mr. Chazbek:

Transmitted herewith is the Geotechnical Materials Report for the subject project. The report was prepared in accordance with the scope of work outlined in our proposal.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning our findings or conclusions, please feel free to contact this office at (408) 452-9000.

Very truly yours,
PARIKH Consultants, Inc.

Gary Parikh, P.E., G.E.
President

Attachment: Geotechnical Design & Materials Reports

"Approved as to impact on State facilities and conformance with applicable State standards and practices, and the technical oversight were performed as described in the California Department of Transportation A&E Consultant Services Manual."

Caltrans_____

Title_____

Date_____

Caltrans_____

Title_____

Date_____

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RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
CONTRA COSTA COUNTY & MARIN COUNTY, CALIFORNIA
04-CC-580 PM 5.0/7.8, 04-MRN-580 PM 0.0/3.3, EA 04-2J6800

1. INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Richmond-San Rafael Bridge Access Improvement Project in Contra Costa County and Marin County, California. The general location of the project site and its approximate limits are shown in Plate 1, Project Location Map.

This report addresses structural pavement sections, overhead sign structures and corrosion investigation recommendations. The investigation included review of readily available soils and geologic literature pertaining to the site, site reconnaissance, obtaining representative samples and logging soil materials encountered in exploratory borings, laboratory testing of the representative samples, performing engineering analyses, and preparation of this report.

The purpose of this report is to document subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, and to recommend design and construction criteria for the project. This report also establishes a geotechnical baseline to be used in assessing the existence and scope of changed site conditions, if any.

The report is intended for use by the project roadway design engineer, construction personnel, bidders, and contractors for information and reference purposes only and should not be construed directly as project specifications.

2. EXISTING FACILITIES AND PROPOSED IMPROVEMENTS

The Richmond-San Rafael Bridge Access Improvement Project ("project") proposes to convert the existing shoulders on the Richmond-San Rafael ("RSR") Bridge to accommodate bicycle and pedestrian access on the upper bridge deck (westbound), and a new vehicular travel lane on the lower deck (eastbound). Bicycle and pedestrian access on the upper deck of the RSR Bridge would be provided by installing a barrier to separate bicyclists and pedestrians from motorists.

The total length of the project is approximately 6 miles [Contra Costa County post mile (PM) R4.98 to Marin County PM 3.3]. Within the project limits there are six existing structures; San Quentin Undercrossing (Main Street) (Br. No. 27-0070), the RSR Bridge (Br. No. 28-0100),

Western Drive Undercrossing (Stenmark Drive) (Br. No. 28-0141R), Scofield Avenue Undercrossing (Br. No. 28-0140 L/R), Marine Street Undercrossing (Br. No. 28-0139), and the Castro Street Undercrossing (Br. No. 28-0290 L/R/S). All proposed improvements are anticipated to be within existing highway and local street rights-of-way, except as noted below in Project Element 3. The project location is shown in Plate 1 and proposed improvements for all project elements are shown in Plate 2, Site Plan.

The project consists of three major components that are interrelated:

- Element 1: Eastbound I-580 travel lane between Marin County and Contra Costa County.
- Element 2: Bicycle/Pedestrian Path in Contra Costa County.
- Element 3: Bicycle/Pedestrian Path on the RSR Bridge and connections to the RSR Bridge.

Project Element 1 – Eastbound I-580 Third Lane (Including RSR Bridge Pilot Project)

Project Element 1 of the proposed project would construct a new third travel lane by converting the existing shoulder of the eastbound lower deck of the RSR Bridge to a travel lane. The new lane will begin immediately downstream from the Main Street EB off-ramp in Marin County and terminate on the Contra Costa County side of the RSR Bridge, slightly downstream of the Marine Street/East Standard Avenue EB off-ramp in Richmond. The Bridge portion of the third lane on the lower deck will operate during peak hours only (as part of the pilot project). The exact hours of operation of the lane will be outlined in the Project Report. The off-Bridge portion of the third lane will operate 24 hours a day, 7 days per week. Electronic and static signs will be used to operate and manage the lane during the hours of operations and are included in the project description below. The third travel lane on the RSR Bridge is part of a pilot project with Project Element 3, which will run for the duration of four years and is intended to test and evaluate the performance and use of the third travel lane. After four years, the third lane on the RSR Bridge will be evaluated to determine if it is to remain a peak period use lane (PPUL), be converted to a full-time use lane, or return to function as a shoulder. All other constructed components of Project Element 1 would be permanent. The EB I-580 third lane would include the following work elements:

1) Modify roadside post mounted signage on EB I-580 and install new roadside signs.
2) Install new electronic signs on the bridge to communicate to drivers when the third travel lane may be used. Electronic Changeable Message Signs (CMS) will indicate whether the lane is open or closed.

3) Modify striping on the EB Main Street/San Quentin off-ramp to extend the existing Sir Francis Drake auxiliary lane beyond the Main Street interchange. The extension of the auxiliary lane necessitates shifting of the median barrier approximately 10 feet northerly to improve stopping sight distance for mainline eastbound traffic, at the approach to the RSR Bridge.
4) Widen Main Street between the eastbound and westbound ramps to accommodate two 5-foot Class II bike lanes, maintaining the 5-foot sidewalk. A Type 7 (L-shaped) retaining wall will be constructed on the west side (southbound side) of Main Street under I-580 (Retaining Wall No.1). This element will be a permanent feature and is not part of the pilot test.
5) Realign the EB Main Street on-ramp to merge with the proposed travel lane. A Type 7 (L-shaped) retaining wall will be constructed along the left side of the ramp (Retaining Wall No.3). Standard construction methods will be used.
6) Reconstruct the southeast corner of the Main Street/WB off-ramp intersection and the northeast corner of the Main Street/EB on-ramp intersection and construct a new sidewalk on the southeast corner of the Main Street/EB on-ramp. A new Type 7 (L-shaped) wall will be constructed at the foot of the embankment slope at the San Quentin Undercrossing. The wall will curve around the corner behind the northeast Main Street sidewalk, onto the north side of the Main Street on-ramp (Retaining Wall No.2). The southeast sidewalk will be constructed along the east side of Main Street, from the sidewalk constructed by Marin Public Works prior to this project (approximately 25 feet south of the EB on-ramp) where it will conform to the right-of-way line. The sidewalk will continue around the southeast corner of the EB on-ramp intersection and along the south side of the EB on-ramp, where a new Golden Gate Transit bus shelter will be constructed. A new Type 5 retaining wall will be constructed along the south side of the Main Street on-ramp to preserve access to the electrical substation at the Caltrans Maintenance Yard (Retaining Wall No.6).
7) Reconstruct the right shoulder in the eastbound direction to create a travel lane from the RSR Bridge to the Marine Street off-ramp (CC PM R5.43).
8) Remove the existing 362.5-foot retaining wall along EB I-580, immediately downstream of the Scofield Avenue Undercrossing. A new soil nail retaining wall will be constructed approximately 15 feet south of the existing edge of pavement (Retaining Wall No.4). The new wall will improve the stopping sight distance along mainline EB I-580. An additional soil nail retaining wall will be constructed 30 feet east of Retaining Wall No. 4, providing additional shoulder and lane width (Retaining Wall No. 7). The new wall will require the removal of 85 feet of the existing retaining wall along EB I-580. Additional shoulder and lane width requires

that the 8-foot by 7-foot tunnel utility running under EB I-580 be extended by 17 feet. This includes 8 feet of tunnel extension and 9 feet of access structure. This utility tunnel is located between Retaining Walls No. 4 and No. 7.

9) Reconfigure the Marine Street off-ramp exit nose to accommodate continuation of the eastbound travel lane and widen the inside of the existing off-ramp to provide additional storage for vehicle queuing.
10) Reconfigure East Standard Avenue between Marine Street and Castro Street to change one of the two westbound lanes to an eastbound lane by reconstructing the existing median barrier approximately 12 feet northerly. A permanent, Type 60 series concrete barrier will also be installed to separate the bicycle and pedestrian path from vehicular traffic. The barrier will extend along the south side of East Standard Avenue between Marine Street and Castro Street to the existing bicycle and pedestrian paths, linking the bicycle facilities on Tewksbury Avenue and Marine Street. Minor sliver widening will be required along East Standard Avenue to accommodate the full street configuration including shoulders.
11) Modify traffic signal and intersection operations, including upgrading, replacing, or adding new controller cabinets, traffic signal posts, and other intersection control equipment at three locations: EB I-580/Marine Street off-ramp, EB East Standard Avenue/Castro Street and WB I-580/Castro Street off-ramp. It is anticipated that any controller cabinets or traffic signal poles would be installed within the existing operational transportation right-of-way.
12) Install standard loop traffic monitoring stations in the pavement of the upper and lower bridge decks.
13) Mount CCTV cameras with eastbound and westbound views along the Bridge. CCTV cameras with eastbound views will begin on I-580 East in Marin County and end at the Marine Street interchange in Contra Costa County. CCTV cameras with westbound views will begin on the westbound RSR Bridge, westerly of the Toll Plaza, and end near the Main Street off-ramp.
14) Install ramp metering at two on-ramp locations: the eastbound Main Street single-lane on-ramp and the eastbound Standard Avenue two-lane on-ramp.

All improvements for Project Element 1 will be within existing local and state right-of-way.

Element 2 – Bicycle/Pedestrian Path in Contra Costa County

The proposed Class I bi-directional bicycle and pedestrian path in Contra Costa County would be constructed along the north side of westbound (WB) I-580 from the Marine Street interchange in

Contra Costa County to Stenmark Drive (formerly Western Drive) and the Toll Plaza where it would then connect to Project Element 3. The Class I bi-directional bicycle and pedestrian path would be implemented along the existing WB I-580 and Stenmark Drive shoulders and would replace the existing one-way Class III bicycle lanes on both EB and WB I-580 between Marine Street and the Toll Plaza. The proposed bi-directional bicycle and pedestrian path would be separated from vehicle traffic by a continuous concrete barrier. Implementation of the path would include the following work elements:

1) Install a Class I bi-directional path for bicycles and pedestrians separated from automobile traffic by a permanent concrete barrier. The path will begin at the existing bike lane and sidewalk on the Marine Street EB off-ramp and continue parallel with WB I-580 to the Stenmark Drive off-ramp.
2) Widen the north side of the existing Stenmark Drive off-ramp to provide an inside shoulder, a vehicle lane, an outside shoulder, a concrete barrier, and a 12-foot bi-directional bicycle/pedestrian path. A new retaining wall will be constructed along the north side of the bi-directional bicycle and pedestrian path (Retaining Wall No.5). The new wall will be set back 14 feet from the existing edge of the travel way. Bicycle/pedestrian path improvements along Stenmark Drive between stations 1 and 3 will be no more than 3 feet deep. Standard construction methods will be used.
3) A gabion wall 8.5 feet high and approximately 54 feet long will be installed on the slope between WB 580 and the curve of the bicycle/pedestrian path, just west of Marine Street. The gabion wall will be embedded 2 feet into the ground, with a 12 degree batter.
4) Install a crosswalk at Stenmark Drive to continue the Class I bi-directional bicycle and pedestrian path further west on the south side of Stenmark Drive, where it will connect to the existing bicycle trail and to the Point Molate path being constructed by East Bay Regional Parks (separate project).
5) Replace existing railings on the Scofield Avenue Undercrossing with a Transportation Security Administration (TSA) approved visual screen (similar to a chain-link fence) or wall to physically and visually block access to the adjacent Chevron fuel pipelines. The visual screen or wall will be designed to prevent the general public from dropping objects onto Chevron's petroleum facilities below.

6) PG&E will relocate utility poles and overhead wires along Stenmark Drive to a location of their choice, within local and state right-of-way (may be underground). Currently the poles are within the footprint of the multi-use path.
7) Project Element 2 is expected to require installation of new roadside signs and relocation or removal of existing signs.

All improvements for Project Element 2 will be within existing local and state right-of-way.

Element 3 – Bicycle/Pedestrian Path on RSR Bridge and Related Connections to RSR Bridge (Pilot Project)

Project Element 3 includes the continuation of the proposed Class I bi-directional bicycle and pedestrian path from the Stenmark Drive off-ramp to East Francisco Boulevard at Grange Avenue. The portion of the bi-directional bicycle and pedestrian path from Stenmark Drive to the Main Street off-ramp would be part of the pilot project that would run for four years, intended to evaluate the performance and use of a bicycle and pedestrian path on the RSR Bridge. After four years, the bi-directional bicycle and pedestrian path on the RSR Bridge may be made permanent, or may return to functioning as a shoulder. All other portions of the bike path would be permanent. Bicycle and pedestrian access improvements are also included in this project element to improve multimodal circulation and connections to the RSR Bridge. Implementation of Project Element 3 would include the following work elements:

1) Install a 10-foot wide Class I bi-directional bicycle and pedestrian path from Stenmark Drive west of the Toll Plaza Maintenance Buildings on an easement through Chevron property, connecting to the bicycle and pedestrian path on the RSR Bridge.
2) Install a 10-foot wide Class I bi-directional bicycle/pedestrian path on the westbound upper deck of the RSR Bridge, separated from motor vehicle traffic by a 42-inch moveable concrete barrier. The 18-inch wide moveable barrier would start near the end of the maintenance facility and continue across the RSR Bridge to the Marin County side of WB I-580. The barrier would be movable to provide emergency access, access for RSR Bridge maintenance, and other safety considerations.
3) Raise the outside bridge railing to approximately 48 inches above the utility tray (approximately 60 inches above the RSR Bridge deck) to provide additional fall protection for bicyclists and pedestrians. Install necessary signage to properly guide pedestrian and bicycle traffic onto and off the bike path.

4) Realign the Main Street off-ramp to continue the Class I bicycle/pedestrian path onto Francisco Boulevard between Main Street and Grange Avenue. From Grange Avenue, bicyclists and pedestrians can connect to other existing off-street and on-street routes, including the Class I San Francisco Bay Trail.
5) Install bike detection systems on the westbound upper deck of the Bridge. The bike detection system for the bicycle/pedestrian path on the Bridge will be located at the Marin side approach to the Bridge at the East Francisco Boulevard off-ramp. The bike detection system for the bicycle/pedestrian path in Contra Costa County will be located near the Toll Plaza.
6) Project Element 3 is expected to require installation of new roadside signage, and relocation or removal of existing signs.

With the exception of the segment of the bicycle/pedestrian path adjacent to the maintenance facility located on an easement to be provided by Chevron, all improvements for Project Element 3 will be located within local and state right-of-way.

3. PERTINENT REPORTS AND INVESTIGATION

Previous investigations, reports and published maps that include the project corridor vary in focus and scale.

California Department of Transportation (Caltrans) Logs of Test Borings (LOTBs)

a) Caltrans, LOTBs for Construction on I-580 in Richmond from Marine Street Undercrossing to 0.1 Mile South of Scofield Avenue Undercrossing - Retaining Wall at P.M. 5.6 (Br. No. 28-302M).

b) Caltrans, LOTBs for Construction on I-580 in Richmond from 0.1 Mile East of Railroad Avenue Overhead to Scofield Avenue Undercrossing – Railroad Avenue Overhead (Br. No. 28-56S).

c) Caltrans, LOTBs for Construction on I-580 in Richmond from 0.1 Mile East of Railroad Avenue Overhead to Scofield Avenue Undercrossing – Castro Street U.C (Br. No. 28-290R&S).

d) Caltrans, LOTBs for Construction on I-580 in Richmond from 0.1 Mile East of Railroad Avenue Overhead to Scofield Avenue Undercrossing – Marine Street Undercrossing (Br. No. 28-139).

e) Caltrans, LOTBs for Construction on I-580 in Richmond from 0.1 Mile East of Railroad Avenue Overhead to Scofield Avenue Undercrossing – Retaining Wall S1-27 (Br. No. 28-RW27).

f) Caltrans, LOTBs for Construction on I-580 in Richmond from 0.1 Mile East of Railroad Avenue Overhead to Scofield Avenue Undercrossing – Retaining Wall SLI 95 (Br. No. 28-RW95).

g) Caltrans, LOTBs for Construction on I-580 in Richmond from 0.1 Mile East of Railroad Avenue Overhead to Scofield Avenue Undercrossing – Retaining Wall BI-15 (Br. No. 28-RW15).

h) Caltrans, LOTBs for Construction on I-580 in Richmond from 0.1 Mile East of Railroad Avenue Overhead to Scofield Avenue Undercrossing – Retaining Wall DI-14 (Br. No. 28-RW14).

i) Caltrans, LOTBs for Scofield Avenue Undercrossing (Widen) (Br. No. 28-140R/L).

j) Caltrans, LOTB for Marine Street Undercrossing, (Br. No. 28-139).

Copies of the as-built LOTBs are included in Appendix A.

4. PHYSICAL SETTING

4.1 Climate

The climate in the San Quintin (Sir Francis Drake EB On-Ramp) to Richmond (Marine Street) area is characterized by mild weather conditions. This consists of mild winters, warm summers and small daily and seasonal temperature ranges. Strong winds blow through the straight crossed by the San Rafael Bridge. Based on the statistical data from “Western Regional Climate Center”, extreme temperature ranges from average minimum temperature of approximately 43°F in January to average maximum temperature of approximately 74°F in September and the average total annual precipitation is around 23 inches in the area. Most of

the rainfall is recorded in January with the average total monthly precipitation of approximately 1.92 inches. July is the month with the least rainfall precipitation of 0.04 inches. Freezing weather may be anticipated. However, it is generally not necessary to design for freeze-thaw conditions.

4.2 Topography and drainage

The terrain elevation of I-580 within the project corridor varies from about sea level to approximately 80 feet above sea level along the alignment where it crosses the flanks of hills. The elevation (bathymetry) of the floor of San Francisco Bay beneath the Richmond-San Rafael Bridge varies from sea level at the west and east shorelines to several tens of meters in the relatively narrow shipping channel located under the eastern portion of the bridge. Run-off water generally drains toward the local storm drainage system which flows into San Francisco Bay.

4.3 Man-Made and Natural Features of Engineering and Construction Significance

Within the project limits there are six existing structures; Main Street/San Quentin Undercrossing, the RSR Bridge, Western Drive Undercrossing, Scofield Avenue Undercrossing, Marine Street Undercrossing, and the Castro Street Undercrossing.

4.4 Regional Geology and Seismicity

The regional geologic framework of the San Francisco Bay Area can be understood through the theory of plate tectonics. Earth's mantle is composed of several large plates that move very slowly (inches per year) relative to each other. The San Andreas Fault System forms the junction of two such plates (the "North American Plate" to the east and the "Pacific Plate" to the west). The "North American Plate" is moving northwest relative to the "Pacific Plate". One result of the relative plate movements is the regional rock deformation that is expressed in the general northwest trend of faults, uplifted ridges, and eroded valleys in the San Francisco Bay Area. Another result of the plate movements is the regional seismicity (earthquakes originating on active faults). Another result of on-going plate movements is slow creep and sudden surface fault rupture where active faults intersect the ground surface.

The project corridor is located within the Coast Ranges Geomorphic Province of California. The bedrock materials that underlie the project corridor include Cretaceous-age metamorphosed

marine sedimentary and volcanic rock units within the Franciscan Complex. The Franciscan rocks were brought together by tectonic subduction which subjected them to great pressure and elevated temperatures. The internal deformation of these rock units was a direct result of the forces that subducted them. The northwest/southeast structural trend of fractures in the bedrock units beneath the project corridor is consistent with the regional structural grain (primarily a northwest/southeast strike) of the Coast Ranges.

During the last ice age, the basin that is now filled by the San Francisco Bay was a large linear valley with small hills, similar to most of the valleys of the Coast Ranges. The rivers of the Central Valley ran out to sea through a canyon that is now the Golden Gate. As the great ice sheets melted, sea level rose 400 feet over the past 5,000 years and valley filled with a mixture of fresh and salt water to form the bay. The small hills became islands and sediments eroded from the surrounding hills were deposited on the floor and along the shores of the bay. One such island, Red Rock Island is located 1430 feet south of the Richmond - San Rafael Bridge. It rises out of the bay to a height of 151 feet. It is surrounded by some of the deepest water in the North San Francisco Bay –up to 60 feet deep.

The western portion of the project is located on Franciscan mélange and artificial fill that was placed upon Bay Mud that accumulated along the shores of San Francisco Bay. The eastern portion of the project is located on Franciscan Sandstone as well as historic artificial fill that was placed upon young alluvial sediments and Bay Mud that were previously deposited along the shore of San Francisco Bay.

5. EXPLORATION

5.1 Drilling and Sampling

Based on the plans and discussions with the design team, 16 borings were drilled at selected locations to depths ranging from 5 feet to 84 feet below the existing ground surface for the proposed improvement. The deeper borings were drilled to about 13.5 feet to 84 feet depth for the proposed retaining walls, and the new overhead sign structure. The shorter borings were drilled to about 5 foot depth for R-value tests samples to evaluate the subgrade conditions along the proposed I-580 widening and bike path areas.

The test borings were advanced with a truck-mounted drill rig with energy ratio of 77% and 84%, by Geo-Ex Subsurface Exploration of Dixon, CA. The bore holes were advanced by hollow stem and solid stem auger drilling method. Selected samples were obtained from 2.5-inch I.D. (Modified California, MC) and 1.4-inch I.D. (Standard Penetration Test, SPT) samplers at various depths. The samplers were driven into subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts are presented on the Log of Test Boring (LOTB) in Appendix A. When correlating standard penetration data, the blow counts for the Modified California Sampler may be converted to equivalent SPT blow counts by multiplying a conversion factor of 0.65. Bulk samples were collected from the soil cuttings.

The samples were sealed and transported to our laboratory for further evaluation and testing. The field investigation was conducted under the supervision of our field engineer who logged the test borings and prepared the samples for subsequent laboratory testing and evaluation. The overall boring program is summarized in the following tables.

TABLE 1 – EXPLORATION PROGRAM

Boring No.	Approx. Location(*)	Approx. Offset(*)	Elevation (ft)	Approx. Boring Depth (ft)
A-15-580-001	"MAINO2" Line 243+00±	30± ft Lt.	32.0±	41.5
A-15-580-002	"MAINO2" Line 243+80±	64± ft Rt.	32.0±	5
A-15-580-003	"MAINO3" Line 244+30±	8± ft Lt.	13.0±	5
A-15-580-004	"CC-E" Line 1003+00±	57± ft Rt.	38.0±	5
A-15-580-005	"CC-E" Line 1013+00±	43± ft Rt.	48.0±	5
A-15-580-006	"CC-E" Line 1021+95±	36± ft Rt.	75.0±	5
A-15-580-007	"CC-M" Line 1030+55±	69± ft Rt.	70.0±	5
A-15-580-008	"MRNEO1" Line 1040+65±	12± ft Lt.	43.0±	5
A-15-580-009	"BP1" Line 1004+60±	54± ft Lt.	38.0±	5
A-15-580-010	"BP1" Line 1006+25±	4± ft Lt.	50.0±	13.5
A-15-580-011	"BP1" Line 1009+35±	5± ft Lt.	40.0±	16.5
A-16-580-012	"CC-M" Line 1023+15±	158± ft Rt.	160.0±	84.0
A-15-580-013	"MAINO4" Line 4+30±	48± ft Lt.	15.0±	25.0
A-15-580-014	"MRN" Line 231+55±	43± ft Rt.	33.0±	25.0
A-15-580-015	"MAINO2" Line 243+16±	44± ft Rt.	18.0±	45.8
A-15-580-016	"MAINO2" Line 245+08±	29± ft Rt.	24.0±	50.3

* The boring locations and elevations were not surveyed and are approximate based on the plans provided by the designer.

The approximate locations of these explorations are shown on the attached Site Plans, Plates 2A

through 2C.

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

5.2 Geologic Mapping

The site is primarily underlain by artificial fill and sandstone. The subject was considered and was determined to be not significant for the project.

5.3 Geophysical Studies

NORCAL Geophysical Consultants, Inc, performed seismic refraction investigation at the top of the rock slope located next to the proposed Retaining Wall No. 4 location. Since the slope is steep, the investigation could not be performed along the slope and it was performed on top of the slope at the Chevron property. NORCAL investigation report is attached in Appendix D.

5.4 Instrumentation

The subject was considered and was determined to be not applicable to the project.

5.5 Exploration Notes

The exploratory borings mainly encountered undivided surficial deposits. Drilling was conducted mainly by using hollow stem augers and solid stem augers for this project. The drilling operation was considered hard because of shallow bed rock.

6. GEOTECHNICAL TESTING

6.1 In-Situ Testing

In-situ testing consists of recording blow counts during sampling (using both Modified California sampler and Standard Penetration Test sampler). Based on our previous experience, when correlating standard penetration data in similar soils, the blow counts for the Modified

California Sampler may be converted equivalent SPT blow counts by multiplying a conversion factor of 0.65. Based on the average values of the SPT-N values for the soil materials encountered in the field exploration, majority of the subsurface soils are classified generally as medium dense to dense clayey gravel and poorly graded gravel with layers of medium stiff lean clay and weathered sandstone. The in-situ test results are presented on the LOTB attached in Appendix A.

6.2 Laboratory Testing

Laboratory tests were performed on selected samples in the laboratory to evaluate the physical and engineering properties of the subsoils. The tests performed for the study include the following: Laboratory determination of Moisture (California Test Method 226), Atterberg Limits (California Test Method 204), Grain Size Analysis (California Test Method 202), Unconfined Compression Test (California Test Method 221), Compressive Strength Test for Rock Core Samples (ASTM C 42), Resistivity and pH Test (California Test Method 643), Sulfate Content (California Test Method 417), Chloride Content (California Test Method 422), and R-value Tests (California Test Method 301). The laboratory test results are attached in Appendix B.

7. GEOTECHNICAL CONDITIONS

7.1 Site Geology

General geologic features pertaining to the site were evaluated by reference to the following published maps:

a) Sims, J.D.; Fox Jr., K.F.; Bartow, J.A.; and Helley, E.J.; 1973; Preliminary Geologic Map of the Solano County and Parts of Napa, Contra Costa, Marin, and Yolo Counties, California; U.S. Geological Survey Miscellaneous Field Studies Map 484 (3 sheets, scale 1:62,500).

b) Nilsen, T.H.; 1975; Preliminary photo-interpretation maps of landslide and other surficial deposits of 56 7.5-minute quadrangles, Alameda, Contra Costa, and Santa Clara Counties, California (with parts of adjoining counties on several maps by John A. Bartow, Virgil A. Frizzell, Jr., and John D. Sims): U.S. Geological Survey Open-File Report 75-277, scale 1:24,000.

c) Graymer, R.W.; Jones, D.L.; and Brabb, E.E.; 1994; Preliminary Geologic Map emphasizing bedrock formations in Contra Costa County, California; U. S. Geological Survey Open-File Report 94-622.

d) Blake, M.C.; Graymer, R.W.; and Jones, D.L.; 2000; Geologic Map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California; U.S. Geological Survey Miscellaneous Field Studies MF 2337, Online Version 1.0 (Digital Database by Soule, A., and Graymer, R.W.) http://pubs.usgs.gov/mf/2000/2337/

e) Graymer, R.W.; Moring, B.C.; Saucedo, G.J.; Wentworth, C.M.; Brabb, E.E.; and Knudsen, K.L.; 2006; Geologic Map of the San Francisco Bay Region; U.S. Geological Survey Scientific Investigations Map 2918. http://pubs.usgs.gov/sim/2006/2918/

Based on these published maps, the project site is mainly underlain by:

- Historic artificial fill over Bay Mud (“Qmf”),
- Holocene to Pleistocene-age Bay Mud (Qhbm”)
- Late Pleistocene-age Alluvial Deposits (“Qpa”),
- Late Cretaceous-age Franciscan sandstone (“Kfs”),
- Late Cretaceous to Jurrasic-age Franciscan mélange (“fsr”)
- Cretaceous-age Franciscan chert (“KJfch”), and
- Jurrasic-age Franciscan greenstone (“Jfgs”).

Localized exposures of Franciscan chert (“Jfch”) and greenstone (“Jfgs”) exist on the small island (Red Rock) located south of the San Rafael Bridge. Borings made for the retrofitting of foundations for the Richmond-San Rafael Bridge encountered various Franciscan rock types beneath various thicknesses of Bay Mud on the floor of San Francisco Bay.

Geology mapped along the project corridor is shown on Plates 4A and 4B.

The following are the general descriptions of the geologic units that have been mapped as underlying portions of the project corridor:

Qmf - **Artificial fill over Marine and Marsh Deposits** (Holocene, historic): Human-placed mixture of varying character, consisting of clay, silt, sand, rock fragments, organic material, and (or) man-made debris that may be engineered or non-engineered.

Qbm – **Bay Mud** (Holocene): layers of silt, sand, and much organic material. Soft-to-very soft where wet. Deposited in marine and salt-water marshes.

Qpa - **Alluvium**, undivided (Pleistocene): Alluvium deposited on fans, terraces, or in basins; composed of sand, gravel, silt, and clay that are poorly sorted.

Kfs – **Franciscan Sandstone and Shale** (Cretaceous):Sandstone and interbedded shale, with minor conglomerate; crops out in alternating sequence of largely medium-thick to very thick sandstone beds with generally minor interbedded shale and predominantly shale with interbedded thin to medium-thick sandstone beds; rock is locally severely sheared or brecciated but lacks tectonic inclusions of other rock types such as greenstone and chert which are common in unit fsr; thicker sandstone beds are medium- to coarse-grained arkosic wacke containing 2 to 25 percent detrital potassium feldspar, but commonly 2 to 5 percent, whereas thinner sandstone beds are fine grained, quartz rich wacke, and contain 0 to 2 percent detrital potassium feldspar; sandstone is light gray where fresh, weathering to buff colors, and shale is commonly dark gray; laumontite veins, calcite veins, and microscopic secondary prehnite and (or) pumpellyite are common in sandstone. Rocks of this unit typically form resistant topography. Bedding may be indistinct to prominent. The cut slope along the north side of the west-bound off-ramp located north of the toll plaza at the east end of the Richmond-San Rafael Bridge exposes bedding planes in Franciscan sandstone that dip steeply (60 to 70 degrees below horizontal) toward the southwest as depicted on the published map.

fsr – **Franciscan mélange**: A tectonic mixture of variably sheared shale and sandstone containing (1) hard tectonic inclusions largely of greenstone, chert, graywacke, and their metamorphosed equivalents, plus exotic high grade metamorphic rocks and serpentinite and (2) variably resistant masses of graywacke, greenstone, and serpentinite up to several miles in longest

dimension, and including minor discrete masses of limestone too small to be shown. Blocks and resistant masses have survived the extensive shearing evident in the mélange's matrix, and range in abundance from less than 1 to 50 percent or more of the rock mass. The degree of shearing in the unit ranges from gouge to unsheared rock, with resistant masses relatively unsheared and matrix sheared. Severely sheared shale is abundant in areas where blocks are abundant. Fresh, relatively unsheared rock is hard, the larger resistant masses are pervasively fractured, and blocks are commonly tough and relatively unfractured. Sandstone is graywacke, grayish green where fresh, weathering to brown, commonly medium to coarse grained, containing abundant angular lithic grains and no detrital potassium feldspar, except rarely as much as 5 percent. Graywacke is locally veined with quartz and carbonate, and usually contains microscopic secondary pumpellyite. Topography of coherent masses resembles that of unit Kfs, whereas highly sheared matrix typically yields subdued, gently-rounded topography.

KJfch - **Chert** (Cretaceous and Jurassic): Chert with shale inter-beds. Chert is thin bedded, closely fractured, and parts along bedding planes; contains tests of radiolaria that range in age from early Jurassic (Tithonian) to Middle Cretaceous (Cenomanian) age (Murchey, 1984); crops out as irregularly shaped bodies as long as 4.5 km. In southern Marin County and parts of San Francisco, chert up to 100 m thick overlies pillow lava and is overlain conformably by a few feet of fine-grained, black shale that grades into overlying greywacke and shale. Vertical beds of red chert are exposed in the eroded flanks of Red Rock Island located south of the Richmond – San Rafael Bridge.

Jfg - **Greenstone** (Jurassic): Consists of pillow lava and less abundant tuff, breccia, and intrusive basalt, diabase, and rare gabbro. Local lenses of thin-bedded radiolarian chert are as thick as 30 m. Fresh rock is hard, relatively unsheared, and ranges from essentially structureless to strongly pillowed; deeply weathered in places;

7.2 Lithology

The site consists of artificial fill, sandstone and mélange rocks. The subject was considered and was determined to be not applicable for the project. Detailed description of subsoil conditions are presented in Section 7.3.

7.2.1 Structure

The site consists of roadway fills and rocks. The subject was considered and was determined to be not applicable for the project.

7.2.2 Existing Slope Stability

The slopes within the project vicinity consist of man-made embankment slopes at the existing I-580 overcrossing abutments at Main Street in Marin County and natural rock slopes in Contra Costa County within the project limit. These existing slopes, typically having gradients of 1.5H:1V or flatter, are covered with vegetations, and generally appear to be in good condition.

7.3 Subsurface Soil Conditions

The subsurface soil conditions are summarized in the following paragraphs.

– *Sign Location OS2-1, approx. "MRN" Line Sta. 227+00*
  Based on the boring data (Boring A-15-580-014), the subsurface soil conditions at the sign location generally consist of moderately to slightly weathered sandstone to the maximum explored depth of 25 feet below the existing grade (approximate elevation of 8.0 feet). The sign location was moved by approximately 360 feet towards west after the boring was drilled. Based on the geology map, we anticipate similar soil/rock profile at the new sign location. However, it should be noted, that these materials could vary in strength and can impact the drilling operation, especially if harder rock conditions are encountered. This may result in a potential claim. It may be prudent to require an exploration ahead of the construction or perform another boring at the new location during the design phase.

– *Retaining Wall No. 1 (RW 1), approx. "MAINO4" Line Sta. 4+40.66 to 6+11.11*
  Based on the boring data of Boring A-15-580-013, the subsurface soil consists of moderately to slightly weathered sandstone to the maximum explored depth of 25 feet

below the existing grade (approximate elevation of -10 feet). Based on the boring data of Boring A-15-580-001, the subsurface soil consists of medium dense to dense clayey gravel and poorly graded gravel with clay and sand interbedded with medium stiff sandy lean clay with gravel to the maximum explored depth of 41.5 feet below the existing grade (approximate elevation of -9.5 feet). As shown on Site Plan, Boring A-15-580-013 was drilled at the southern corner of Main Street off-ramp and Main Street, which is at the beginning of the proposed retaining wall. Boring A-15-580-001 was drilled at the eastbound I-580, which is southeast of Main Street and at the top of the embankment. Based on the Geologic Map (Plate 3A), RW 1 is located in the boarder of artificial fill and bedrock geologic units. Therefore, based on the boring logs and geologic map, artificial fill and bedrock are anticipated at RW 1 location.

– *Retaining Wall No. 2 (RW 2), approx. "MAINO2" Line Sta. 241+68.54 to 242+09.74 & Retaining Wall No. 3 (RW 3), approx. "MAINO2" Line Sta. 242+64 to 243+96.91*
  Based on the boring data of Boring A-15-580-001, the subsurface soil consists of medium dense to dense clayey gravel and poorly graded gravel with clay and sand interbedded with medium stiff sandy lean clay with gravel to the maximum explored depth of 41.5 feet below the existing grade (approximate elevation of -9.5 feet). RW 3 is located in existing roadway embankment. As shown on Site Plan, Boring A-15-580-001 was drilled at the eastbound I-580, at the top of the embankment.

– *Retaining Wall No. 4 (RW 4), approx. "CC-E" Line Sta. 1021+22.90 to "CC-M" Line Sta. 1026+71.51 & Retaining Wall No. 7 (RW 7), approx. "CC-M" Line Sta. 1026+93.56 to 1028+25.08*
  Based on the as-built boring data of Retaining Wall No. 317, the subsurface soil consists of fractured greywacke was encountered to the maximum explored depth of 3 feet and 9 feet below the existing grade (approximate elevation 67 feet and 63 feet). Based on the as-built boring data of Retaining Wall at PM 5.6, the upper part of the material is primarily exhibiting the nature of sandy soil. Based on the as-built plans, we understand that as-built borings at PM 5.6 were drilled on top of the slope close to Marine Street off-ramp.

  One boring, R-15-580-012, was drilled during our current investigation at the top of the hill inside Chevron property. Based on the boring data, the subsurface soil consist of lean clay with some weathered claystone/sandstone severely fragile to the depth of 8 feet

(approximate elevation 152 feet) underlain by highly weathered sandstone to slightly weathered sandstone to the depth of 84 feet (approximate elevation 76 feet).

The geophysical investigation performed on top of the office hill confirmed the rocky material at the location.

- *Retaining Wall No. 5 (RW 5), approx. "WST" Line Sta. 1005+60.21 to 1010+60.96*
  Based on the boring data of Boring A-15-580-010 and A-15-580-011, the subsurface soil consists of intensely to slightly weathered sandstone to the maximum explored depth of 13.5 feet and 16.5 feet below the existing grade (approximate elevation of 36.5 feet and 24.8 feet).

- *Retaining Wall No. 6 (RW 6), approx. "MAIN02" Line Sta. 242+04.21 to 245+42.00*
  Based on the boring data of Boring A-15-580-015 and A-15-580-016, the subsurface soil consists of loose to very dense silty sand with gravel, silty gravel with sand and poorly graded gravel with silt and sand interbedded with soft lean clay and fat clay to the depth of 23 to 40 feet below the existing grade (approximate elevation of -5 to -16 feet). Weathered sandstone was encountered below the sand and gravel layers.

- *Retaining Wall No. 8 (RW 8) on the slope between WB 580 and the curve of the bicycle/pedestrian path, just west of Marine Street*
  Based on the as-built boring data of Marine Street undercrossing, the subsurface soil, below undercrossing elevation, consists of medium stiff clays at shallow depths overlying stiff to hard clay and silty clay, and compact to dense silt and clayey silt with scattered weathered gravel and coarse sand up to the elevation between -10 feet and -28 feet, underlain by weathered sandstone and clayey shale. As noted above these borings were drilled at the underpass elevation (boring elevations are between 11 feet and 20 feet). The proposed gabion earth wall will be on the abutment embankment. The embankment should be a compacted fill embankment of Route 580 construction. Therefore compacted fill properties can be assumed for embankment fill for the gabion wall design.

- *R-value Sample Locations*
  Based on the shallow R-value boring data, the subsurface soil consists of sandy lean clay and sandy lean clay with gravel to the maximum explored depth of 5 feet below the existing grade.

Detailed descriptions of the materials encountered in the exploratory borings are presented in the LOTB in Appendix A "Log of Test Borings". It should be noted that these descriptions and related information depict subsurface conditions only at the locations indicated and on the particular date noted on the LOTB. Because of the variability from place to place within soil/rock in general, subsurface soil 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 due to field limitations. Also, the passage of time may result in a change in the soil conditions at the locations due to environmental changes.

7.4 Water

7.4.1 Surface Water

The terrain along Interstate 580 slopes downward towards the bay. The surface water/drainage generally follows the ground topography and is discharged to the local drainage systems and ocean.

7.4.1.1 Scour

The subject was considered and was determined to be not applicable for the proposed project since no open water body passes through the site.

7.4.1.2 Erosion

The subject was considered and was determined to be not applicable for the project.

7.4.2 Groundwater

Based on the as-built Log of Test Borings, the groundwater was encountered between approximate elevations of +90.8 feet and +93.4 feet (As-built Borings of Retaining Wall at P.M. 5.6) and approximate depths between 7 feet and 18 feet below the existing ground during drilling in Contra Costa County. That appeared to be a location on the hill side. Near the bay, the natural groundwater level is expected to be close to Elev. ~0.

Groundwater was encountered at elevation 7.0, -1 and 6 feet in Borings A-15-580-001, A-15-580-015 and A-15-580-016, respectively, in Marin County. Near the bay, the natural groundwater level is expected to be close to Elev. ~0.

The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, variations in yearly rainfall, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other environmental factors that may not be present at the time of the investigation.

Note that the explorations were performed during the worst drought period that California has experienced. Ground water elevation could significantly vary in the event of a 'normal' rainfall period or following an El Nino period. Also, since groundwater may take time to recharge or react to such changes, the potential fluctuations due the extreme conditions as noted above may or may not be observed during construction.

7.5 Project Site Seismicity

7.5.1 Ground Motions

The project site is located in a seismically active part of northern California. Many faults existing in northern California are capable of producing earthquakes and may cause strong ground shaking at the site.

Maximum moment magnitudes (MMax) of some of the closest faults in the area are based on the Caltrans ARS Online (v2.3.06) Report. These maximum moment magnitudes represent the largest earthquake a fault is capable of generating and is related to the seismic moment. The earthquake data of the active faults in the project vicinity are summarized below.

TABLE 2 – ARS DATA

Fault (Caltrans Fault ID)	Maximum Credible Earthquake Magnitude (Mmax)	Fault Type	Approximate Rupture Distance & Horizontal Distance to the Surface Projection from Site, Rrup/Rx (km)
Hayward (North) (123)	7.3	Strike-Slip	7.9/0.6
San Andreas (North Coast) 2011 CFM (80)	8	Strike-Slip	19.2/19.2
San Andreas (Peninsula) 2011 CFM (134)	8	Strike-Slip	19.4/18.3
San Gregorio fault (San Gregorio section) (127)	7.4	Strike-Slip	21.6/21.6
Rodgers Creek (103)	7.3	Strike-Slip	25.9/18.6
Hayward (South) (137)	7.3	Strike-Slip	34.4/7.7
Green Valley 2011 CFM	6.8	Strike-Slip	32.7/29.2
Calaveras (No) 2011 CFM (130)	6.9	Strike-Slip	35.7/21.8

* Distances are based on Caltrans ARS online and only for ground motion estimation purpose. Not recommended to locate faults for site specific studies.

7.5.2 Ground Rupture

Since no active faults pass through the site, the potential for fault rupture is relatively low.

8. GEOTECHNICAL ANALYSIS AND DESIGN

8.1 Dynamic Analysis

8.1.1 Parameter Selection

The recommended response spectrum was determined based on new Caltrans ARS Online (Ver2.3.06) for the development of response spectra for design. The development of the design ARS curve is based on several input parameters, including site location (longitude/latitude), average shear wave velocity for the top 30 m/100 feet (V_{S30m}), and other site parameters, such as fault characteristics, site-to-fault distances. The attached Fault Map (Plate 3) presents the locations of the fault systems relative to the project site.

The design methods incorporate both deterministic and probabilistic seismic hazards to produce the Design Response Spectrum. According to the design manual, we have also compared with the probabilistic response spectrum from 2008 USGS Deaggregation Hazard (beta) web site for the 5% in 50 years probability of exceedance (or 975 year return period) at periods of 0, 0.3, 1 and 3 seconds.

Average shear wave velocity for the top 30m (100 feet) at the site is estimated by using established correlations and the procedure provided in the recent "Methodology for Developing Design Response Spectrum for use in Seismic Design Recommendations" (November 2012) publication by Caltrans.

<u>Retaining Wall Nos. 1, 2, 3 & 6 (Located at northwest of Main St Undercrossing, East corner of the intersection of Main St and Eastbound on-ramp and Main St Eastbound on-ramp)</u>

Borings A-15-580-001, A-15-580-013, A-15-580-015 & A-15-580-016 data were used to calculate average shear wave velocities.

- Site Location: 37.9434ºN/ 122.4805ºW

- Estimated V_{S30m} = 450 m/s
- Anticipated Peak Ground Acceleration = 0.628g
- The recommended ARS curve is governed by Caltrans online probabilistic ARS.

The recommended design curve is presented on Plate 5A & 5B.

Retaining Wall Nos. 4, 5 & 7 (located East of Scofield Ave UC and at Western Ave Off-Ramp)

Borings A-15-580-010 & A-15-580-011 and as-built boring data were used to calculate average shear wave velocities.

- Site Location: 37.9321ºN/ 122.3991ºW
- Estimated V_{S30m} = 510 m/s
- Anticipated Peak Ground Acceleration = 0.677g
- The recommended ARS curve is governed by Caltrans online probabilistic ARS.

The recommended design curve is presented on Plate 6A & 6B.

Retaining Wall No. 8 (located at Marine St Undercrossing)

As-built boring data were used to calculate average shear wave velocities.

- Site Location: 37.9315ºN/ 122.3917ºW
- Estimated V_{S30m} = 275 m/s
- Anticipated Peak Ground Acceleration = 0.698g
- The recommended ARS curve is governed by Caltrans online probabilistic ARS.

The recommended design curve is presented on Plates 7A & 7B.

