Ann M. Stillman
Interim Director

County Government Center
555 County Center, 5th Floor
Redwood City, CA 94063
650-363-4100 T
650-361-8220 F
www.smcgov.org

May 6, 2022

COUNTY OF SAN MATEO

HIGGINS CANYON ROAD BRIDGE AT MILLS CREEK EMBANKMENT AND WINGWALL REPAIRS PROJECT

TOTAL PROJECT APPROXIMATELY 60 FEET IN LENGTH WITH APPURTENANT WORK THERETO IN SAN MATEO COUNTY

COUNTY PROJECT NO. RW909
PROJECT FILE NO. E5003

ADDENDUM NO. 1

TO ALL PLAN HOLDERS:

The following **Addendum No. 1** to the above referenced project, dated April 15, 2022, shall be included in the project plans and specifications.

1. Sheet 4 shall be replaced in the Project Plans:

   Replace Plan Sheet 4 of 20 with Sheet 4 of 20 (REV).

2. Page iv of the Table of Contents (Tablecon.E5003) Section shall be replaced in the Project Specifications:

   Replace page iv of the Tablecon Section with page iv (rev).

3. Page 48 of the Special Provisions (SP.E5003) Section shall be replaced in the Project Specifications:

   Replace page 48 of the SP.E5003 Section with page 48 (rev).

4. Appendix G shall be added to the Project Specifications:

   Appendix G cover sheet and Geotechnical Report (53 pages in total) shall be added to the Project Specifications after Appendix F.

Please sign and return the attached "Receipt of Addendum No. 1" form. The "Receipt of Addendum No. 1" form MUST be received in this office no later than 4:00 PM, Wednesday, May 11, 2022 or the bid will NOT be considered. The Receipt of Addendum can be emailed to Krzysztof Lisaj's attention email at klisaj@smcgov.org, with carbon copy to alum@smcgov.org and mmanalo@smcgov.org.

If you have any questions or require additional information, please contact Michelle Manalo, Anthony Lum, or Krzysztof Lisaj of our office at (650) 363-4100. They can also be reached by e-mail at:

mmanalo@smcgov.org
alum@smcgov.org
klisaj@smcgov.org

Very truly yours,

Ann M. Stillman
Interim Director of Public Works

AMS:KL:AL:MM
\\dpw.sanmateocounty.ads\data\Users\design\C3D\E5003000_Higgins Canyon Slip-Out\01 Bid Process\Addendums\Addendum #1\Addendum No.1 Cover Letter.doc

Encl.- "Receipt of Addendum No. 1" Form
Revised Plan Sheet 4 of 20 (rev)
Revised Page iv (rev) of the TableCon.E5003 Section
Revised Page 48 (rev) of the SP.E5003 Section
Appendix G Cover Sheet and Geotechnical Report dated 8/8/2019

cc: Krzysztof Lisaj, Principal Civil Engineer, Engineering and Construction
Anthony Lum, Senior Civil Engineer, Project Development and Design
Michelle Manalo Mason, Associate Engineer, Project Development and Design

Ann M. Stillman
Interim Director

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May 6, 2022

COUNTY OF SAN MATEO

HIGGINS CANYON ROAD BRIDGE AT MILLS CREEK EMBANKMENT AND WINGWALL REPAIRS PROJECT

COUNTY PROJECT NO. RW909
PROJECT FILE NO. E5003

RECEIPT OF ADDENDUM NO. 1

I, ____, an authorized representative for ____, have received **Addendum No. 1** for the Higgins Canyon Road Bridge at Mills Creek Embankment and Wingwall Repairs Project from an authorized representative of the County of San Mateo, which is to be included in the Specifications for the above referenced project.