8.1.2 Liquefaction Potential

Liquefaction is a phenomenon in which saturated soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged, cohesionless sands and silts of low relative density are the type of soils which usually are susceptible to liquefaction - the susceptibility

increases with decreasing relative density (reflected by the number of blows to drive a sampler), and decreasing fines content. Accepted procedures for the assessment for liquefaction potential for cohesionless soils have evolved over the years through research and field observations (Youd et al, 2001). As indicated by recent advances in soil liquefaction engineering (Bray, 2006), for soils with sufficient fines content so as to separate the coarser particles and control behavior, liquefaction appears to occur in soils where these fines are either non-plastic or are low plasticity silts and/or silty clays (PI<12%, and LL<37%), and with high water content relative to their liquid limit (w> 0.85LL).

We have evaluated the liquefaction potential along the project limit based on the boring data. The detail discussions of each segment are summarized in the following paragraphs.

– *Sign Location OS2-1, approx. "MRN" Line Sta. 227+00*
  Based on the Boring data of A-14-580-001, moderately to slightly weathered sandstone was encountered at he proposed sign location. Therefore liquefaction potential will be relatively low at this location.

– *Retaining Wall No. 1 (RW 1), approx. "MAINO4" Line Sta. 104+42 to 106+07*
  As discussed in Section 7.3, RW 1 is located in the boarder of artificial fill and bedrock geologic units. Therefore, based on the boring logs and geologic map, both artificial fill and bedrock are anticipated along the RW 1 location. Based on the analysis of A-15-580-001 data, potential liquefiable poorly graded gravel with clay and sand, and clayey gravel layers were identified between elevations 8 and -0.5 feet, and -5 and -8.5 feet. The estimated post-liquefaction settlement is up to 2.5 inches.

– *Retaining Wall No. 2 (RW 2), approx. "MAINO2" Line Sta. 241+63 to 241+99 & Retaining Wall No. 3 (RW 3), approx. "MAINO2" Line Sta. 243+00 to 243+95*
  Based on the boring data of A-14-580-001, the subsurface soil consists of medium dense to dense clayey gravel and poorly graded gravel with clay and sand interbedded with medium still sandy lean clay with gravel. Based on the analysis of A-15-580-001 data, potential liquefiable poorly graded gravel with clay and sand, and clayey gravel layers were identified between elevations 8 and -0.5 feet, and -5 and -8.5 feet. The estimated post-liquefaction settlement is up to 2.5 inches.

- *Retaining Wall No. 4 (RW 4), approx. "CC-E" Line Sta. 1021+23 to "CC-M" Line Sta. 1026+72 & Retaining Wall No. 7 (RW 7), approx. "CC-M" Line Sta. 1026+94 to 1028+25*

  Based on the as-built boring data and boring A-16-580-012, the subsurface consists of fractured greywacke (weathered sandstone with prominent to dominant clay particles), and weathered sandstone and based on our analysis, the liquefaction potential is relatively low.

- *Retaining Wall No. 5 (RW 5), approx. "WAT" Line Sta. 1005+60 to 1010+64*

  Based on the boring data of A-14-580-010 and A-14-580-011, the subsurface consists of intensely to slightly sandstone and based on our analysis, the liquefaction potential is relatively low.

- *Retaining Wall No. 6 (RW 6), approx. "MAIN02" Line Sta. 242+04 to 245+30*

  Based on the boring data of Boring A-15-580-015 and A-15-580-016, the subsurface soil consists of loose to very dense silty sand with gravel, silty gravel with sand and poorly graded gravel with silt and sand interbedded with soft lean clay and fat clay to the depth of 23 to 40 feet below the existing grade (approximate elevation of -5 to -16 feet). The underlying rock formation appears to slope towards the east. Based on the analysis, potential liquefiable silty sand with gravel and silty gravel with sand layers were identified between elevations 16 and 5 feet, and 1 and -16 feet. The estimated post-liquefaction settlement is up to 6 inches.

- *Retaining Wall No. 8 (RW 8) along Bike Path at Marine St Undercrossing*

  Based on the as-built boring data of Marine Street undercrossing, the subsurface soil, below undercrossing elevation, consists of medium stiff clays at shallow depths overlying stiff to hard clay and silty clay, and compact to dense silt and clayey silt with scattered weathered gravel and coarse sand up to the elevation between -10 feet and -28 feet, underlain by weathered sandstone and clayey shale. Based on our analysis, the liquefaction potential is relatively low.

8.2 Cuts and Excavations

Based on the plans and profiles provided to us, cuts and excavations are planned for RW 4, RW 5, RW 7 and RW 8 construction. Minor cuts are expected at RW 1, RW 2 and RW 3.

8.2.1 Stability

Since the proposed cuts and excavation are planned along the existing rock slope at RW 4, RW 5 and RW 7, the stability should not be a concern for the cut slopes of maximum 2H:1V. For the temporarily cut slopes, steeper than 1.5H:1V, the contractor has to verify the soil conditions during construction and modify the slope if necessary based on the actual soil conditions in the field.

Slope stability analysis was performed at RW 8 on the proposed slope configuration after earth gabion wall construction. Based on our analysis, the stability appears to be satisfactory for both static condition (F.S greater than 1.5) and seismic condition (F.S. greater than 1.1). Slope stability analysis outputs are attached in Appendix C.

8.2.2 Rippability

The site is generally underlain by artificial fill and shallower bedrock. As discussed in Section 7.3, shallow bedrock was encountered during our filed investigation. A geophysical investigation was performed by NORCAL, and the results are attached in Appendix D. Based on the investigation, a seismic wave velocity of about 7000 ft/sec was measured. Extracted pages from CATERPILLER “Handbook of Ripping” were attached in Appendix E to help evaluating rippability.

8.2.3 Grading Factor

The on-site native soil meeting the project specifications may be used as engineered fill. For preliminary estimate, a grading factor of 0.9 may be assumed based on previous experience. This estimate should not be used for bidding or construction estimates.

8.3 Embankments

Based on the plans and profiles, no major embankments are anticipated for the project. Therefore, significant future settlements are not anticipated. RW 2 is proposed near to the toe of the existing I-580 embankment near to the Main Street Undercrossing. RW 3 is proposed on the existing embankment along eastbound on-ramp. Based on the provided cross sections, we understand that existing embankment is approximately 2H: 1V slope.

As discussed in Section 8.1.2, potential liquefiable gravel and sand layers were encountered at the embankment location. Slope stability analysis was performed for the embankment behind proposed RW 2.

The stability of the embankments were evaluated under two conditions: (1) static condition (immediately after construction) and (2) seismic condition (pseudo-static with a seismic coefficient (k) and residual shear strength for liquefaction condition). For determining the horizontal seismic coefficient for pseudo-static analysis, we have referred to the "Guidelines for Structures Foundation Reports manual" (Ver. 2.0, 2006, updated December 2009), which recommends that the seismic factor equal to one third of the horizontal peak acceleration and not exceeding 0.2g. The adopted horizontal peak acceleration is 0.2g.

Per Duncan and Wright (2005), short-term, undrained shear strength (non-liquefied) parameters are recommended for analysis under seismic condition since liquefaction generally is not expected to occur in conjunction with the peak ground acceleration. However, according to the latest Caltrans guideline of lateral spreading, we have conducted the pseudo-static analysis with the residual shear strengths (Sr) for the liquefiable soils. The residual soil strengths (S_r) of the liquefiable soils were estimated per Kramer and Wang (2007) as cited in Caltrans guideline.

Based on our analysis, the stability of the embankment appears to be satisfactory for both static condition (F.S. greater than 1.5) and seismic condition (F.S. greater than 1.1).

8.4 Corrosion

The corrosion investigation for this project was performed in general accordance with the provisions of California Test Method 643. Chemical tests were performed on selected samples to evaluate the corrosion potential of the subsurface soil. The corrosion test results are summarized in the following table.

TABLE 3 – SUMMARY OF CORROSION TEST RESULTS

Location	Sample No.	Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
A-15-580-001	3	11	1390	7.73	18.2	18.7
A-15-580-011	1	3	1540	6.94	22.1	34.1
A-15-580-013	1	3	2220	7.41	11.6	26.1
A-15-580-014	1	5	4560	8.05	5.1	9.0
A-15-580-015	2	6	2090	7.65	12.9	47.6
A-15-580-016	3	16	350	7.56	485.4	360.8

Note: Please refer to "Boring Program" (Table 1) or the Site Plans in Appendix A for the boring locations.

According to Caltrans Corrosion Guidelines, November 2012 (Version 2.0), Caltrans considers a site to be corrosive to foundation element if one of the following conditions exists for the representative soil samples taken at the site:

- Chloride concentration is greater than or equal to 500 ppm,
- Sulfate concentration is greater than or equal to 2000 ppm,
- pH is 5.5 or less.

Based on the test results, the on-site soils are classified as non-corrosive per Caltrans corrosion guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructures. The minimum cement factor and cover thickness should be per Caltrans Bridge Design Specifications (Section 8.22).

8.4.1 Drainage Pipes

For selection of pipe material for drainage applications, AltPipe program (Version 7.0) is used by Caltrans to assist designers. The analyses performed by AltPipe are based on the procedures and California Test Methods described in Chapter 850 of the Highway Design Manual (HDM). AltPipe program incorporates current requirements from the HDM supplemented by Caltrans Design Information Bulletin No. 83 (D.I.B. No. 83, June 30, 2003) for abrasion potential for material selection.

The AltPipe program is intended for final design by the civil or hydraulic designer. In addition to soil corrosivity data, the input requires data such as Abrasion Level, 2- to 5-year Flow Velocity and cover height that should be determined while finalizing the

drainage design. Based on discussion with the designer, the AltPipe analyses were performed by Parikh with the design information provided. The design is based on HDM (2011) Chapter 850 Physical Standards.

Based on the AltPipe analyses, the following is a brief summary of the results. The AltPipe program output is attached in Appendix C of the report. The designer needs to refer to the output for detail information and requirements.

- Various steel pipes and steel spiral rib pipes may be allowed; the thickness ranges from 0.052 inch to 0.138 inch.
- Plastic pipes are allowed for majority of the system.
- Reinforced Concrete Pipes (RCP) are allowed. The required cover thickness, cement content and water content need to be verified with the AltPipe output.

AltPipe Outputs are attached in Appendix C.

8.5 Minor Structure Foundations

8.5.1 Single Post Overhead Signs

Based on the information provided by the designer, one new overhead sign is planned as part of the proposed improvement. According to the plans provided, the sign will be a Single Post Types VI on round pedestal pile foundation at approximate “MRN” Line Sta. 227+00.

The boring data (A-14-004) indicates that the subsoil consists of weathered sandstone to the maximum depth explored of 25 feet (elevation 8 feet). Ground water was not encountered during field investigation. The liquefaction potential should be relatively low. The sign location was moved by approximately 360 feet towards west after the boring was drilled. Based on the geology map, we anticipate similar soil/rock profile at the new sign location. However, it should be noted, that these materials could vary in strength and can impact the drilling operation, especially if harder rock conditions are encountered. This may result in a potential claim. It may be prudent to require an exploration ahead of the construction or perform another boring at the new location during the design phase.

The sign posts will be supported on 5 feet diameter CIDH piles and the foundation depths will be determined according to Caltrans Standard Plans Sheet S8. Based on the standard plan, recommended foundation depth for the proposed sign post is 22 feet.

9. STRUCTURAL PAVEMENT

New pavement will be constructed on existing grade and on import borrow materials. For the pavement design, we have collected bulk samples along the project limit. Eleven (11) bulk samples were collected between the depths of 0 to 5 feet below the existing surface to determine the design R-value. The test results are summarized in the following table.

TABLE 4 – SUMMARY OF R-VALUE TEST RESULTS

Boring No.	Location	Description	R-Value
A-15-580-001	"MAINO2" Line 243+00±	Sandy Lean Clay (CL)	19
A-15-580-002	"MAINO2" Line 243+80±	Sandy Lean Clay with Gravel (CL)	23
A-15-580-003	"MAINO3" Line 244+30±	Sandy Lean Clay (CL)	14
A-15-580-004	"CC-E" Line 1003+00±	Sandy Lean Clay with Gravel (CL)	30
A-15-580-005	"CC-E" Line 1013+00±	Sandy Lean Clay (CL)	16
A-15-580-006	"CC-E" Line 1021+95±	Sandy Lean Clay (CL)	11
A-15-580-007	"CC-E" Line 1021+95±	Sandy Lean Clay with Gravel (CL)	20
A-15-580-008	"MRNEO1" Line 1040+65±	Sandy Lean Clay with Gravel (CL)	26
A-15-580-009	"BP1" Line 1004+60±	Silt (ML)	50
A-15-580-010	"BP1" Line 1006+25±	Lean Clay (CL)	12
A-15-580-011	"BP1" Line 1009+35±	Sandy Lean Clay (CL)	14

*Note: The locations of the borings were not surveyed and are approximate based on the plans provided by the designer.

As shown in the above table, R-value varies between 11 and 50 at the proposed new pavements section area. There are only two locations that have R-value more than 30, which are apparently higher than other test results. This appears to be a localized spot, which may not be representative. An R-value of 10 was used for the pavement design to account for the subgrade variation. In addition, the recommended minimum R-value for Aggregate Base (AB, Class 2) is 78 and Aggregate Subbase (AS, Class 2) is 50.

The Traffic Index (TI) values for each segment of roadway used for the pavement design were provided by the designer. The pavement design was performed in accordance to standard Caltrans procedures as outlined in Highway Design Manual, Section 630. The recommended flexible and rigid structural pavement sections are tabulated in the following tables.

TABLE 5 – RECOMMENDED FLEXIBLE STRUCTURAL PAVEMENT SECTIONS (20-YR DESIGN LIFE)

Location	T.I	R-value Used in Design	Flexible Structural Pavement Section (ft)															
			Option 1*						Option 2				Option 3				Option 4	
			RHMA-G	HMA	ATPB	HMA	LCB	AS	RHMA-G	HMA	LCB	AS	RHMA-G	HMA	AB	AS	RHMA-G	Full Depth HMA
Main Line	12	10	-	-	-	-	-	-	0.15	0.65	0.35	1.30	0.15	1.40**	0.35**	-	0.20	1.35
	12.5		-	-	-	-	-	-	0.15	0.70	0.35	1.35	0.15	1.40**	0.35**	-	0.20	1.40
I-580 (EB) Main Street/San Quentin On-Ramp	10		-	-	-	-	-	-	-	-	-	-	0.15	0.40	0.75	1.10	-	-
I-580 (EB) Marine Street Off-Ramp	10.5		-	-	-	-	-	-	-	-	-	-	0.15	0.40	0.85	1.15	-	-
I-580 (WB) Western Drive Off-Ramp	7		-	-	-	-	-	-	-	-	-	-	0.15	0.20	0.55	0.70	0.20	0.65
Main Street	9.5		-	-	-	-	-	-	-	-	-	-	0.15	0.35	0.75	1.00	0.20	1.00
Francisco Blvd	9.5		-	-	-	-	-	-	-	-	-	-	-	0.50	0.75	1.00	-	1.20
Bike Path	5		-	-	-	-	-	-	-	-	-	-	-	0.25	0.35	0.45	0.20	0.40
I-580 (WB) Main Street Off-Ramp	8		-	-	-	-	-	-	-	-	-	-	0.15	0.25	0.65	0.80		-
E. Standard Ave (Westbound)	11		0.15	0.30	0.25	0.10	0.35	0.65	-	-	-	-	-	-	-	-	-	-
E. Standard Ave (Eastbound)	9		0.15	0.30	0.25	-	0.35	0.65	-	-	-	-	-	-	-	-	-	-
Castro Street (Southbound)	11		-	-	-	-	-	-	-	-	-	-	0.15	0.45	0.85	1.20	0.20	1.20
Castro Street (Northbound)	9		-	-	-	-	-	-	-	-	-	-	0.15	0.30	0.75	0.95	0.20	0.95
I-580 (EB) Sir Francis Drake On-Ramp	9		-	-	-	-	-	-	-	-	-	-	0.15	0.30	0.75	0.95	0.20	0.95

* Depth of HMA has been adjusted to match the existing pavement above ATPB layer.

** Depth of HMA and AB has been adjusted to match the existing pavement as requested by designer.

TI - Traffic Index
RHMA-G – Rubberized Hot Mix Asphalt – gap-graded
HMA - Hot Mix Asphalt (Type A)
ATPB - Asphalt Treated Permeable Base
AB - Aggregate Base (Class 2) with R-value of 78
LCB - Lean Concrete Base
AS - Class 2 Aggregate Subbase

Evaluation of As-Built Sections

Based on the as-built plan (Contract No 04-0C4204, dated 07/20/2001), we understand that ATPB and above ATPB layers of eastbound right lane (between PM 5.25 and 5.6) and both lanes (between PM 5.6 and toll booth) were replaced with AC. Caltrans Memorandum (dated March 30, 2010) of Structural Section Recommendations, which includes coring data, did not indicate presence of ATPB. The as-built plan is attached in Appendix F. Therefore ATPB layer is not required for the proposed widening area along main line. Based on the construction plans (Contract No. 04-1A3204, dated 05/25/2011), we also understand that existing east bound shoulder between Scofield Ave undercrossing and Marine Street off-ramp was replaced with 1.55' HMA and 0.35' Class 2 AB. We were told by the designer that this was already constructed and the as-built plans are not available yet. Considering the TI and design R-value, this shoulder section of 1.55' HMA and 0.35' AB is adequate for traffic lane. It is our understanding that existing shoulder between Marine Street off-ramp and Marine Street UC shoulder will be replaced with 1.55' HMA and 0.35' AB for the proposed lane widening.

Based on the as-built plans (Contract No. 04-108764, dated 6/16/92), we understand that E. Standard Ave (westbound and eastbound) consists of 0.25' ATPB layer. Therefore, it is recommended to place 0.25' ATPB layer for the widening to match the existing condition.

Hot Mix Asphalt (HMA, Type A) and Aggregate Bases (AB, Class 2) should conform to the Caltrans Standard Specifications in Section 39 and 26 respectively. Pavement subgrade and structural sections should be prepared and compacted in accordance with the project specification and Caltrans standard. Based on Caltrans Standard Specifications (2010), Rubberized Asphalt Concrete (RHMA) should be spread and compacted only if the atmospheric temperature is at least 55 degree F and the surface temperature at least 60 degree F.

10. MATERIAL SOURCES

There are several commercial sources of asphalt, concrete, and aggregate products in the area. The following table lists available commercial suppliers in the area.

TABLE 6 – SOURCES OF ASPHALT AND AGGREGATE MATERIAL

Source	Location	Approx. Haul Dist. (one way, miles)
Cemex	402 Wright Ave, Richmond, CA 94804	2.9
Readymix	808 Gilman St, Berkeley. CA 94710	16.3
Rich Readimix Concrete	101 Rich St, Greenbrae, CA 94904	14.5

11. MATERIAL DISPOSAL

Based on our understanding, the project will require disposal of the excess materials from the excavation for the Retaining Wall No. 4 & 5.

12. CONSTRUCTION CONSIDERATIONS

12.1 Construction Advisories

These sections are written primarily for the engineer responsible for the preparation of plans and specifications. Since these sections identify potential construction issues related to the project, it may also be of use to the Agency's representatives involved in monitoring of construction activity. The field investigation performed by us primarily addresses design issues and was not planned specifically to identify construction issues.

The project site is located along the existing I-580. Traffic control is required to maintain traffic flow along I-580. Several underground utilities exist at the site. The contractor should verify the utility lines, be aware of the existing conditions and plan the construction activities accordingly.

In our opinion, conventional equipment may be used to excavate the on-site soil materials. The material to be excavated for RW 4 & 5 may consist of weathered and un-weathered sandstone. Extracted pages from CATERPILLER "Handbook of Ripping" were attached in Appendix E to determine rippability. Localized subgrade pumping may be encountered during earthwork construction depending on the weather, moisture condition of the subsurface soils, and surface drainage conditions. Equipment mobility may also be difficult if the subgrade is wet. In which case, the subgrade soils may require reworking, aeration, or over-excavation and replacing with dry granular fill to facilitate earthwork construction. It is possible that unknown old buried utilities or abandoned structures, concrete rubble etc. are located along the alignment. It might require special equipment and additional efforts to remove these buried objects.

Prospective contractors for the project must evaluate construction-related issues on the basis of their own knowledge and experience in the local area, on the basis of similar projects in other localities, or on the basis of field investigation on the site performed by them, taking into account their proposed construction methods and procedures. In addition, construction

activities related to excavation and lateral earth support must conform to safety requirements of OSHA and other applicable municipal and State regulatory agencies.

Construction Considerations for Cast-In-Drilled-Hole (CIDH) Piles. We understand that CIDH pile is considered for the overhead sign foundation. Based on the boring log, moderately and slightly weathered sandstone was encountered at the proposed overhead sign location. Due to the presence of the weathered bedrock, caving is expected which may require additional drilling and cleaning effort and may increase the concrete volume for the piles. It is prudent to make the contractor aware of these conditions so that he takes appropriate steps to comply with the standards and maintain the integrity of the CIDH piles. The use of temporary steel casing should be anticipated at all times to maintain the integrity of the piles. Prior to construction, the qualifications and previous work experience of the potential sub-contractors should be reviewed. The intent is to help select qualified contractors to reduce construction issues.

Caltrans standard specifications and standard special provisions (SSP) for "Cast-in-Place Concrete Piling" should be used for the construction of CIDH piles. Vertical inspection pipes for acceptance testing should be provided in all CIDH piles that are 24 inches in diameter or larger, except when the holes are dry or when the holes are dewatered without the use of temporary casing to control groundwater. The acceptance test should include Gamma-Gamma Logging and may also include crosshole sonic logging. Gamma-Gamma Logging should be performed in accordance with California Test Method 233 (CT233) to check the homogeneity of the CIDH piles. CT233 defines pile rejection criteria based on the statistical principles of mean and three standard deviations to analyze the homogeneity of a pile. Anomalies detected should be evaluated by the designer for their significance and potential impact on design and to see if mitigation plans are required. Details of the acceptance testing and Gamma-Gamma Logging are contained in Caltrans SSP and CT233.

All piles excavations should be observed by the geotechnical engineer or regulatory agency prior to the placement of reinforcement and concrete so that if conditions differ from those anticipated, appropriate recommendations can be made.

12.2 Construction Consideration that Influence Specifications

The contractor should verify the conditions of the existing utility lines. These locations should not be used for stockpiling of borrow or excavated materials. Any conflicts with proposed construction should also be reviewed prior to construction.

12.3 Hazardous Waste Considerations

The project environmental study report should be referred to for details about any potential hazardous materials within the project site.

12.4 Differing Site Conditions

The soil conditions described in this report are based on available boring data. It should be noted that these borings depict subsurface conditions only at the locations drilled. Because of the variability from place to place within soils in general, and the nature of geologic depositions, subsurface conditions could change between the explored locations.

Early communication should be made between the Resident Engineer, the Contractor, and the Geotechnical Engineer as soon as conditions that differ from those established in this report are recognized by any of the parties. Additional recommendations could be provided if such conditions arise.

13. RECOMMENDATIONS AND SPECIFICATIONS

13.1 Summary of Recommendations

If the designer has questions or concerns with any of these recommendations, or, if conditions are found to be different during construction, the Geotechnical Engineer who prepared this report should be contacted. Additional fieldwork, analysis or changes in recommendations may be required. These services may be provided under a separate authorization, as necessary. A concise summary of the geotechnical recommendations is presented below:

- Based on the investigation, the subsoils generally consist of medium dense to very dense clayey gravel and poorly graded gravel with clay and sand interbedded with medium stiff sandy lean clay and intensely to slightly weathered sandstone. (Ref: Section 7.3)
- Groundwater was encountered at elevation 7.0 feet in Boring A-15-580-001 in Marin

County. Near the bay, the natural groundwater level is expected to be close to Elev. ~0. (Ref: Section 7.4.2)

- The anticipated Peak Ground Acceleration for the Marin County and Contra Costa County project site are 0.628g and 0.677g, respectively. Liquefaction potential is considered medium to high at the proposed retaining wall location in Marin County. (Ref: Section 8.1).
- No major Embankments are anticipated. Therefore, significant future settlements are not anticipated. (Ref: Section 8.3)
- Refer to Table 5 for the design structural pavement sections. The structural pavement sections were calculated based on R-value of 10. The design TI's were provided by the designer. (Ref: Section 9).

13.2 Recommended Materials Specifications

13.2.1 Standard Specifications

Unless otherwise stated in the special provisions, all materials specifications should conform to Caltrans Standard Specifications, 2010 edition, including but not limited to the following: Earthwork, Structure Backfill, Pervious Backfill Material, Reinforcing Geofabric, Thermoplastic Pipes, Asphalt Concrete, Aggregate Base, Aggregate Subbase, etc.

13.2.2 Special Provisions

Imported Borrow:

Imported material should be in accordance with the specifications set forth in Caltrans Section 19. In particular, for new roadway construction, the material placed within 2.5 ft of the finish pavement subgrade should meet the following requirements:

1. Free of organic or other deleterious materials.
2. Minimum R-value of 15 to meet the pavement design criteria.

Aggregate Base: Class 2 aggregate base shall conform to the provisions in Section 26 of the Standard Specifications and to these Special Provisions. It shall also be clean and free from organic matter and other deleterious substances. The percentage composition by weight of Class 2 aggregate base shall conform to the following grading as determined by California Test Method No. 202.

14. 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 field exploration and the assumption that the soil conditions do not deviate from observed conditions. 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 findings 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 soil 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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Project Manager

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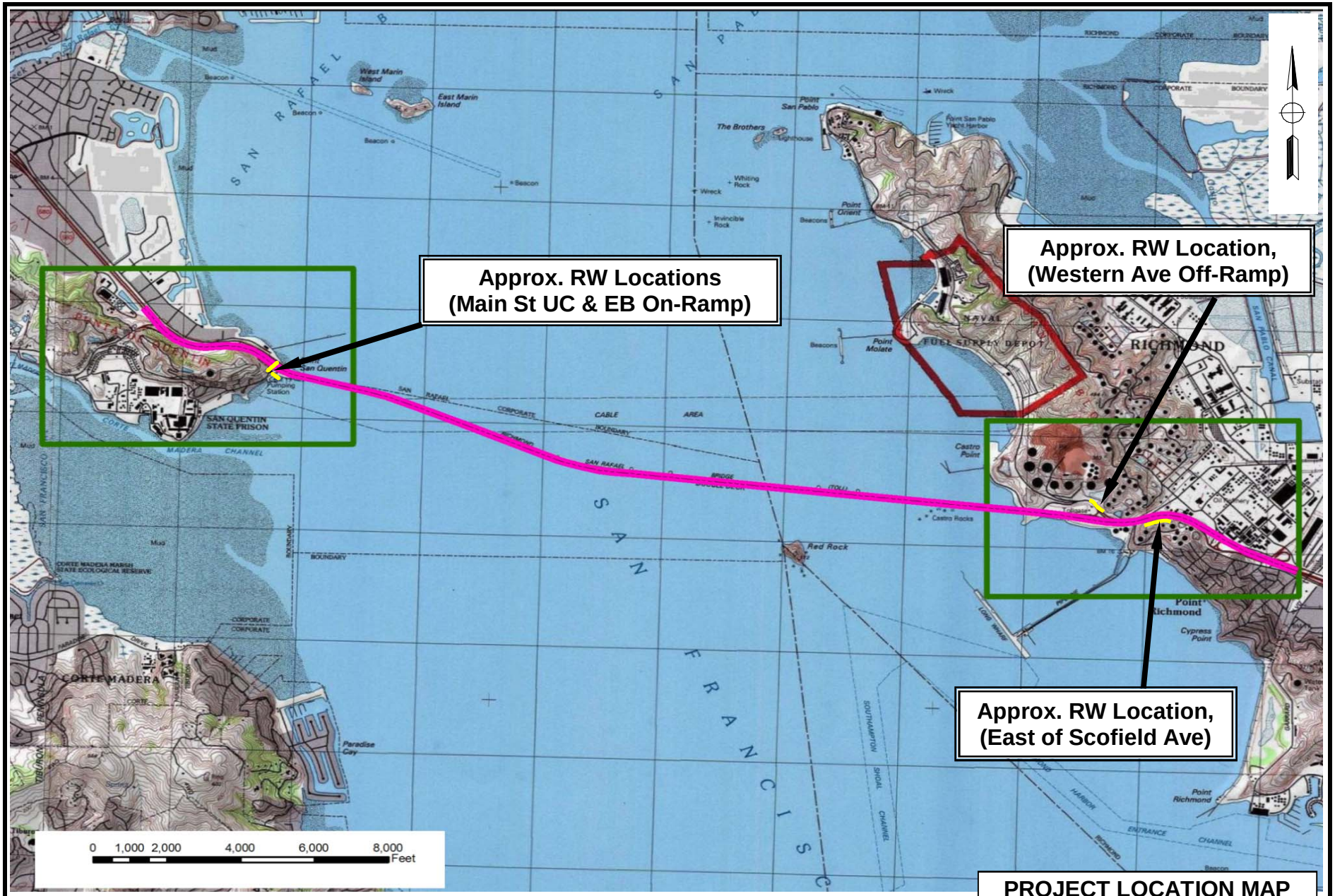

PROJECT LOCATION MAP

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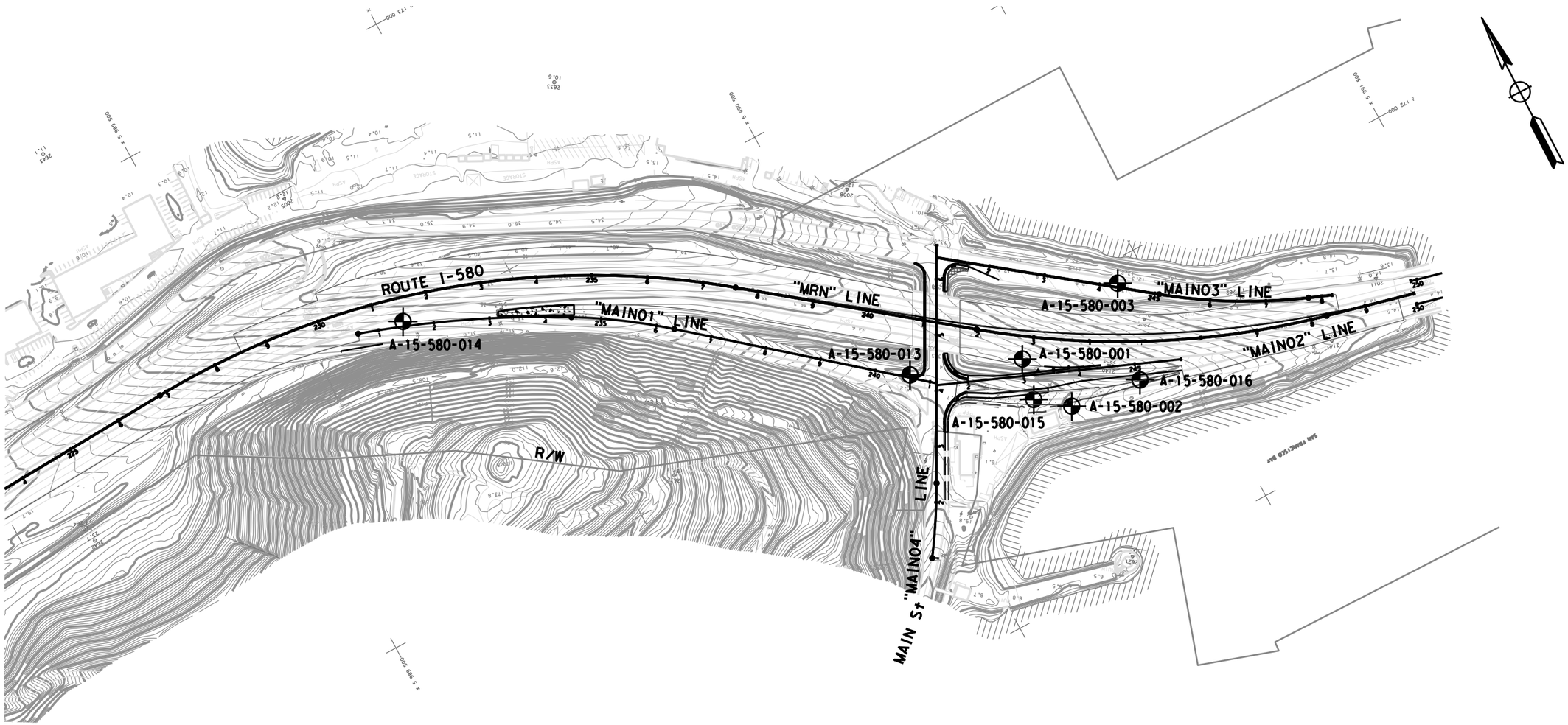
RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
MARIN AND CONTRA COSTA COUNTIES, CALIFORNIA

JOB NO.: 2014-125-GDR	PLATE NO.: 1



SITE PLAN

LEGEND

A-15-580-001

Approx. Boring Location Drilled By Parikh Consultants, Inc. in 2015

SCALE: 1 inch = 200 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by HNTB, Inc.

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RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
RICHMOND CITY, CALIFORNIA

PROJECT NO.:2014-125-GDR | PLATE NO: 2A

SITE PLAN

LEGEND

A-15-580-001

Approx. Boring Location Drilled By Parikh Consultants, Inc. in 2015

SCALE: 1 inch = 200 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by HNTB, Inc.

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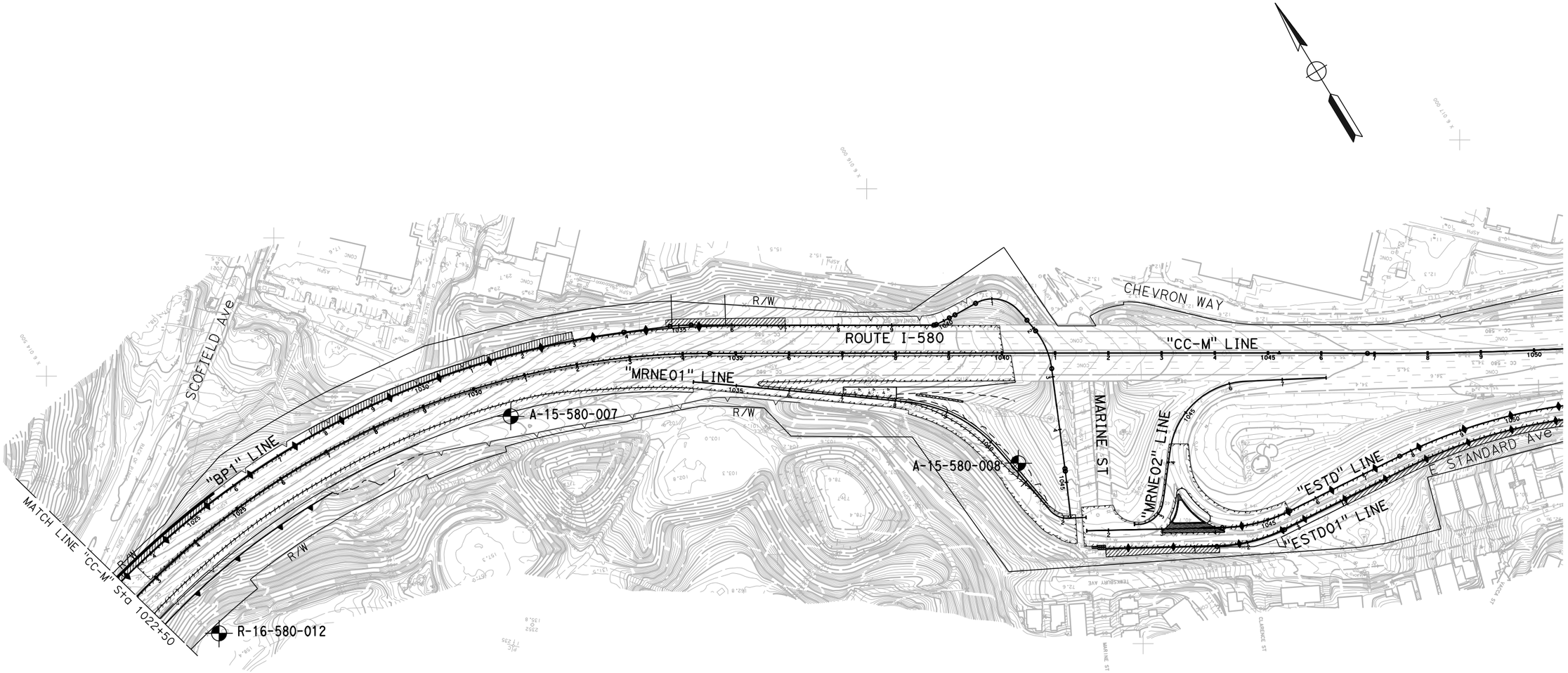
RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
RICHMOND CITY, CALIFORNIA

PROJECT NO.:2014-125-GDR | PLATE NO: 2B



SITE PLAN

LEGEND

A-15-580-001

Approx. Boring Location Drilled By Parikh Consultants, Inc.

SCALE: 1 inch = 200 feet

Note: All units are in feet unless otherwise specified
Reference Map was provided by HNTB, Inc.

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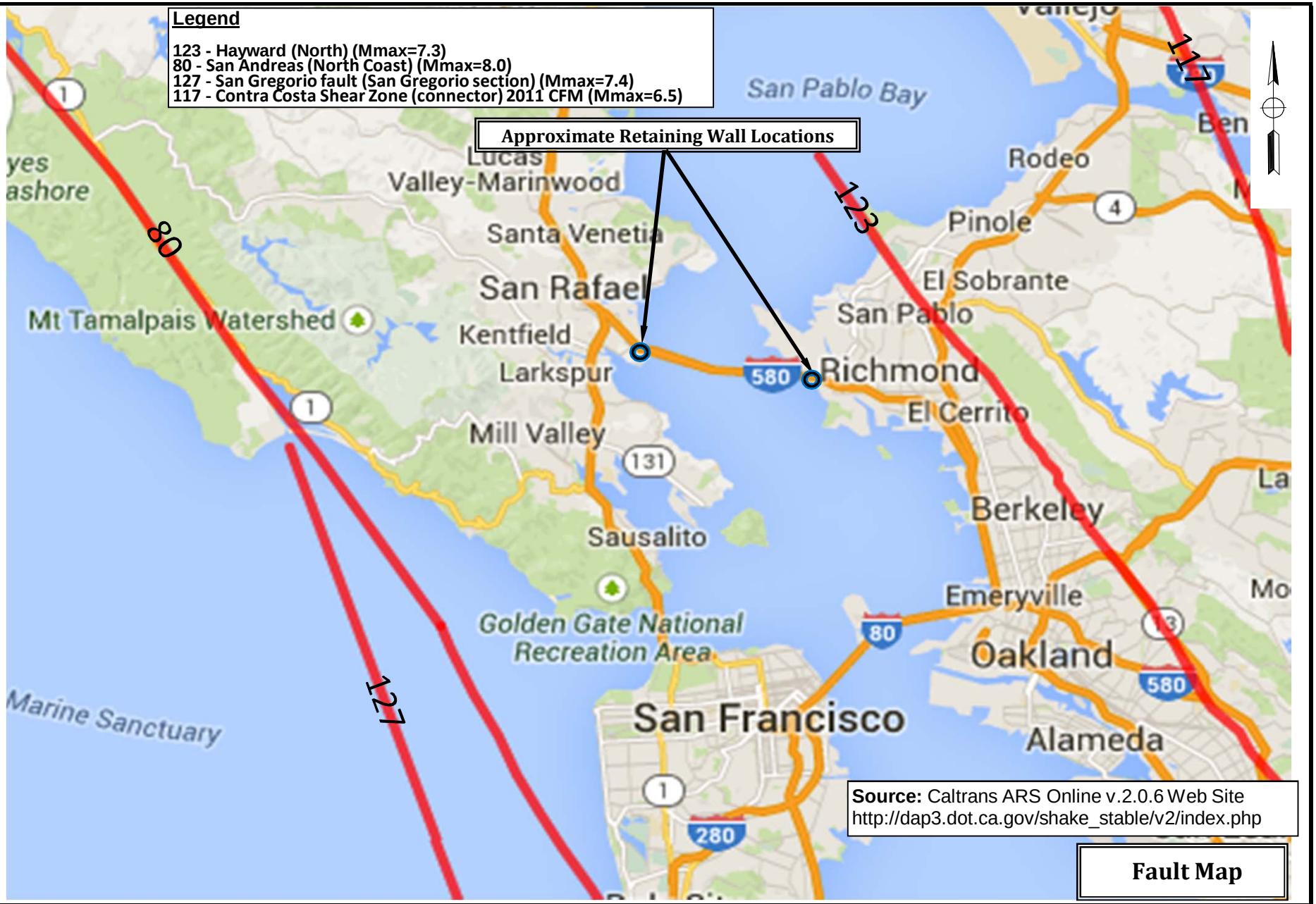
RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
RICHMOND CITY, CALIFORNIA

PROJECT NO.:2014-125-GDR | PLATE NO: 2C



Fault Map

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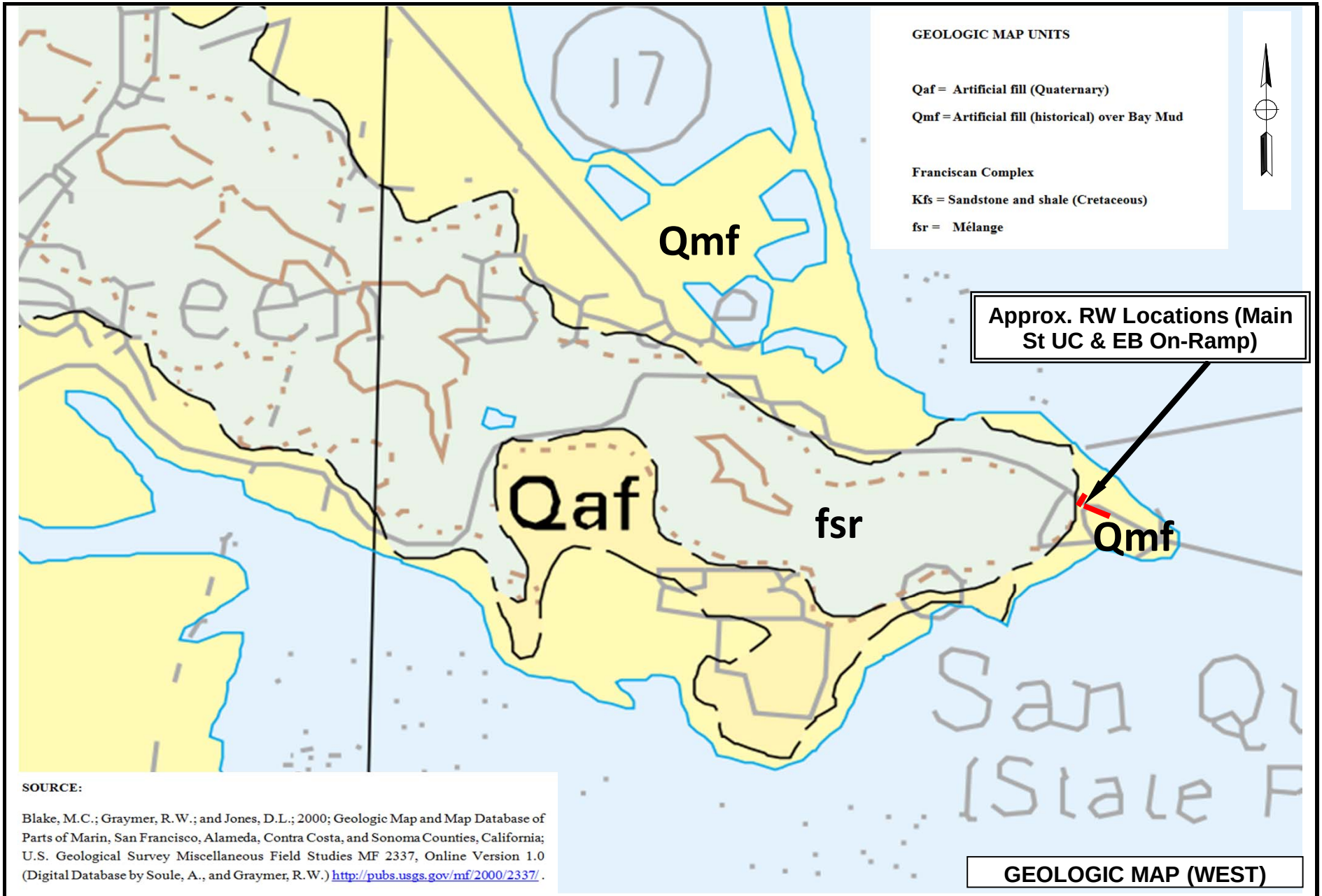
RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
MARIN AND CONTRA COSTA COUNTIES, CALIFORNIA

JOB NO.: 2014-125-GDR	PLATE NO.: 3



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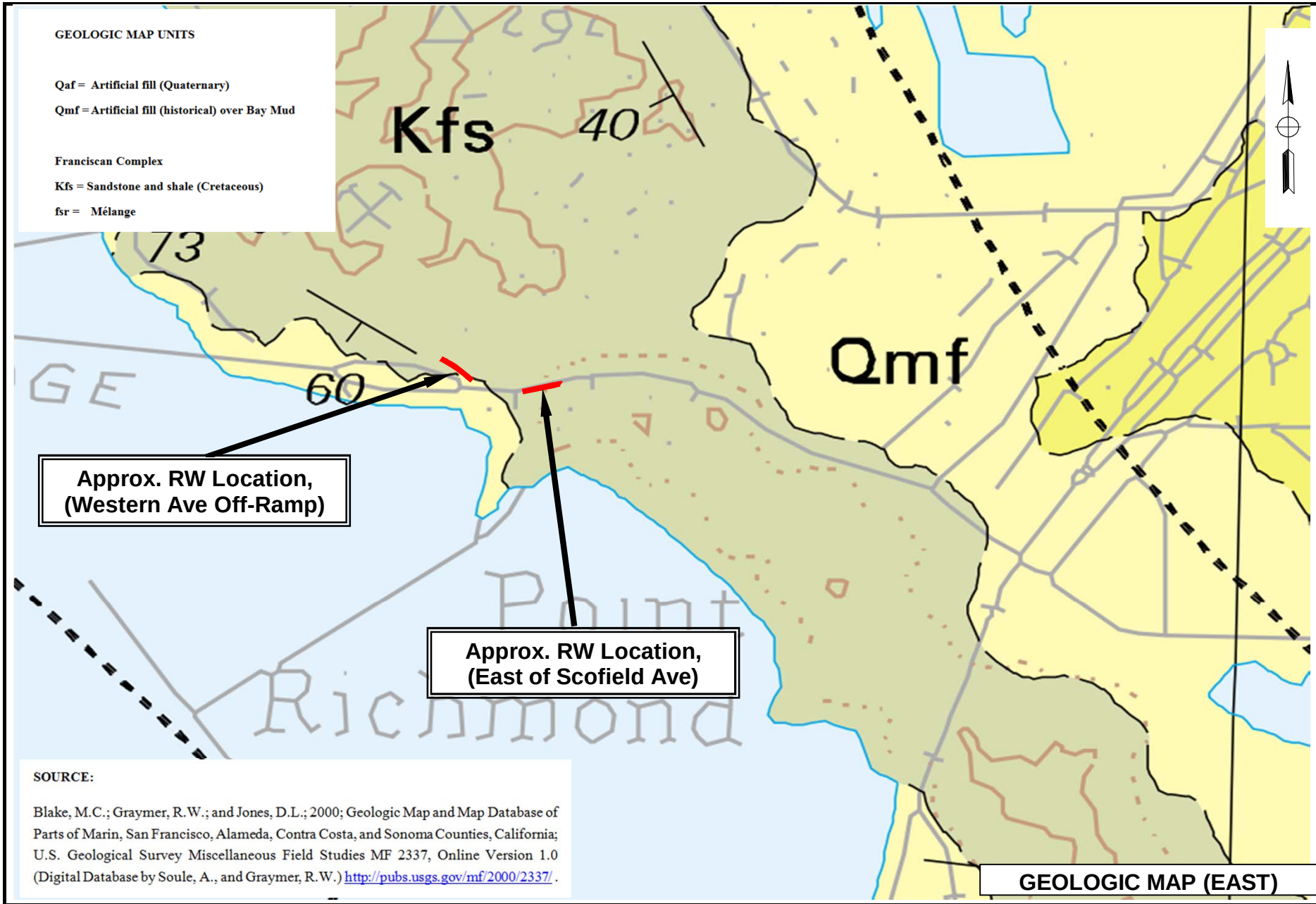
RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
MARIN AND CONTRA COSTA COUNTIES, CALIFORNIA

JOB NO.: 2014-125-GDR	PLATE NO.: 4A



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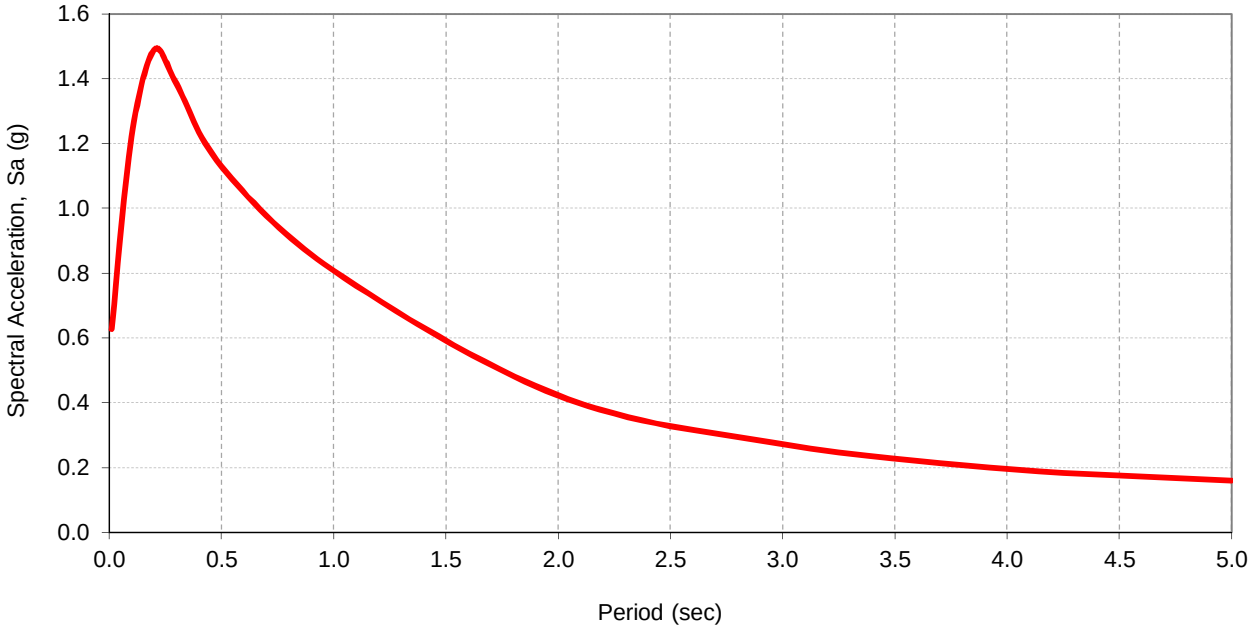
RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
MARIN AND CONTRA COSTA COUNTIES, CALIFORNIA

JOB NO.: 2014-125-GDR	PLATE NO.: 4B

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

(5% Damping)



Site Information

Latitude:	37.9434
Longitude	-122.4805
V_{S30} (m/s) =	450
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	14.8

Governing Curve:

Caltrans Online Probabilistic ARS

Recommended Response Spectrum

Period (sec)	Caltrans Online Probabilistic Spectral Acceleration (g)	Adjusted for Near Fault Effect	Adjusted For Basin Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.627	1	1	0.627
0.1	1.222	1	1	1.222
0.2	1.489	1	1	1.489
0.3	1.384	1	1	1.384
0.5	1.13	1	1	1.130
1.0	0.673	1.2	1	0.808
2.0	0.352	1.2	1	0.422
3.0	0.227	1.2	1	0.272
4.0	0.163	1.2	1	0.196
5.0	0.133	1.2	1	0.160

Source:

1. Caltrans ARS Online tool (V.2, http://dap3.dot.ca.gov/ARS_Online/)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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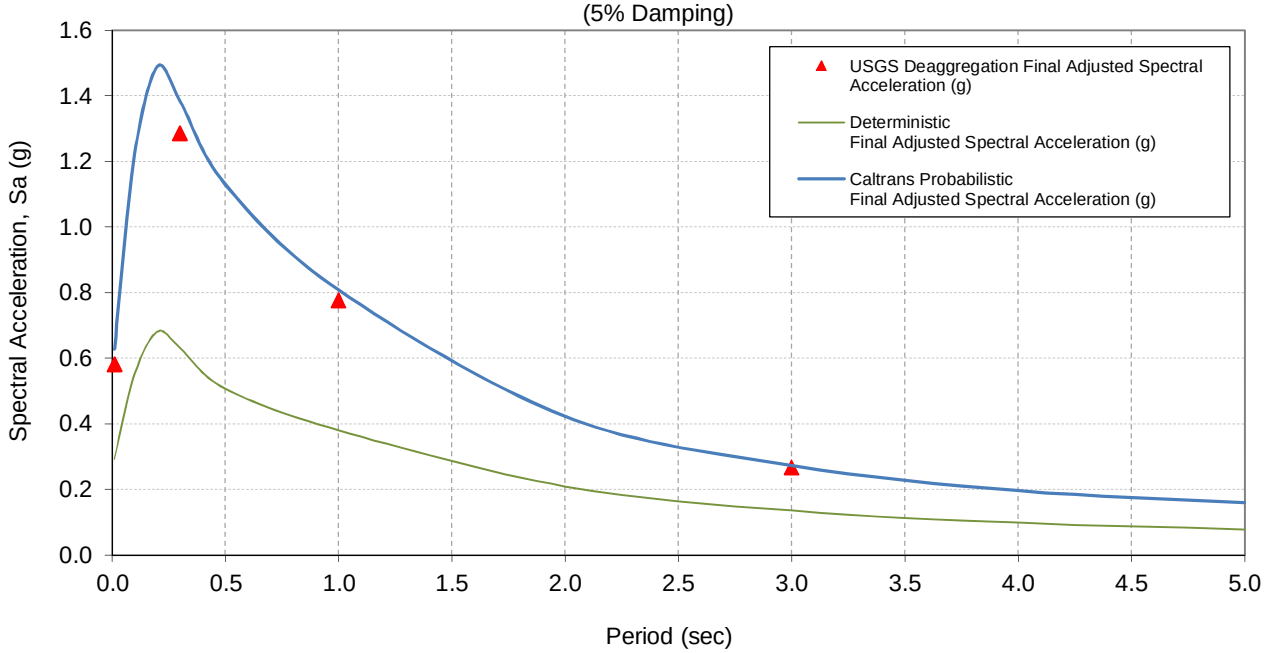
Richmond-San Rafael Bridge Access Improvement Project
Retaining Walls 1, 2, 3 & 6

Project No.: 2014-125-PGR | **Plate No.: 5A**

ACCELERATION RESPONSE SPECTRUM COMPARISON

(Deterministic & Probablistic Curves)
(5% Damping)



Site Information

Latitude:	37.9434
Longitude	-122.4805
V_{S30} (m/s) =	450
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	14.8

Period (sec)	Deterministic Final Adjusted Spectral Acceleration (g)	Caltrans Probabilistic Final Adjusted Spectral Acceleration (g)	USGS Deaggregation Final Adjusted Spectral Acceleration (g)
0.0	0.293	0.627	0.581
0.1	0.552	1.222	
0.2	0.681	1.489	
0.3	0.631	1.384	1.286
0.5	0.507	1.130	
1.0	0.380	0.808	0.776
2.0	0.209	0.422	
3.0	0.135	0.272	0.267
4.0	0.098	0.196	
5.0	0.078	0.160	

Source:

1. Caltrans ARS Online tool (V.2, http://dap3.dot.ca.gov/ARS_Online/)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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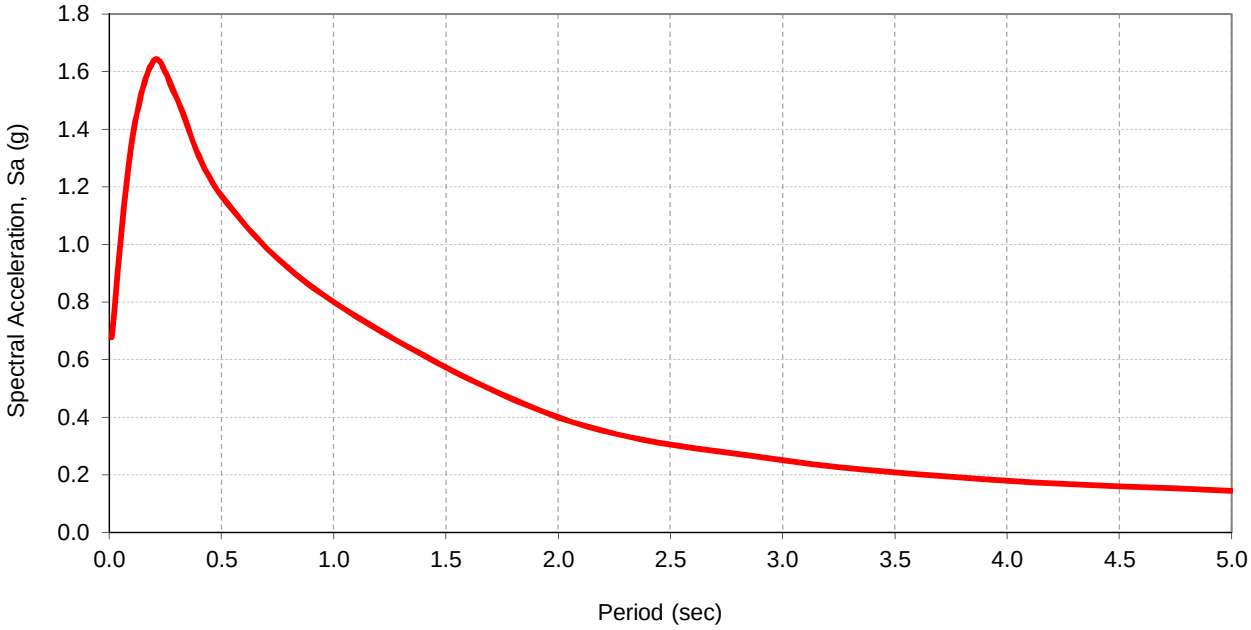
Richmond-San Rafael Bridge Access Improvement Project
Retaining Walls 1, 2, 3 & 6

Project No.: 2014-125-PGR | **Plate No.: 5B**

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

(5% Damping)



Site Information

Latitude:	37.9321
Longitude	-122.3991
V_{S30} (m/s) =	510
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	7.4

Governing Curve:

Caltrans Online Probabilistic ARS

Recommended Response Spectrum

Period (sec)	Caltrans Online Probabilistic Spectral Acceleration (g)	Adjusted for Near Fault Effect	Adjusted For Basin Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.677	1	1	0.677
0.1	1.351	1	1	1.351
0.2	1.639	1	1	1.639
0.3	1.508	1	1	1.508
0.5	1.169	1	1	1.169
1.0	0.666	1.2	1	0.799
2.0	0.333	1.2	1	0.400
3.0	0.209	1.2	1	0.251
4.0	0.149	1.2	1	0.179
5.0	0.12	1.2	1	0.144

Source:

1. Caltrans ARS Online tool (V.2, http://dap3.dot.ca.gov/ARS_Online/)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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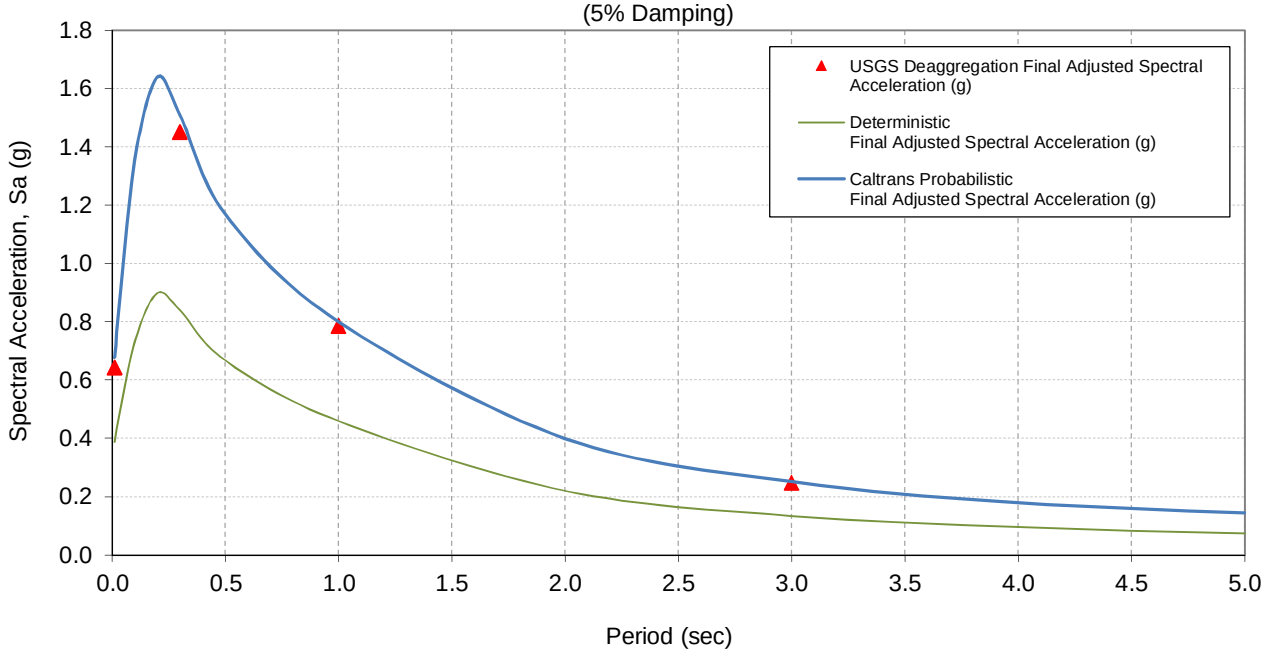
Richmond-San Rafael Bridge Access Improvement Project
Retaining Walls 4, 5 & 7

Project No.: 2014-125-PGR | **Plate No.: 6A**

ACCELERATION RESPONSE SPECTRUM COMPARISON

(Deterministic & Probablistic Curves)
(5% Damping)



Site Information

Latitude:	37.9321
Longitude	-122.3991
V_{S30} (m/s) =	510
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	7.4

Period (sec)	Deterministic Final Adjusted Spectral Acceleration (g)	Caltrans Probabilistic Final Adjusted Spectral Acceleration (g)	USGS Deaggregation Final Adjusted Spectral Acceleration (g)
0.0	0.388	0.677	0.642
0.1	0.730	1.351	
0.2	0.898	1.639	
0.3	0.839	1.508	1.451
0.5	0.667	1.169	
1.0	0.459	0.799	0.785
2.0	0.220	0.400	
3.0	0.134	0.251	0.247
4.0	0.095	0.179	
5.0	0.073	0.144	

Source:

1. Caltrans ARS Online tool (V.2, http://dap3.dot.ca.gov/ARS_Online/)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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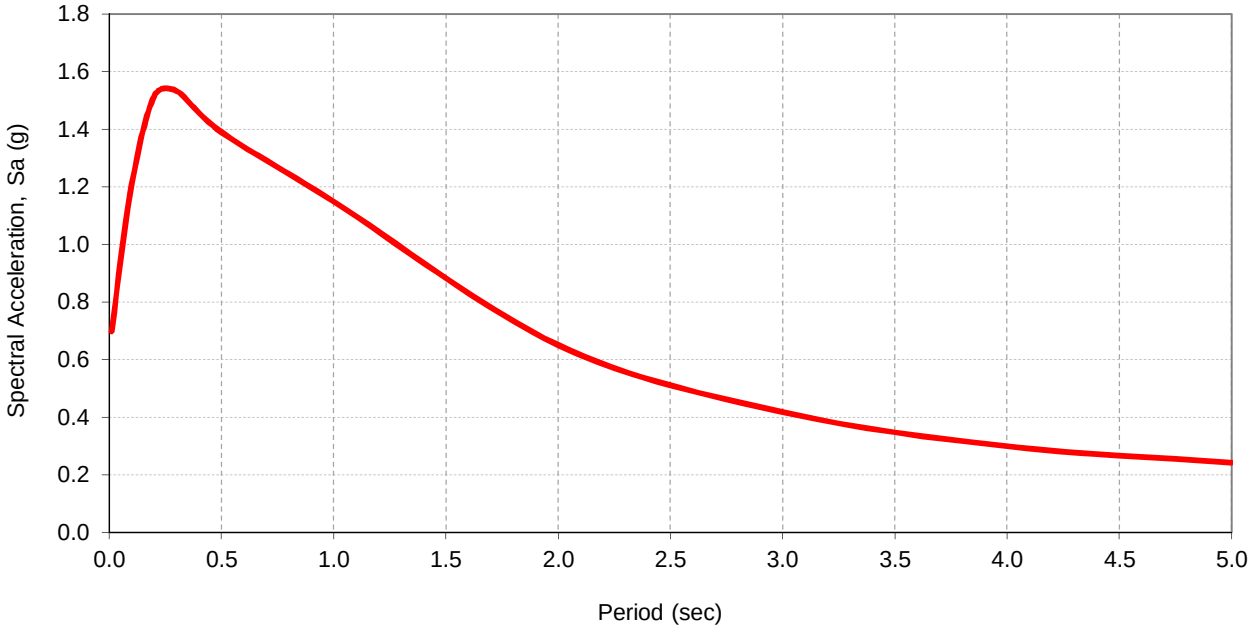
Richmond-San Rafael Bridge Access Improvement Project
Retaining Walls 4, 5 & 7

Project No.: 2014-125-PGR | **Plate No.: 6B**

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

(5% Damping)



Site Information

Latitude:	37.9315
Longitude	-122.3917
V_{S30} (m/s) =	275
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	6.9

Governing Curve:

Caltrans Online Probabilistic ARS

Recommended Response Spectrum

Period (sec)	Caltrans Online Probabilistic Spectral Acceleration (g)	Adjusted for Near Fault Effect	Adjusted For Basin Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.698	1	1	0.698
0.1	1.207	1	1	1.207
0.2	1.514	1	1	1.514
0.3	1.532	1	1	1.532
0.5	1.389	1	1	1.389
1.0	0.957	1.2	1	1.148
2.0	0.542	1.2	1	0.650
3.0	0.348	1.2	1	0.418
4.0	0.249	1.2	1	0.299
5.0	0.202	1.2	1	0.242

Source:

1. Caltrans ARS Online tool (V.2, http://dap3.dot.ca.gov/ARS_Online/)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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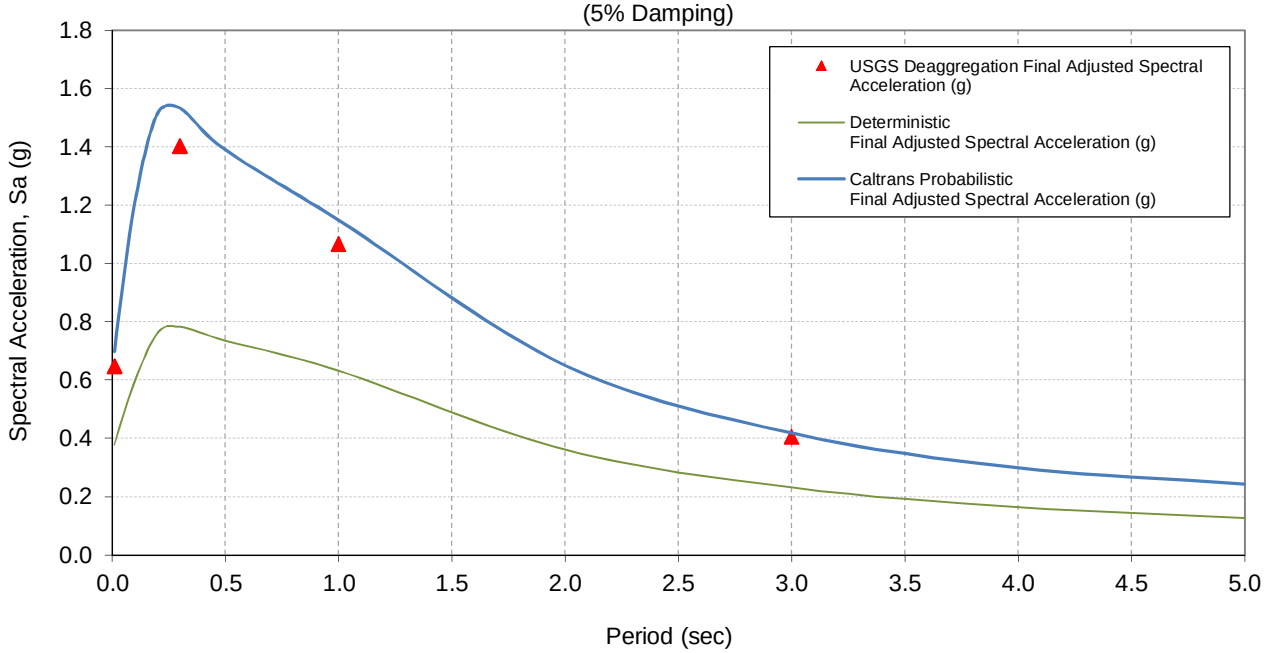
Richmond-San Rafael Bridge Access Improvement Project
Retaining Wall 8

Project No.: 2014-125-FDN | **Plate No.: 7A**

ACCELERATION RESPONSE SPECTRUM COMPARISON

(Deterministic & Probablistic Curves)
(5% Damping)



Site Information

Latitude:	37.9315
Longitude	-122.3917
V_{S30} (m/s) =	275
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	6.9

Period (sec)	Deterministic Final Adjusted Spectral Acceleration (g)	Caltrans Probabilistic Final Adjusted Spectral Acceleration (g)	USGS Deaggregation Final Adjusted Spectral Acceleration (g)
0.0	0.378	0.698	0.647
0.1	0.595	1.207	
0.2	0.760	1.514	
0.3	0.782	1.532	1.402
0.5	0.735	1.389	
1.0	0.632	1.148	1.067
2.0	0.361	0.650	
3.0	0.231	0.418	0.406
4.0	0.164	0.299	
5.0	0.126	0.242	

Source:

1. Caltrans ARS Online tool (V.2, http://dap3.dot.ca.gov/ARS_Online/)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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Richmond-San Rafael Bridge Access Improvement Project
Retaining Wall 8

Project No.: 2014-125-FDN | **Plate No.: 7B**

APPENDIX A

Log of Test Borings

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	CC/Mrn	580	5.0/7.8, 0.0/3.2		

GEOTECHNICAL PROFESSIONAL — DATE 4/14/16

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. 666 — Exp. 12/31/17 — GEOTECHNICAL — STATE OF CALIFORNIA

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

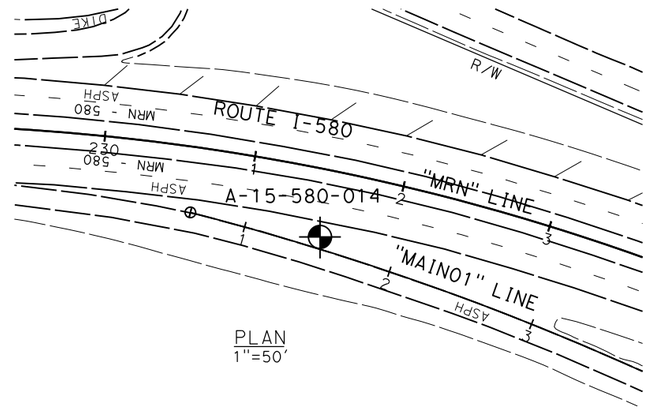
NOTES:

Standard Penetration Test Sampler: I.D. = 1.4"; O.D. = 2" Modified California Sampler: I.D. = 2.5"; O.D. = 3" Hammer Assembly: A 140 lb hammer with a 30" drop (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (2010)

See Caltrans 2010 Standard Plans A10F, A10G and A10H for Soil and Rock Legend.

All dimensions are in feet unless otherwise shown



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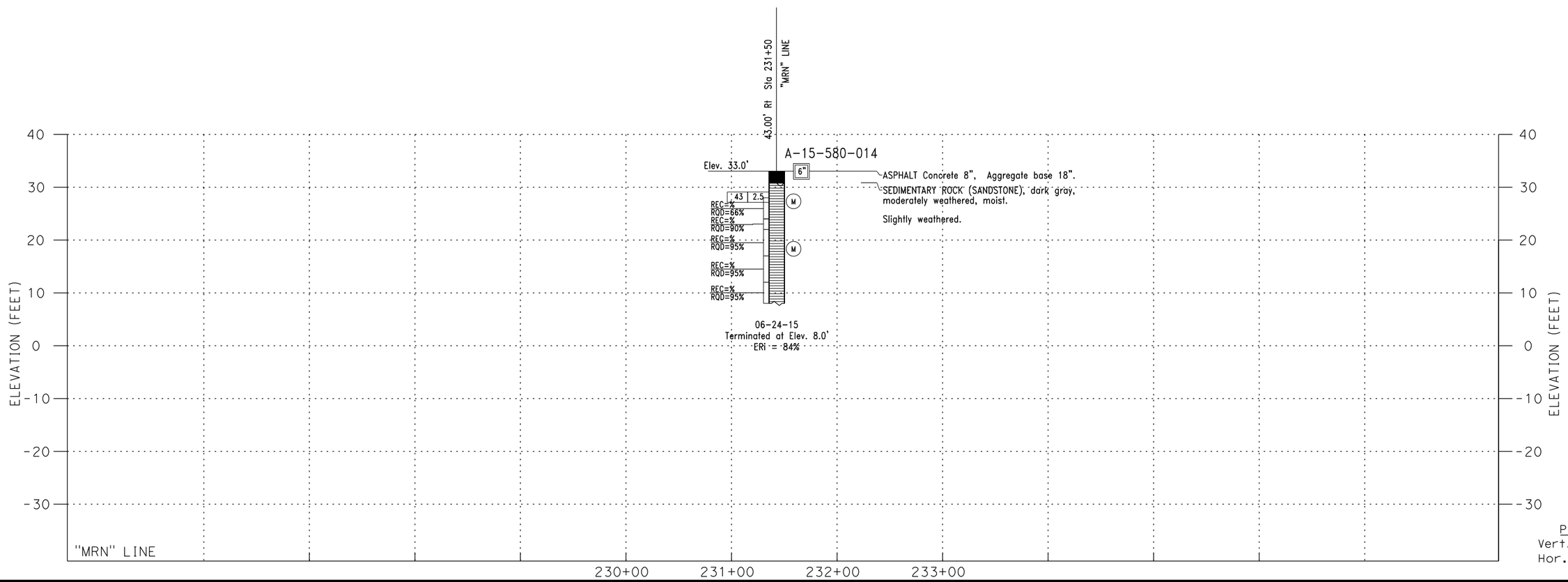

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

X DESIGN OVERSIGHT	DRAWN BY	KIM OUYANG	L.S. BHANGOO FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	KANDEEP SARAVANAPAVAN PROJECT ENGINEER
X SIGN OFF DATE	CHECKED BY	KANDEEP SARAVANAPAVAN	DATE: JUNE & DECEMBER 2015 TO MARCH 2016		BRIDGE NO. / POST MILES

RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT

LOG OF TEST BORINGS 1 OF 5

GS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT:
PROJECT NUMBER & PHASE:
CONTRACT NO.:

DISREGARD PRINTS BEARING EARLIER REVISION DATES → REVISION DATES | SHEET | OF

FILE => 1 of 5.dgn

USERNAME => ogouthier DATE PLOTTED => 4/14/2016 TIME PLOTTED => 4:03:26 PM

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	CC/Mrn	580	5.0/7.8, 0.0/3.2		

GEOTECHNICAL PROFESSIONAL — DATE 4/14/16

PLANS APPROVAL DATE

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

NOTES:

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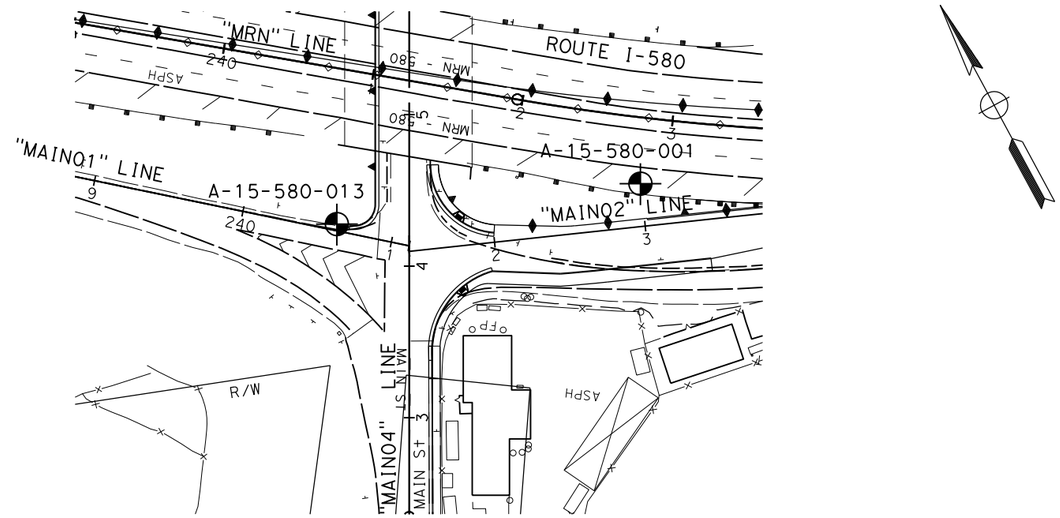
This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (2010)

See Caltrans 2010 Standard Plans A10F, A10G and A10H for Soil and Rock Legend.

All dimensions are in feet unless otherwise shown



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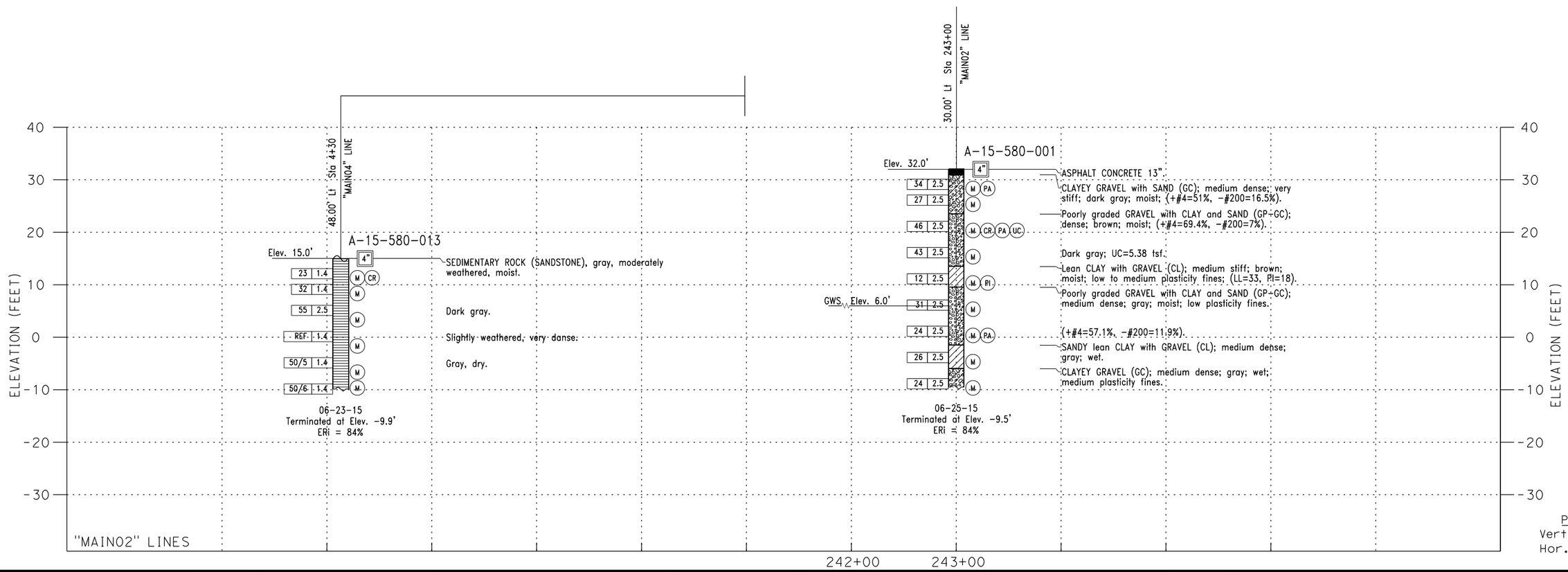

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

X DESIGN OVERSIGHT	DRAWN BY	KIM OUYANG	L.S. BHANGOO FIELD INVESTIGATION BY:
X SIGN OFF DATE	CHECKED BY	KANDEEP SARAVANAPAVAN	DATE: JUNE & DECEMBER 2015 TO MARCH 2016

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

KANDEEP SARAVANAPAVAN
PROJECT ENGINEER

BRIDGE NO.
POST MILES

RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT

LOG OF TEST BORINGS 2 OF 5

GS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT:
PROJECT NUMBER & PHASE:
CONTRACT NO.:

DISREGARD PRINTS BEARING EARLIER REVISION DATES

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	CC/Mrn	580	5.0/7.8, 0.0/3.2		

GEOTECHNICAL PROFESSIONAL — DATE 4/22/16

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. 666 — Exp. 12/31/17 — GEOTECHNICAL — STATE OF CALIFORNIA

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

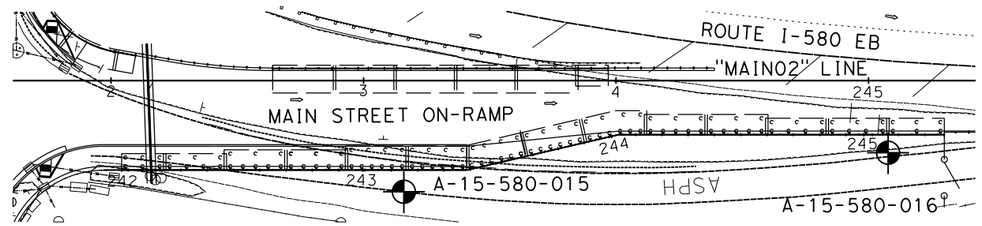
NOTES:

Standard Penetration Test Sampler: I.D. = 1.4"; O.D. = 2" Modified California Sampler: I.D. = 2.5"; O.D. = 3" Hammer Assembly: A 140 lb hammer with a 30" drop (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (2010)

See Caltrans 2010 Standard Plans A10F, A10G and A10H for Soil and Rock Legend.

All dimensions are in feet unless otherwise shown



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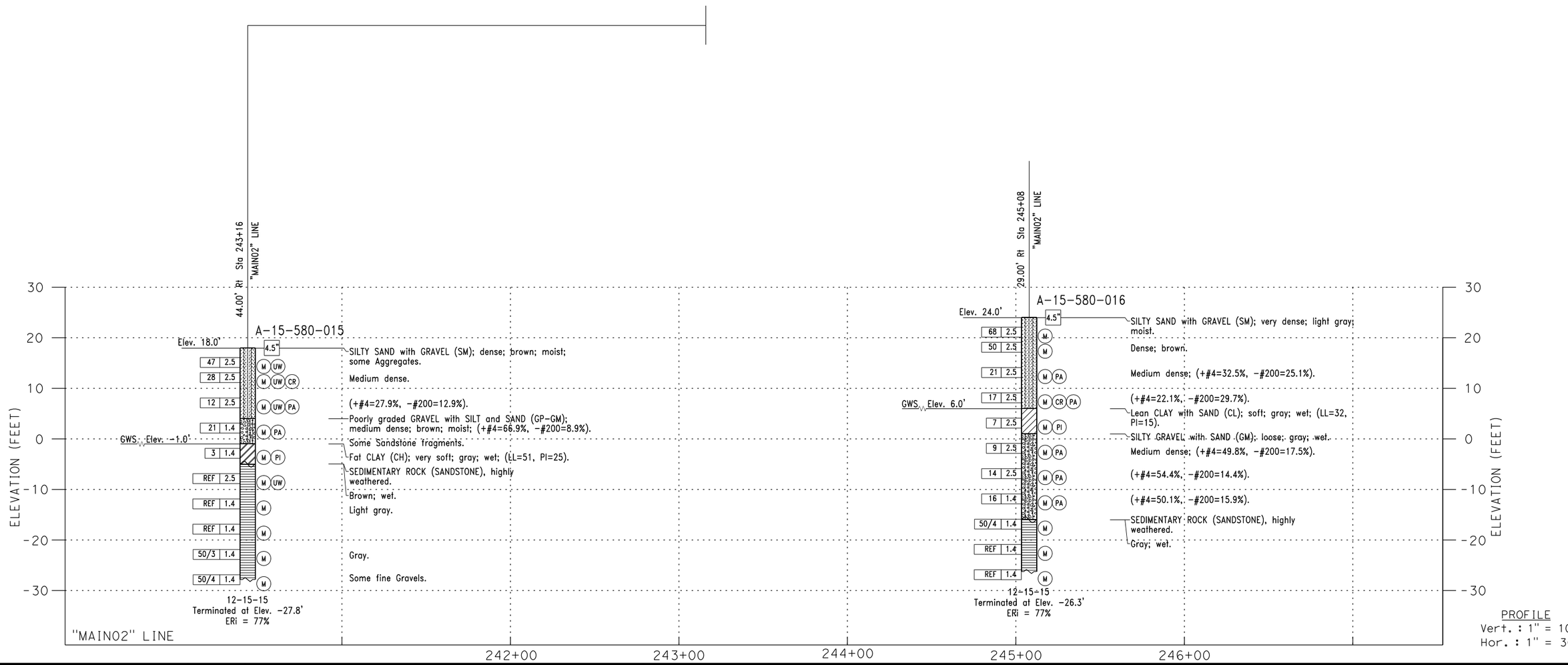

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 30'

X DESIGN OVERSIGHT	DRAWN BY	KIM OUYANG	L.S. BHANGOO FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	KANDEEP SARAVANAPAVAN PROJECT ENGINEER
X SIGN OFF DATE	CHECKED BY	KANDEEP SARAVANAPAVAN	DATE: JUNE & DECEMBER 2015 TO MARCH 2016		BRIDGE NO. / POST MILES

RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT

LOG OF TEST BORINGS 3 OF 5

GS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT:
PROJECT NUMBER & PHASE:
CONTRACT NO.:

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

FILE => 3 of 5.dgn

USERNAME => ogouthier DATE PLOTTED => 4/22/2016 TIME PLOTTED => 1:33:06 PM

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	CC/Mrn	580	5.0/7.8, 0.0/3.2		

GEOTECHNICAL PROFESSIONAL — DATE 4/14/16

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. 666 — Exp. 12/31/17 — GEOTECHNICAL — STATE OF CALIFORNIA

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

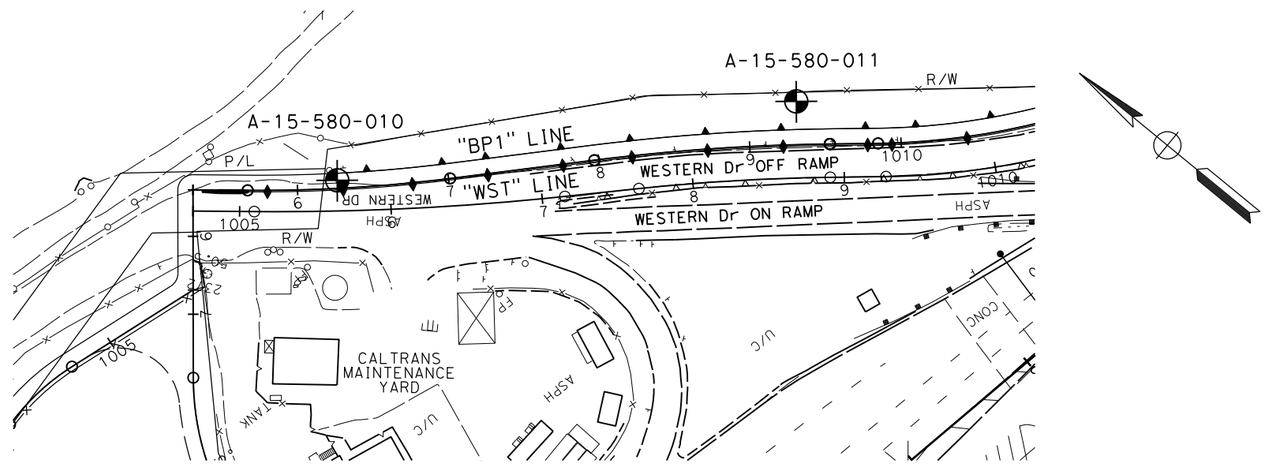
NOTES:

Standard Penetration Test Sampler: I.D. = 1.4"; O.D. = 2" Modified California Sampler: I.D. = 2.5"; O.D. = 3" Hammer Assembly: A 140 lb hammer with a 30" drop (Automatic Hammer)

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (2010)

See Caltrans 2010 Standard Plans A10F, A10G and A10H for Soil and Rock Legend.