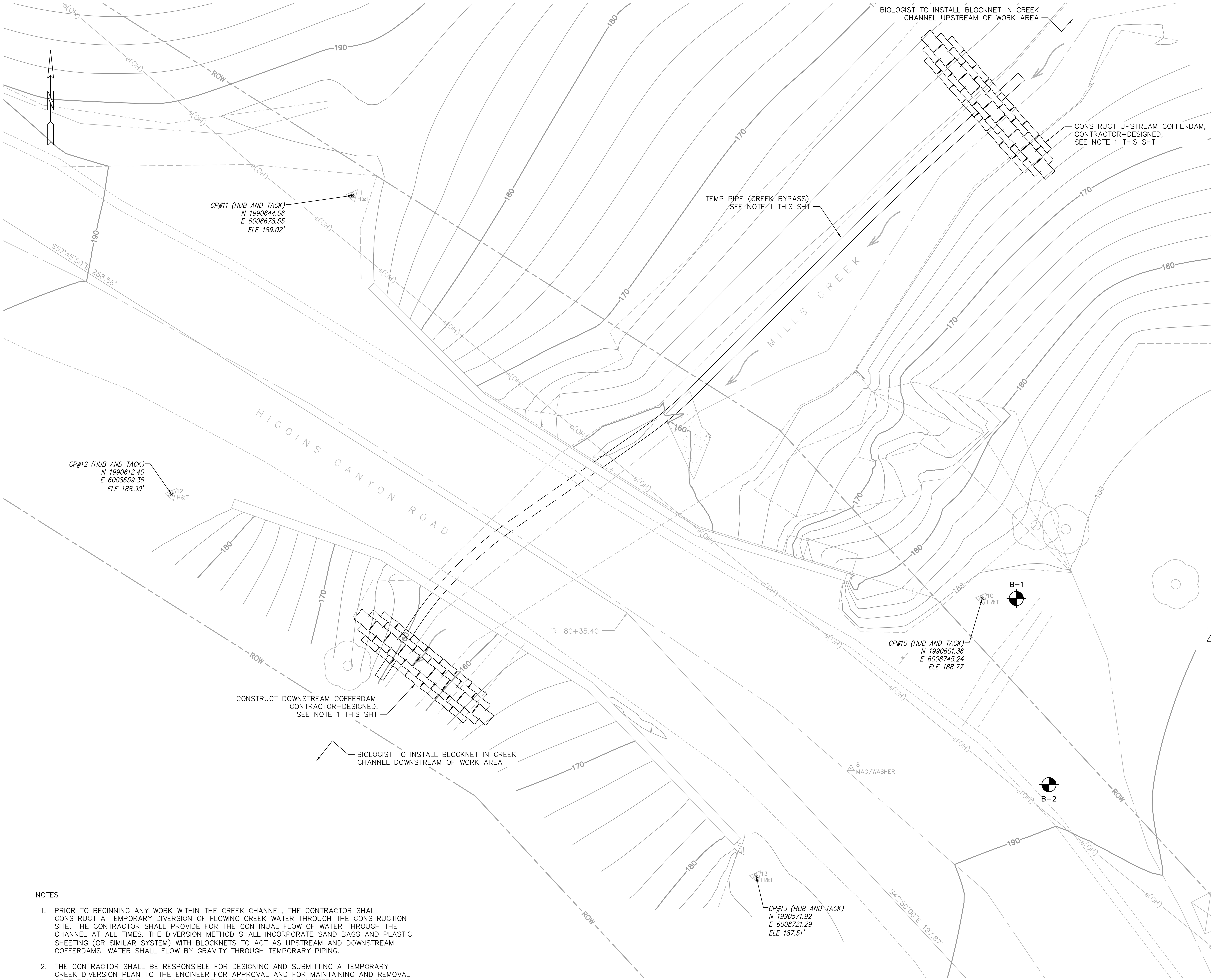
This form must be signed and received in the offices of the County of San Mateo, Department of Public Works ***no later than*** **4:00 P.M., Wednesday, May 11, 2022.**

"Contractor"

____
(Print)

____
(Signature)

____
(Date)



APPROVED:

DATE: 04/15/2022

ANN MADER STILLMAN, DIRECTOR OF PUBLIC WORKS
R. C. E. # 47882 / EXPIRES 12-31-2023

REGISTERED PROFESSIONAL ENGINEER ANN MADER STILLMAN C 47882 EXPIRES 12/31/23 CIVIL STATE OF CALIFORNIA

DIVERSION NOTES

THE DIVERSION PLAN SHOWN IS SCHEMATIC. GENERAL REQUIREMENTS ARE PROVIDED BELOW. THE FULL REQUIREMENTS OF THE DIVERSION AND DEWATERING PLAN ARE SPECIFIED IN THE PROJECT TECHNICAL SPECIFICATIONS.

1. GENERAL

1.1 DEWATER THE PROJECT SITE AS REQUIRED TO FACILITATE IN-STREAM CONSTRUCTION AND TO REDUCE POTENTIAL IMPACTS TO WATER QUALITY DOWNSTREAM OF THE PROJECT SITE.

1.2. CONFIRM THAT A FAVORABLE LONG TERM WEATHER FORECAST (1 WEEK, MIN.) IS OBSERVED PRIOR TO PLACEMENT OF DIVERSION STRUCTURES.

1.3. PRIOR TO PLACEMENT OF DIVERSION STRUCTURE, REMOVE FISH FROM THE PROJECT REACH, IN ACCORDANCE WITH SECTION 2.

1.4. DIVERT FLOW ONLY WHEN THE DIVERSION CONSTRUCTION IS OTHERWISE COMPLETE. FOLLOWING ENGINEER'S APPROVAL OF THE COMPLETED WORK, REMOVE DIVERSION BEGINNING AT THE DOWNSTREAM LIMIT, IN AN UPSTREAM DIRECTION.

2. FISH REMOVAL

2.1 FISH RELOCATION SHALL BE COORDINATED WITH THE COUNTY'S BIOLOGIST PRIOR TO INSTALLING THE TEMPORARY CREEK DIVERSION SYSTEM. PROVIDE NO LESS THAN 72 HOUR NOTIFICATION TO THE COUNTY IN ADVANCE OF TEMPORARY CREEK DIVERSION WORK.

2.2. BLOCK NETS SHALL BE PROVIDED AND INSTALLED BY THE BIOLOGIST. BLOCK NETS SHALL BE MAINTAINED BY THE CONTRACTOR BOTH UPSTREAM AND DOWNSTREAM OF THE DIVERSION, THROUGHOUT THE PERIOD OF CONSTRUCTION. MAINTENANCE INCLUDES PERIODIC REMOVAL OF ACCUMULATED DEBRIS, AS NECESSARY TO ENSURE FUNCTION. BLOCK NETS SHALL BE REMOVED BY THE BIOLOGIST AFTER THE DIVERSION IS REMOVED AND THE IN CHANNEL WORK AREA IS RE-WATERED.

3. DIVERSION SYSTEM

3.1. INSTALL A SEALED, TEMPORARY DIVERSION DAM CONSTRUCTED USING GRAVEL FILLED BAGS TO CAPTURE AND DIVERT STREAM FLOW UPSTREAM OF THE PROJECT SITE. THE DAM AND METHOD OF SEALING SHALL BE PLACED AT AN APPROPRIATE DEPTH TO CAPTURE SUBSURFACE STREAM FLOW, AS NEEDED TO DEWATER THE STREAMBED. GRAVEL SHALL BE WASHED PRIOR TO PLACEMENT IN BAGS. THE USE OF SAND WILL NOT BE ALLOWED. NO OTHER DIVERSION METHOD SHALL BE USED WITHOUT AUTHORIZATION OF THE ENGINEER. IF AN ALTERNATE DIVERSION METHOD IS PREFERRED BY THE CONTRACTOR, THE CONTRACTOR SHALL SUBMIT A PLAN TO THE ENGINEER FOR APPROVAL, DETAILING THE DESIRED DIVERSION METHOD.

3.2. THE DIVERSION STRUCTURE SHALL BE CONSTRUCTED AS SHOWN ON THE PLANS AND AS DIRECTED BY THE ENGINEER IN THE FIELD.

3.3. IN THE EVENT OF A SIGNIFICANT STORM, THE CONTRACTOR SHALL BE PREPARED TO TAKE NECESSARY MEASURES TO INSURE SAFE PASSAGE OF STORM WATER FLOW THROUGH THE PROJECT AREA, WITHOUT DAMAGE TO EXISTING STRUCTURES, OR INTRODUCTION OF EXCESSIVE SEDIMENT. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ALL TEMPORARY EROSION CONTROL B.M.P.'S.

1 3.4. THE DIVERSION SHALL BE CAPABLE OF CONVEYING A MINIMUM OF 10 CFS (ADULT HIGH FLOW) WITH LESS THAN 6 INCHES OF HEAD OVER THE TOP OF PIPE AT THE INLET, AND SHALL BE A MINIMUM DIAMETER OF 10", WITH A MANNING'S ROUGHNESS NOT EXCEEDING .012.

4. DEWATERING OF CONSTRUCTION AREAS

4.1. THE CONTRACTOR SHALL SUPPLY ALL NECESSARY PUMPS, PIPING, FILTERS, SHORING, AND OTHER TOOLS AND MATERIALS NECESSARY FOR DEWATERING. IF A PUMPED SYSTEM IS RELIED UPON TO ENSURE DOWNSTREAM WATER QUALITY, A BACKUP PUMP OF EQUAL CAPACITY SHALL BE PROVIDED AT ALL TIMES AND THE PUMP MUST BE CONTINUOUSLY MONITORED.

4.2. DEWATERING ACTIVITIES WHICH MAY BE REQUIRED FOR CONSTRUCTION PURPOSES SHALL COMPLY WITH WATER QUALITY STANDARDS ISSUED BY THE CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD.

4.3. DISCHARGE OF WATER FROM THE DEWATERED CONSTRUCTION SITE, EITHER BY GRAVITY OR PUMPING, SHALL BE PERFORMED IN A MANNER THAT PREVENTS EXCESSIVE TURBIDITY FROM ENTERING THE RECEIVING WATERWAYS AND PREVENTS SCOUR AND EROSION OUTSIDE OF THE CONSTRUCTION SITE. PUMPED WATER SHOULD BE PRE-FILTERED WITH A GRAVEL PACK AROUND SUMPS FOR SUBSURFACE FLOWS AND A SILT FENCE AROUND PUMPS FOR SURFACE FLOW. PUMPED WATER SHALL BE DISCHARGED INTO ISOLATED LOCAL DEPRESSIONS, FILTER BAGS, SETTLING (BAKER) TANKS, OR TEMPORARY SEDIMENT BASINS, AS NECESSARY TO MEET WATER QUALITY REQUIREMENTS. WHERE WATER TO BE DISCHARGED INTO MILLS CREEK WILL CREATE EXCESSIVE TURBIDITY, THE WATER SHALL BE ROUTED THROUGH A SEDIMENT INTERCEPTOR OR OTHER FACILITIES TO REMOVE SEDIMENT FROM WATER.