All dimensions are in feet unless otherwise shown



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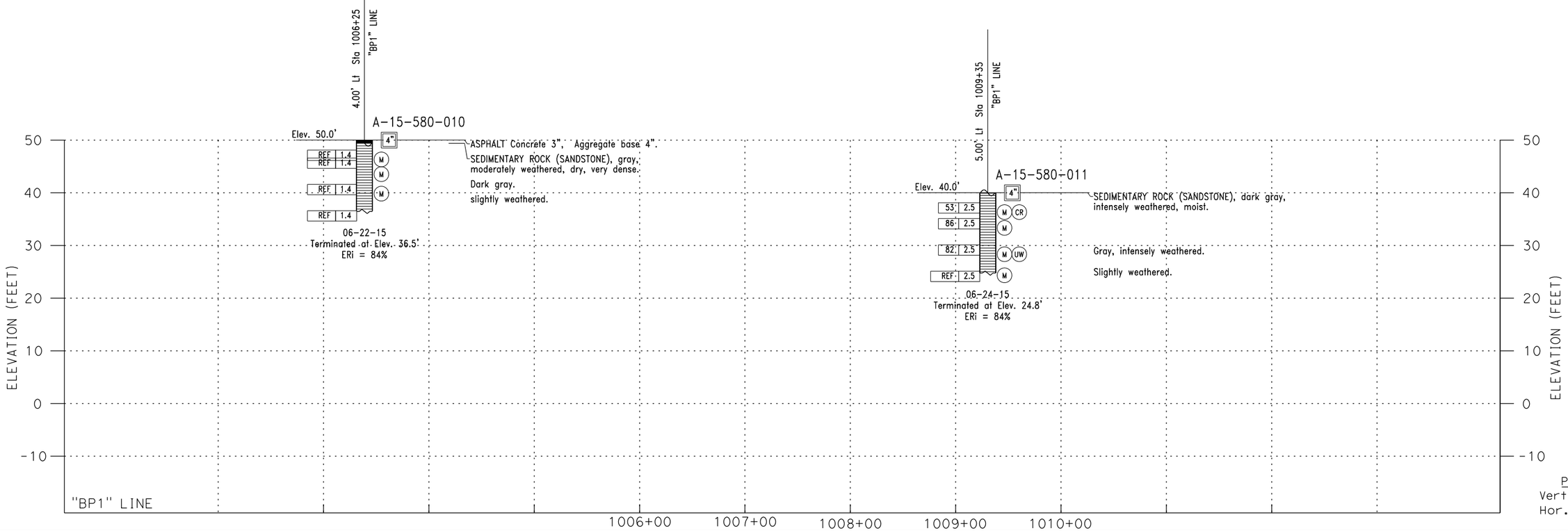

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

X DESIGN OVERSIGHT	DRAWN BY	KIM OUYANG	L.S. BHANGOO FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	KANDEEP SARAVANAPAVAN PROJECT ENGINEER
X SIGN OFF DATE	CHECKED BY	KANDEEP SARAVANAPAVAN	DATE: JUNE & DECEMBER 2015 TO MARCH 2016		

BRIDGE NO. / POST MILES

RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT

LOG OF TEST BORINGS 4 OF 5

GS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT:
PROJECT NUMBER & PHASE:
CONTRACT NO.:

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES — SHEET — OF

FILE => 4 of 5.dgn

USERNAME => oguthier DATE PLOTTED => 4/14/2016 TIME PLOTTED => 4:06:28 PM

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	CC/Mrn	580	5.0/7.8, 0.0/3.2		

GEOTECHNICAL PROFESSIONAL — DATE 4/14/16

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER — GARY PARIKH — No. 666 — Exp. 12/31/17 — GEOTECHNICAL — STATE OF CALIFORNIA

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

NOTES:

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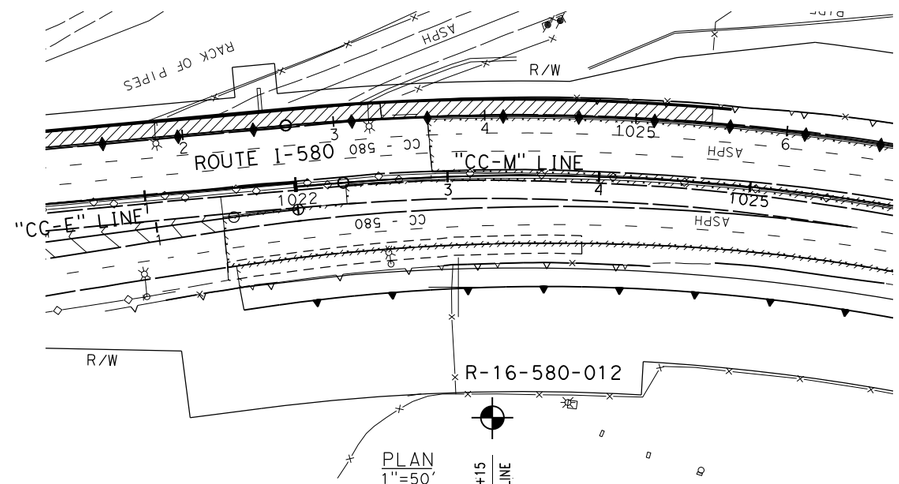
This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock, Logging, Classification, and Presentation Manual (2010)

See Caltrans 2010 Standard Plans A10F, A10G and A10H for Soil and Rock Legend.

All dimensions are in feet unless otherwise shown



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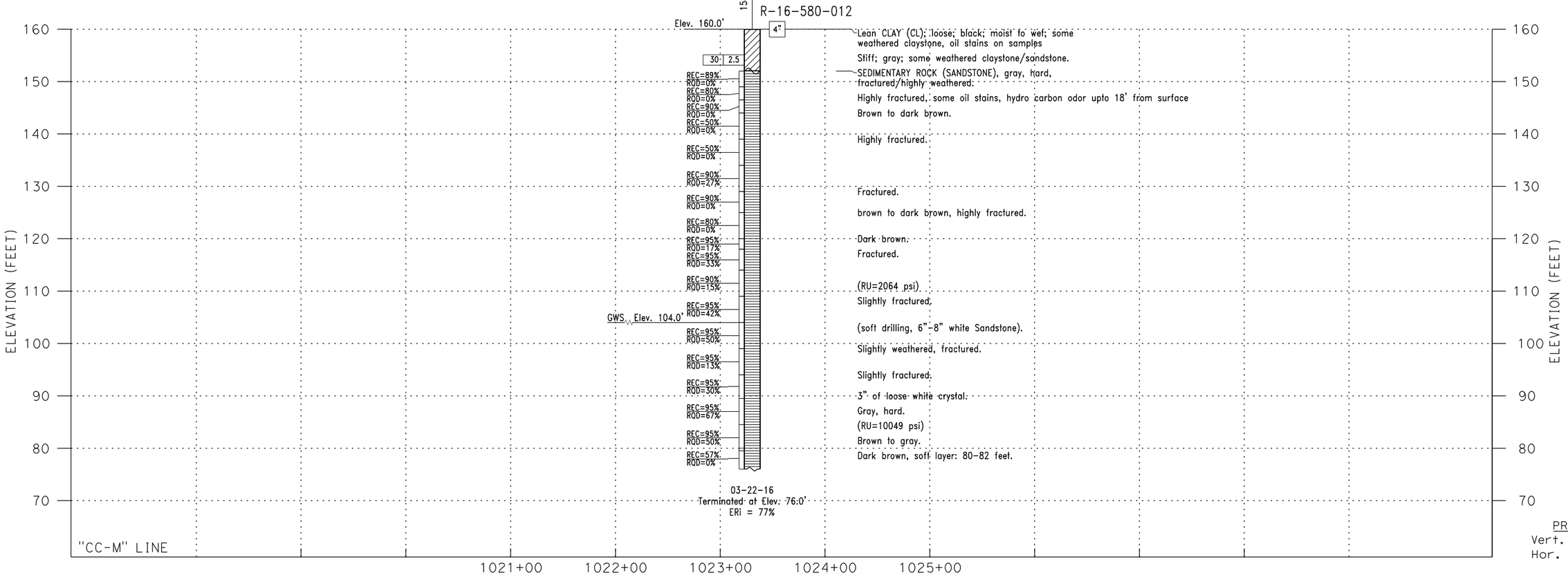

PROFILE
Vert. : 1" = 10'
Hor. : 1" = 50'

X DESIGN OVERSIGHT	DRAWN BY	KIM OUYANG	L.S. BHANGOO FIELD INVESTIGATION BY:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	KANDEEP SARAVANAPAVAN PROJECT ENGINEER
X SIGN OFF DATE	CHECKED BY	KANDEEP SARAVANAPAVAN	DATE: JUNE & DECEMBER 2015 TO MARCH 2016		

BRIDGE NO. / POST MILES

RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT

LOG OF TEST BORINGS 5 OF 5

GS GEOTECHNICAL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 03/14/12)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT:
PROJECT NUMBER & PHASE:
CONTRACT NO.:

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

FILE => 5 of 5.dgn

USERNAME => ogouthier DATE PLOTTED => 4/15/2016 TIME PLOTTED => 10:44:16 AM

APPENDIX B

Laboratory Test Data

APPENDIX B

LABORATORY TESTS

Classification Tests
The field classifications of the samples were verified through visual examination in the laboratory and laboratory testing according to the Unified Soil Classification System (ASTM Test Method 2488). 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 2216-92. 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 and Laboratory Summary Sheet, Plate No: B-2A & B-2B, "Laboratory Test Summary"

Atterberg Limits
The Atterberg Limits were determined for selected samples that had been sieved through No. 40 sieve. These results were used to classify the soils, as well as to obtain an indication of the effective strength characteristics and expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM Test Method D 4318-93. The results of these tests are presented on Plate No: B-3, "Plasticity Chart".

Grain Size Classification
Grain size classification tests (ASTM Test Method D422-63) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plate No: B-4A & B-4B, "Grain Size Distribution Curves".

Corrosion Test
Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistively tests were performed according to California Test Method 643. The tests were performed by Sunland Analytical. The test results are presented on Plate No: B-5A through B-5F.

R-value Tests
R-value tests were performed on selected bulk samples. The tests were performed according to California Test Method 301. The test results are presented on Plate No: B-6A through B-6K.

Compressive Strength Test of Rock Core
Compressive Strength test of rock core were performed on selected core samples. The tests were performed according to ASTM Test Method C 42. The test results are presented on Plate No: B-7A & B-7B.

Borehole	Sample Number	Depth	Classi-fication	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Shear Strength (tsf)
A-15-580-001	1	3.0	GC	5.2	-				51.0	16.5	
A-15-580-001	2	6.0	GC	4.6	-						
A-15-580-001	3	11.0	GP-GC	2.6	-				69.4	7.0	
A-15-580-001	4	16.0	GP-GC	3.9	-						
A-15-580-001	5	21.0	CL	17.9	-	33	18	15			
A-15-580-001	6	26.0	GP-GC	5.9	-						
A-15-580-001	7	31.0	GP-GC	13.1	-				57.1	11.9	
A-15-580-001	8	36.0	CL	8.6	-						
A-15-580-001	9	41.0	GC	15.4	-						
A-15-580-010	1	3.0	-	1.3	-						
A-15-580-010	2	4.5	-	4.9	-						
A-15-580-010	3	9.5	-	1.9	-						
A-15-580-010	4	14.5	-	-	-						
A-15-580-011	1	3.0	-	7.8	-						
A-15-580-011	2	6.0	-	6.0	-						
A-15-580-011	3	11.0	-	15.2	116.0						
A-15-580-011	4	15.0	-	5.2	-						
A-15-580-013	1	3.0	-	5.5	-						
A-15-580-013	2	6.0	-	4.3	-						
A-15-580-013	3	11.0	-	9.4	-						
A-15-580-013	4	16.0	-	1.5	-						
A-15-580-013	5	21.0	-	2.6	-						
A-15-580-013	6	24.0	-	4.1	-						
A-15-580-014	1	5.0	-	6.2	-						
A-15-580-014	CORE 1	7.5	-	-	-						
A-15-580-014	CORE 2	10.0	-	-	-						
A-15-580-014	CORE 3	14.0	-	0.5	-						
A-15-580-014	CORE 4	19.0	-	-	-						
A-15-580-014	CORE 5	23.5	-	-	-						
A-15-580-015	1	3.0	SM	9.2	120.7						
A-15-580-015	2	6.0	SM	8.6	123.9						
A-15-580-015	3	11.0	SM	9.2	95.7				27.9	12.9	
A-15-580-015	4	16.0	GP-GM	8.6	-				66.9	8.9	
A-15-580-015	5	21.0	CH	30.5	-	51	26	25			
A-15-580-015	6	26.0	-	8.2	122.4						
A-15-580-015	7	31.0	-	6.5	-						
A-15-580-015	8	36.0	-	5.7	-						
A-15-580-015	9	41.0	-	8.3	-						
A-15-580-015	10	46.0	-	10.4	-						
A-15-580-016	1	3.0	SM	2.8	-						
A-15-580-016	2	6.0	SM	3.8	-						
A-15-580-016	3	11.0	SM	6.5	-				32.5	25.1	
A-15-580-016	4	16.0	SM	5.7	-				22.1	29.7	

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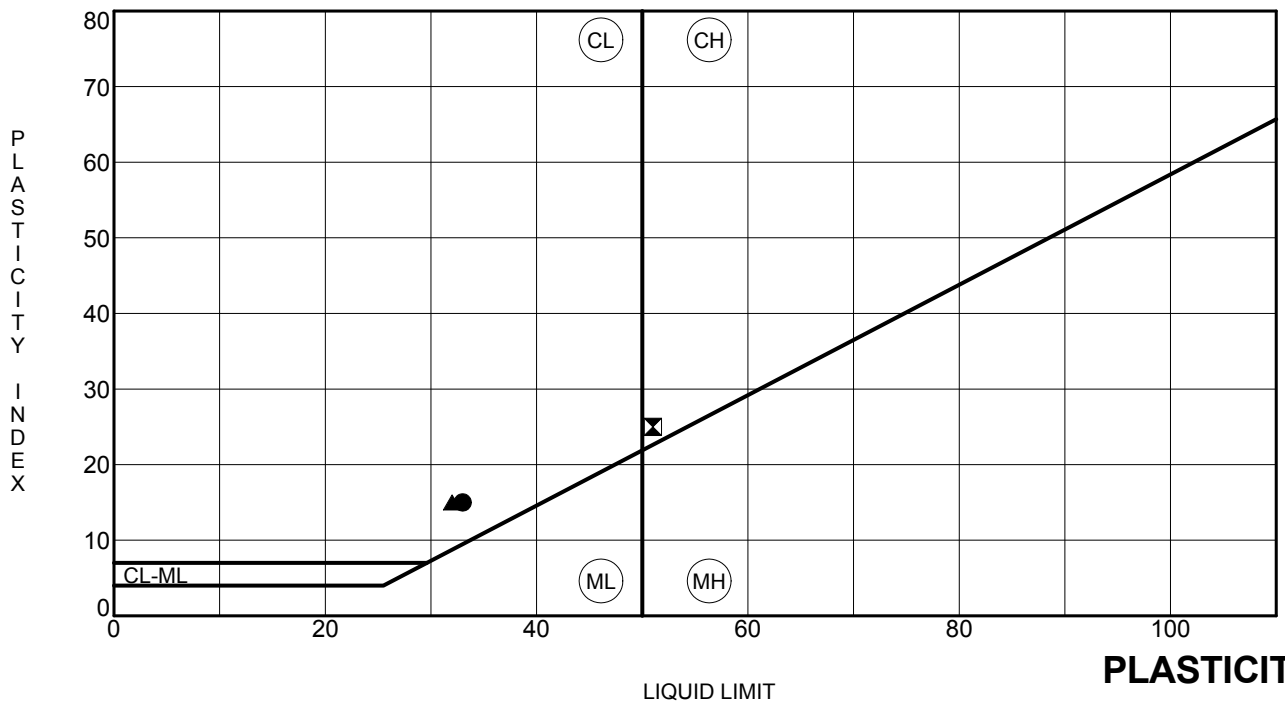
RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMEMT PROJECT
RICHMOND & SAN RAFAEL, CALIFORNIA

JOB NO: 2014-125-GDR | **PLATE NO:** B-2A

Borehole	Sample Number	Depth	Classi-fication	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Shear Strength (tsf)
A-15-580-016	5	21.0	CL	12.1	-	32	17	15			
A-15-580-016	6	26.0	GM	13.1	-				49.8	17.5	
A-15-580-016	7	31.0	GM	9.2	-				54.4	14.4	
A-15-580-016	8	36.0	GM	9.1	-				50.1	15.9	
A-15-580-016	9	41.0	-	11.2	-						
A-15-580-016	10	46.0	-	9.9	-						
A-15-580-016	11	51.0	-	6.1	-						



PLASTICITY

	BOREHOLE	SAMPLE #	DEPTH	LL	PL	PI	Fines	Classification
●	**A-15-580-001**	**5**	**21.0**	**33**	**18**	**15**		**Lean CLAY with GRAVEL**
⧗	**A-15-580-015**	**5**	**21.0**	**51**	**26**	**25**		**Fat CLAY**
▲	**A-15-580-016**	**5**	**21.0**	**32**	**17**	**15**		**Lean CLAY with SAND**

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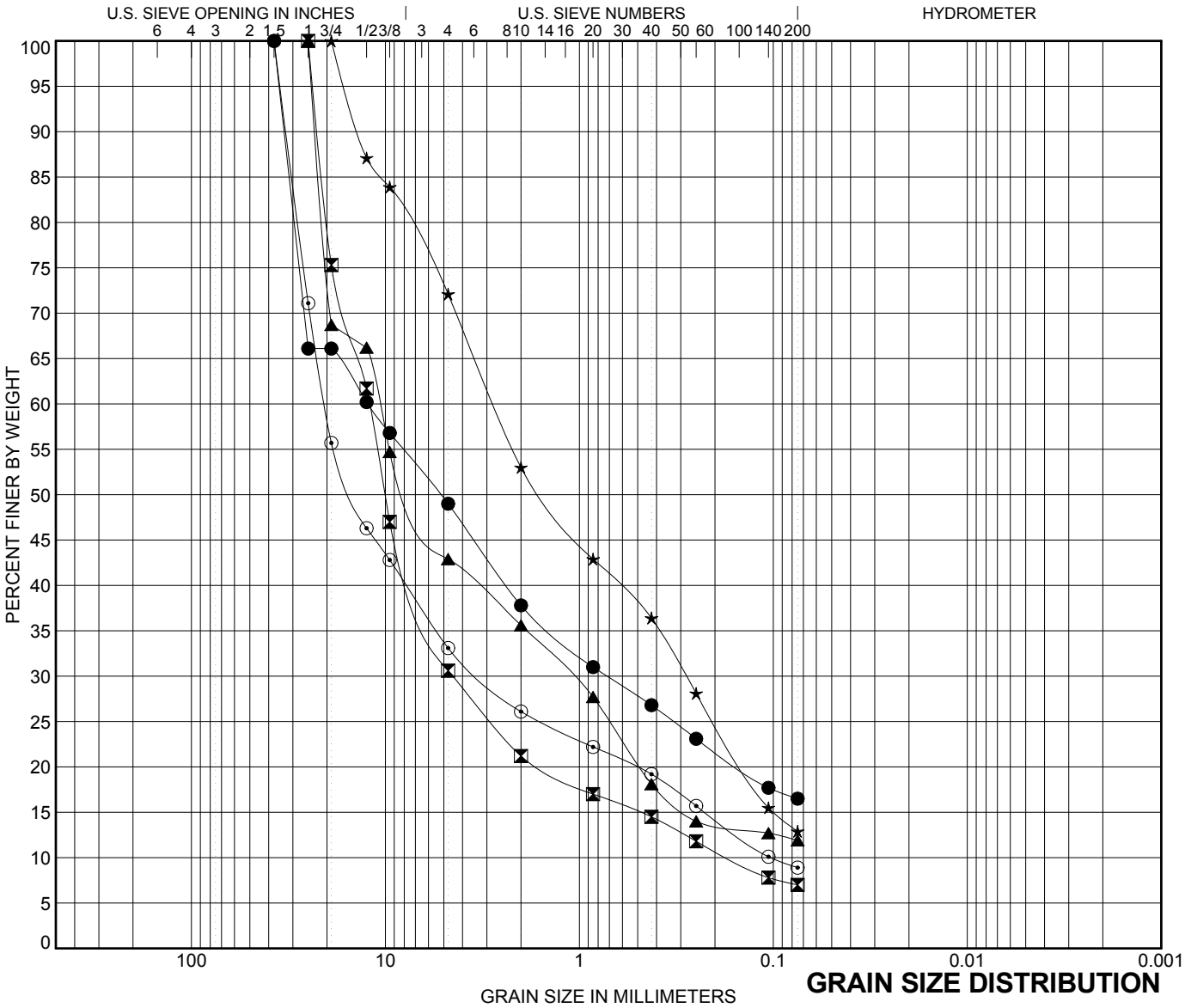
RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMEMT PROJECT
RICHMOND & SAN RAFAEL, CALIFORNIA

JOB NO: 2014-125-GDR | **PLATE NO: B-3**



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	A-15-580-001	1	3.0	CLAYEY GRAVEL with SAND					
⊠	A-15-580-001	3	11.0	Poorly graded GRAVEL with CLAY and SAND				9.82	71.26
▲	A-15-580-001	7	31.0	Poorly graded GRAVEL with CLAY and SAND				3.34	326.91
★	A-15-580-015	3	11.0	SILTY SAND with GRAVEL					
⊙	A-15-580-015	4	16.0	Poorly graded GRAVEL with SILT and SAND				4.96	199.18

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-15-580-001	1	3.0	37.5	12.3	0.721		51.0	32.5	16.5	
⊠	A-15-580-001	3	11.0	25	12.11	4.495	0.17	69.4	23.6	7.0	
▲	A-15-580-001	7	31.0	25	10.781	1.09		57.1	31.0	11.9	
★	A-15-580-015	3	11.0	19	2.746	0.282		27.9	59.2	12.9	
⊙	A-15-580-015	4	16.0	37.5	20.513	3.238	0.103	66.9	24.2	8.9	

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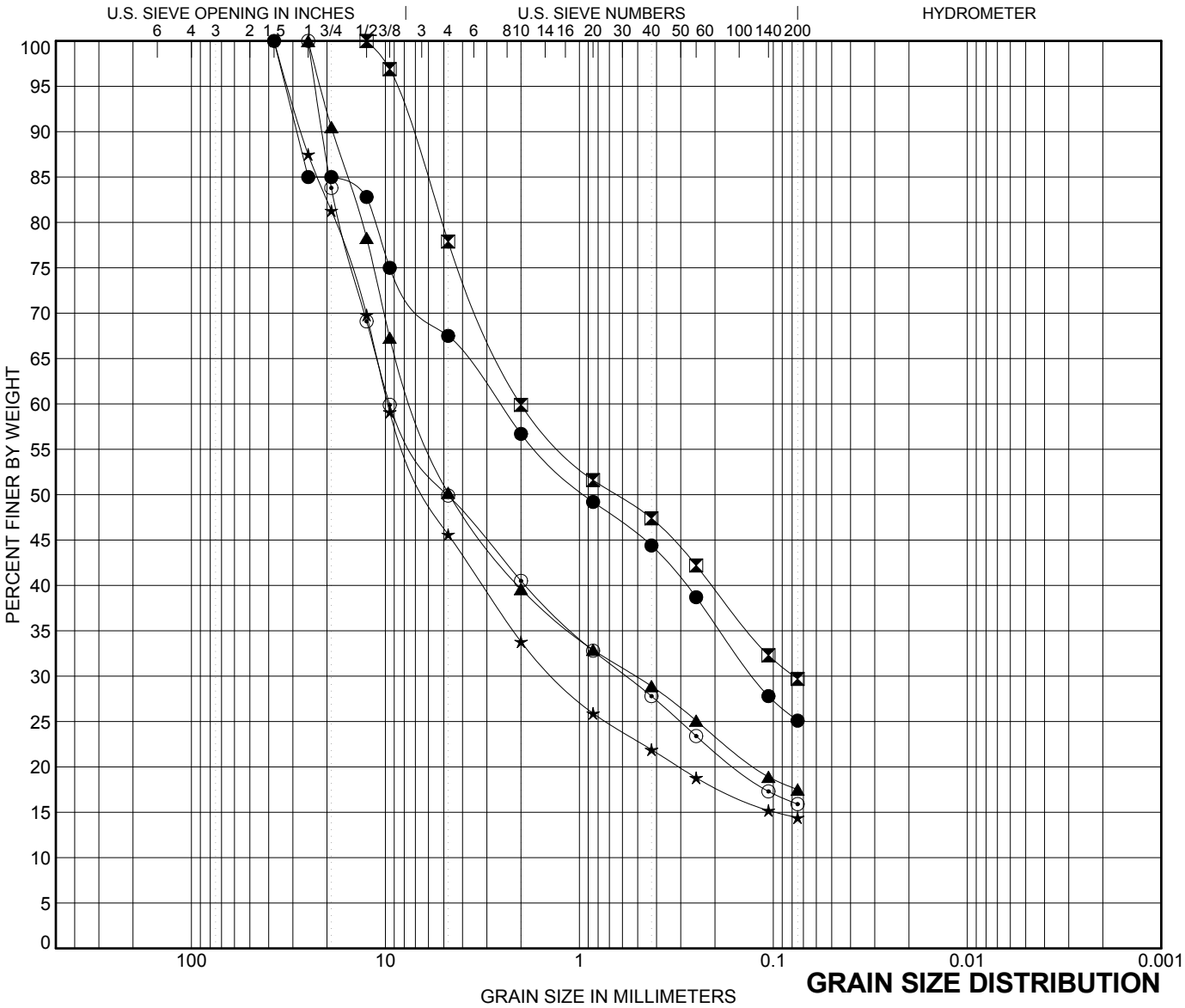
RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMEMT PROJECT
RICHMOND & SAN RAFAEL, CALIFORNIA

JOB NO: 2014-125-GDR | **PLATE NO: B-4A**



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	A-15-580-016	3	11.0	SILTY SAND with GRAVEL					
⊠	A-15-580-016	4	16.0	SILTY SAND with GRAVEL					
▲	A-15-580-016	6	26.0	SILTY GRAVEL with SAND					
★	A-15-580-016	7	31.0	SILTY GRAVEL with SAND					
⊙	A-15-580-016	8	36.0	SILTY GRAVEL with SAND					

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	A-15-580-016	3	11.0	37.5	2.605	0.126		32.5	42.4	25.1	
⊠	A-15-580-016	4	16.0	12.5	2.01	0.078		22.1	48.2	29.7	
▲	A-15-580-016	6	26.0	25	7.067	0.514		49.8	32.7	17.5	
★	A-15-580-016	7	31.0	37.5	9.722	1.325		54.4	31.2	14.4	
⊙	A-15-580-016	8	36.0	25	9.528	0.577		50.1	34.0	15.9	

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Practicing in the Geosciences

RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMEMT PROJECT
RICHMOND & SAN RAFAEL, CALIFORNIA

JOB NO: 2014-125-GDR | **PLATE NO: B-4B**

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/05/2015
Date Submitted 07/31/2015

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

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

The reported analysis was requested for the following location:
Location : 2014-125-GDR Site ID : A15-001-3@11 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 70149-146243.

EVALUATION FOR SOIL CORROSION

Soil pH	7.73	
Minimum Resistivity	1.39 ohm-cm (x1000)	
Chloride	18.2 ppm	00.00182 %
Sulfate	18.7 ppm	00.00187 %

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

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/05/2015
Date Submitted 07/31/2015

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

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

The reported analysis was requested for the following location:
Location : 2014-125-GDR Site ID : A15-011-1@3 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 70149-146244.

EVALUATION FOR SOIL CORROSION

Soil pH	6.94	
Minimum Resistivity	1.54 ohm-cm (x1000)	
Chloride	22.1 ppm	00.00221 %
Sulfate	34.1 ppm	00.00341 %

METHODS

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

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/05/2015
Date Submitted 07/31/2015

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

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

The reported analysis was requested for the following location:
Location : 2014-125-GDR Site ID : A15-013-1@3 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 70149-146245.

EVALUATION FOR SOIL CORROSION

Soil pH	7.41	
Minimum Resistivity	2.22 ohm-cm (x1000)	
Chloride	11.6 ppm	00.00116 %
Sulfate	26.1 ppm	00.00261 %

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

Plate No: B-5C

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/05/2015
Date Submitted 07/31/2015

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

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

The reported analysis was requested for the following location:
Location : 2014-125-GDR Site ID : A15-014-1@5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 70149-146246.

EVALUATION FOR SOIL CORROSION

Soil pH	8.05	
Minimum Resistivity	4.56 ohm-cm (x1000)	
Chloride	5.1 ppm	00.00051 %
Sulfate	9.0 ppm	00.00090 %

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

K

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 01/20/2016
Date Submitted 01/15/2016

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

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

The reported analysis was requested for the following location:
Location : 2014-125-FDN Site ID : A15-580-015
Thank you for your business.

* For future reference to this analysis please use SUN # 71111-148298.

EVALUATION FOR SOIL CORROSION

Soil pH	7.65	
Minimum Resistivity	2.09 ohm-cm (x1000)	
Chloride	12.9 ppm	00.00129 %
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

Plate No: B-5E

K

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 01/20/2016
Date Submitted 01/15/2016

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

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

The reported analysis was requested for the following location:
Location : 2014-125-FDN Site ID : A15-580-016.
Thank you for your business.

* For future reference to this analysis please use SUN # 71111-148299.

EVALUATION FOR SOIL CORROSION

Soil pH	7.56	
Minimum Resistivity	0.35 ohm-cm (x1000)	
Chloride	485.4 ppm	00.04854 %
Sulfate	360.8 ppm	00.03608 %

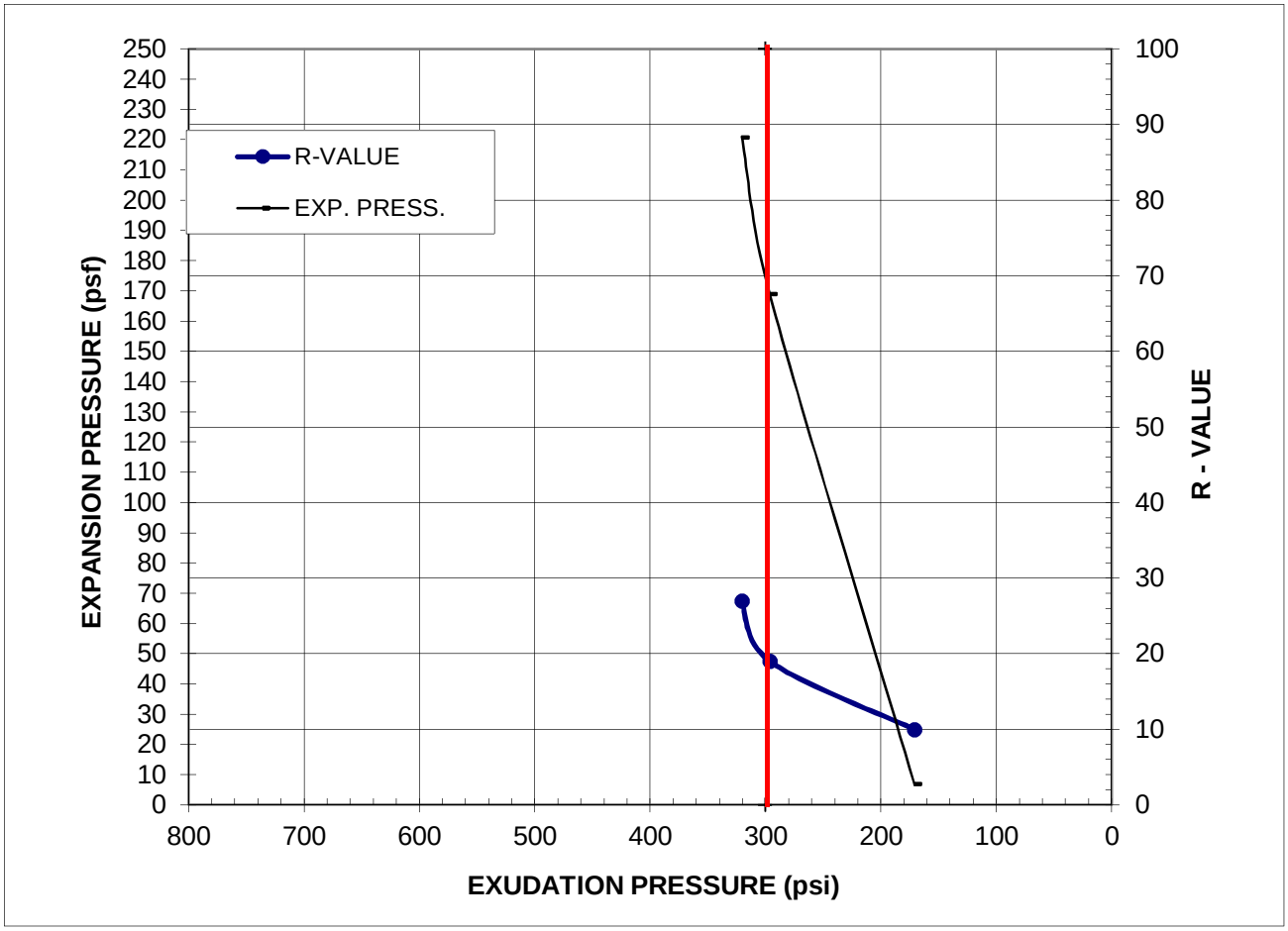
METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Plate No: B-5F

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/7/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-001	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Sandy Lean Clay (CL)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	171.02	296.4	320.5
Expansion Pressure, psf	7	168.9	220.8
R-Value	10	19	27
Moisture Content at Test, %	9.4	8.5	8.0
Dry Density at Test, pcf	130.8	135.0	136.5

R-Value @ 300 psi Exudation Pressure = 19

Expansion Pressure @300 psi Exudation, psf = **170**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

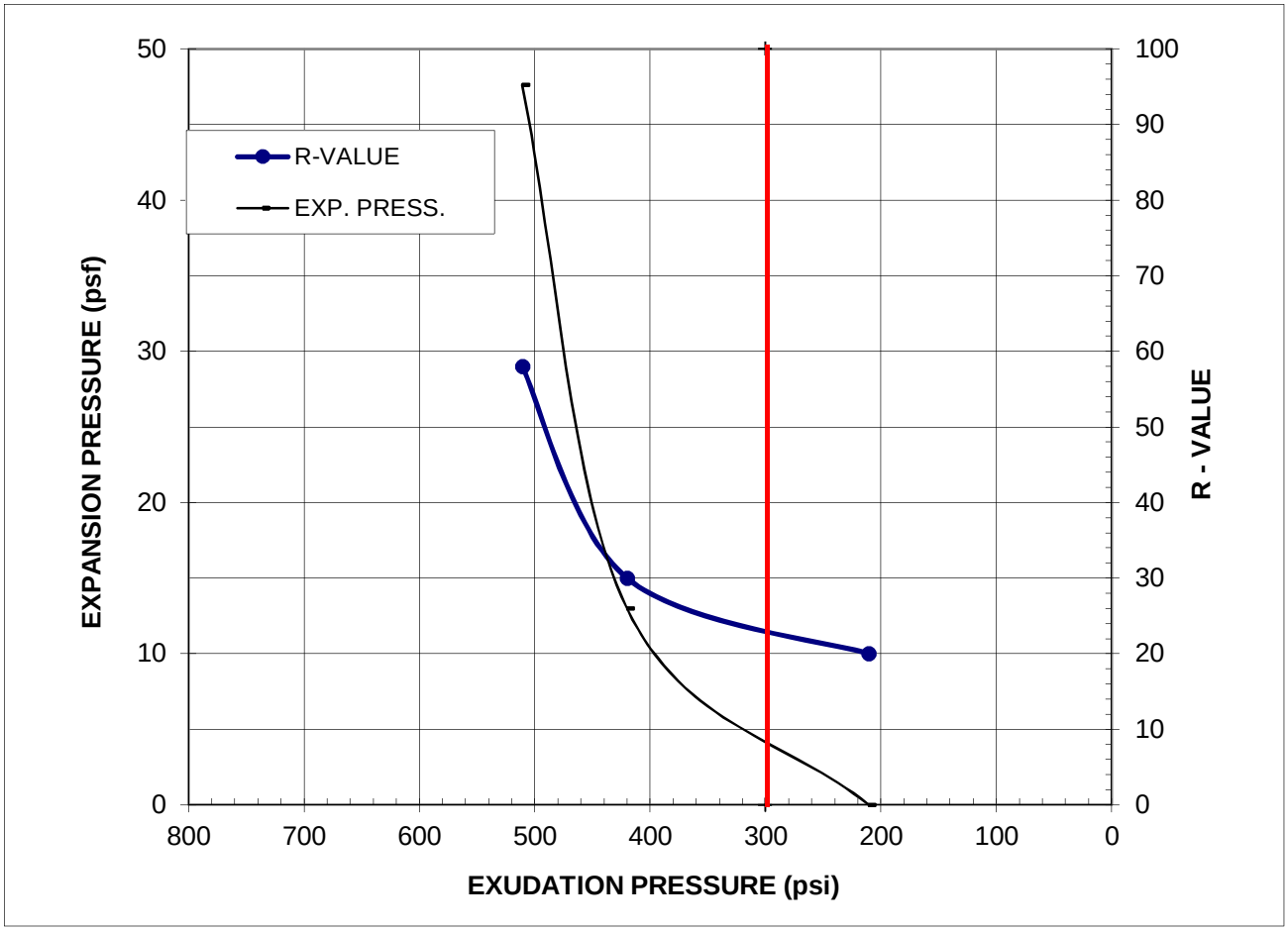
Plate No: B-6A

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/8/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-002	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Sandy Lean Clay with Gravel (CL)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	210.46	420.07	510.87
Expansion Pressure, psf	0	12.99	47.63
R-Value	20	30	58
Moisture Content at Test, %	9.1	8.2	7.4
Dry Density at Test, pcf	132.7	136.1	137.8

R-Value @ 300 psi Exudation Pressure = 23

Expansion Pressure @300 psi Exudation, psf = **8**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

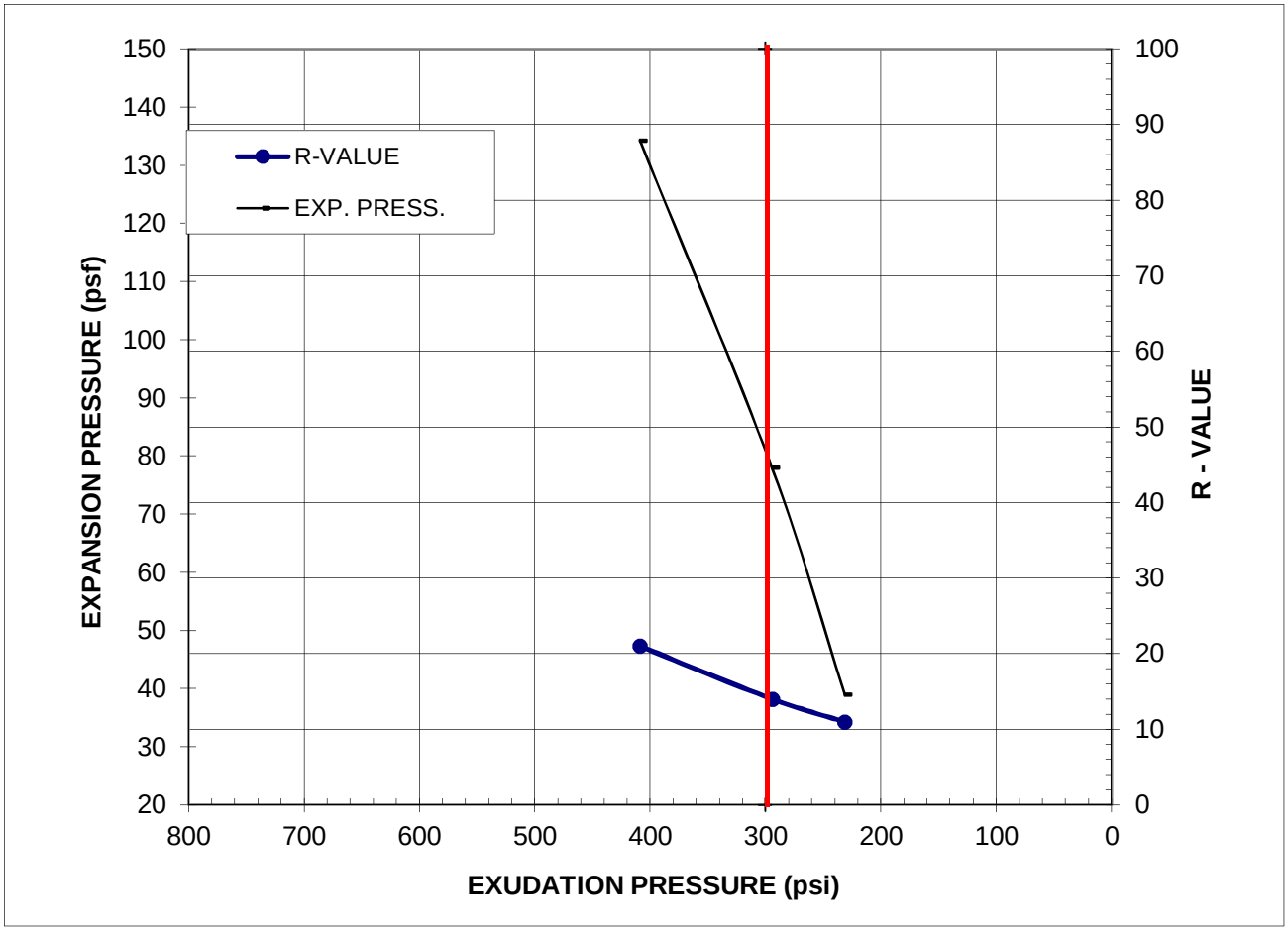
Plate No: B-6B

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/8/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-003	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Sandy Lean Clay (CL)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	231.4	294.35	408.9
Expansion Pressure, psf	38.97	77.94	134.23
R-Value	11	14	21
Moisture Content at Test, %	12.1	11.2	10.3
Dry Density at Test, pcf	124.4	126.2	129.1

R-Value @ 300 psi Exudation Pressure = 14

Expansion Pressure @300 psi Exudation, psf = **79**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

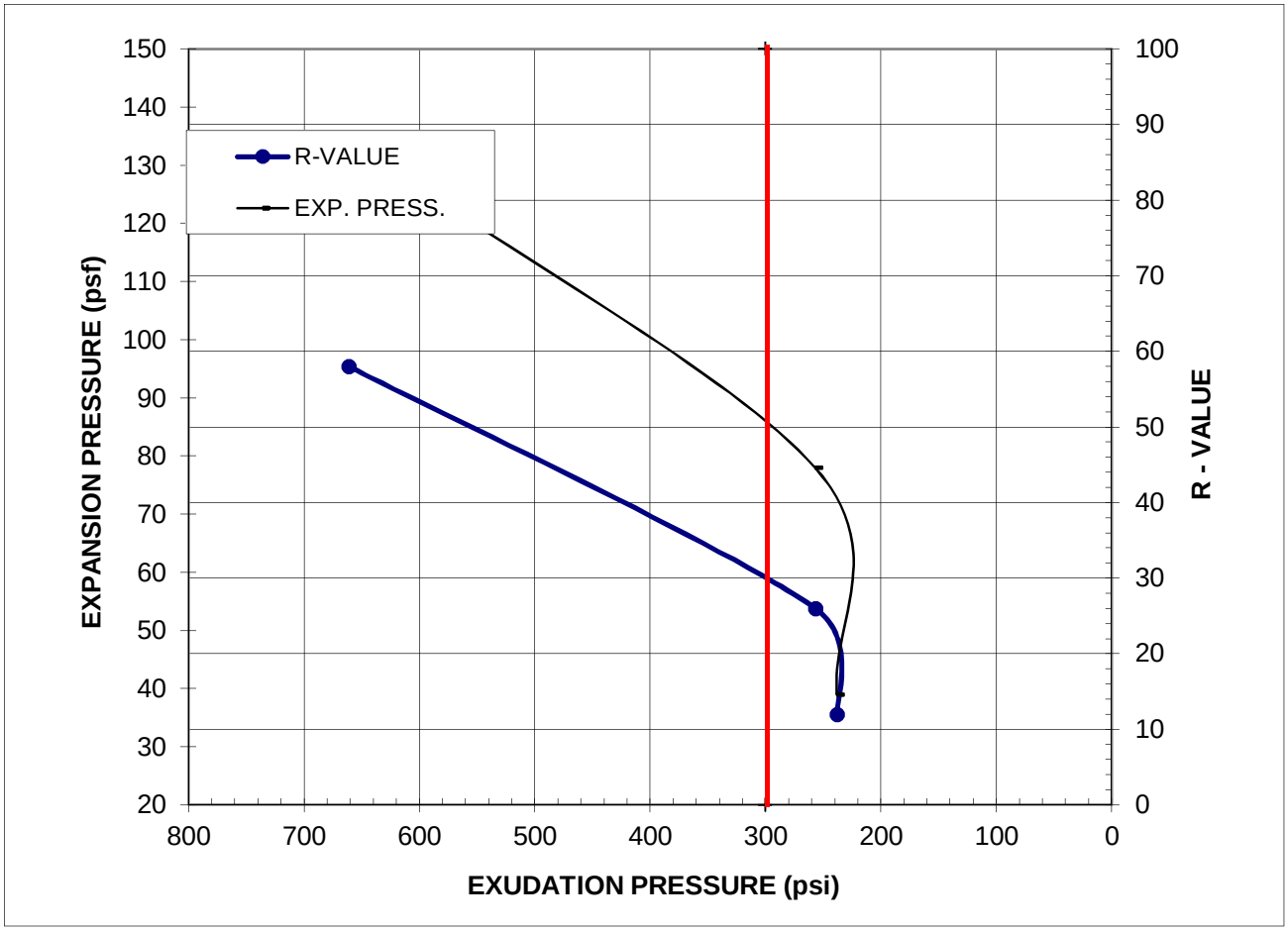
Plate No: B-6C

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/8/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-004	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Sandy Lean Clay with Gravel (CL)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	238.11	256.8	661.3
Expansion Pressure, psf	38.97	77.94	134.23
R-Value	12	26	58
Moisture Content at Test, %	9.6	8.7	7.8
Dry Density at Test, pcf	130.3	134.4	137.1

R-Value @ 300 psi Exudation Pressure = 30

Expansion Pressure @300 psi Exudation, psf = **85**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

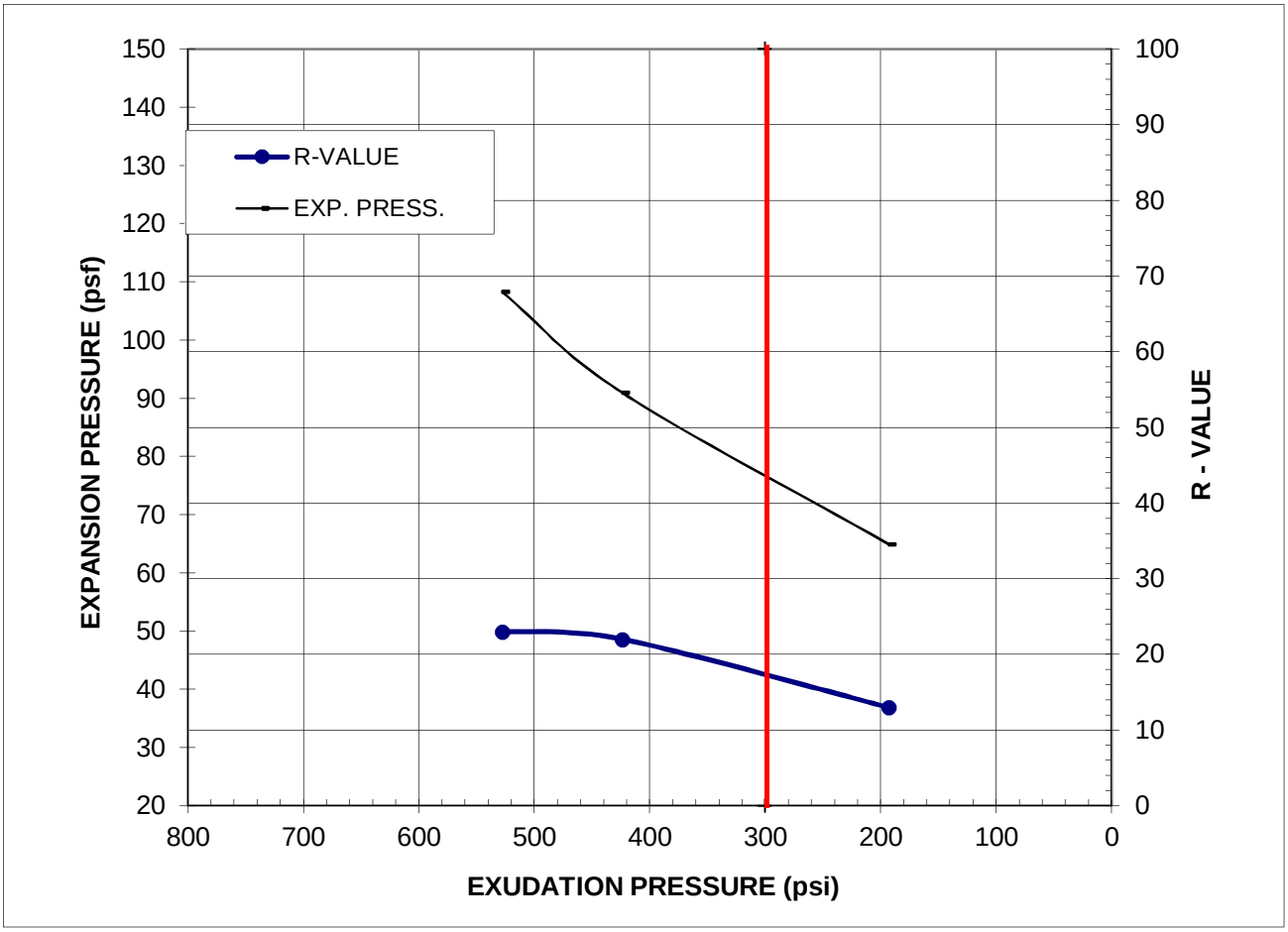
Plate No: B-6D

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/9/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-005	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Sandy Lean Clay (CL)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	193.17	423.7	527.67
Expansion Pressure, psf	64.95	90.93	108.25
R-Value	13	22	23
Moisture Content at Test, %	10.2	9.3	8.8
Dry Density at Test, pcf	129.5	132.4	133.4

R-Value @ 300 psi Exudation Pressure = 16

Expansion Pressure @300 psi Exudation, psf = **76**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

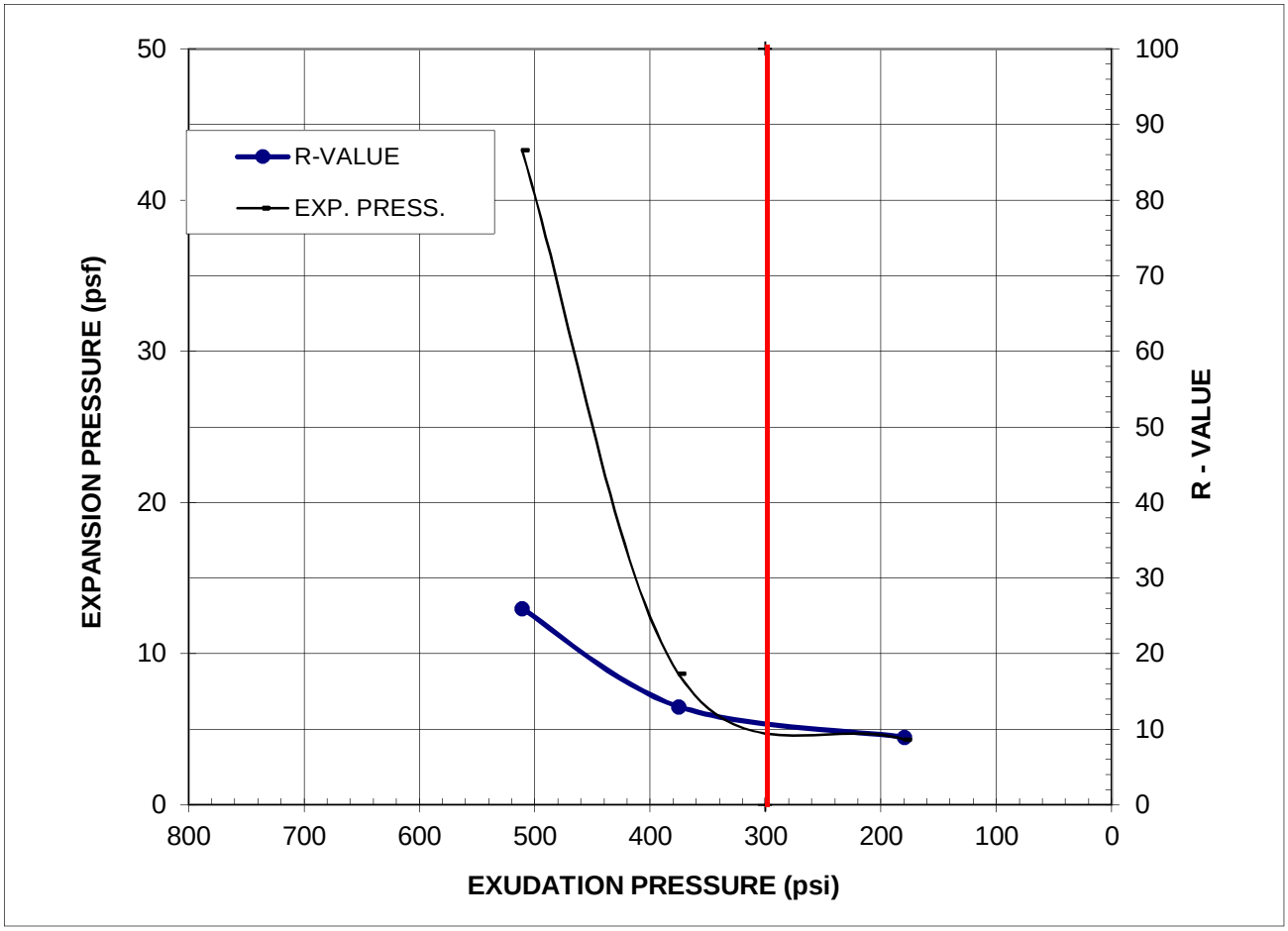
Plate No: B-6E

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/12/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-006	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Sandy Lean Clay (CL)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	179.9	375.3	511.06
Expansion Pressure, psf	4.33	8.66	43.3
R-Value	9	13	26
Moisture Content at Test, %	10.3	9.4	8.6
Dry Density at Test, pcf	132.6	129.7	127.5

R-Value @ 300 psi Exudation Pressure = 11

Expansion Pressure @300 psi Exudation, psf = **8**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

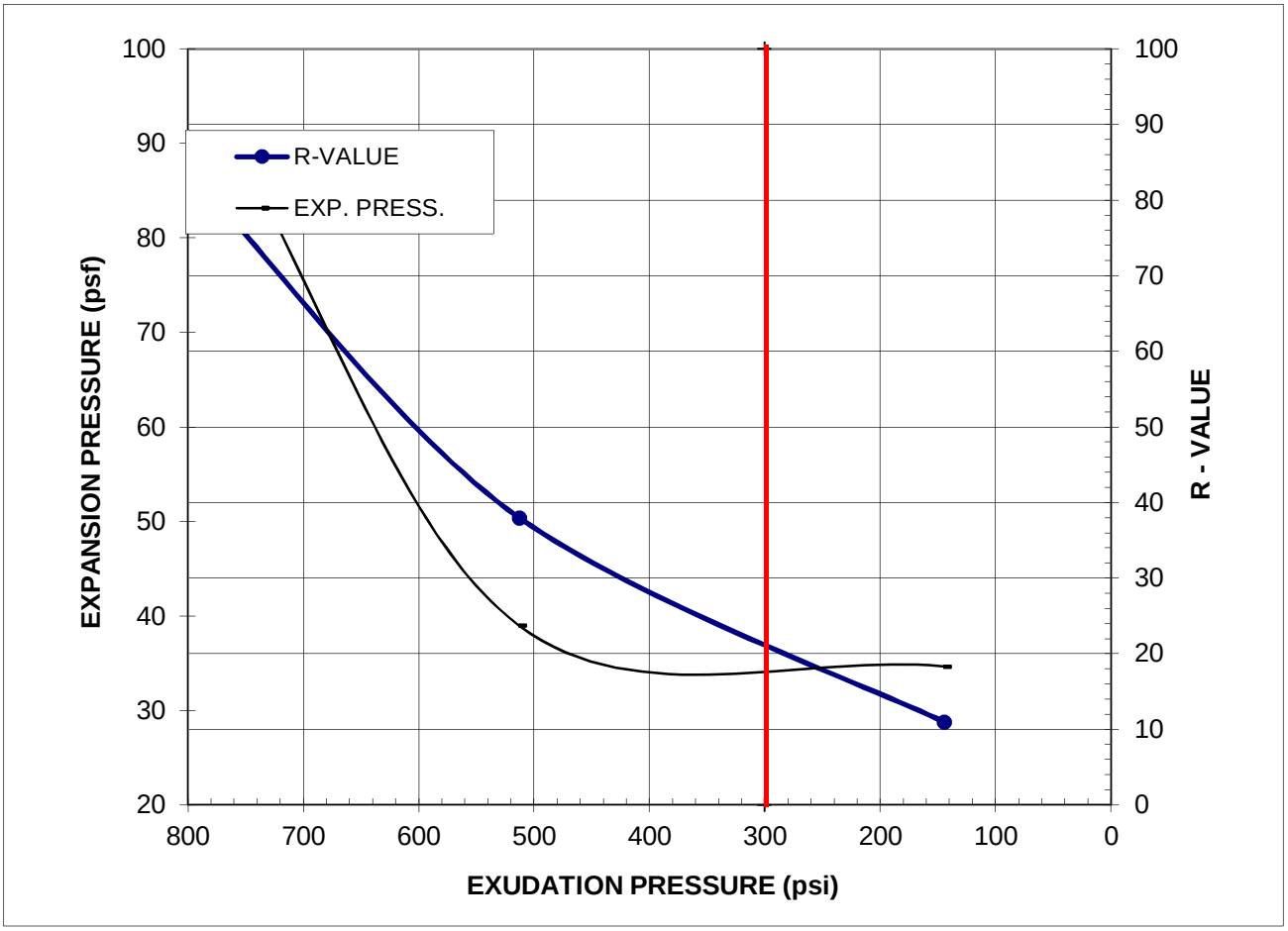
Plate No: B-6F

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement	Date:	7/13/2015
Client:	HNTB	Project #:	2014-125-GDR
Sample #:	A-15-007 Depth: 0'-5'	Lab #:	
Location / Source:	Onsite / Native	Sample Date:	
Material :	Sandy Lean Clay With Gravel (CL)	Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	144.84	513	765.17
Expansion Pressure, psf	34.64	38.97	90.93
R-Value	11	38	78
Moisture Content at Test, %	8.0	7.2	6.3
Dry Density at Test, pcf	134.4	136.9	139.0

R-Value @ 300 psi Exudation Pressure = 20

Expansion Pressure @300 psi Exudation, psf = **34**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

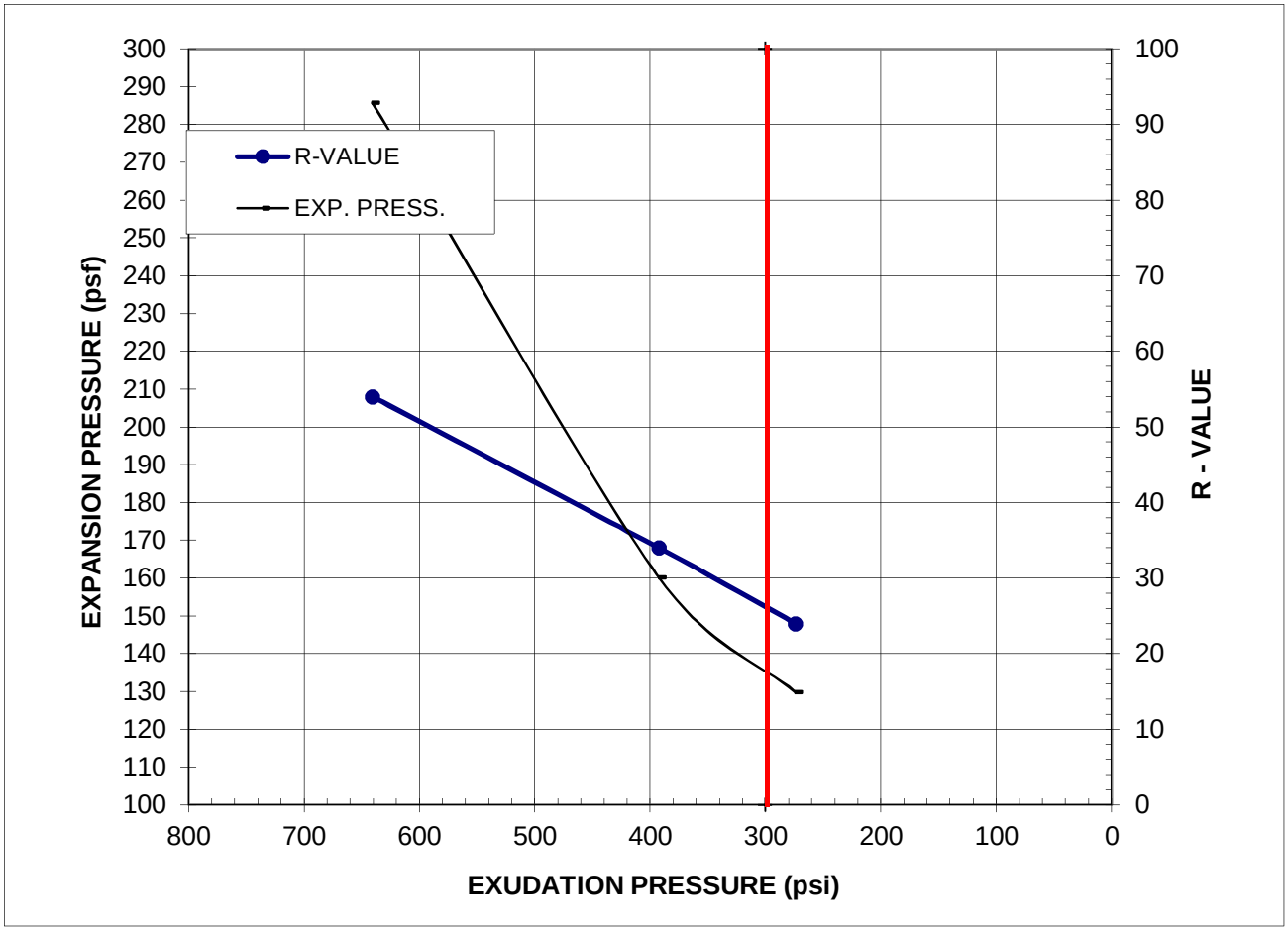
Plate No: B-6G

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement	Date:	7/13/2015
Client:	HNTB	Project #:	2014-125-GDR
Sample #:	A-15-008 Depth: 0'-5'	Lab #:	
Location / Source:	Onsite / Native	Sample Date:	
Material :	Sandy Lean Clay With Gravel (CL)	Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	274.29	392.53	640.8
Expansion Pressure, psf	129.9	160.2	285.78
R-Value	24	34	54
Moisture Content at Test, %	11.5	10.6	9.7
Dry Density at Test, pcf	122.9	125.1	127.5

R-Value @ 300 psi Exudation Pressure = 26

Expansion Pressure @300 psi Exudation, psf = **17**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

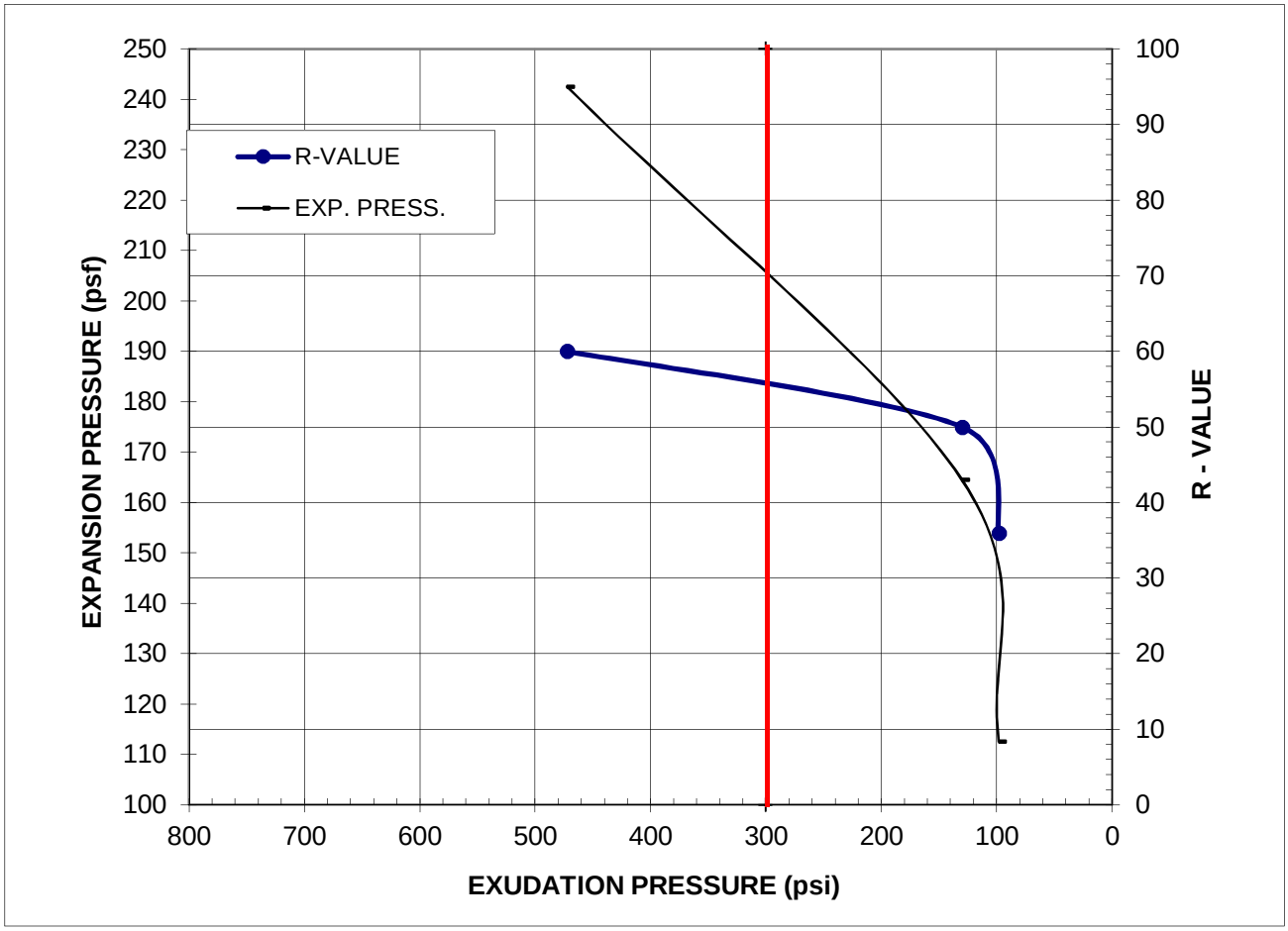
Plate No: B-6H

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/15/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-009	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Silt (ML)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	98.1	130.06	472.3
Expansion Pressure, psf	112.6	164.5	242.5
R-Value	36	50	60
Moisture Content at Test, %	22.0	20.8	18.6
Dry Density at Test, pcf	92.7	96.3	97.1

R-Value @ 300 psi Exudation Pressure = 50

Expansion Pressure @300 psi Exudation, psf = **205**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

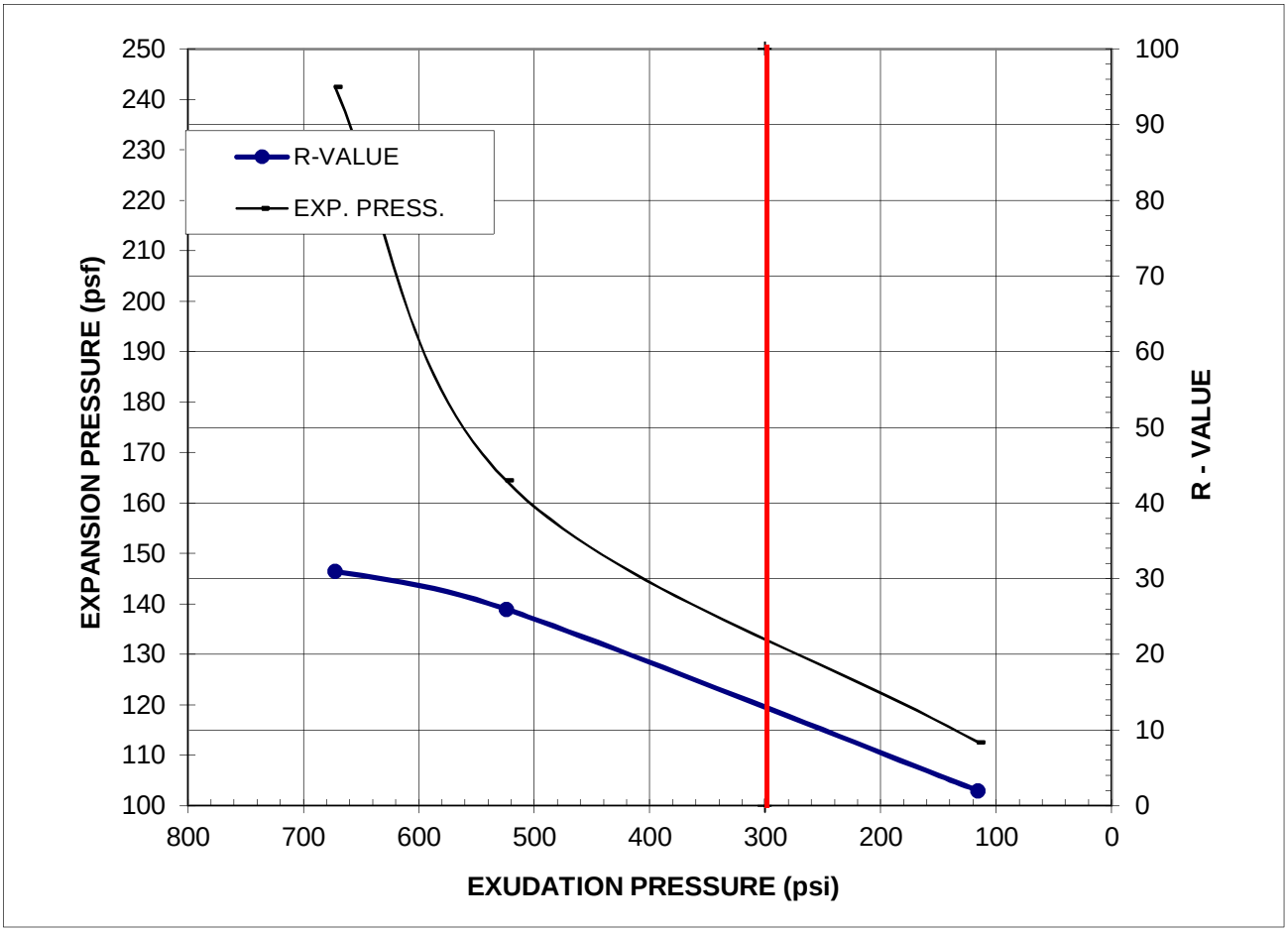
Plate No: B-6l

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/15/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-010	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Lean Clay (CL)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	115.9	524.58	672.7
Expansion Pressure, psf	112.6	164.5	242.5
R-Value	2	26	31
Moisture Content at Test, %	16.9	13.2	12.2
Dry Density at Test, pcf	111.9	126.5	127.7

R-Value @ 300 psi Exudation Pressure = 12

Expansion Pressure @300 psi Exudation, psf = **132**

Minimum R-Value Requirement:

Comments:

Report By: Nasir Ahmad

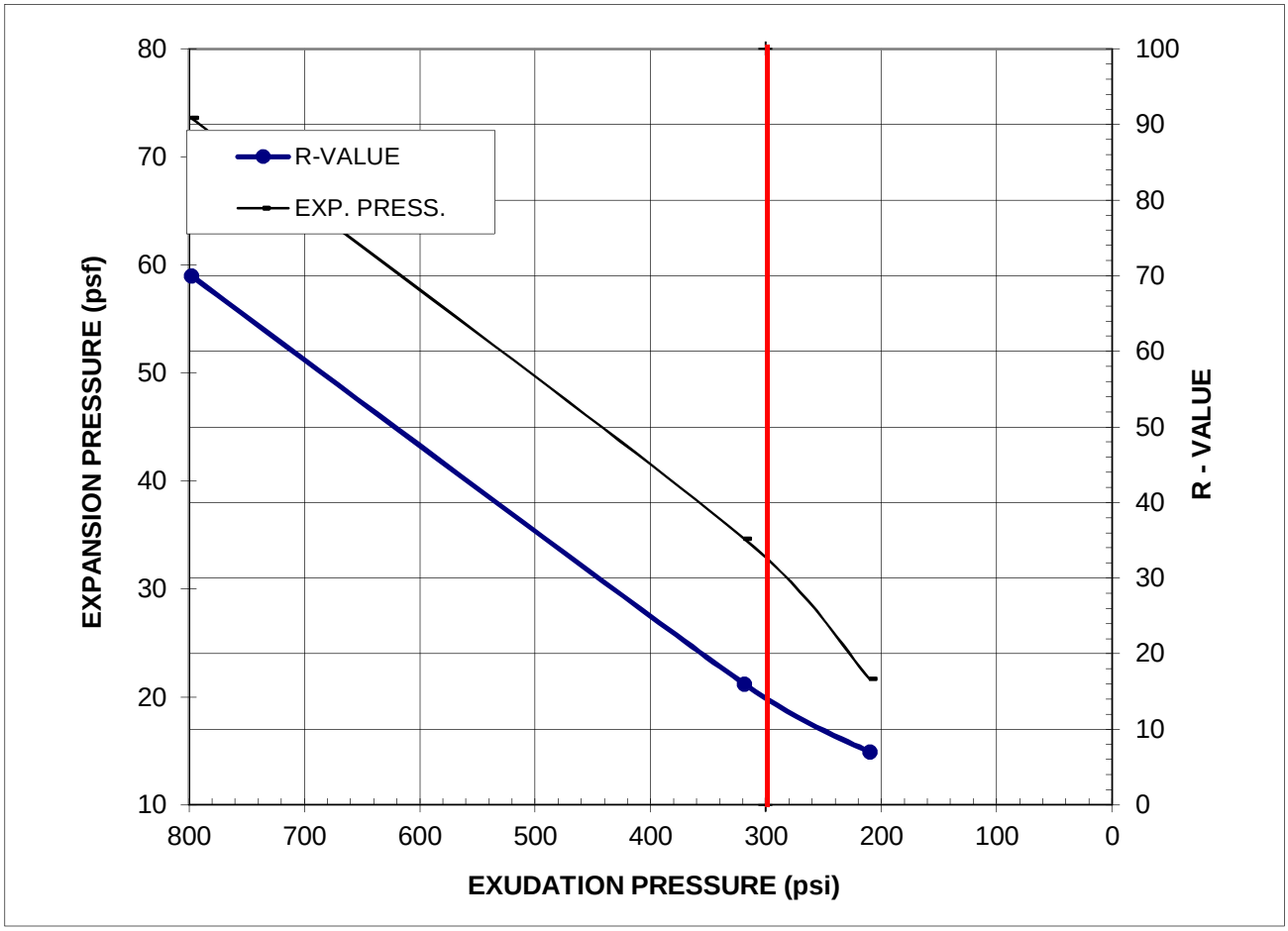
Plate No: B-6J

RVALUE with calcs pdp

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	I-580 Improvement			Date:	7/15/2015
Client:	HNTB			Project #:	2014-125-GDR
Sample #:	A-15-011	Depth:	0'-5'	Lab #:	
Location / Source:		Onsite / Native		Sample Date:	
Material :	Sandy Lean Clay (CL)			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	210	318.63	798.1
Expansion Pressure, psf	21.65	34.64	73.61
R-Value	7	16	70
Moisture Content at Test, %	11.2	10.3	7.6
Dry Density at Test, pcf	124.2	129.0	130.8

R-Value @ 300 psi Exudation Pressure = 14

Expansion Pressure @300 psi Exudation, psf = **33**

Minimum R-Value Requirement:

Comments:

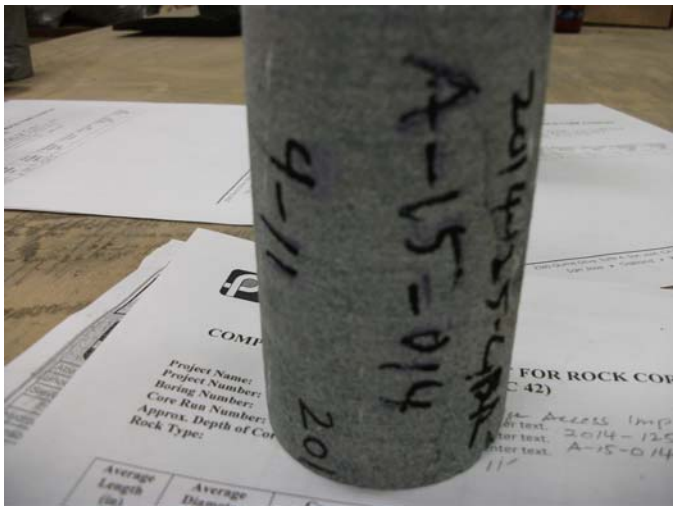
Report By: Nasir Ahmad

Plate No: B-6K

COMPRESSIVE STRENGTH TEST FOR ROCK CORE SAMPLES (ASTM C 42)

Project Name: SRS Bridge Access Improvement
Project Number: 2014-125-GDR
Boring Number: A-15-014
Core Run Number:
Approx. Depth of Core Sample (ft): 9-11'
Rock Type:

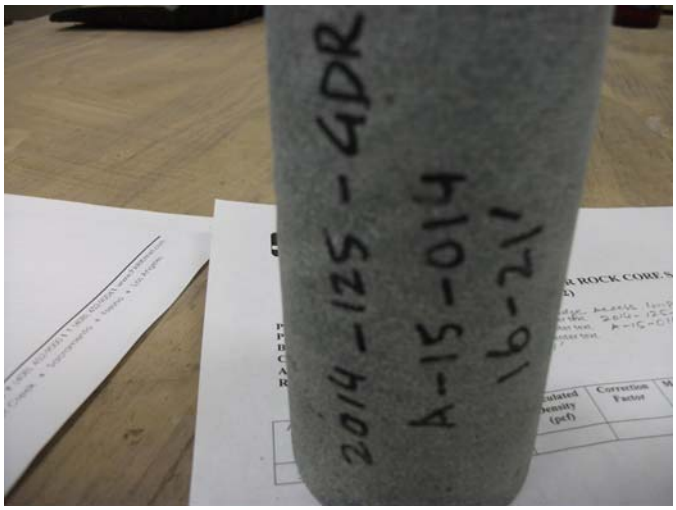
Average Length (in)	Average Diameter (in)	Core Weight (gms)	Calculated Density (pcf)	Correction Factor	Max. Load (lbs)	Compressive Strength (psi)	Corrected Strength (psi)
5.04	2.4	998	166	1	23800	5264	5264



COMPRESSIVE STRENGTH TEST FOR ROCK CORE SAMPLES (ASTM C 42)

Project Name: SRS Bridge Access Improvement
Project Number: 2014-125-GDR
Boring Number: A-15-014
Core Run Number:
Approx. Depth of Core Sample (ft): 16-21'
Rock Type:

Average Length (in)	Average Diameter (in)	Core Weight (gms)	Calculated Density (pcf)	Correction Factor	Max. Load (lbs)	Compressive Strength (psi)	Corrected Strength (psi)
5.02	2.39	999	168.9	1	41830	9329	9329



COMPRESSIVE STRENGTH TEST FOR ROCK CORE SAMPLES (ASTM C 42)

Project Name: Richmond San Rafael Bridge Improvement
Project Number: 2014-125-FDN
Boring Number: A-15-580-012
Core Run Number: 10
Approx. Depth of Core Sample (ft): 49
Rock Type: Meta-Sandstone
Test Date: 4/5/16

Average Length (in)	Average Diameter (in)	Core Weight (gms)	Calculated Density (pcf)	Correction Factor	Max. Load (lbs)	Compressive Strength (psi)	Corrected Strength (psi)
5.0	2.39	984.5	166.8	1	9255	2064	2064

COMPRESSIVE STRENGTH TEST FOR ROCK CORE SAMPLES (ASTM C 42)

Project Name: Richmond San Rafael Bridge Improvement
Project Number: 2014-125-FDN
Boring Number: A-16-580-012
Core Run Number: 15
Approx. Depth of Core Sample (ft): 73
Rock Type: Meta-Sandstone
Test Date: 4/5/16

Average Length (in)	Average Diameter (in)	Core Weight (gms)	Calculated Density (pcf)	Correction Factor	Max. Load (lbs)	Compressive Strength (psi)	Corrected Strength (psi)
4.9	2.4	949	163.3	1	45438	10049	10049

APPENDIX C

Calculations

PAVEMENT DESIGN CALCULATION

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *Main Line*

Design Case: AC over LCB over AS

Design TI=	***12***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	***78***	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0.0032*TI*(100-R_{BS}) =$		3.46	
$GE_{AC+LCB} = 0.0032*TI*(100-R_{AS}) =$		1.92	
$GE_{AC} = 0.4*GE_{AC+LCB} =$		0.77	
=>	$GE'_{AC} =$	0.97	(add 0.2 ft safety factor)
	AC thickness =	0.58	ft
=>	AC Thickness=	**0.80**	ft (round up to the nearest 0.05 ft)
	$G_{f,\ AC}$=	1.88	
	GE_{AC}=	1.50	
$GE_{LCB} = (GE_{AC+LCB})-GE_{AC} =$		0.42	
=>	GE_{AC+AB}=	0.62	(add 0.2 ft safety factor)
	$G_{f,\ LCB}$=	1.90	
	LCB thickness=	0.33	
=>	LCB Thickness=	**0.35**	ft (round up to the nearest 0.05 ft)
	GE_{LCB}=	0.67	
$GE_{AS} = GE_{TOTAL}-GE_{LCB}-GE_{AC} =$		1.29	
=>	AS Thickness=	**1.30**	ft (round up to the nearest 0.05 ft)

Design Section:

AC	0.80	ft
LCB	0.35	ft
AS	1.30	ft
Base Soil		

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *Main Line*

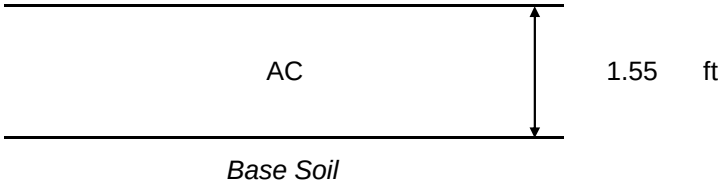
Design Case: Full depth AC

Design TI= ***12*** (20- Yr Pavement Design Life)
R_{BS}= ***10***

$GE_{AC} = 0.0032*TI*(100-R_{BS}) =$ 3.46
=> GE'_{AC}= 3.56 (add 0.1 ft safety factor)
=> AC Thickness= 1.53

=> AC Thickness= **1.55** ft (round up to the nearest 0.05 ft)

Design Section:



PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *Main Line*

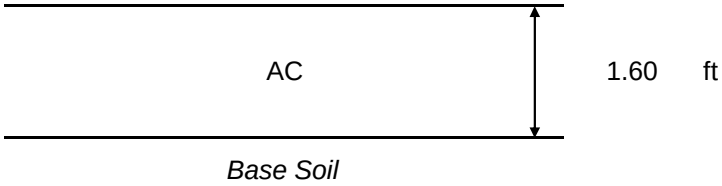
Design Case: Full depth AC

Design TI= ***12.5*** (20- Yr Pavement Design Life)
R_{BS}= ***10***

$GE_{AC} = 0.0032*TI*(100-R_{BS}) =$ 3.60
=> GE'_{AC}= 3.70 (add 0.1 ft safety factor)
=> AC Thickness= 1.60

=> AC Thickness= **1.60** ft (round up to the nearest 0.05 ft)

Design Section:



PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *Main Line*

Design Case: AC over LCB over AS

Design TI=	***12.5***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	***78***	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0.0032*TI*(100-R_{BS}) =$		3.60	
$GE_{AC+LCB} = 0.0032*TI*(100-R_{AS}) =$		2.00	
$GE_{AC} = 0.4*GE_{AC+LCB} =$		0.80	
=>	GE'_{AC} =	1.00	(add 0.2 ft safety factor)
	AC thickness =	0.60	ft
=>	AC Thickness=	**0.85**	ft (round up to the nearest 0.05 ft)
	$G_{f,\ AC}$=	1.88	
	GE_{AC}=	1.59	
$GE_{LCB} = (GE_{AC+LCB})-GE_{AC} =$		0.41	
=>	GE_{AC+AB}=	0.61	(add 0.2 ft safety factor)
	$G_{f,\ LCB}$=	1.90	
	LCB thickness=	0.32	
=>	LCB Thickness=	**0.35**	ft (round up to the nearest 0.05 ft)
	GE_{LCB}=	0.67	
$GE_{AS} = GE_{TOTAL}-GE_{LCB}-GE_{AC} =$		1.34	
=>	AS Thickness=	**1.35**	ft (round up to the nearest 0.05 ft)

Design Section:

AC	0.85	ft
LCB	0.35	ft
AS	1.35	ft
Base Soil		

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *I-580 (EB) Main St/San Quentin On-Ramp*

Design Case: AC over AB over AS

Design TI=	***10***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$ 2.88

$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$ 0.70

=> GE'_{AC} = 0.90 (add 0.2 ft safety factor)

AC thickness = 0.51 ft

=> AC Thickness= **0.55** ft (round up to the nearest 0.05 ft)

$G_{f,\ AC}$= 1.81

GE_{AC}= 1.00

$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$ 1.60

=> GE_{AC+AB}= 1.80 (add 0.2 ft safety factor)

$GE_{AB} = GE_{AC+AB} - GE_{AC} =$ 0.80

=> AB thickness= 0.73

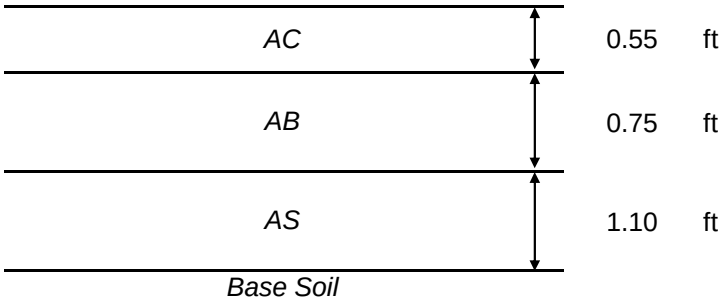
=> AB Thickness= **0.75** ft (round up to the nearest 0.05 ft)

GE_{AB}= 0.83 $G_{f,\ AB}$=1.1

$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$ 1.06

=> AS Thickness= **1.10** ft (round up to the nearest 0.05 ft)

Design Section:



0.55 ft

0.75 ft

1.10 ft

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *I-580 (EB) Marine Off-Ramp*

Design Case: AC over AB over AS

Design TI=	***10.5***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$		3.02	
$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$		0.74	
=>	GE'_{AC} =	0.94	(add 0.2 ft safety factor)
	AC thickness =	0.54	ft
=>	AC Thickness=	**0.55**	ft (round up to the nearest 0.05 ft)
	$G_{f,\ AC}$=	1.77	
	GE_{AC}=	0.97	
$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$		1.68	
=>	GE_{AC+AB}=	1.88	(add 0.2 ft safety factor)
$GE_{AB} = GE_{AC+AB} - GE_{AC} =$		0.91	
=>	AB thickness=	0.82	
=>	AB Thickness=	**0.85**	ft (round up to the nearest 0.05 ft)
	GE_{AB}=	0.94	$G_{f,\ AB}$=1.1
$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$		1.12	
=>	AS Thickness=	**1.15**	ft (round up to the nearest 0.05 ft)

Design Section:

AC	0.55	ft
AB	0.85	ft
AS	1.15	ft
Base Soil		

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *E. Standard Ave (Eastbound)*

Design Case: AC over LCB over AS

Design TI=	***9***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	***78***	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0.0032*TI*(100-R_{BS}) =$		2.59	
$GE_{AC+LCB} = 0.0032*TI*(100-R_{AS}) =$		1.44	
$GE_{AC} = 0.4*GE_{AC+LCB} =$		0.58	
=>	GE'_{AC} =	0.78	(add 0.2 ft safety factor)
	AC thickness =	0.41	ft
=>	AC Thickness=	**0.45**	ft (round up to the nearest 0.05 ft)
	$G_{f,\ AC}$=	1.89	
	GE_{AC}=	0.85	
$GE_{LCB} = (GE_{AC+LCB})-GE_{AC} =$		0.59	
=>	GE_{AC+AB}=	0.79	(add 0.2 ft safety factor)
	$G_{f,\ LCB}$=	1.90	
	LCB thickness=	0.42	
=>	LCB Thickness=	**0.45**	ft (round up to the nearest 0.05 ft)
	GE_{LCB}=	0.86	
$GE_{AS} = GE_{TOTAL}-GE_{LCB}-GE_{AC} =$		0.89	
=>	AS Thickness=	**0.90**	ft (round up to the nearest 0.05 ft)