NOTES

1. PRIOR TO BEGINNING ANY WORK WITHIN THE CREEK CHANNEL, THE CONTRACTOR SHALL CONSTRUCT A TEMPORARY DIVERSION OF FLOWING CREEK WATER THROUGH THE CONSTRUCTION SITE. THE CONTRACTOR SHALL PROVIDE FOR THE CONTINUAL FLOW OF WATER THROUGH THE CHANNEL AT ALL TIMES. THE DIVERSION METHOD SHALL INCORPORATE SAND BAGS AND PLASTIC SHEETING (OR SIMILAR SYSTEM) WITH BLOCKNETS TO ACT AS UPSTREAM AND DOWNSTREAM COFFERDAMS. WATER SHALL FLOW BY GRAVITY THROUGH TEMPORARY PIPING.
2. THE CONTRACTOR SHALL BE RESPONSIBLE FOR DESIGNING AND SUBMITTING A TEMPORARY CREEK DIVERSION PLAN TO THE ENGINEER FOR APPROVAL AND FOR MAINTAINING AND REMOVAL OF THE SYSTEM. THE PLAN SHALL INCLUDE JUSTIFICATION OF ALL COFFERDAM AND PIPE SIZING.
3. CONTRACTOR SHALL COORDINATE WITH THE PROJECT BIOLOGIST IN THE FIELD TO CONFIRM THE APPROPRIATE LOCATIONS FOR THE DIVERSION PIPE, BLOCKNETS AND COFFERDAMS, AND MAKE FIELD ADJUSTMENTS, AS NEEDED.

5 0 5 10
SCALE FEET

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CE&G CAL ENGINEERING & GEOLOGY
6455 Almaden Expwy.
Suite 100
San Jose, CA 95120
Phone: (408) 440-4542

APPROVED DATE: APRIL 15, 2022	
CHRIS HOCKETT, P.E., G.E., PRINCIPAL ENGINEER	
CAL ENGINEERING & GEOLOGY, INC.	
R.C.E. # 71938, R.G.E. # 2928 / EXP 12-31-23	

REGISTERED PROFESSIONAL ENGINEER CHRISTOPHER A. HOCKETT No. Geo GE 2928 No. Civil C 71938 CIVIL GEOTECHNICAL STATE OF CALIFORNIA

COUNTY OF SAN MATEO PUBLIC WORKS

REVISION	DATE		
1	5/4/2022	DESIGNED BY: KF	DRAWN BY: EV
		CHECKED BY: CH	

HIGGINS CANYON ROAD BRIDGE AT MILLS CREEK
EMBANKMENT AND WINGWALL REPAIRS
CREEK PROTECTION AND BYPASS PLAN

SCALE: 1"=5'
DATE: APR 2022
FILE NO.: E5003

ANN MADER STILLMAN, DIRECTOR OF PUBLIC WORKS
SAN MATEO COUNTY

555 COUNTY CENTER, 5th FLOOR
REDWOOD CITY, CALIFORNIA 94063

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES 0 1 2 3 4

SHEET 4 OF 20 (REV)

TABLE OF CONTENTS (Continued)

specified in these Special Provisions, and as directed by the Engineer. Reference is made to Appendix F of these Special Provisions.

B. Execution

The Contractor shall provide for the continual flow of creek water at all times without disturbance. The diversion method shall incorporate cleaned washed gravel sandbags and plastic sheeting or inflatable dams to act as upstream and downstream cofferdams. Water shall be allowed to flow by gravity through temporary piping.

The Contractor shall design the temporary creek diversion for a minimum flow rate of ~~150~~ **10** cubic feet per second. **The Contractor shall upsize the temporary bypass system as needed and as directed by the Engineer for no additional compensation.**

C. Submittals

The Contractor shall submit a temporary creek diversion plan to the Engineer at the pre-construction meeting. The plan shall include calculations to justify all cofferdam and pipe sizing, and timing of cofferdam relocation, as applicable. The Engineer will have seven calendar days to review the temporary creek diversion plan.

D. Measurement and Payment

Full compensation for all work involved for this item, “Temporary Creek Diversion System,” shall be as specified in Section 9 of the Standard Specifications and these Special Provisions.