Design Section:

AC	0.45	ft
LCB	0.45	ft
AS	0.90	ft
Base Soil		

LCB and AS layer thickness were adjusted considering 0.25' ATPB layer to match existing pavement. Adjusted section is 0.45' HMA over 0.25' ATPB over 0.35 LCB over 0.65 AS

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *E. Standard Ave (Westbound)*

Design Case: AC over LCB over AS

Design TI=	***11***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	***78***	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0.0032*TI*(100-R_{BS}) =$		3.17	
$GE_{AC+LCB} = 0.0032*TI*(100-R_{AS}) =$		1.76	
$GE_{AC} = 0.4*GE_{AC+LCB} =$		0.70	
=>	GE'_{AC} =	0.90	(add 0.2 ft safety factor)
	AC thickness =	0.53	ft
=>	AC Thickness=	**0.55**	ft (round up to the nearest 0.05 ft)
	$G_{f,\ AC}$=	1.73	
	GE_{AC}=	0.95	
$GE_{LCB} = (GE_{AC+LCB})-GE_{AC} =$		0.81	
=>	GE_{AC+AB}=	1.01	(add 0.2 ft safety factor)
	$G_{f,\ LCB}$=	1.90	
	LCB thickness=	0.53	
=>	LCB Thickness=	**0.55**	ft (round up to the nearest 0.05 ft)
	GE_{LCB}=	1.05	
$GE_{AS} = GE_{TOTAL}-GE_{LCB}-GE_{AC} =$		1.17	
=>	AS Thickness=	**1.20**	ft (round up to the nearest 0.05 ft)

Design Section:

AC	0.55	ft
LCB	0.55	ft
AS	1.20	ft
Base Soil		

LCB and AS layer thickness were adjusted considering 0.25' ATPB layer to match existing pavement. Adjusted section is 0.45' HMA over 0.25' ATPB over 0.10' HMA over 0.35 LCB over 0.65 AS

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *Bike Path*

Design Case: AC over AB over AS

Design TI=	***5***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$ 1.44

$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$ 0.35

=> GE'_{AC} = 0.55 (add 0.2 ft safety factor)

AC thickness = 0.22 ft

=> AC Thickness= **0.25** ft (round up to the nearest 0.05 ft)

$G_{f,\,AC}$= 2.54

GE_{AC}= 0.63

$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$ 0.80

=> GE_{AC+AB}= 1.00 (add 0.2 ft safety factor)

$GE_{AB} = GE_{AC+AB} - GE_{AC} =$ 0.37

=> AB thickness= 0.33

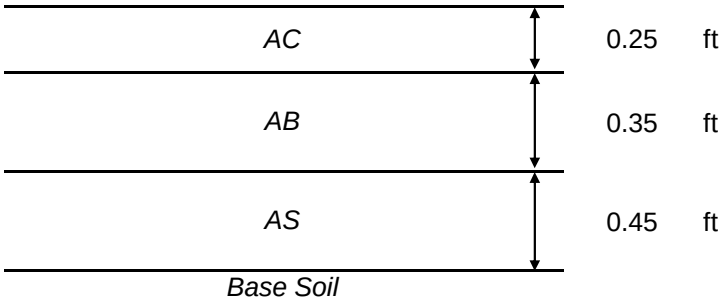
=> AB Thickness= **0.35** ft (round up to the nearest 0.05 ft)

GE_{AB}= 0.39 $G_{f,\,AB}$=1.1

$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$ 0.42

=> AS Thickness= **0.45** ft (round up to the nearest 0.05 ft)

Design Section:



0.25 ft

0.35 ft

0.45 ft

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

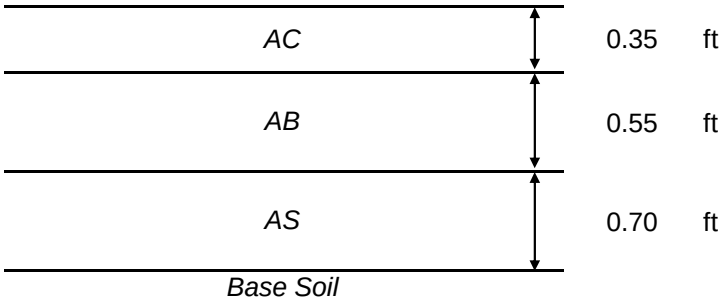
PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *I-580 (WB) Western Dr Off-Ramp*

Design Case: AC over AB over AS

Design TI=	***7***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$		2.02	
$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$		0.49	
=>	$GE'_{AC} =$	0.69	(add 0.2 ft safety factor)
	AC thickness =	0.32	ft
=>	AC Thickness=	**0.35**	ft (round up to the nearest 0.05 ft)
	$G_{f,\ AC}$=	2.14	
	GE_{AC}=	0.75	
$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$		1.12	
=>	GE_{AC+AB}=	1.32	(add 0.2 ft safety factor)
$GE_{AB} = GE_{AC+AB} - GE_{AC} =$		0.57	
=>	AB thickness=	0.52	
=>	AB Thickness=	**0.55**	ft (round up to the nearest 0.05 ft)
	GE_{AB}=	0.61	$G_{f,\ AB}$=1.1
$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$		0.66	
=>	AS Thickness=	**0.70**	ft (round up to the nearest 0.05 ft)

Design Section:



0.35 ft

0.55 ft

0.70 ft

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *I-580 (WB) Main St Off-Ramp*

Design Case: AC over AB over AS

Design TI=	***8***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$ 2.30

$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$ 0.56

=> GE'_{AC} = 0.76 (add 0.2 ft safety factor)

AC thickness = 0.38 ft

=> AC Thickness= **0.40** ft (round up to the nearest 0.05 ft)

$G_{f,\ AC}$= 2.00

GE_{AC}= 0.80

$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$ 1.28

=> GE_{AC+AB}= 1.48 (add 0.2 ft safety factor)

$GE_{AB} = GE_{AC+AB} - GE_{AC} =$ 0.68

=> AB thickness= 0.62

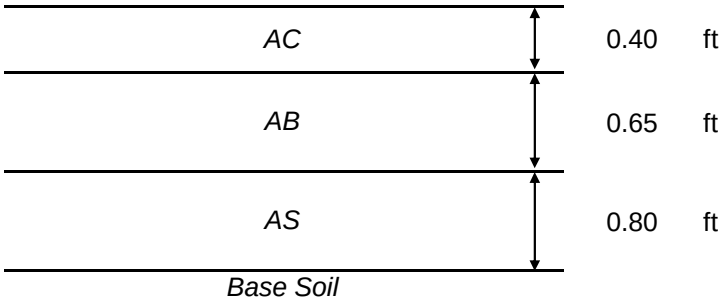
=> AB Thickness= **0.65** ft (round up to the nearest 0.05 ft)

GE_{AB}= 0.72 $G_{f,\ AB}$=1.1

$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$ 0.79

=> AS Thickness= **0.80** ft (round up to the nearest 0.05 ft)

Design Section:



0.40 ft

0.65 ft

0.80 ft

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *Main St*

Design Case: AC over AB over AS

Design TI=	***9.5***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$		2.74	
$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$		0.67	
=>	$GE'_{AC} =$	0.87	(add 0.2 ft safety factor)
	AC thickness =	0.47	ft
=>	AC Thickness=	**0.50**	ft (round up to the nearest 0.05 ft)
	$G_{f,\ AC}$=	1.84	
	GE_{AC}=	0.92	
$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$		1.52	
=>	GE_{AC+AB}=	1.72	(add 0.2 ft safety factor)
$GE_{AB} = GE_{AC+AB} - GE_{AC} =$		0.80	
=>	AB thickness=	0.73	
=>	AB Thickness=	**0.75**	ft (round up to the nearest 0.05 ft)
	GE_{AB}=	0.83	$G_{f,\ AB}$=1.1
$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$		0.99	
=>	AS Thickness=	**1.00**	ft (round up to the nearest 0.05 ft)

Design Section:

AC	0.50	ft
AB	0.75	ft
AS	1.00	ft
Base Soil		

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *Main Street*

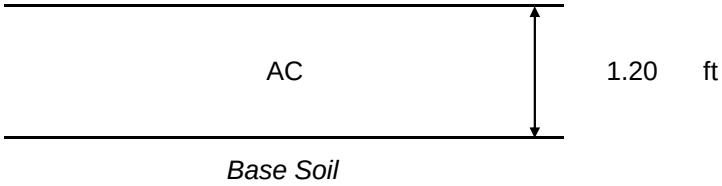
Design Case: Full depth AC

Design TI= ***9.5*** (20- Yr Pavement Design Life)
R_{BS}= ***10***

$GE_{AC} = 0.0032*TI*(100-R_{BS}) =$ 2.74
=> GE'_{AC}= 2.84 (add 0.1 ft safety factor)
=> AC Thickness= 1.18

=> AC Thickness= **1.20** ft (round up to the nearest 0.05 ft)

Design Section:



PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *Bike Path*

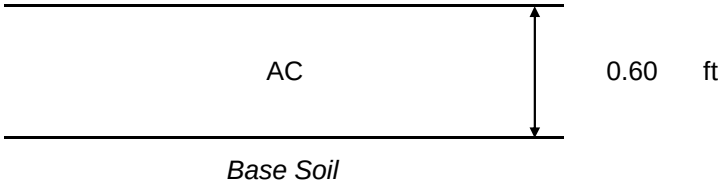
Design Case: Full depth AC

Design TI= ***5*** (20- Yr Pavement Design Life)
R_{BS}= ***10***

$GE_{AC} = 0.0032*TI*(100-R_{BS}) =$ 1.44
=> GE'_{AC}= 1.54 (add 0.1 ft safety factor)
=> AC Thickness= 0.59

=> AC Thickness= **0.60** ft (round up to the nearest 0.05 ft)

Design Section:



PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *I-580 (WB) Western Drive Off-Ramp*

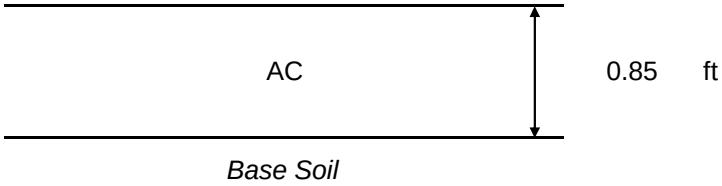
Design Case: Full depth AC

Design TI= ***7*** (20- Yr Pavement Design Life)
R_{BS}= ***10***

$GE_{AC} = 0.0032*TI*(100-R_{BS}) =$ 2.02
=> GE'_{AC}= 2.12 (add 0.1 ft safety factor)
=> AC Thickness= 0.85

=> AC Thickness= **0.85** ft (round up to the nearest 0.05 ft)

Design Section:



PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *Castro Street (Southbound)*

Design Case: AC over AB over AS

Design TI=	***11***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$ 3.17

$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$ 0.77

=> GE'_{AC} = 0.97 (add 0.2 ft safety factor)

AC thickness = 0.56 ft

=> AC Thickness= **0.60** ft (round up to the nearest 0.05 ft)

$G_{f,\,AC}$= 1.78

GE_{AC}= 1.07

$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$ 1.76

=> GE_{AC+AB}= 1.96 (add 0.2 ft safety factor)

$GE_{AB} = GE_{AC+AB} - GE_{AC} =$ 0.89

=> AB thickness= 0.81

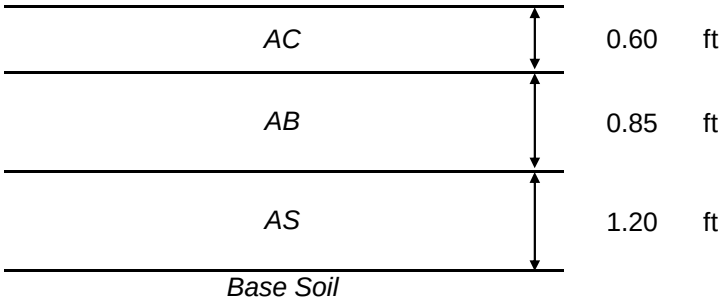
=> AB Thickness= **0.85** ft (round up to the nearest 0.05 ft)

GE_{AB}= 0.94 $G_{f,\,AB}$=1.1

$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$ 1.16

=> AS Thickness= **1.20** ft (round up to the nearest 0.05 ft)

Design Section:



0.60 ft

0.85 ft

1.20 ft

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *Castro Street (Northbound)*

Design Case: AC over AB over AS

Design TI=	***9***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$ 2.59

$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$ 0.63

=> GE'_{AC} = 0.83 (add 0.2 ft safety factor)

AC thickness = 0.44 ft

=> AC Thickness= **0.45** ft (round up to the nearest 0.05 ft)

$G_{f,\ AC}$= 1.89

GE_{AC}= 0.85

$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$ 1.44

=> GE_{AC+AB}= 1.64 (add 0.2 ft safety factor)

$GE_{AB} = GE_{AC+AB} - GE_{AC} =$ 0.79

=> AB thickness= 0.72

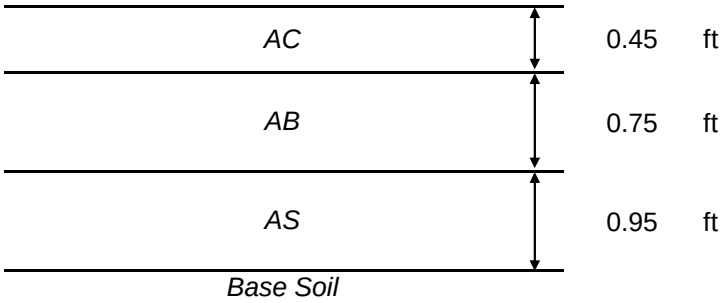
=> AB Thickness= **0.75** ft (round up to the nearest 0.05 ft)

GE_{AB}= 0.83 $G_{f,\ AB}$=1.1

$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$ 0.92

=> AS Thickness= **0.95** ft (round up to the nearest 0.05 ft)

Design Section:



0.45 ft

0.75 ft

0.95 ft

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *Castro Street (Southbound)*

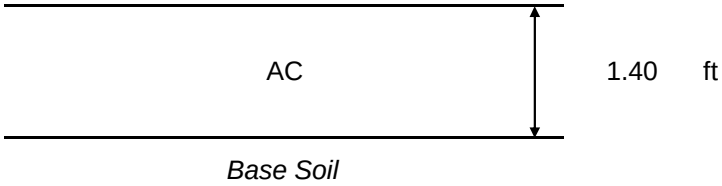
Design Case: Full depth AC

Design TI= ***11*** (20- Yr Pavement Design Life)
R_{BS}= ***10***

$GE_{AC} = 0.0032*TI*(100-R_{BS}) =$ 3.17
=> GE'_{AC}= 3.27 (add 0.1 ft safety factor)
=> AC Thickness= 1.39

=> AC Thickness= **1.40** ft (round up to the nearest 0.05 ft)

Design Section:



PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *Castro Street (Northbound)*

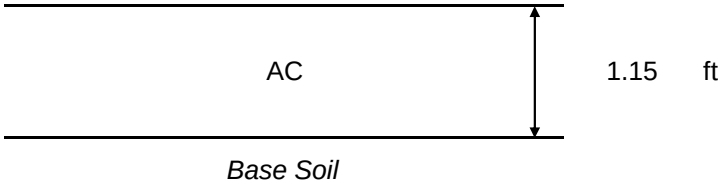
Design Case: Full depth AC

Design TI= ***9*** (20- Yr Pavement Design Life)
R_{BS}= ***10***

$GE_{AC} = 0.0032*TI*(100-R_{BS}) =$ 2.59
=> GE'_{AC}= 2.69 (add 0.1 ft safety factor)
=> AC Thickness= 1.11

=> AC Thickness= **1.15** ft (round up to the nearest 0.05 ft)

Design Section:



PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-108-GDR*
STRUCTURE.: *I-580 (EB) Sir Francis Drake On-Ramp*

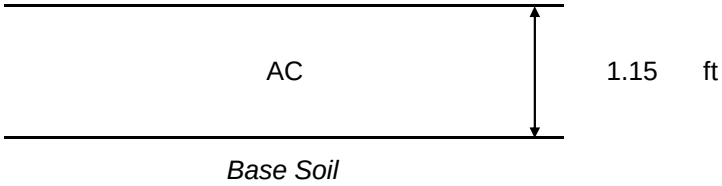
Design Case: Full depth AC

Design TI= ***9*** (20- Yr Pavement Design Life)
R_{BS}= ***10***

$GE_{AC} = 0.0032*TI*(100-R_{BS}) =$ 2.59
=> GE'_{AC}= 2.69 (add 0.1 ft safety factor)
=> AC Thickness= 1.11

=> AC Thickness= **1.15** ft (round up to the nearest 0.05 ft)

Design Section:



PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Richmond- San Rafael Bridge Access Improvement Project*
PROJECT NO.: *2014-125-GDR*
STRUCTURE.: *I-580 (EB) Sir Francis Drake On-Ramp*

Design Case: AC over AB over AS

Design TI=	***9***	(20- Yr Pavement Design Life)
R_{BS}=	***10***	
R_{AB}=	78	
R_{AS}=	50	(Class 4 AS)

$GE_{TOTAL} = 0..0032*TI*(100-R_{BS}) =$ 2.59

$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$ 0.63

=> GE'_{AC} = 0.83 (add 0.2 ft safety factor)

AC thickness = 0.44 ft

=> AC Thickness= **0.45** ft (round up to the nearest 0.05 ft)

$G_{f,\ AC}$= 1.89

GE_{AC}= 0.85

$GE_{AB+AC} = 0.0032*TI*(100-R_{AS}) =$ 1.44

=> GE_{AC+AB}= 1.64 (add 0.2 ft safety factor)

$GE_{AB} = GE_{AC+AB} - GE_{AC} =$ 0.79

=> AB thickness= 0.72

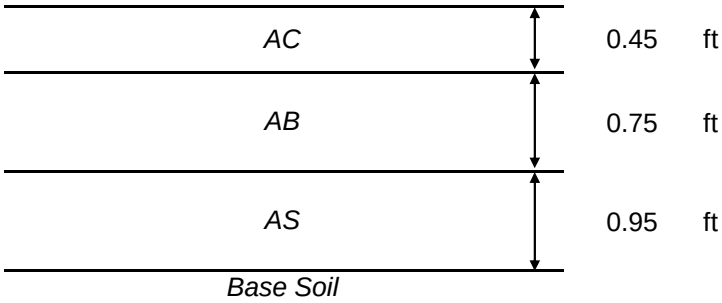
=> AB Thickness= **0.75** ft (round up to the nearest 0.05 ft)

GE_{AB}= 0.83 $G_{f,\ AB}$=1.1

$GE_{AS} = GE_{TOTAL} - GE_{AB} - GE_{AC} =$ 0.92

=> AS Thickness= **0.95** ft (round up to the nearest 0.05 ft)

Design Section:



0.45 ft

0.75 ft

0.95 ft

LIQUEFACTION ANALYSIS

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Richmond San Rafael Bridge Access Improvement Project***
PROJECT NO. ***2014-125-GDR***
BORING NO. ***A-15-580-001***

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

FAULT INFO
Hayward (North)
a_{max} (g)= 0.63
FAULT M_w = 7.3

GW DEPTH (ft)= 26
BOREHOLE DIA (in)= 4
HAMMER ENERGY = 84%
CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 26 *(below OG)*
MSF = 1.07

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*								*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	4.0	1	3.0	1	34	MC	22.1	23.2	375.0	1.7	39.4	17%	44.6		375.0	375.0	1.0	0.4	1.0	1.0			
4.0	8.5	2	6.0	1	27	MC	17.6	19.7	750.0	1.6	32.1		32.1		750.0	750.0	1.0	0.4	1.0	1.0			
8.5	13.0	3	11.0	1	46	MC	29.9	35.6	1375.0	1.2	42.9	7%	43.4		1375.0	1375.0	1.0	0.4	1.0	1.0			
13.0	18.0	4	16.0	1	43	MC	28.0	37.2	2000.0	1.0	37.2		37.2		2000.0	2000.0	1.0	0.4	1.0	1.0			
18.0	22.5	5	21.0	2	12	MC	7.8	10.4	2625.0	0.9	9.1												
22.5	28.0	6	26.0	1	31	MC	20.2	28.2	3250.0	0.8	22.1		22.1	0.2	3250.0	3250.0	0.9	0.4	0.8	1.0	(0.57)	1.30%	0.86
28.0	33.5	7	31.0	1	24	MC	15.6	21.8	3563.0	0.7	16.4	12%	18.4	0.2	3875.0	3563.0	0.9	0.4	0.8	1.0	(0.43)	1.49%	0.98
33.5	38.0	8	36.0	2	26	MC	16.9	23.7	3876.0	0.7	17.0											-	-
38.0	41.5	9	41.0	1	24	MC	15.6	21.8	4189.0	0.7	15.1		15.1	0.2	5125.0	4189.0	0.8	0.4	0.8	1.0	(0.33)	1.73%	0.73

Total = 2.57

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
5. Post-liquefaction settlement is estimated per Tokimatsu and Seed (1987).

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

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Richmond San Rafael Bridge Access Improvement Project***
PROJECT NO. ***2014-125-GDR***
BORING NO. ***A-15-580-010***

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

FAULT INFO
Hayward (North)
a_{max} (g)= 0.68
FAULT M_w = 7.3

GW DEPTH (ft)= 14
BOREHOLE DIA (in)= 4
HAMMER ENERGY = 84%
CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 14 *(below OG)*
MSF = 1.07

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*								*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	3.0	1	3.0	1	100	SPT	100.0	126.0	375.0	1.7	214.2	17%	229.3		375.0	375.0	1.0	0.4	1.0	1.0			
3.0	7.0	2	6.0	1	100	SPT	100.0	134.4	750.0	1.6	219.5		219.5		750.0	750.0	1.0	0.4	1.0	1.0			
7.0	10.0	3	11.0	1	100	SPT	100.0	142.8	1375.0	1.2	172.2	7%	173.8		1375.0	1375.0	1.0	0.4	1.0	1.0			
10.0	13.5	4	16.0	1	100	SPT	100.0	159.6	1875.2	1.0	164.8		164.8		2000.0	1875.2	1.0	0.5	1.0	1.0	NON-LIQ.		

Total = 0.00

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
5. Post-liquefaction settlement is estimated per Tokimatsu and Seed (1987).

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

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Richmond San Rafael Bridge Access Improvement Project***
PROJECT NO. ***2014-125-GDR***
BORING NO. ***A-15-580-011***

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

FAULT INFO
Hayward (North)
a_{max} (g)= 0.68
FAULT M_w = 7.3

GW DEPTH (ft)= 16
BOREHOLE DIA (in)= 4
HAMMER ENERGY = 84%
CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 16 *(below OG)*
MSF = 1.07

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*								*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	3.0	1	3.0	1	63	MC	41.0	43.0	375.0	1.7	73.1	17%	80.2		375.0	375.0	1.0	0.4	1.0	1.0			
3.0	8.0	2	6.0	1	86	MC	55.9	62.6	750.0	1.6	102.2		102.2		750.0	750.0	1.0	0.4	1.0	1.0			
8.0	12.0	3	11.0	1	82	MC	53.3	63.4	1375.0	1.2	76.5	7%	77.3		1375.0	1375.0	1.0	0.4	1.0	1.0			
12.0	15.3	4	16.0	1	100	MC	65.0	86.5	2000.0	1.0	86.5		86.5		2000.0	2000.0	1.0	0.4	1.0	1.0	NON-LIQ.		

Total = 0.00

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
5. Post-liquefaction settlement is estimated per Tokimatsu and Seed (1987).

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

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Richmond San Rafael Bridge Access Improvement Project***
PROJECT NO. ***2014-125-GDR***
BORING NO. ***A-15-580-013***

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

FAULT INFO
Hayward (North)
a_{max} (g)= 0.63
FAULT M_w = 7.3

GW DEPTH (ft)= 26
BOREHOLE DIA (in)= 4
HAMMER ENERGY = 84%
CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 26 *(below OG)*
MSF = 1.07

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*								*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	4.0	1	3.0	1	23	SPT	23.0	29.0	375.0	1.7	49.3	17%	55.0		375.0	375.0	1.0	0.4	1.0	1.0			
4.0	8.0	2	6.0	1	32	SPT	32.0	43.0	750.0	1.6	70.2		70.2		750.0	750.0	1.0	0.4	1.0	1.0			
8.0	13.0	3	11.0	1	55	MC	35.8	42.5	1375.0	1.2	51.3	7%	51.9		1375.0	1375.0	1.0	0.4	1.0	1.0			
13.0	17.0	4	16.0	1	100	SPT	100.0	159.6	2000.0	1.0	159.6		159.6		2000.0	2000.0	1.0	0.4	1.0	1.0			
17.0	22.0	5	21.0	1	100	SPT	100.0	159.6	2625.0	0.9	139.3		139.3		2625.0	2625.0	1.0	0.4	0.9	1.0			
22.0	25.0	6	26.0	1	100	SPT	100.0	168.0	3250.0	0.8	131.8		131.8		3250.0	3250.0	0.9	0.4	0.8	1.0	NON-LIQ.		

Total = 0.00

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
5. Post-liquefaction settlement is estimated per Tokimatsu and Seed (1987).

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

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Richmond San Rafael Bridge Access Improvement Project***
PROJECT NO. ***2014-125-GDR***
BORING NO. ***A-15-580-014***

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

FAULT INFO
Hayward (North)
a_{max} (g)= 0.63
FAULT M_w = 7.3

GW DEPTH (ft)= 26
BOREHOLE DIA (in)= 4
HAMMER ENERGY = 84%
CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 26 *(below OG)*
MSF = 1.07

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*								*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	4.0	1	3.0	1	43	MC	28.0	29.3	375.0	1.7	49.9	17%	55.6		375.0	375.0	1.0	0.4	1.0	1.0			
4.0	8.0	2	6.0	1	100	SPT	100.0	134.4	750.0	1.6	219.5		219.5		750.0	750.0	1.0	0.4	1.0	1.0			
8.0	13.0	3	11.0	1	100	SPT	100.0	142.8	1375.0	1.2	172.2	7%	173.8		1375.0	1375.0	1.0	0.4	1.0	1.0			
13.0	17.0	4	16.0	1	100	SPT	100.0	159.6	2000.0	1.0	159.6		159.6		2000.0	2000.0	1.0	0.4	1.0	1.0			
17.0	22.0	5	21.0	1	100	SPT	100.0	159.6	2625.0	0.9	139.3		139.3		2625.0	2625.0	1.0	0.4	0.9	1.0			
22.0	25.0	6	26.0	1	100	SPT	100.0	168.0	3250.0	0.8	131.8		131.8		3250.0	3250.0	0.9	0.4	0.8	1.0	NON-LIQ.		

Total = 0.00

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
5. Post-liquefaction settlement is estimated per Tokimatsu and Seed (1987).

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

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***RSR Bridge Access Improvement***
PROJECT NO. ***2014-125-GDR***
BORING NO. ***A-15-580-015***

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

FAULT INFO
Hayward (North)
a_{max} (g)= 0.63
FAULT M_w = 7.3

GW DEPTH (ft)= 19
BOREHOLE DIA (in)= 4
HAMMER ENERGY = 77%
CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 8 *(below OG)*
MSF = 1.07

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*								*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	4.0	1	3.0	1	47	MC	30.6	29.4	375.0	1.7	50.0		50.0		375.0	375.0	1.0	0.4	1.0	1.0			
4.0	8.0	2	6.0	1	28	MC	18.2	18.7	750.0	1.6	30.5		30.5		750.0	750.0	1.0	0.4	1.0	1.0			
8.0	13.0	3	11.0	1	12	MC	7.8	8.5	1375.0	1.2	10.3		10.3	0.1	1375.0	1187.8	1.0	0.5	1.0	1.0	(0.27)	2.31%	1.39
13.0	19.0	4	16.0	1	21	SPT	21.0	30.7	2000.0	1.0	30.7		30.7		2000.0	1500.8	1.0	0.5	1.0	1.0	NON-LIQ.		
19.0	23.0	5	21.0	2	3	SPT	3.0	4.4	2500.2	0.9	3.9												
23.0	28.0	6	25.0	1	100	MC	65.0	83.4	2750.6	0.9	71.1		71.1		3125.0	2064.2	0.9	0.6	0.9	1.0	NON-LIQ.		
28.0	33.0	7	30.0	1	100	SPT	100.0	154.0	3063.6	0.8	124.4		124.4		3750.0	2377.2	0.9	0.6	0.8	1.0	NON-LIQ.		
33.0	38.0	8	35.0	1	100	SPT	100.0	154.0	3378.6	0.8	118.5		118.5		4377.0	2692.2	0.9	0.6	0.8	1.0	NON-LIQ.		
38.0	43.0	9	40.5	1	100	SPT	100.0	154.0	3730.9	0.7	112.8		112.8		5072.5	3044.5	0.8	0.6	0.8	1.0	NON-LIQ.		
43.0	45.8	10	45.5	1	100	SPT	100.0	154.0	4056.4	0.7	108.1		108.1		5710.0	3370.0	0.8	0.6	0.8	1.0	NON-LIQ.		

Total = 1.39

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
5. Post-liquefaction settlement is estimated per Tokimatsu and Seed (1987).

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

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***RSR Bridge Access Improvement***
PROJECT NO. ***2014-125-GDR***
BORING NO. ***A-15-580-016***

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

FAULT INFO
Hayward (North)
a_{max} (g)= 0.63
FAULT M_w = 7.3

GW DEPTH (ft)= 18

BOREHOLE DIA (in)= 4
HAMMER ENERGY = 77%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 14 *(below OG)*

MSF = 1.07

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*								*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	4.0	1	3.0	1	68	MC	44.2	42.5	375.0	1.7	72.3		72.3		375.0	375.0	1.0	0.4	1.0	1.0			
4.0	8.0	2	6.0	1	50	MC	32.5	33.4	750.0	1.6	54.5		54.5		750.0	750.0	1.0	0.4	1.0	1.0			
8.0	13.0	3	11.0	1	21	MC	13.7	14.9	1375.0	1.2	18.0		18.0		1375.0	1375.0	1.0	0.4	1.0	1.0			
13.0	18.0	4	16.0	1	17	MC	11.1	13.5	2000.0	1.0	13.5		13.5	0.1	2000.0	1875.2	1.0	0.4	1.0	1.0	(0.37)	1.88%	1.13
18.0	23.0	5	21.0	2	7	MC	4.6	5.5	2437.8	0.9	5.0												
23.0	28.0	6	26.0	1	9	MC	5.9	7.5	2750.8	0.9	6.4		6.4	0.1	3250.0	2501.2	0.9	0.5	0.9	1.0	(0.16)	3.29%	1.97
28.0	33.0	7	31.0	1	14	MC	9.1	11.7	3063.8	0.8	9.4		9.4	0.1	3875.0	2814.2	0.9	0.5	0.9	1.0	(0.20)	2.46%	1.48
33.0	40.0	8	36.0	1	16	SPT	16.0	24.6	3379.8	0.8	19.0		19.0	0.2	4503.0	3130.2	0.9	0.5	0.8	1.0	(0.35)	1.46%	1.22
40.0	44.0	9	40.5	1	100	SPT	100.0	154.0	3666.5	0.7	113.7		113.7		5070.5	3416.9	0.8	0.5	0.8	1.0	NON-LIQ.		
44.0	47.0	10	45.0	1	100	SPT	100.0	154.0	3958.2	0.7	109.5		109.5		5643.0	3708.6	0.8	0.5	0.8	1.0	NON-LIQ.		
47.0	50.3	11	50	1	100	SPT	100.0	154.0	4289.2	0.7	105.2		105.2		6286.0	4039.6	0.8	0.5	0.7	1.0	NON-LIQ.		

Total = 5.80

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
5. Post-liquefaction settlement is estimated per Tokimatsu and Seed (1987).

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

SLOPE STABILITY ANALYSIS

RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
MARIN COUNTY, CALIFORNIA
RETAINING WALL 2

Staic Analysis

Name: Clay 1 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 1,300 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Sand 2 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 ° Phi-B: 0 ° Piezometric Line: 1
Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 34 ° Phi-B: 0 ° Piezometric Line: 1
Name: Sand 1 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 33 ° Phi-B: 0 ° Piezometric Line: 1
Name: Retaining Wall Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 10,000 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Clay 2 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 2,500 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Sand 3 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 ° Phi-B: 0 ° Piezometric Line: 1

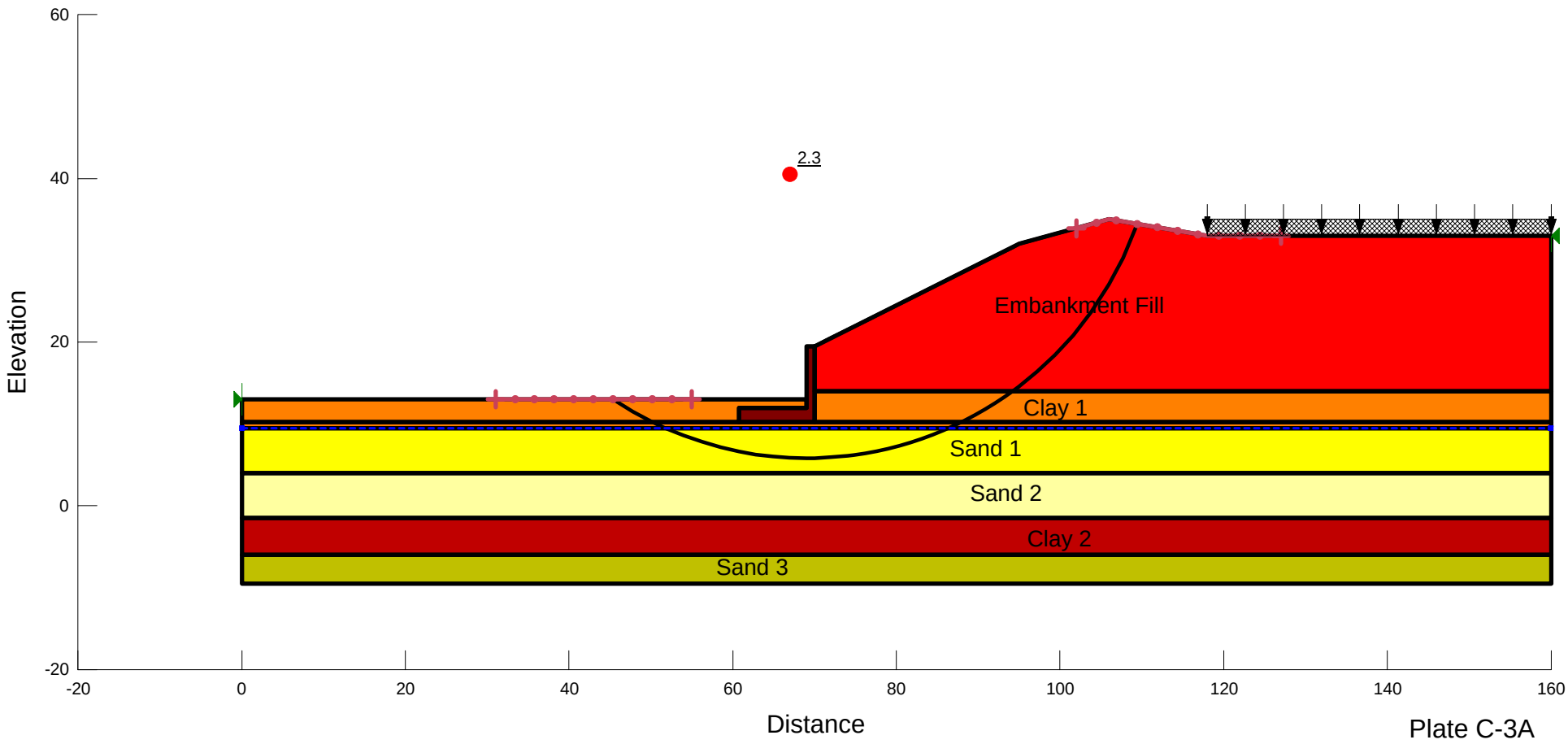


Plate C-3A

RICHMOND-SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
MARIN COUNTY, CALIFORNIA

RETAINING WALL 2

Pseudostatic Analysis

Horz Seismic Coef.: 0.2

Name: Clay 1 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 1,300 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Liquefiable Sand 2 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 750 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 34 ° Phi-B: 0 ° Piezometric Line: 1
Name: Liquefiable Sand 1 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 1,350 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Retaining Wall Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 10,000 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Clay 2 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 2,500 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1
Name: Liquefiable Sand 3 Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 750 psf Phi': 0 ° Phi-B: 0 ° Piezometric Line: 1

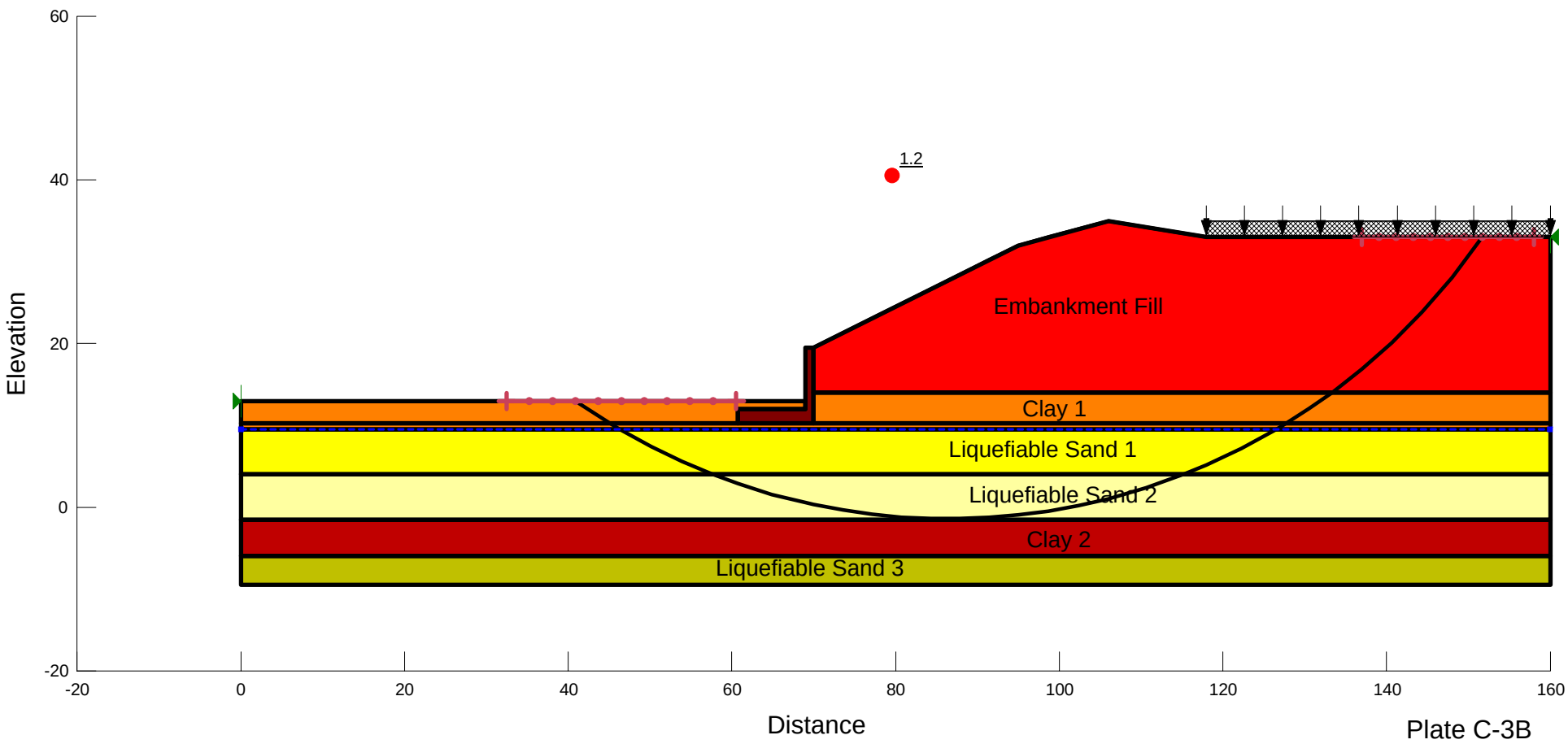


Plate C-3B

RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
EA NO. 04-2J6800
RETAINING WALL NO. 8 (GABIAN WALL)

Name: Embankment Fill Unit Weight: 120 pcf Cohesion': 100 psf Phi': 30 ° Phi-B: 0 °
Name: Earth Gabion Unit Weight: 125 pcf Cohesion': 100 psf Phi': 30 ° Phi-B: 0 °

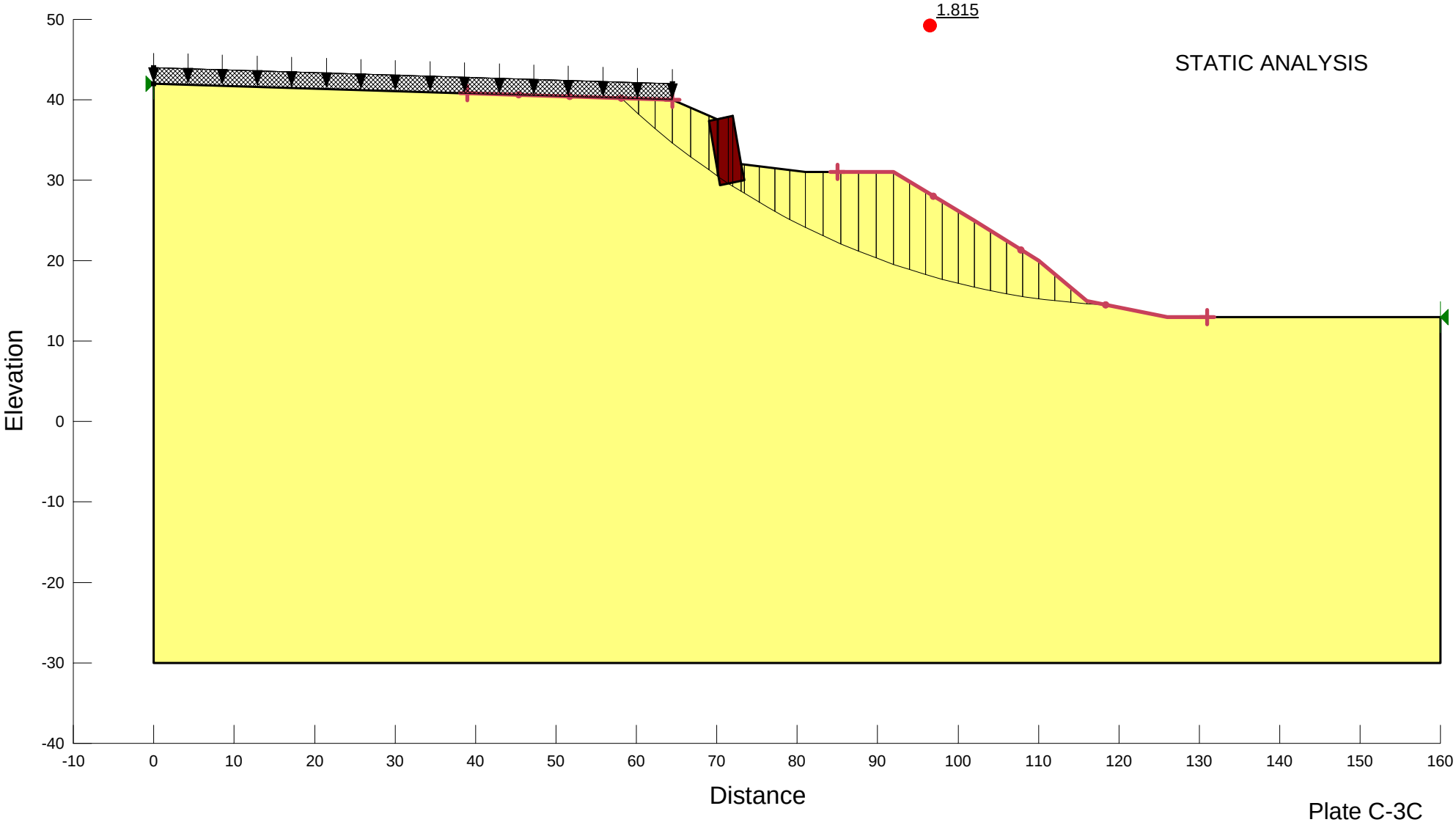


Plate C-3C

RICHMOND - SAN RAFAEL BRIDGE ACCESS IMPROVEMENT PROJECT
EA NO. 04-2J6800
RETAINING WALL NO. 8 (GABIAN WALL)

Name: Embankment Fill Unit Weight: 120 pcf Cohesion': 100 psf Phi': 30 ° Phi-B: 0 °
Name: Earth Gabion Unit Weight: 125 pcf Cohesion': 100 psf Phi': 30 ° Phi-B: 0 °

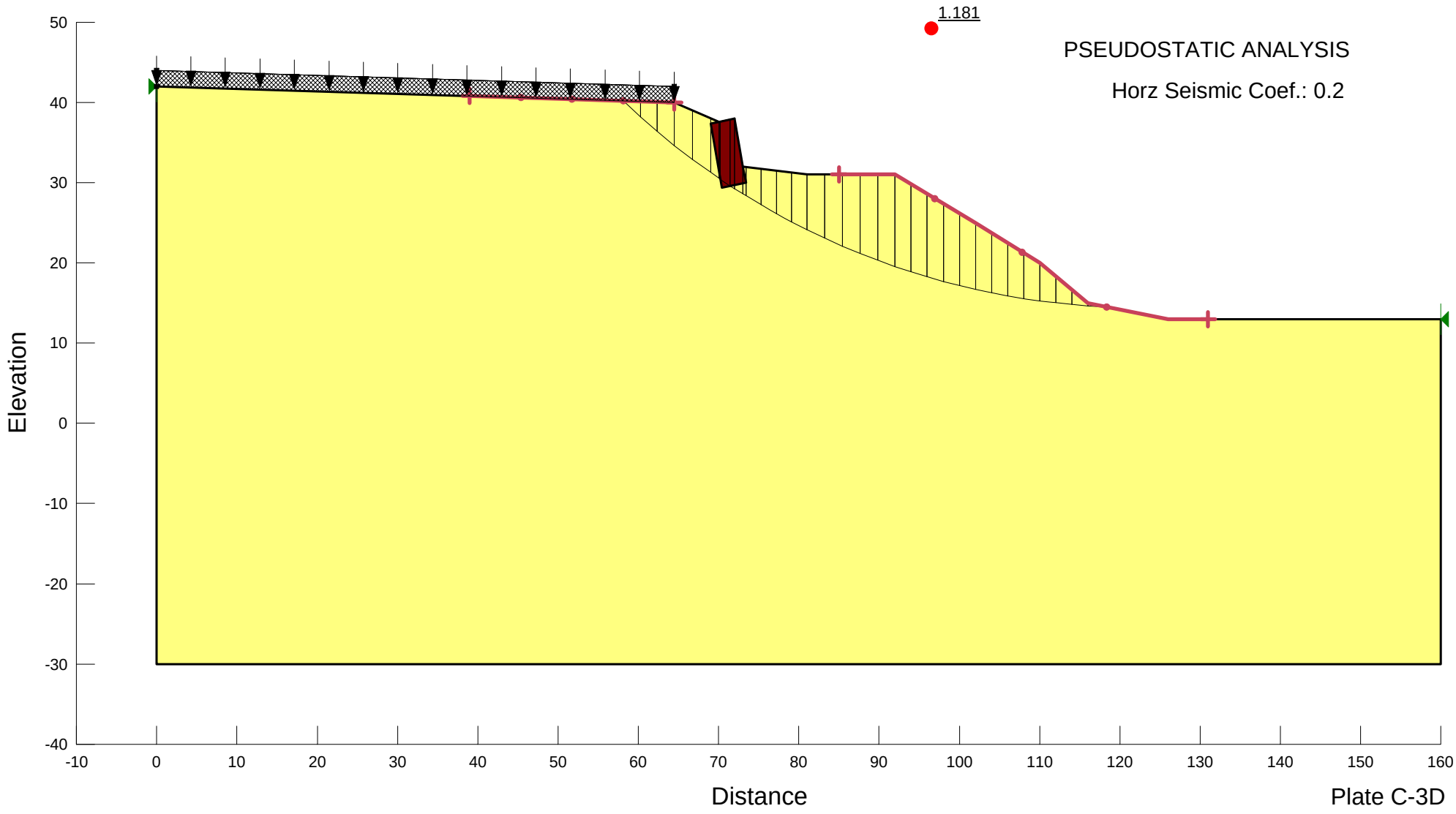


Plate C-3D

AltPipe Output

AltPipe

Project EA: 04-2J6800
Project Engineer: Kandeep Saravanapavan
Location: Marin County , CA
Description: Pipe Details for Package A & Package B in Marin County.