END OF SECTION

Appendix G

Cal Engineering & Geology's
Geotechnical Report dated 8/8/2019

CE&G
Pragmatic Expertise™

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6455 Almaden Expwy., Suite 100| San José | CA 95120
23785 Cabot Blvd., Suite 321 | Hayward | CA 94545
www.caleng.com

GEOTECHNICAL DESIGN REPORT

HIGGINS CANYON ROAD LANDSLIDE REPAIR PROJECT

CE&G DOCUMENT: 190360.001

AUGUST 8, 2019

Prepared for:

San Mateo County Department of Public Works
555 County Center
Redwood City, California 94063

Dan Peluso, P.E., G.E.
Principal Engineer
C49562, GE2367

REGISTERED PROFESSIONAL ENGINEER
DANIEL J. PELUSO
GE 2367
GEOTECHNICAL
STATE OF CALIFORNIA

Kevin Loeb, P.G.
Project Geologist

PROFESSIONAL GEOLOGIST
KEVIN P. LOEB
No. 9665
STATE OF CALIFORNIA

TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 GENERAL

San Mateo County has continuously suffered extensive roadway damage due to a series of storms. Along Higgins Canyon Road near the one-lane bridge over Mills Creek, a slide occurred immediately adjacent to another landslide that reportedly occurred in 2015; the recent slide appears to have enlarged the older slide and encroached to the edge of the roadway and adjacent to the bridge wingwall, resulting in a loss of foundation material for the north side of the roadway embankment. To repair the slip-out, a retaining wall is recommended to support the road embankment for Higgins Canyon Road. An underpinning system is also proposed to support the existing southeast wingwall adjacent to the south bridge abutment using a system of cast-in-drilled-hole (CIDH) piles and pile caps.

In support of the San Mateo County Public Works Department (County), Cal Engineering & Geology's (CE&G) work included compiling and reviewing available relevant geotechnical and geologic data; performing a preliminary field reconnaissance, a field exploration, laboratory testing program and a geotechnical engineering analysis which consists of developing geotechnical design recommendations for the proposed improvements; and preparing this report. The work has been completed to collect geotechnical data and provide a geotechnical investigation, engineering analysis and geotechnical design recommendations for the design team to design a steel soldier pile and concrete lagging retaining wall to be constructed at the site for the slope stabilization and an underpinning system for the exposed wingwall.

1.2 PROJECT DESCRIPTION

During heavy rains in early February 2019, a landslide occurred beneath Higgins Canyon Road. The location of the project is shown on the Site Location Map (Figure 1). Heavy rains have caused large amount of runoff, saturating the slope soils and causing the water level to rise in the creek. It seems that the sliding slope and removal of foundation materials of the wingwall were due to the saturated embankment fill and adjacent slope soils. The following photos from the CE&G reconnaissance visit show the damage.

View of the landslide looking down from the south roadway shoulder. Note the exposed communication conduit.

View of the landslide looking south along the roadway.

View of the southeast wingwall looking west.


View of the undermined portion of the southeast wingwall.

In developing the scope of the design services, several slope repair designs to stabilize the slope have been considered. Among these preliminary design alternatives are: 1) a soldier beam wall with tiebacks and concrete laggings; 2) a soil nail wall system with shotcrete temporary facing and permanent concrete facing; and 3) a mechanically stabilized reinforced retaining wall. An underpinning system is considered to support the exposed wingwall.

After preliminary analysis and discussions with the County, a soldier pile wall with tiebacks and concrete lagging was deemed most appropriate for design to stabilize the slope, with an underpinning system to stabilize the wingwall foundation.

1.3 PURPOSE AND SCOPE OF SERVICES

The investigation completed by CE&G was undertaken to assess the existing surface and subsurface conditions in the immediate vicinity of the proposed project, and to develop geotechnical design recommendations for the proposed improvements.

The scope of work completed for the geotechnical investigation and report included:

1. Meetings and consultation with San Mateo County personnel and management of geotechnical explorations.

2. Completion of an office study to identify and evaluate relevant geologic and geotechnical information available for the site, including published geologic maps, and unpublished geotechnical information in our files regarding the site and vicinity.

3. Geologic reconnaissance to observe and map current site conditions, and to mark for USA (Underground Service Alert)

4. Subsurface exploration using a track-mounted drill rig, in accordance with a Drilling Permit facilitated by the County.

5. Performance of a laboratory testing program to determine key engineering index properties of selected earth materials.

6. Completion of engineering analyses to develop and evaluate alternative geotechnical approaches to restore the roadway embankment, and to develop parameters for the design of the retaining wall.