Steel Pipes

2–5 Year Flow Velocity (ft/sec)		2		3		4		5		6		7	
Pipe Diameter (in)		**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**
Pipe Type	**Coat**	**Minimum Thickness (in)**											
Corrugated Steel Pipe - Helical Corrugations - 2⅔" X ½" Corrugations	GAL		0.109		0.109		0.109		0.109		0.109		0.109
	BC	0.079	0.079		0.109		0.109		0.109		0.109		0.109
	BCI	0.064	0.064	0.079	0.079	0.079	0.079	0.079	0.079	0.079	0.079		0.109
	PA		0.052		0.052								
	PS	0.052	0.064	0.052	0.064	0.064	0.064	0.064	0.064	0.079	0.079	0.079	0.079
Corrugated Steel Pipe - Annular Corrugations - 2⅔" X ½" Corrugations	BCI												
	PA												
	PS						0.064		0.064				
Steel Spiral Rib Pipe - ¾" X 1" Ribs At 11½" Pitch	BCI												
	PA												
	PS												
Steel Spiral Rib Pipe - ¾" X 1" Ribs At 8½" Pitch	BCI												
	PA												
	PS												
Steel Spiral Rib Pipe - ¾" X ¾" Ribs At 7½" Pitch	BCI												
	PA												
	PS												
	CSS												

Plastic Pipes

2–5 Year Flow Velocity (ft/sec)	2		3		4		5		6		7	
Pipe Diameter (in)	**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**
Pipe Type	**Availability**											
PVC Corrugated	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable
HDPE Corrugated - Type S	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable
HDPE Corrugated - Type C	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable

Reinforced Concrete Pipes

2–5 Year Flow Velocity (ft/sec)	2		3		4		5		6		7	
Pipe Diameter (in)	**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**	**12**	**18**
Steel Cover (in)	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
Sacks of Cement	5	5	5	5	5	5	5	5	5	5	5	5
Percentage Water	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1

Other Information

2–5 Year Flow Velocity (ft/sec)	2		3		4		5		6		7	
Soil pH	7.65	7.65	7.65	7.65	7.65	7.65	7.65	7.65	7.65	7.65	7.65	7.65
Minimum Soil Resistivity (ohm-cm)	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400
Sulfate Concentration (ppm)	50	50	50	50	50	50	50	50	50	50	50	50
Chloride Concentration (ppm)	20	20	20	20	20	20	20	20	20	20	20	20
Abrasion Level	2	2	2	2	2	2	2	2	3	3	3	3
2–5 Year Flow Velocity (ft/sec)	2	2	3	3	4	4	5	5	6	6	7	7
Design Service Life (years)	50	50	50	50	50	50	50	50	50	50	50	50
Height of Cover (ft)**	5	5	5	5	5	5	5	5	5	5	5	5

*The results were obtained from AltPipe Version 6.08 © State of California
**The minimum height of cover is 2 feet per Caltrans Highway Design Manual, Section 856 and Caltrans Standard Plan D88

AltPipe

Project EA: 04-2J6800
Project Engineer: Kandeep Saravanapavan
Location: Contra Costa County , CA
Description: Pipe Details for Package A & Package B in Contra Costa County.

Steel Pipes

2–5 Year Flow Velocity (ft/sec)		2		3		4		7		11		11	4	5
Pipe Diameter (in)		18	24	18	24	18	24	18	24	18	24	18	24	24
Pipe Type	Coat	Minimum Thickness (in)												
Corrugated Steel Pipe - Helical Corrugations - 2⅔" X ½" Corrugations	GAL		0.138		0.138		0.138		0.138		0.138		0.138	0.138
	AL2		0.138		0.138		0.138		0.138		0.138		0.138	0.138
	BC		0.138		0.138		0.138		0.138		0.138		0.138	0.138
	BCI	0.109	0.109	0.109	0.109		0.138		0.138		0.138		0.138	0.138
	PS	0.064	0.064	0.079	0.079	0.109	0.109		0.138		0.138		0.109	0.109
Corrugated Steel Pipe - Annular Corrugations - 2⅔" X ½" Corrugations	BCI													
	PA													
	PS	0.064	0.064		0.079									
Steel Spiral Rib Pipe - ¾" X 1" Ribs At 11½" Pitch	BCI		0.109		0.109									
	PA													
	PS		0.064		0.079		0.109						0.109	0.109
Steel Spiral Rib Pipe - ¾" X 1" Ribs At 8½" Pitch	BCI		0.109		0.109									
	PA													
	PS		0.064		0.079		0.109						0.109	0.109
Steel Spiral Rib Pipe - ¾" X ¾" Ribs At 7½" Pitch	GAL		0.138		0.138		0.138		0.138		0.138		0.138	0.138
	AL2		0.138		0.138		0.138		0.138		0.138		0.138	0.138
	BC		0.138		0.138		0.138		0.138		0.138		0.138	0.138
	BCI		0.109		0.109		0.138		0.138		0.138		0.138	0.138
	PS		0.064		0.079		0.109		0.138		0.138		0.109	0.109

Aluminum Pipes

2–5 Year Flow Velocity (ft/sec)	2		3		4		7		11		11	4	5
Pipe Diameter (in)	18	24	18	24	18	24	18	24	18	24	18	24	24
Pipe Type	Availability												
Corrugated Aluminum Pipe Annular Corrugations - 2⅔" X ½" Corrugations	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060
Corrugated Aluminum Pipe Helical Corrugations - 2⅔" X ½" Corrugations	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060
Aluminum Spiral Rib Pipe - ¾" X 1" Ribs At 11½" Pitch		0.060		0.060		0.060		0.060		0.060		0.060	0.060
Aluminum Spiral Rib Pipe - ¾" X ¾" Ribs At 7½" Pitch		0.060		0.060		0.060		0.060		0.060		0.060	0.060

Plastic Pipes

2–5 Year Flow Velocity (ft/sec)	2		3		4		7		11		11	4	5
Pipe Diameter (in)	18	24	18	24	18	24	18	24	18	24	18	24	24
Pipe Type	Availability												
PVC Corrugated	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable
HDPE Corrugated - Type S	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable
HDPE Corrugated - Type C	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable	Allowable			

Reinforced Concrete Pipes

2–5 Year Flow Velocity (ft/sec)	2		3		4		7		11		11	4	5
Pipe Diameter (in)	18	24	18	24	18	24	18	24	18	24	18	24	24
Steel Cover (in)	0.75	1	0.75	1	0.75	1	0.75	1	0.75	1	0.75	1	1
Sacks of Cement	5	5	5	5	5	5	5	5	5	5	5	5	5
Percentage Water	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1	11.1

Other Information

2–5 Year Flow Velocity (ft/sec)	2		3		4		7		11		11	4	5
Soil pH	6.9	6.9	6.9	6.9	6.9	6.9	6.9	6.9	6.9	6.9	6.9	6.9	6.9
Minimum Soil Resistivity (ohm-cm)	1500	1500	1500	1500	1500	1500	1500	1500	1500	1500	1500	1500	1500
Sulfate Concentration (ppm)	40	40	40	40	40	40	40	40	40	40	40	40	40
Chloride Concentration (ppm)	25	25	25	25	25	25	25	25	25	25	25	25	25
Abrasion Level	2	2	2	2	2	2	3	3	3	3	3	2	2
2–5 Year Flow Velocity (ft/sec)	2	2	3	3	4	4	7	7	11	11	11	4	5
Design Service Life (years)	50	50	50	50	50	50	50	50	50	50	50	50	50
Height of Cover (ft)**	5	5	5	5	5	5	5	5	5	5	6	11	7

*The results were obtained from AltPipe Version 6.08 © State of California
**The minimum height of cover is 2 feet per Caltrans Highway Design Manual, Section 856 and Caltrans Standard Plan D88

APPENDIX D

Geophysical Study Report by NORCAL

July 30, 2015

Parikh Consultants, Inc.
2360 Qume Drive, Suite A
San Jose, California 95131

Subject: Seismic Refraction Investigation
Interstate 580 Retaining Wall
Richmond, California

NORCAL Job No: 15-426.22

Attention: Mr. Kandeep Saravanapavan

This report presents the findings of a seismic refraction (SR) investigation performed by NORCAL Geophysical Consultants as part of the retaining wall improvement project on eastbound Interstate 580 approximately 2,000 feet east of the Richmond Bridge Toll Plaza. The scope of the project includes widening the freeway and replacing the retaining wall with one approximately 15 feet further into the slope on the south side of the roadway. The survey was performed on June 29th, 2015 by NORCAL Professional Geophysicist David T. Hagin PGp 1033 and Senior Geophysical Technician Travis W. Black. Access to the site was via the Chevron Richmond Refinery. Logistical support was provided onsite by Mr. Robert Vanderlaan of Chevron.

1.0 SITE DESCRIPTION

The site consists of a steep cut slope above the existing retaining wall on the southern side of Interstate 580. The seismic survey was initially proposed to be conducted on the slope; however after field evaluation of the steepness of the slope, due to safety considerations it was determined jointly by Parikh and NORCAL personnel that the seismic survey would be conducted at the top of the slope adjacent to a chain link fence (Plate 1). In order to properly engineer the construction of the new retaining wall, it is desired to know the thickness of overburden and the depth to more competent material. Additionally, it is desired to estimate the excavatibility (rippability) of the rock.

The purpose of this investigation was to evaluate the shallow sub-surface conditions near the location of the planned excavation by measuring the seismic p-wave velocity values. These data will be used to evaluate the thickness of overburden and rock hardness with respect to rippability.

2.0 METHODOLOGY

The SR method is used to determine the compressional acoustic primary wave velocity (seismic velocity) of subsurface materials. The seismic velocity of fill, sediments, and rock are dependent

on physical properties such as compaction, density, hardness, and induration. However, other factors such as bedding, fracturing, and saturation also affect seismic velocity. Typically, low velocities are indicative of loose, dry soils, poorly compacted fill material, poorly to semi-consolidated sediments, or alternatively, deeply weathered and/or highly fractured rock. Moderate velocities usually indicate dense and highly compacted or saturated sedimentary deposits or fill, and/or moderately weathered and fractured rock. High velocities typically represent slightly weathered to unweathered (fresh) rock with little fracturing. A more detailed description of the SR methodology is provided in Appendix A.

3.0 FIELD SURVEY AND DATA PROCESSING

3.1 Data Acquisition

The geophysical survey entailed the acquisition of a single SR line extending along the top of the slope above the existing retaining wall, as shown on Plate 1. The placement of the line was determined jointly by Parikh and NORCAL personnel, and based on the planned area of excavation. The seismic line consisted of two geophone spreads overlapping by 12 geophone stations. Each spread was comprised of 24 geophones coupled to the ground surface at eight foot intervals and seven shot points distributed in a collinear array. The two end shot points were located four stations beyond each end of the geophone spreads in order to assure adequate depth of investigation. Two additional shot points were located two stations beyond each end of the spreads and the remaining shot points were evenly spaced within the spread, yielding a total line length of 344 feet.

3.2 Instrumentation

The SR data were recorded using a ***Geometrics Geode***, 24-bit digital seismic recording system and ***Oyo Geospace*** digital-grade geophones with a natural frequency of 10 Hz. We produced seismic energy at each shot point by striking an aluminum plate placed on the ground surface with a 16-pound sledge hammer. An accelerometer attached to the hammer transmitted a triggering pulse to the seismograph to begin recording each time the plate was struck. Several strikes were performed and stacked at each shot point to ensure an acceptable signal to noise ratio. The locations and elevations of the geophones and shot-points were determined using field mapping techniques, stadia rod and hand level and a ***Trimble Geo 7X*** GPS receiver.

3.3 Data Processing

The refraction data were processed in-house using ***SeisImager***, specialized software developed by Geometrics, Inc. of San Jose, California. We then used the program ***Surfer 12*** by Golden Software to graphically illustrate the subsurface distribution of seismic velocities. This consisted of generating a color-contoured seismic velocity cross-section (profile).

4.0 RESULTS AND INTERPRETATIONS

The results of the seismic refraction survey are illustrated by the seismic velocity profile shown on Plate 2. The vertical axis represents elevation (above mean sea level) and the horizontal axis represents survey stationing (distance along the line). The profile shows the ground surface and color contours representing the distribution of seismic velocity values according to the color scale shown at the bottom of the plate.

4.1 Seismic Velocities

Low seismic velocity values of less than about 4,500 feet per second (ft/s) are interpreted to represent soil or fill "overburden" or possibly intensely weathered rock (tan, yellow and green). Moderate seismic velocity values ranging from 4,500 to 6,000 ft/s are interpreted to likely represent moderately weathered and/or fractured rock (blue). The highest seismic velocity values vary between 6,000 and slightly greater than 7,000 ft/s; they are interpreted to represent less weathered and/or fractured rock (magenta).

4.2 Seismic Refraction Profile - Line 1

The seismic profile indicates that a continuous surficial layer of low to moderate velocity material approximately ten feet thick extends across the entire line. This layer thickens slightly between stations 100 and 170 to a maximum of approximately 15 feet. Below this layer we note a rapid transition to higher seismic velocity values. The underlying material appears very uniform and extends to the bottom of the profile, with seismic velocity values that vary between 6,000 and 7,200 ft/s. The maximum depth of exploration is approximately 50 feet and the maximum seismic velocity values measured were approximately 7,200 ft/s.

4.3 Rippability

Seismic velocity charts relating seismic velocity to excavation characteristics have been developed from field tests by others. These charts list the seismic velocity of various types of rock and their relative ease of excavation using different types of rippers. Caterpillar Tractor Company publishes a performance manual that lists ripper performance charts for the D8R, D9R, D10T and D11T tractors. The following information in Table A was obtained from a performance chart for a D8R ripper (Caterpillar Performance Handbook, Edition 36, April 2006). We present the velocity range for metamorphic rock, as the California Geological Survey (CGS) 2010 geologic map indicates that local bedrock is the Franciscan Formation.

Table A: CAT D8R Ripper Performance in Metamorphic Rock

PERFORMANCE	VELOCITY RANGE (FT/S)
Rippable	Less than 6,300
Marginally Rippable	6,300 to 8,200
Non-Rippable	Greater than 8,200

We compared the measured seismic velocity values to the various ranges listed in the Caterpillar Performance Handbook. All of the measured seismic velocity values fall within the rippable or marginally rippable ranges for the selected equipment. Although the actual equipment used during excavation may vary from the referenced equipment, this chart may serve as a relative guideline to site rippability conditions.

5.0 LIMITATIONS

This information should only be used as a general guide to rippability as many other factors also contribute to the evaluation of rock rippability. These factors include rock jointing and fracture patterns, the experience of the equipment operator, and the equipment and excavation methods selected. Also, the computed velocities measured along each line are an average; therefore, there may be localized zones where the velocities may be higher or lower than indicated. Since the accuracy of our findings is subject to these limitations, it should be noted that subsurface conditions may vary from those depicted in the final results. A more detailed discussion of the limitations with regard to the seismic refraction method is presented in Appendix A.

It should also be noted that the seismic refraction technique is based on the assumption that seismic velocity increases with depth. Any layers representing a decrease in velocity with depth, otherwise known as a velocity inversion, will not be defined and will result in the over-estimation of the depth of deeper, higher velocity layers. In addition, relatively thin layers might not be individually resolved and might, instead, be lumped together with other layers. Hard and soft zones within a given seismic layer will tend to be averaged into the velocity of that layer. Finally, there is not necessarily a one-to-one relationship between lithologic layers and seismic layers. It is entirely possible that two different types of material could have the same seismic velocity. Alternatively, a change in velocity can occur within a single lithologic unit.

6.0 STANDARD OF CARE

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

Thank you for the opportunity to participate on this project.

Sincerely,

NORCAL Geophysical Consultants, Inc.

David T. Hagin
Professional Geophysicist PGp 1033

DTH/KGB/tt
Enclosures: Plates 1 and 2
Appendix A - Seismic Refraction Survey

Appendix A

SEISMIC REFRACTION SURVEY

Appendix A

SEISMIC REFRACTION (SR)

METHODOLOGY

The seismic refraction method provides information regarding the seismic velocity structure of the subsurface. An impulsive (mechanical or explosive) source is used to produce compressional (P) wave seismic energy. The P-waves propagate into the earth and are refracted along interfaces caused by an increase in velocity. A portion of the P-wave energy is refracted back to the surface where it is detected by sensors (geophones) that are coupled to the ground surface in a collinear array (spread). The detected signals are recorded on a multi-channel seismograph and are analyzed to determine the shot point-to-geophone travel times. These data can be used along with the corresponding shot point-to-geophone distances to determine the depth, thickness, and velocity of subsurface seismic layers.

The seismic refraction technique is based on several assumptions. Paramount among these are:

- seismic velocity increases with depth, and,
- the velocity of each seismic layer is uniform over the length of the given spread.

In cases where these assumptions do not hold, the accuracy of the technique decreases. For example, if a low velocity layer occurs between two layers of higher velocity, the low velocity layer will not be detected and the depth to the underlying high velocity layer will be erroneously large. Also, if the velocity of a seismic layer varies laterally within a spread, those variations will be interpreted as fluctuations in the elevation of the underlying seismic layer.

It should be noted that apparent velocities can be affected by the orientation of bedding planes with respect to the direction of the seismic profile. Apparent velocities of rock are typically slower when measured along lines oriented perpendicular to bedding planes of steeply dipping rock than those measured along lines oriented parallel.

INSTRUMENTATION

Data acquisition is initiated along each SR line by producing seismic energy using a mechanical source. Mechanical sources produce energy by impacting a metal strike plate on the ground surface with either a 12-16 pound sledge hammer or an elastic-band driven weight drop. The resulting seismic wave forms are recorded using a Geometrics 24-channel engineering seismograph and Mark Products geophones with a natural frequency of 10 Hz. The data are recorded on hard copy records (seismograms) as well as on computer disks for future processing. The seismograms display the amount of time it takes for a compression (P) wave to travel from a given shot point to each geophone in a spread.

DATA ANALYSIS

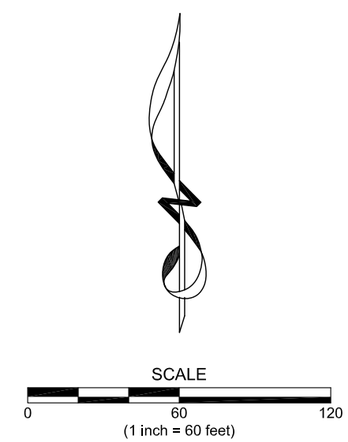
The seismic data are downloaded to a computer and processed using the software ***Seisimager*** by Geometrics, Inc. This is an interactive program that is used to determine the shot point to geophone travel times, and to compute a 2D model based on those times. Once the travel times for a given line are determined, the programs time-term algorithm is used to compute a preliminary 2D seismic model. This model is then used as input for the programs tomographic routine. Using this procedure, the program divides the starting model into a network of cells and assigns velocities to those cells based on the starting model. The program then traces the refracted seismic travel paths through those cells and computes the associated travel times. It then compares the computed travel times with the measured times and adjusts the velocities of the appropriate cells to improve the fit. The software is programmed to continue this procedure for twenty iterations. Typically, at the end of the twenty iterations the travel times associated with the computed model match the observed travel times to an accuracy of one milli-second (mS) or better. Once a satisfactory model is computed, the software contours the model velocities to produce seismic velocity vs. depth and distance cross-sections (profiles).

LIMITATIONS

In general, there are limitations unique to the SR method. These limitations are primarily based on assumptions that are made by the data analysis routine. First, the data analysis routine assumes that the velocities along the length of each spread are uniform. If there are localized zones within each layer where the velocities are higher or lower than indicated, the analysis routine will interpret these zones as changes in the surface topography of the underlying layer. A zone of higher velocity material would be interpreted as a low in the surface of the underlying layer. Zones of lower velocity material would be interpreted as a high in the underlying layer.

Second, the data analysis routine assumes that the velocity of subsurface materials increase with depth. Therefore, if a layer exhibits velocities that are slower than those of the material above it, the slower layer will not be resolved. Also, a velocity layer may simply be too thin to be detected. Due to these and other limitations inherent to the SR method, the results of the SR survey should be considered only as approximations of the subsurface conditions. The actual conditions may vary locally.

VICINITY MAP



LEGEND	
	SEISMIC REFRACTION LINE

NORCAL

SITE LOCATION MAP
SEISMIC REFRACTION SURVEY
INTERSTATE 580 RETAINING WALL

LOCATION: RICHMOND, CALIFORNIA

CLIENT: PARIKH CONSULTANTS

NORCAL GEOPHYSICAL CONSULTANTS INC.

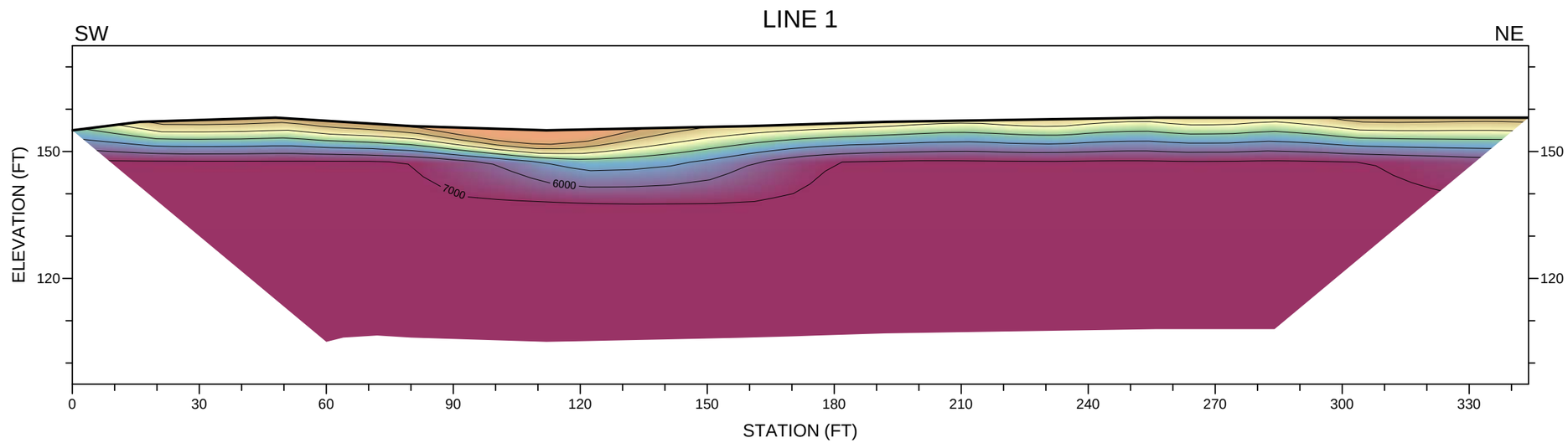
JOB #: 15-426.22

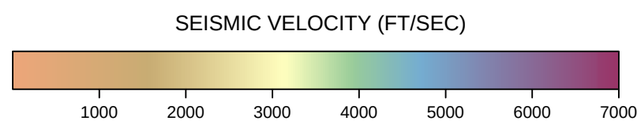
DATE: JUL. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DTH

PLATE 1





NORCAL

SEISMIC REFRACTION PROFILE
LINE 1
INTERSTATE 580 RETAINING WALL

LOCATION: RICHMOND, CALIFORNIA

CLIENT: PARIKH CONSULTANTS

JOB #: 15-426.22

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: JUL. 2015

DRAWN BY: G.RANDALL

APPROVED BY: DTH

PLATE
2

APPENDIX E

Pages from CATERPILLER “Handbook of Ripping”

Rippability Investigation & Prediction Service

Although visible laminations, faults, and fractures may indicate rippability and are usually helpful, conditions which are not visible are also important. That's because surface features give only a clue as to what lies underneath. To determine rippability when a field trial is not feasible, a method of estimating underlying characteristics is required.

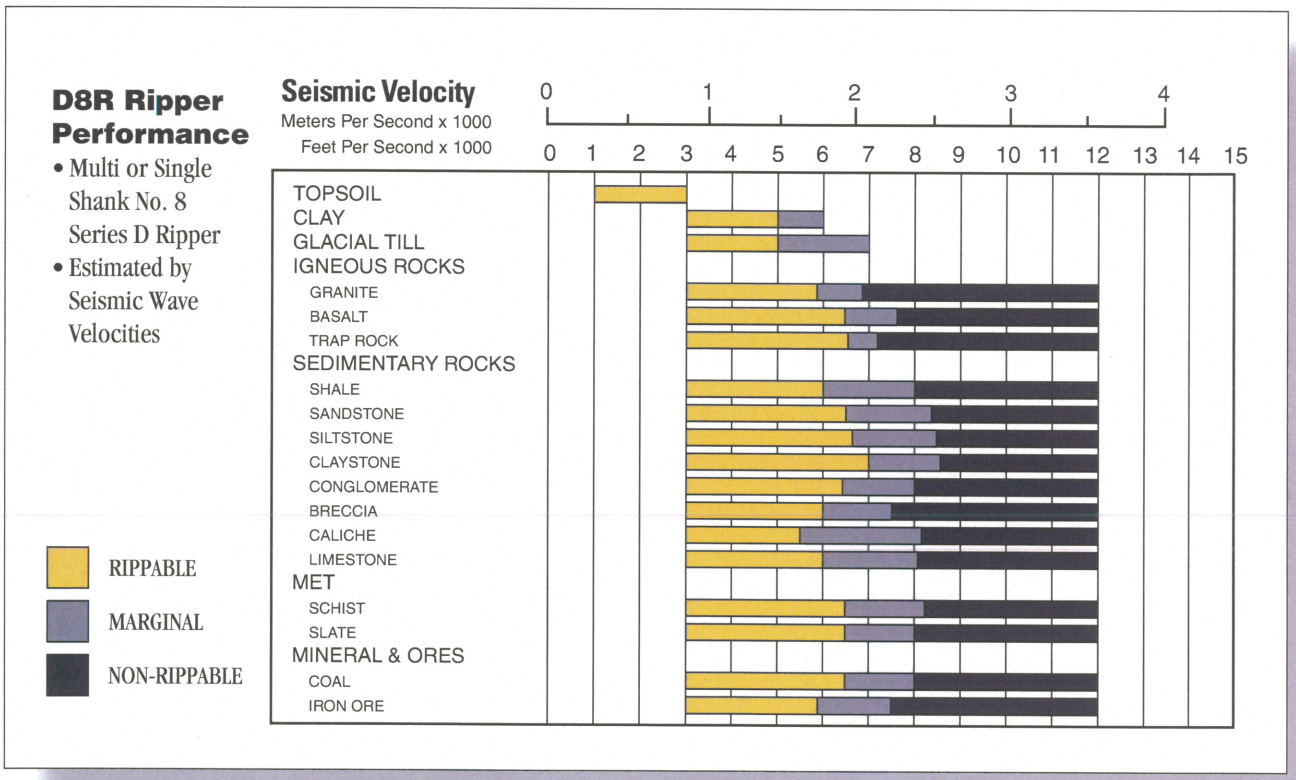
Caterpillar has developed a systematic analysis procedure to predict the rippability of a rock formation which combines new technology with geological and ripping experience. Our process for gathering the information necessary to make a prediction is called the Rippability Investigation and Prediction (RIP) service and is available through Caterpillar research. (Contact your district office.) The service consists of three steps:

1. Rock analysis
2. Site inspection
3. Seismic analysis

Rock Mechanics Analysis

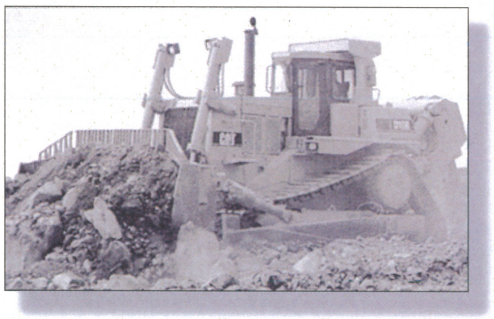
A rock mechanics analysis is the first phase of the RIP service and requires that a fresh rock sample be submitted to our lab for analysis along with other pertinent information about the site. (Minimum sample size should be 10" x 10" x 10".)

Rippers



Geological Site Inspection

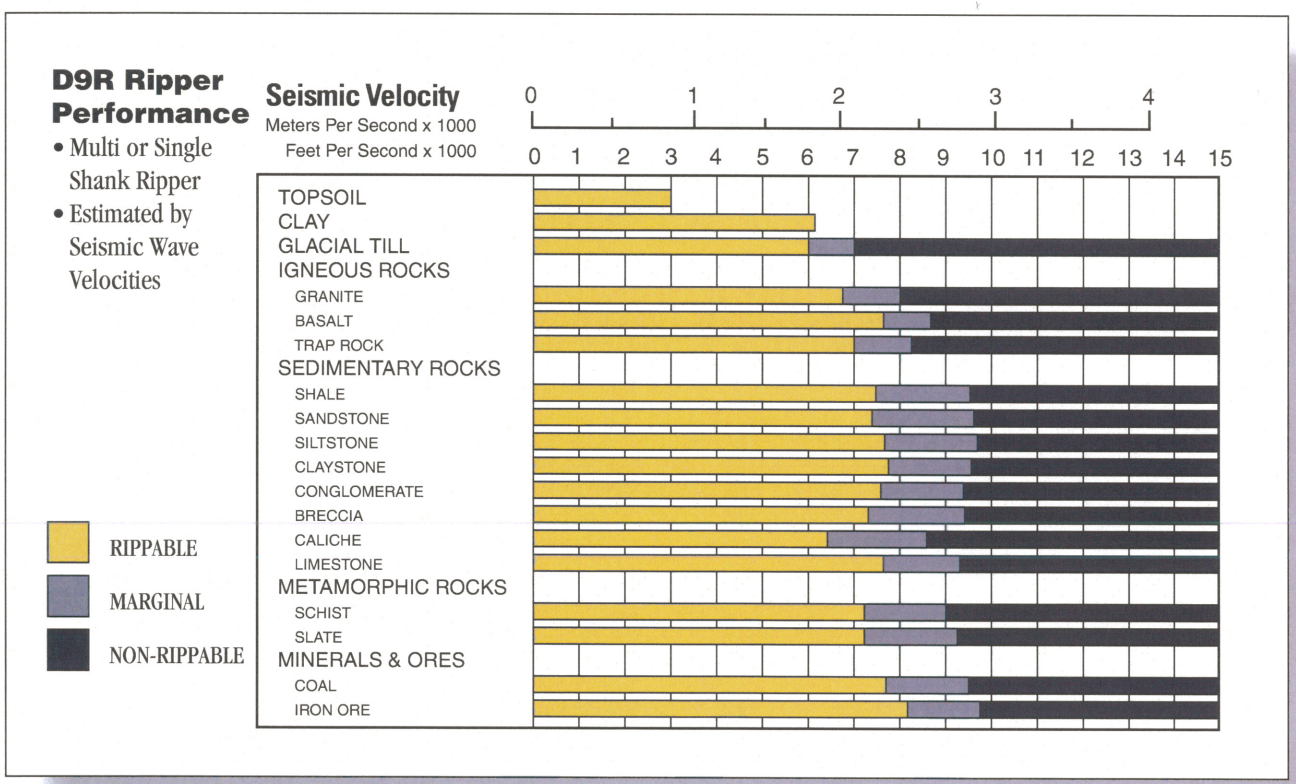
The second phase of the RIP service consists of a site visit by Caterpillar personnel which includes a geological inspection. During the site inspection, the rock formation in question is examined for in-place rock mass characteristics that may affect a ripping tractor's performance. These may include rock type, degree of weathering, bedding features, joint characteristics, and many other pertinent geological features.



Seismic Evaluation

The third phase of the RIP service includes a seismic evaluation. Caterpillar introduced the use of the refraction seismograph in 1958 as an aid to determine rippability of materials. The instrument functions by measuring seismic velocity, an indicator of the degree of consolidation of rock formations. Caterpillar continues to offer this service, along with many independent firms.

Rippers



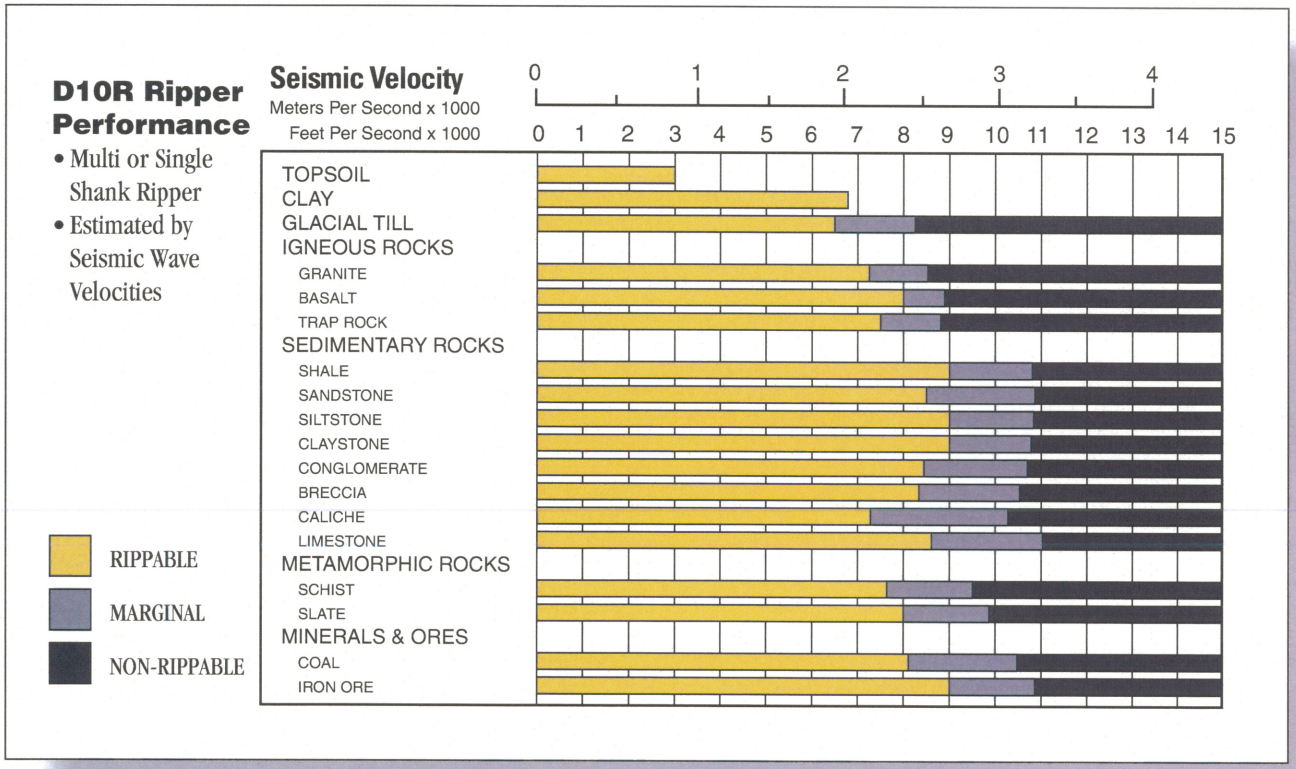
Rippability Investigation & Prediction Service

Caterpillar Systematic Analysis

Not all material conditions are visible from the surface. To determine the rippability of below-the-surface material and formations, Caterpillar Inc. developed a systematic analysis procedure based on technology and field experience. The service consists of three steps:

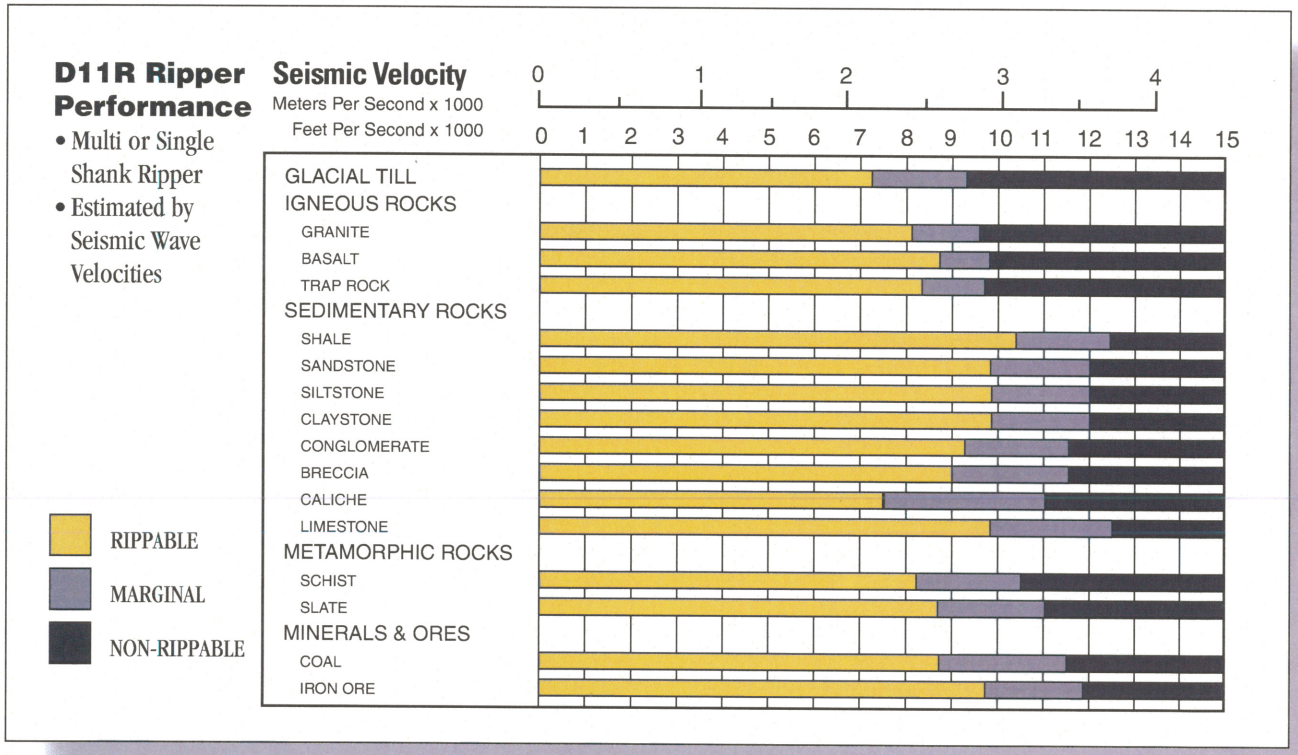
1. Rock analysis
2. Site inspection
3. Seismic evaluation

Rippers



CATERPILLAR
D11R

Rippers


D11R Ripper Performance
• Multi or Single Shank Ripper
• Estimated by Seismic Wave Velocities
Seismic Velocity
Meters Per Second x 1000
Feet Per Second x 1000
0 1 2 3 4
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
GLACIAL TILL
IGNEOUS ROCKS
GRANITE
BASALT
TRAP ROCK
SEDIMENTARY ROCKS
SHALE
SANDSTONE
SILTSTONE
CLAYSTONE
CONGLOMERATE
BRECCIA
CALICHE
LIMESTONE
METAMORPHIC ROCKS
SCHIST
SLATE
MINERALS & ORES
COAL
IRON ORE
RIPPABLE
MARGINAL
NON-RIPPABLE

APPENDIX F

As-Built Plan

ACIM-580-2(040)8E

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	CC	580	0.0/10.0	1	16

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

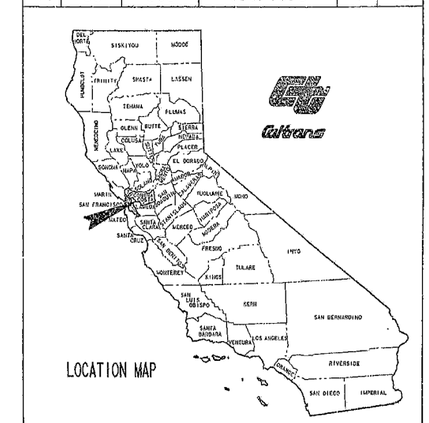
PROJECT PLANS FOR CONSTRUCTION ON
STATE HIGHWAY
IN CONTRA COSTA COUNTY
IN RICHMOND
FROM ALAMEDA COUNTY LINE TO THE EAST END OF RICHMOND - SAN RAFAEL BRIDGE

To be supplemented by Standard Plans dated July, 1999

INDEX OF SHEETS

Sheet No.
1 Title and Location Map
2 Typical Cross Sections and Construction Details
3-13 Construction Area Signs Plan
14 Pavement Delineation Plan
15 Summary of Quantities
16 Electrical Plans

THE STANDARD PLANS LIST APPLICABLE TO THIS CONTRACT IS INCLUDED IN THE NOTICE TO CONTRACTORS AND SPECIAL PROVISIONS BOOK.