7. Preparation of a geotechnical investigation report.

2.0 SITE CONDITIONS AND DESCRIPTION

A topographic survey of the site was prepared by Ruggeri-Jensen-Azar (RJA) dated April 22, 2019 for the County and provided to CE&G. Site topography is shown on Figure 2. The location of the landslide is next to a wing wall of the bridge, which is built along Higgins Canyon Road crossing Mills Creek. This bridge consists of a narrow "two-lane" paved roadway about 15 feet in width. Higgins Canyon Road in the site vicinity is oriented approximately south-north. The slide is approximately 20 feet in width, with material sliding more than 25 feet. Elevations at the site of the landslide range from about 188 feet above sea level (asl) at the top of the landslide to 162 feet asl at the edge of the creek channel below. Natural slopes below the roadway have a gradient of about 1 horizontal to 1 vertical.

The site is in moderately steep, hilly terrain with Mills Creek flowing below. The site is in a moderately densely forested area, with a mix of various native trees interspersed with a brushy understory. Excessive surface and/or subsurface water runoff appears to have been involved in the occurrence of the landslide. Due to the erosion from fluctuating water level, the wingwall adjacent to the landslide was undermined and the conduit going through the bridge was exposed. Therefore, the site is highly vulnerable to erosion and erosion protection may be desirable to prevent further problems.

The recent slide occurs at a location beneath the previous reported landslide in 2015 and has a broadened width.

3.0 GEOLOGY

3.1 GEOLOGIC SETTING

The Higgins Canyon Road site lies within the Santa Cruz Mountains, within the Coast Ranges geomorphic province of California (Figure 3). This province is characterized by northwest-southeast trending mountain ranges and intervening valleys such as that occupied by San Francisco Bay and the Santa Clara Valley. The Santa Cruz Mountains are one such range, marking a mountain-range scale regional uplift southwest of the San Andreas fault. The geologic setting is shown on the Regional Geologic Map (Figure 3).

3.1.1 Bedrock Geology

Regional geologic mapping by Brabb and others (1998), shows the area as being comprised of coarse-grained alluvium and pockets of colluvium, which are underlain by Purisima Formation (Pliocene and Upper Miocene) bedrock. The Purisima Formation is described as typically consisting of "predominantly gray and greenish-gray to buff, fine-grained sandstone, siltstone, and mudstone, but also includes some porcelaneous shale and mudstone, chert, silty mudstone, and volcanic ash" (Brabb and others, 1998). Structures mapped by Brabb and others (1998) in the vicinity of the project site show the Purisima Formation generally dipping 28° to 41° to the southwest. The Holocene age colluvium is described as "loose to firm, friable, unsorted sand, silt, clay, gravel, rock debris, and organic material in varying proportions" and the alluvium is described as "poorly consolidated gravel, sand, and silt, coarser-grained at heads of old fans" (Brabb and others, 1998).

The regional geology has also been mapped by Graymer and others (2006), which shows the site mapped as undifferentiated sedimentary rocks of Pliocene and/or early Miocene age.

3.2 GEOHAZARD MAPPING

3.2.1 State and Regional Geohazard Mapping

The California Geological Survey (CGS), previously the Division of Mines and Geology, setup a Special Studies Zones map for the Half Moon Bay 7.5-minute Quadrangle (CGS, 1976). The project site does not lie within an Alquist-Priolo Earthquake Fault Zone. A Summary of Distribution of Slides and Earthflows in San Mateo County map prepared by Wentworth and others (1997) show the project site as being one of few landslides, but with areas of mostly landslides lying above it.

3.2.2 Local Geohazard Mapping

San Mateo County's Local Hazard Mitigation Plan (LHMP), dated July 2016, shows a series of generalized hazard zone maps, including, but not limited to, liquefaction and landslide potential/hazard zones. These maps were compiled using data generated by the U.S. Geological Survey (USGS), California Geological Survey (CGS), and the Association of Bay Area Governments Resilience Program (ABAG). The site is mapped in an area designated as having a "low to moderate" liquefaction susceptibility and does not appear to be in an area of potential landslide hazards and steep slopes (County of San Mateo, Local Hazard Mitigation Plan [https://planning.smcgov.org/local-hazard-mitigation-plan], accessed July 2019).

3.3 REGIONAL GROUNDWATER

Groundwater within the hillslope areas encompassing the site is likely variable, with the water table commonly sloping downhill toward the closest drainage axis. We did not identify long-term springs and seeps in the site vicinity, although seasonal expressions of these are likely present seasonally.

3.4 SEISMICITY

3.4.1 Active Faults

The Higgins Canyon Road site is located within the greater San Francisco Bay Area, which is recognized as one of the more seismically active regions of California. The right-lateral strike-slip San Andreas fault system controls the northwest-southeast structural grain of the Coast Ranges and the Bay Area. The fault system marks the major boundary between two of earth's tectonic plates, the Pacific Plate on the west and the North American Plate on the east. The Pacific Plate is moving north relative to the North American plate at approximately 40 mm/yr in the Bay Area (WGCEP, 2003).