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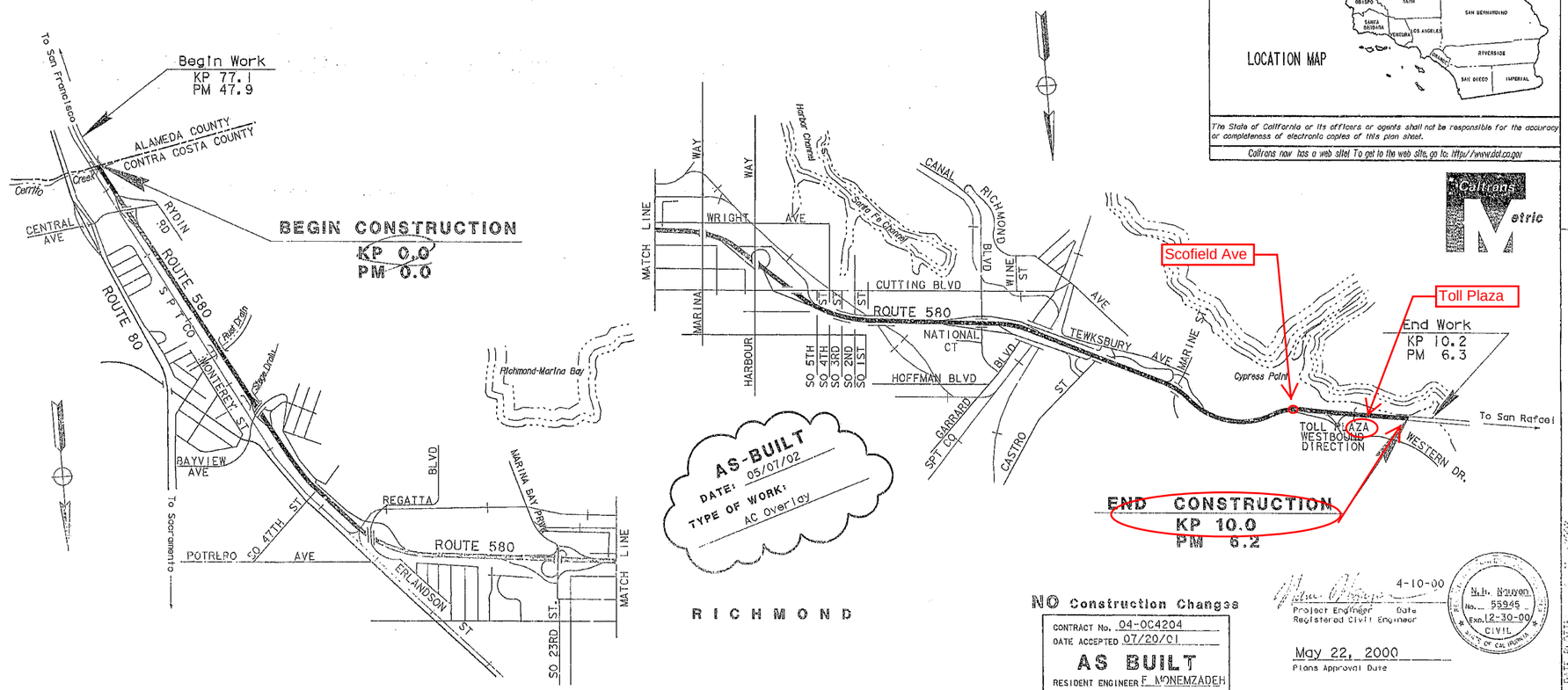
Caltrans now has a web site! To get to the web site, go to: http://www.dot.ca.gov

Caltrans Metric



NO Construction Changes

CONTRACT No. 04-0C4204
DATE ACCEPTED 07/20/01
AS BUILT
RESIDENT ENGINEER F. MONEMZADEH

4-10-00
Project Engineer Date
Registered Civil Engineer

N.H. Nguyen
No. 55945
Exp. 12-30-00
CIVIL
STATE OF CALIFORNIA

May 22, 2000
Plans Approval Date

The Contractor shall possess the Class (or classes) of license as specified in the ''Notice to Contractors''.

Contract No. 04-0C4204

FOR REDUCED PLANS ORIGINAL SCALE IS IN MILLIMETERS 0 20 40 60 80	USERNAME => cbunce DGN FILE => 4oc4204a.dgn	CU 04609	EA 0C4201

ENG. NGUYEN | DATE 10-11 | PROJECT NAME Eli Academia | DATE PLOTTED 05-09-02 | TIME PLOTTED

04-0C4204

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	CC	580	0.0/10.0	2	16

REGISTERED CIVIL ENGINEER 04-10-00

5-22-00
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER N.H. Nguyen No. 55945 Exp. 12-30-00 CIVIL STATE OF CALIFORNIA

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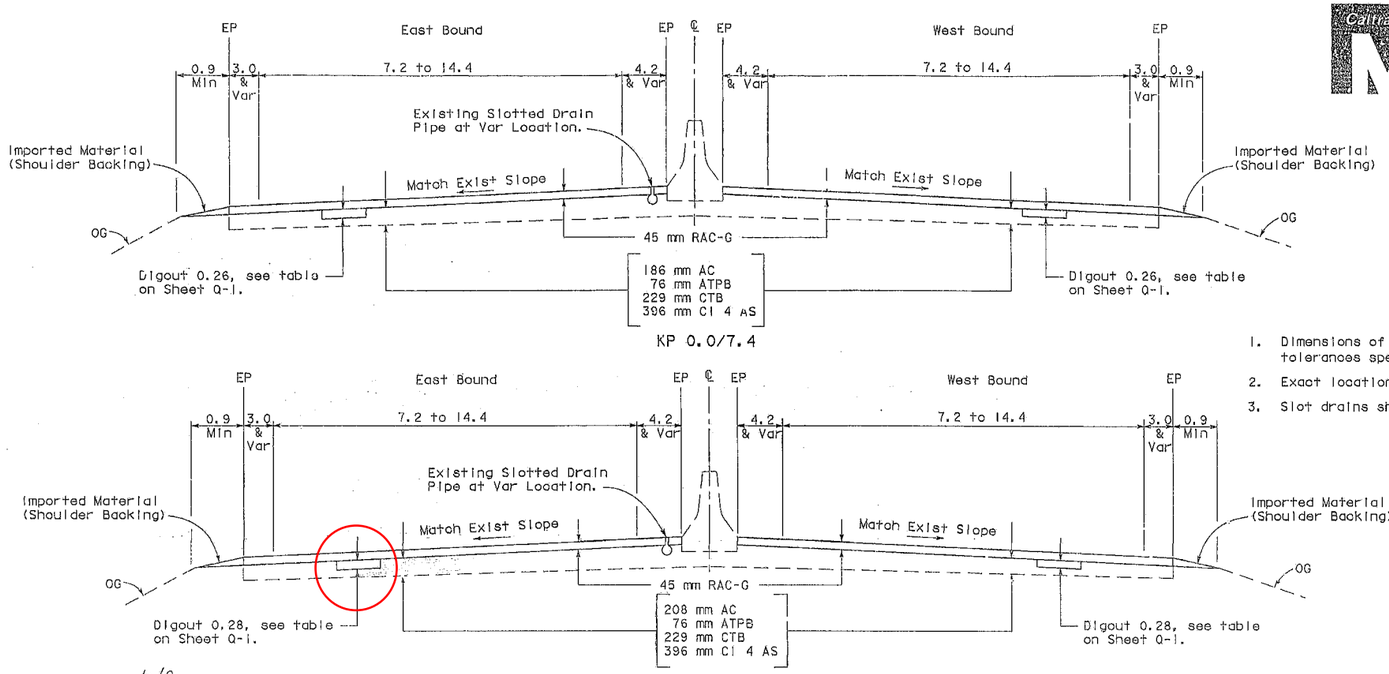
Caltrans now has a web site! To get to the web site, go to: http://www.dot.ca.gov



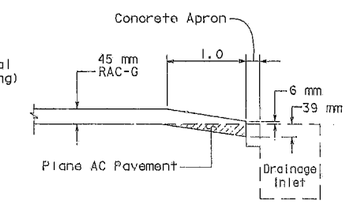
TYPICAL CROSS SECTIONS

LEGEND

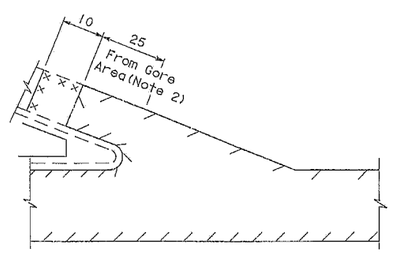
x/x/x	Plane AC Pavement
/ / /	AC Surfacing
RAC-G	Rubberized Asphalt Concrete (Type G)

NOTES

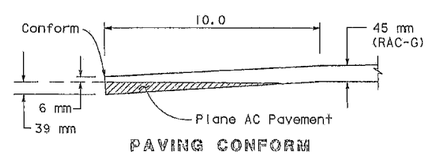
1. Dimensions of the structural sections are subject to tolerances specified in the Standard Specifications.
2. Exact location will be determined by the Resident Engineer.
3. Slot drains shall not be covered.



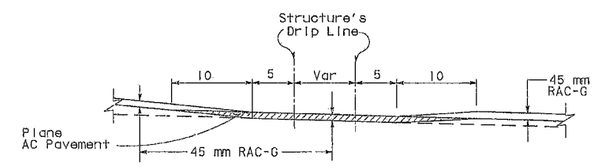
CONFORM AT DRAINAGE INLET LOCATION



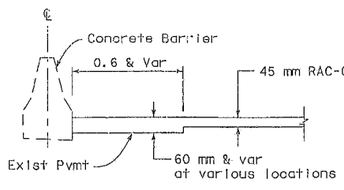
RAMP CONFORM



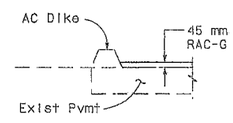
PAVING CONFORM ON MAIN LINE & RAMPS



AC PAVEMENT PLANNING & PLACING UNDER STRUCTURES



PAVING AT CONCRETE BARRIER



PAVING CONFORM AT AC DIKE

NO Construction Changes

CONT No. 04-0C4204
DATE ACCEPTED 07/20/01
AS BUILT
RESIDENT ENGINEER F. MONEMZADEH

TYPICAL CROSS SECTION AND CONSTRUCTION DETAILS

All Dimensions are in Meters unless otherwise shown

NO SCALE

X-1

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans PROJECT DEVELOPMENT

PROJECT ENGINEER NAM NGUYEN

DATE	REVISED BY	DATE REVISED

CALCULATED/DESIGNED BY | CHECKED BY

FOR REDUCED PLANS ORIGINAL SCALE IS IN MILLIMETERS 0 20 40 60 80

USERNAME => cburks
DGN FILE => 4oc4201-01.cab

CU 04609 | EA 0C4201

05-09-12 11:34:04 PM

AC LEVELING SUMMARY

Location	Eastbound Width (m) x Length (m)	Westbound Width (m) x Length (m)	AC Leveling TONNE
Median Shoulder KP 0.32/2.58, (PM 0.2/1.6)	4.5 x 2260		1220
Median Shoulder Lane #1 & Lane #2, KP 4.6 (PM 2.9)	12 x 60		90
Median Shoulder KP 1.93/2.42, (PM 1.2/1.5)		4.2 x 490	250
Median Shoulder Lane #1 & Lane #2, KP 0.64 (PM 0.4)		12 x 100	150
		Total	1710

ROADWAY QUANTITIES SUMMARY

Location	Rubberized Asphalt Concrete (Type G) (RAC-G)	Asphaltic Emulsion (Paint Binder)	Shoulder Backing	Imported Material (Shoulder Backing)
	Tonne	Tonne	Sta	m3
KP 0.0/10.0 (PM 0.0/6.2)	38000	460	64	200
Buchanan to Central Ave.	2464	11		
TOTAL	40464	471	64	200

DIG-OUT LOCATION SUMMARY - EASTBOUND

Location	Eastbound Lane #1	Lane #2	Lane #3	Auxiliary Lane	Replace AC Surfacing
	Width (m) x Length (m) x Thickness (m)				m3
Before Toll Booth	3.6 x 30 x .28	3.6 x 60 x .28	3.6 x 210 x .28		302.4
KP 9.0 (PM 5.6)	3.6 x 45 x .28	3.6 x 110 x .28			156.7
KP 8.85 (PM 5.5)		1.2 x 170 x .28			57.1
KP 8.45 (PM 5.25)		3.6 x 150 x .28			151.2
KP 8.0 (PM 5.0)	3.6 x 375 x .28	3.6 x 375 x .28	3.6 x 375 x .28		1134.0
KP 7.4 (PM 4.6)	3.6 x 225 x .28	3.6 x 225 x .28	3.6 x 225 x .28		680.4
KP 6.76 (PM 4.2)				3.6 x 36 x .26	33.7
KP 6.6 (PM 4.1)	3.6 x 6 x .26				5.6
KP 6.1 (PM 3.8)			3.6 x 27 x .26		25.3
KP 5.6 (PM 3.5)			3.6 x 9 x .26		8.4
KP 5.0 (PM 3.1)			3.6 x 21 x .26		19.7
KP 4.8 (PM 3.0)			3.6 x 12 x .26		11.3
KP 4.3 (PM 2.7)		3.6 x 42 x .26	3.6 x 42 x .26		78.6
KP 4.2 (PM 2.6)				3.6 x 75 x .26	70.2
KP 4.0 (PM 2.5)			3.6 x 9 x .26		8.4
KP 3.6 (PM 2.25)	3.6 x 6 x .26				5.6
KP 3.37 (PM 2.1)	3.6 x 36 x .26				33.7
Total					2781.9

DIG-OUT LOCATION SUMMARY - WESTBOUND

Location	Westbound Lane #1	Lane #2	Lane #3	Auxiliary Lane	Replace AC Surfacing
	Width (m) x Length (m) x Thickness (m)				m3
KP 3.4 (PM 2.13)	3.6 x 15 x .26	3.6 x 15 x .26	3.6 x 15 x .26		42.1
KP 3.8 (PM 2.38)			3.6 x 15 x .26		14.0
KP 3.93 (PM 2.44)		3.6 x 21 x .26			19.7
KP 5.3 (PM 3.3)			1.2 x 42 x .26		13.1
KP 6.28 (PM 3.9)			1.2 x 14 x .26		4.4
KP 7.1 (PM 4.4)		3.6 x 14 x .26			13.1
KP 7.32 (PM 4.55)			3.6 x 33 x .28		33.3
KP 8.37 (PM 5.2)	3.6 x 30 x .28		3.6 x 30 x .28		60.5
Toll Booths	3.6 x 75 x .28 on all seven lanes				529.2
Total					729.4

Resident Engineer Responsible for *Construction Changes*

CONTRACT No. 04-0C4204
DATE ACCEPTED 07/20/01
AS BUILT
RESIDENT ENGINEER F MONEMZADEH
SIGNATURE F. Monemzadeh DATE 05/08/02
CHECKED BY EB DATE 5/20/02

Caltrans Metric

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	CC	580	0.0/10.0	15	16

Nam Nguyen 04-10-00
REGISTERED CIVIL ENGINEER
5-22-00
PLANS APPROVAL DATE
REGISTERED PROFESSIONAL ENGINEER N.H. Nguyen No. 55945 Exp. 12-30-00 CIVIL STATE OF CALIFORNIA

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PLANE AC LOCATION SUMMARY

Location and Description	Width (m) x Length (m)	Plane AC Pavement (m2)
Bridge Approach & Departure KP 0.00 (PM 0.00)	35 x 10	700
Under Structure KP 0.38 (PM 0.240)	35 x 60	2100
Bridge Approach & Departure KP 1.16 (PM 0.72)	35 x 60	2100
Bridge Approach & Departure KP 1.89 (PM 1.17)	35 x 10	700
Under Structure KP 1.95 (PM 1.21)	35 x 60	2100
Under Structure KP 3.36 (PM 2.09)	45 x 130	5850
Under Structure KP 4.65 (PM 2.89)	45 x 70	3150
Under Structure KP 5.38 (PM 3.35)	45 x 60	2700
Under Structure KP 5.79 (PM 3.60)	45 x 60	2700
Under Structure KP 6.10 (PM 3.79)	45 x 70	3150
Bridge Approach & Departure KP 6.58 (PM 4.09)	45 x 10	900
Bridge Approach & Departure KP 7.47 (PM 4.64)	45 x 10	900
Bridge Approach & Departure KP 7.80 (PM 4.85)	40 x 10	800
Bridge Approach & Departure KP 8.22 (PM 5.11)	40 x 10	800
Bridge Approach & Departure KP 8.44 (PM 5.43)	40 x 10	800
Bridge Approach & Departure KP 9.33 (PM 5.80)	30 x 10	600
Bridge Approach & Departure KP 9.60 (PM 6.00)	30 x 10	600
On Ramps Various Locations	10 x 10	4000
Plane AC @ Miscellaneous Locations		1750
Buchanan to Central Ave.		408
Total		35408

SUMMARY OF QUANTITIES

DATE	REVISED BY	DATE REVISED

CALCULATED/DESIGNED BY
CHECKED BY

PROJECT ENGINEER NAM NGUYEN

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans PROJECT DEVELOPMENT

DATE PLOTTED 05-09-02
TIME PLOTTED

FOR REDUCED PLANS ORIGINAL SCALE IS IN MILLIMETERS 0 20 40 60 80

USERNAME = cburke
DGN FILE = 04c4201cl.dgn

CU 04609 EA 0C4201

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

ACBHIM-580-1(052)E

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	CC	580	5.5/6.2	1	301

PROJECT PLANS FOR CONSTRUCTION ON
STATE HIGHWAY

IN CONTRA COSTA COUNTY
IN RICHMOND
FROM 0.3 MILE EAST OF SCOFIELD AVENUE UNDERCROSSING
TO 0.2 MILE WEST OF WESTERN DRIVE UNDERCROSSING

TO BE SUPPLEMENTED BY STANDARD PLANS DATED MAY 2006

INDEX OF PLANS

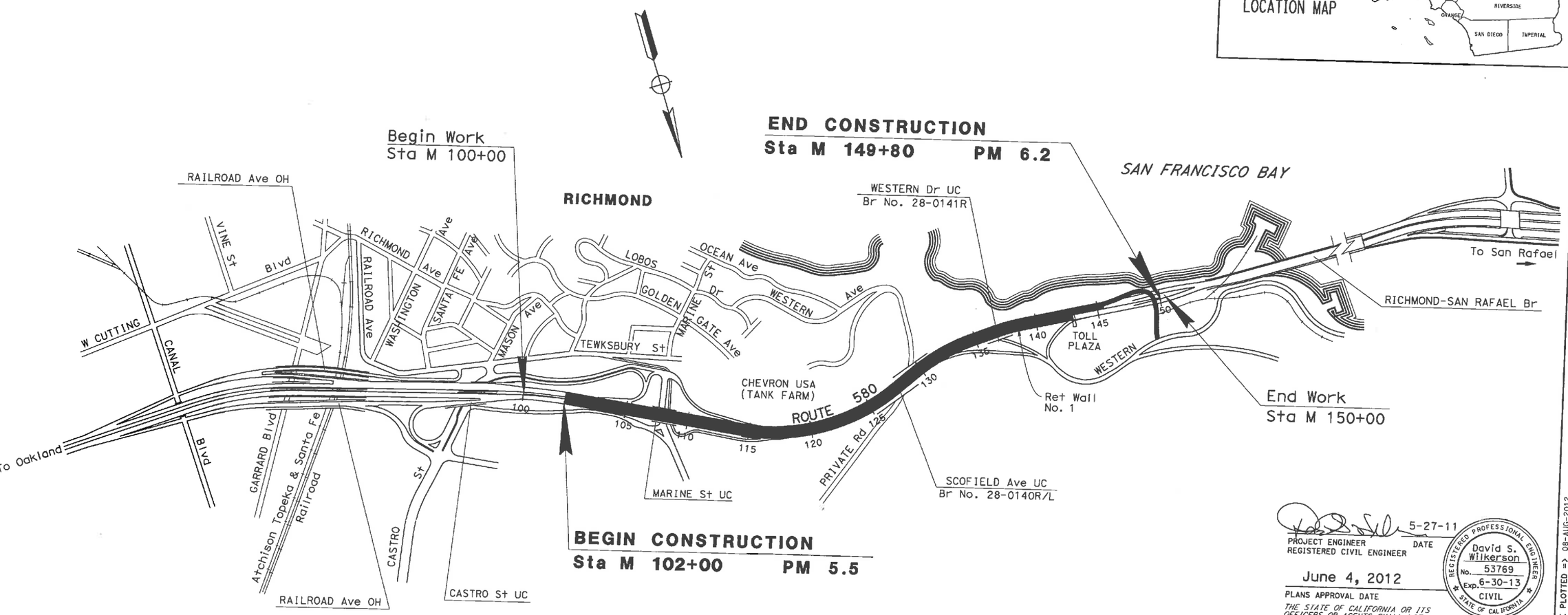
SHEET No.	DESCRIPTION
1	TITLE AND LOCATION MAP
2-6	TYPICAL CROSS SECTIONS
7-10	LAYOUTS
11-15	CONSTRUCTION DETAILS
16-17	EROSION CONTROL PLANS
18-26	DRAINAGE PLANS, PROFILES, DETAILS AND QUANTITIES
27-28	UTILITY PLANS
29-35	CONSTRUCTION AREA SIGNS
36-72	STAGE CONSTRUCTION, DETAILS, AND QUANTITIES
73-79	TEMPORARY DRAINAGE PLANS, PROFILES, DETAILS AND QUANTITIES
80-85	PAVEMENT DELINEATION PLANS, DETAILS, AND QUANTITIES
86-93	SIGN PLANS, DETAILS, AND QUANTITIES
94	SUMMARY OF QUANTITIES
95-96	HIGHWAY PLANTING
97-116	ELECTRICAL PLANS
117-163	REVISED AND NEW STANDARD PLANS

STRUCTURE PLANS

164-255	SCOFIELD AVENUE UNDERCROSSING Br No. 28-0140
256-282	WESTERN DRIVE UNDERCROSSING Br No. 28-0141
283-301	RETAINING WALL No 28E0055

THE STANDARD PLANS LIST APPLICABLE TO THIS CONTRACT IS INCLUDED IN THE NOTICE TO BIDDERS AND SPECIAL PROVISIONS BOOK.

LOCATION MAP



PROJECT MANAGER CHERYL NEVARES

DESIGN ENGINEER JAY FONG

5-27-11
PROJECT ENGINEER REGISTERED CIVIL ENGINEER DATE

REGISTERED PROFESSIONAL ENGINEER David S. Wilkerson No. 53769 Exp. 6-30-13 CIVIL STATE OF CALIFORNIA

June 4, 2012
PLANS APPROVAL DATE

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THE CONTRACTOR SHALL POSSESS THE CLASS (OR CLASSES) OF LICENSE AS SPECIFIED IN THE "NOTICE TO BIDDERS."

NO SCALE

CONTRACT No.	04-1A3204
PROJECT ID	0400000483

LAST REVISION 05-25-11

DATE PLOTTED => 08-AUG-2012
TIME PLOTTED => 06:19

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	CC	580	5.5/6.2	2	301

REGISTERED CIVIL ENGINEER 5-27-11 DATE

6-4-12
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Jay Kin-Kwok Fong No. 72112 Exp. 6-30-12 CIVIL STATE OF CALIFORNIA

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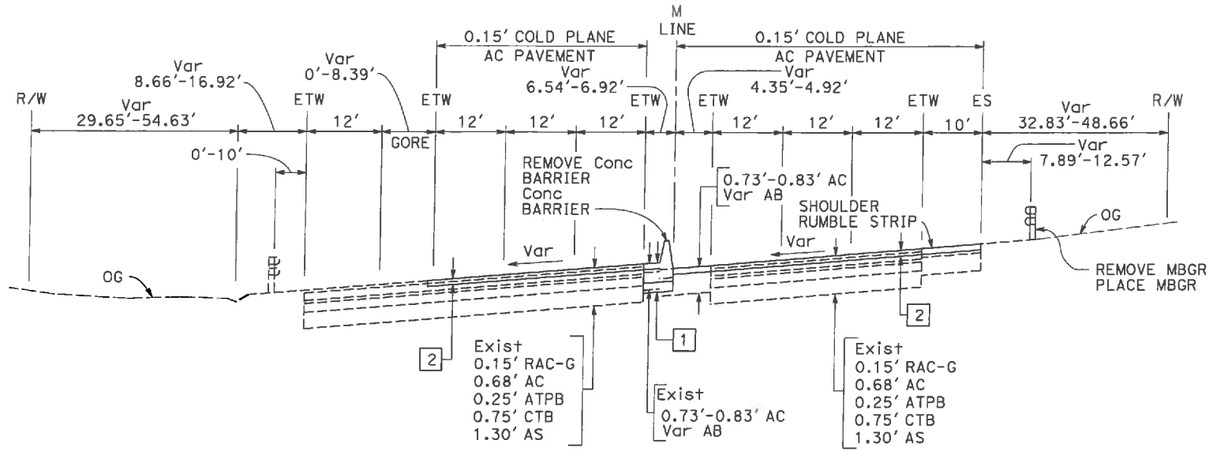
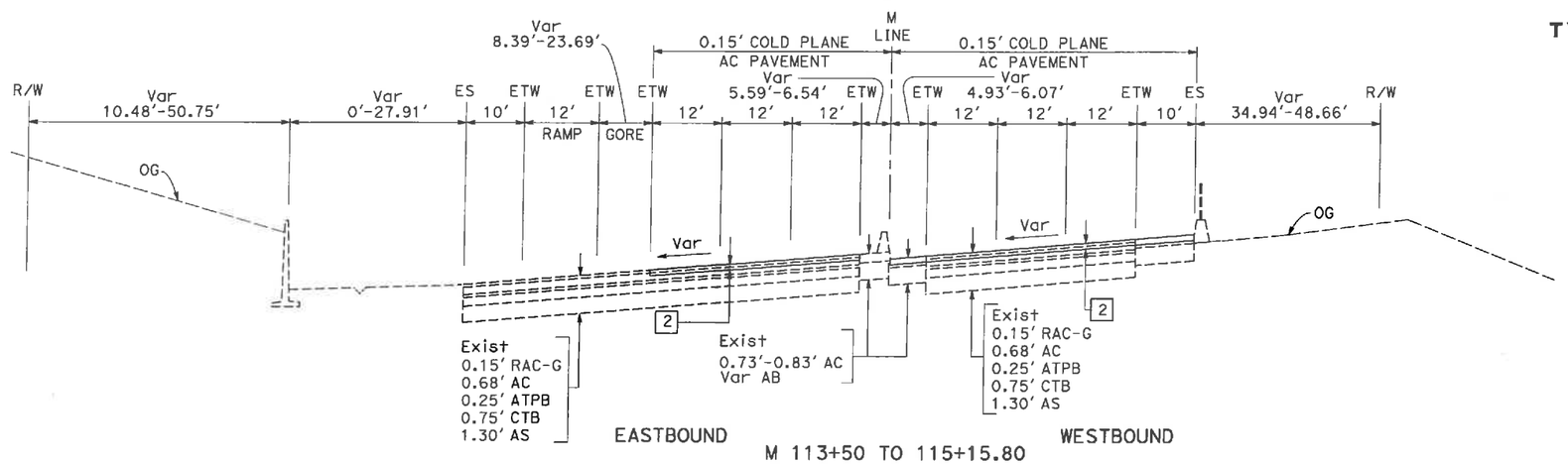


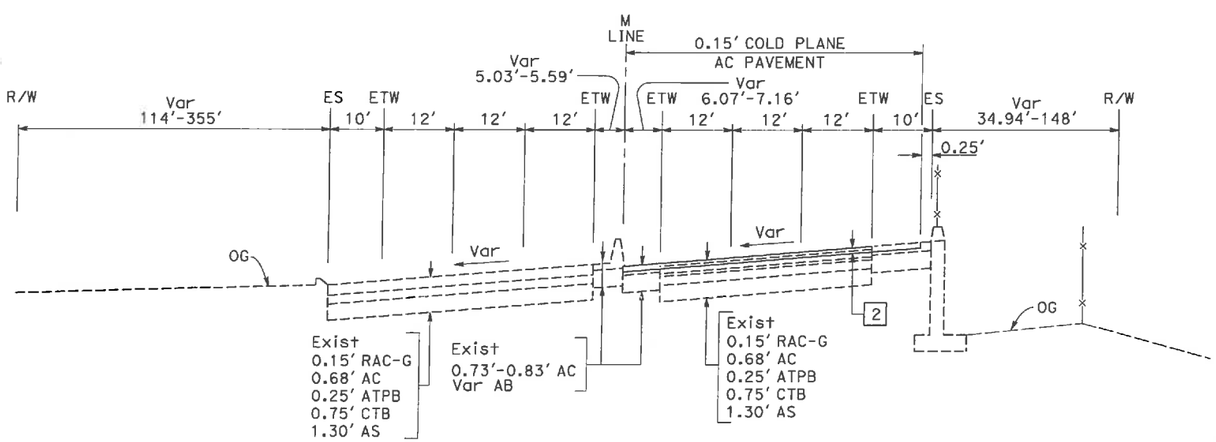
EASTBOUND WESTBOUND

M 115+15.80 TO 117+74.35



EASTBOUND WESTBOUND

M 113+50 TO 115+15.80



EASTBOUND WESTBOUND

M 109+00 TO 113+50

NOTES:

1. DIMENSIONS OF THE PAVEMENT STRUCTURES (STRUCTURAL SECTIONS) ARE SUBJECT TO TOLERANCES SPECIFIED IN THE STANDARD SPECIFICATIONS.
2. SUPERELEVATIONS AS SHOWN OR AS DIRECTED BY THE ENGINEER.
3. FOR LOCATION, TYPE AND LIMITS OF CONCRETE BARRIER, MBGR, AND SHOULDER RUMBLE STRIPS, SEE LAYOUT SHEETS.
4. FOR LIMITS OF COLD PLANE AC, SEE LAYOUT SHEETS.
5. FOR TYPICAL BIKE PATH CROSS SECTION, SEE C SHEETS.

TYPICAL PAVEMENT STRUCTURE SECTIONS

1 — 1.55' HMA (TYPE A)
0.35' CI 2 AB

2 — 0.15' HMA (TYPE A)

DESIGN DESIGNATION

ADT (2010)	78,000	D	51%
ADT (2030)	103,000	T	4.8%
DHV	5,500	V	70 MPH
ESAL	517,450	TI_{20}	13.5

TYPICAL CROSS SECTIONS

NO SCALE

X-1

REVISED BY	DATE REVISED
JF	12/15/10

CALCULATED-DESIGNED BY: JAY FONG
CHECKED BY: DAVID WILKERSON
FUNCTIONAL SUPERVISOR: VINCE BONNER

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans
DESIGN

LAST REVISION 05-19-11
DATE PLOTTED => 06-JUN-2012
TIME PLOTTED => 10:39

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	CC	580	5.5/6.2	3	301

REGISTERED CIVIL ENGINEER 5-27-11 DATE

6-4-12
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Jay Kin-Kwok Fong No. 72112 Exp. 6-30-12 CIVIL STATE OF CALIFORNIA

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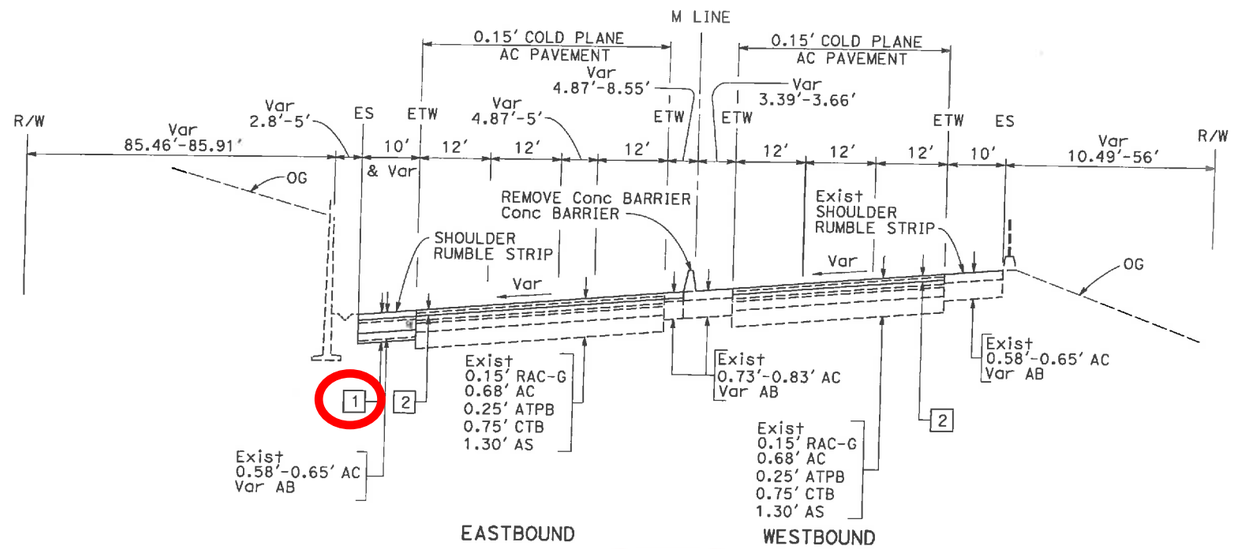
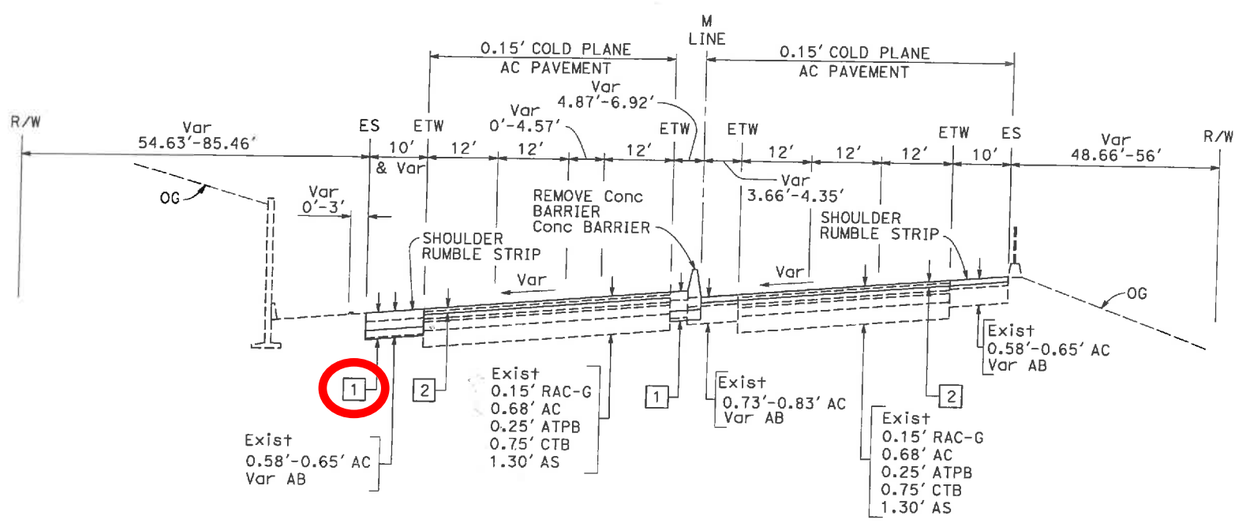


M 125+60 to 126+02.70



M 117+74.35 TO 125+60

TYPICAL CROSS SECTIONS

NO SCALE

FOR NOTES, ABBREVIATIONS AND LEGEND, SEE SHEET X-1

X-2

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans DESIGN

FUNCTIONAL SUPERVISOR: VINCE BONNER
CALCULATED-DESIGNED BY: JAY FONG
CHECKED BY: DAVID WILKERSON
REVISED BY: JF
DATE REVISED: 12/15/10

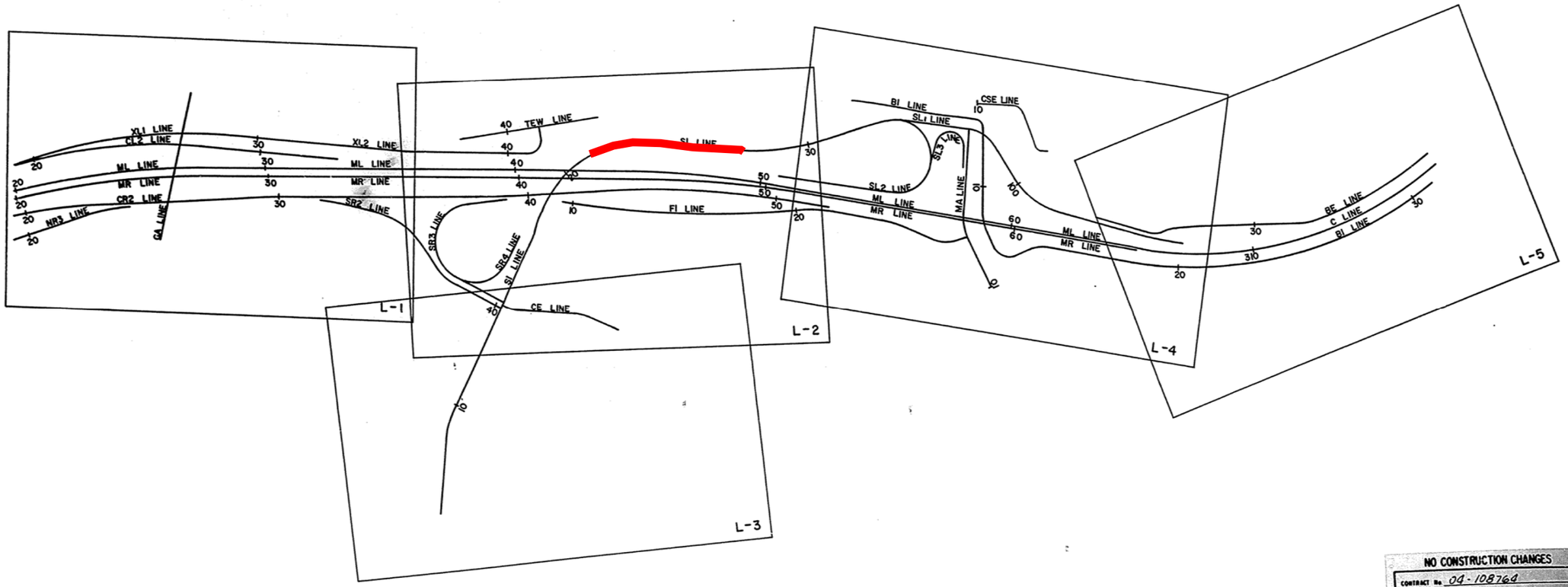
LAST REVISION 05-24-11
DATE PLOTTED => 06-JUN-2012
TIME PLOTTED => 12:42

DIST.	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	CC	580	T4.7/5.8	8	296

Victor R. Zeuzem
REGISTERED CIVIL ENGINEER

6-13-88
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
V. R. ZEUZEM
No. 26675
Exp. 6-30-92
CIVIL
STATE OF CALIFORNIA



NO CONSTRUCTION CHANGES
CONTRACT No. 04-108764
DATE ACCEPTED 6-16-92
AS BUILT
RESIDENT ENGINEER B. Braga

KEY LINE INDEX

SCALE : 1" = 200'

DATE	REVISED BY
	DATE REVISED

CALCULATED/DESIGNED BY
CHECKED BY

PROJECT ENGINEER
V. R. ZEUZEM

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans

FORM DC-OE-02-PF (REV. 1/86)

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES 0 1 2 3

CU 04174 EA 108761

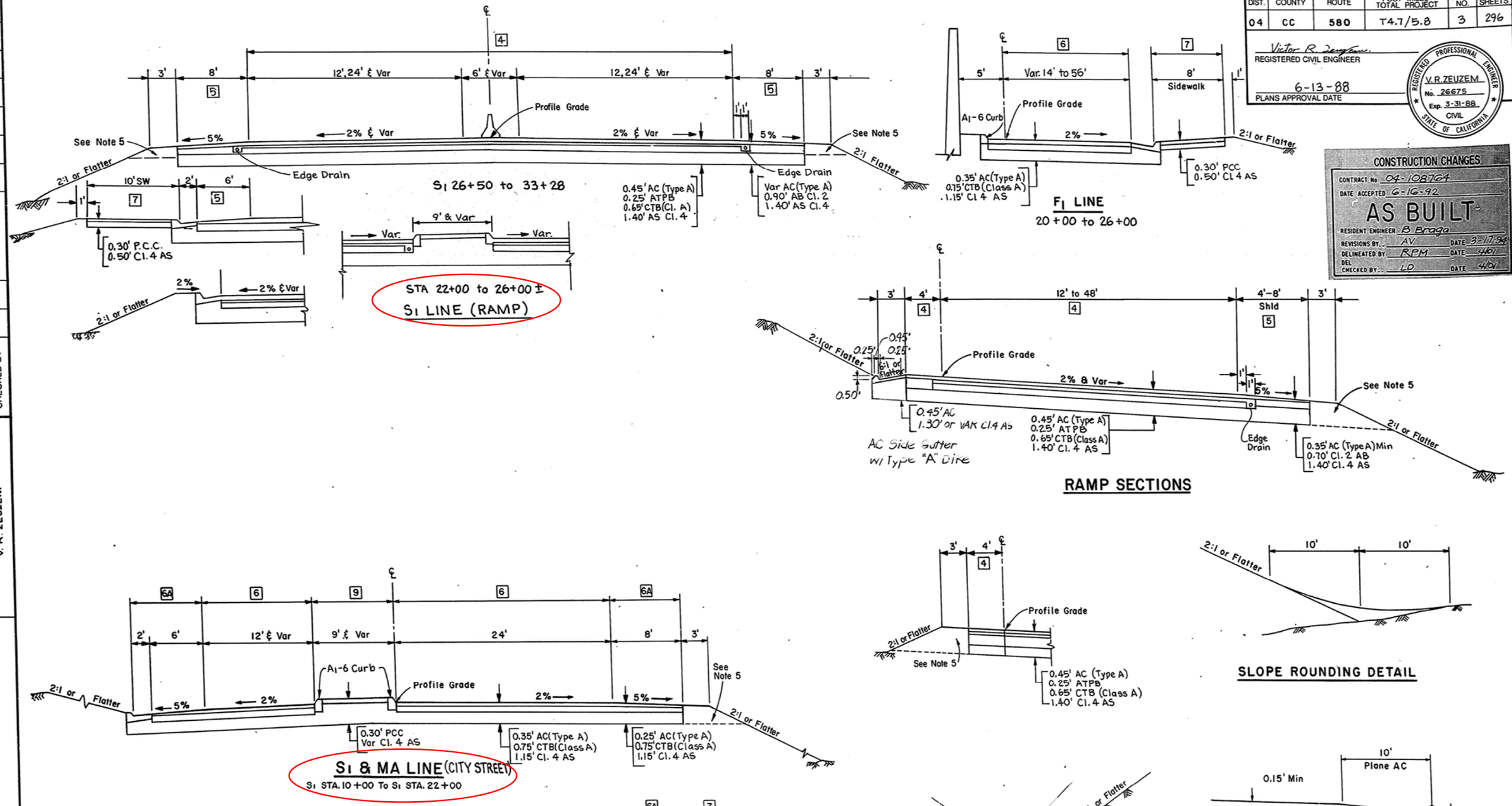
DIST.	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	CC	580	T4.7/5.8	3	296

Victor R. Zeuzem
REGISTERED CIVIL ENGINEER

6-13-88
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
V. R. ZEUZEM
No. 26675
Exp. 3-31-88
CIVIL
STATE OF CALIFORNIA

CONSTRUCTION CHANGES
CONTRACT No. 04-108764
DATE ACCEPTED 6-16-92
AS BUILT
RESIDENT ENGINEER B. Braga
REVISIONS BY AV DATE 3-17-94
DELINEATED BY RPM DATE 4/01
DEL.
CHECKED BY LD DATE 4/01



TYPICAL CROSS SECTIONS
NO SCALE

X-2

PROJECT ENGINEER V. R. ZEUZEM

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans

DATE	REVISED BY	DATE REVISED

CALCULATED/DESIGNED BY
CHECKED BY

FORM DC-OE-92-PF (REV. 1/86)

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES 0 1 2 3

CU 04174 EA 108761