The transform boundary between these two plates has resulted in a broad zone of multiple, subparallel faults within the North American Plate, along which right-lateral strike-slip faulting predominates. In this broad transform boundary, the San Andreas Fault accommodates less than half of the average total relative plate motion. Much of the remainder in the greater south Bay Area is distributed across faults such as the San Gregorio, Monte Vista-Shannon, Cascade, Zayante-Vergeles, Sargent, and Berrocal fault zones.

Since the Higgins Canyon Road site is in the seismically active San Francisco Bay Area, it will likely experience significant ground shaking (moment magnitude greater than 7.0) from one or more of the nearby active faults during the design lifetime of the project. Some major seismic sources in the San Francisco Bay area and their distances from the site are summarized in Table 3-1. Seismogenic (capable of generating significant earthquakes) earthquake faults near the site include the San Gregorio, San Andreas, and Monte Vista-Shannon faults.

Table 3-1. Distances to Selected Major Active Faults

Fault Name	Distance and Direction from Site to Surface Fault Traces
San Gregorio	4.4 km southwest
Pilarcitos	4.7 km northeast
San Andreas	7.6 km northeast
Monte Vista-Shannon	18.4 km southeast
Hayward (southern segment)	37.8 km northeast
Calaveras	49.2 km northeast

3.4.2 Liquefaction and Seismic Densification

Soil liquefaction is a phenomenon in which saturated, cohesionless soils (generally sands) lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated, clean, loose, fine-grained sands and silts. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type and relative density; 3) overburden pressure; and 4) depth to ground water.

Based on subsurface information collected during this investigation, we judge the potential for liquefaction at this site to be low. Bedrock materials unlikely to liquify were encountered below the water table. Loose to medium dense clayey sand with some fines were encountered, but above the water table.

Seismic densification is the densification of unsaturated, loose to medium dense granular soils due to strong vibration such as that resulting from earthquake shaking. There is a potential for seismic densification of some of the on-site alluvial soils, which is discussed in more detail below. We note that the proposed repair would most likely remove a portion of these soils in the area of the repair.

4.0 SITE INVESTIGATION

4.1 PREVIOUS INVESTIGATIONS

As noted above, we are not aware of previous geotechnical investigations at the site. According to information from San Mateo County, there was a reported landslide, which was located at the east upper side of the recent landslide, occurring in 2015.

4.2 SITE RECONNAISSANCE

CE&G performed field reconnaissance of the site in advance of and on the day of our subsurface investigation.

Bedrock was exposed in the north side of creek, but not found in the south side of creek, which is the area beneath landslide. Due to the landslide and erosion, the conduit going through the bridge was exposed and the foundation of the wing wall was undermined. A temporary swale has been dug out approximately 8 feet south from the top of landslide to prevent potential sheet-flow from draining over the scarp. The landslide is approximately 20 feet in width and 26 feet high. While much of the landslide mass has been eroded and carried downstream during heavy creek flows, we estimate the form of the landslide was that of a slump-earth flow. There was no evidence observed that would suggest deep-seated landsliding.

Considering the observation that the foundation of the wing wall was undermined, and the conduit was exposed, a much higher water level within the creek channel was present around the time of the landslide. The erosion of foundation soils are possibly attributed to both surface and subsurface water movement and fluctuation.

4.3 SUBSURFACE EXPLORATION

4.3.1 Scope of Explorations

Two geotechnical borings were drilled at the top of the damaged slope. The locations of the completed borings were marked in the field and recorded by measuring with a tape from established points of reference. The approximate boring locations are shown on Figure 2, Site Plan.

The geotechnical borings were drilled by Britton Exploration, LLC on April 2, 2019, using a track-mounted CME-55 drill rig equipped with 8-inch hollow stem augers. Surface conditions at the borings were soil near the outboard shoulder of the roadway. Both borings B-1 and B-2 was drilled to a total depth of approximately 31.5 feet below ground surface (bgs).

Upon completion, the borings were backfilled with cement grout. The surface of the borings were restored to match the existing grade. Drilling spoils were distributed unobtrusively on site.

4.3.2 Logging and Sampling

The materials encountered in the borings were logged in the field by an engineering geologist. The soils were visually classified in the field, office, and laboratory according to the Unified Soil Classification System (USCS) in general accordance with ASTM D2487 and D2488.

During the drilling operations, soil samples were obtained using one of the following sampling methods:

- California Modified (CM) Sampler; 3.0-inch outer diameter (O.D.), 2.5-inch inner diameter (I.D.) (ASTM D1586)

- Standard Penetration Test (SPT) Split Spoon Sampler; 2.0-inch O.D., 1.375-inch I.D. (ASTM D1586)

The samplers were driven 18 inches (unless otherwise noted on the boring logs) with a 140-pound cable drop hammer dropping 30 inches. The number of blows required to drive the SPT or CM sampler 6 inches were recorded for each sample. The results are included on the boring logs in Appendix A. The blow counts included on the boring logs are uncorrected and represent the field values.

Soil samples obtained from the borings were packaged and sealed in the field to reduce the potential for moisture loss and disturbance. The samples were taken to Cooper Testing Labs, in Palo Alto, for laboratory testing and storage.

4.3.3 Subsurface Conditions Encountered

Subsurface soil conditions encountered in our borings were generally consistent with regional geologic mapping.

The ground surface at each boring consisted of soil with low grasses. From the ground surface, approximately 5 to 7 feet of medium stiff sandy lean clay was underlain by 10 to 18 feet of medium dense clayey sand materials. Beneath the alluvial deposits, bedrock

(Purisima Formation) consisting of sandstone and shale was encountered to the maximum depths explored. Both borings B-1 and B-2 terminated at a depth of 31.5 feet.

For a more detailed description of the soils encountered in the borings, the logs of the borings and laboratory test results are included in Appendices A and B.

4.3.4 Groundwater Conditions Encountered

Groundwater was encountered in both Borings B-1 and B-2, at a depth of about 23 ft and 25 ft, respectively.

4.4 GEOTECHNICAL LABORATORY TESTING

Laboratory testing was performed to obtain information regarding the physical and index properties of selected samples recovered from the exploratory borings. Tests performed included natural moisture content, dry unit weight, grain size distribution with #200 sieve wash analysis, liquid and plastic limits test, unconsolidated-undrained triaxial test, and corrosivity test. Tests were completed in general conformance with applicable ASTM standards. The results of the laboratory tests are summarized on the boring logs in Appendix A and in Appendix B.

5.0 CONCLUSIONS AND DISCUSION

5.1 GENERAL SUMMARY

Based on the results of our investigation, it is our opinion the site is geologically and geotechnically suitable for a retaining wall to stabilize the slope, and underpinning to reinforce the wing wall, provided the recommendations presented in this report are followed.

Geotechnical recommendations for design and construction of the proposed improvements are presented in the "Recommendations" section of this report.

5.2 LANDSLIDING

As previously described, no evidence of deep-seated landsliding was detected at the site. Relatively restricted shallow sloughing (landsliding) of alluvium and the uppermost, severely weathered bedrock appears to have been involved in the landslide. Such shallow instability was likely associated with the concentration of surface runoff from the roadway onto the slope below the roadway and the water level rising within the Mills Creek channel.

In our judgment, the potential for deep-seated landsliding (involving bedrock) to adversely affect the site improvements is low under static conditions, and low to moderate under seismic conditions.

We also judge the potential for shallow-seated landsliding (under static and seismic conditions) to adversely affect the site improvements to be low, provided site improvements are appropriately designed and constructed and surface runoff is appropriately managed.

5.3 SEISMIC HAZARDS

Large magnitude earthquakes and strong ground shaking are likely to affect the project area within the design lifetime of the proposed improvements. Peak ground shaking parameters are presented below in Section 6.2 and should be considered in the design of the proposed improvements. Local ground-modifying effects of high intensity ground shaking are considered secondary seismic effects. Our review of these processes is presented below.

- In our judgment the potential for fault ground rupture or coseismic faulting to significantly affect the proposed improvements is low.

- In our judgment the potential for ridgetop fissuring, ridgetop shattering, ridgetop spreading or other seismically induced ground deformation to significantly affect the proposed improvements is low.

- In our judgment the potential for soil liquefaction to significantly affect the proposed project is low due to the absence of loose to medium dense granular soils below the groundwater table.

5.4 CORROSION

Corrosion testing was performed on one soil sample at this location in general accordance with Caltrans methods. Testing results are presented below:

Table 5-1: Corrosion Testing Results

Boring (depth in feet)	Resistivity (Ohm-cm)	Chloride (mg/kg)	Sulfate (mg/kg)	pH
B-2 (6)	2593	29	131	7.5

Caltrans Corrosion Guidelines, January 2015, identifies a site to be corrosive for structural elements if one or more of the following conditions exist:

- Chloride concentration is 500 ppm or greater;
- Sulfate concentration is 2000 ppm or greater;
- pH is 5.5 or less.

A minimum resistivity value for soil and/or water less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion. Based on the results of the laboratory testing performed, the soil sample tested had values for Chloride, Sulfate, pH that do not meet the Caltrans criteria for a corrosive site. The resistivity of the tested soil sample was above the 1000 ohm-cm threshold defined.

According to ACI 318 Section 4.3, Table 4.3.1:

- Sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible (no restrictions on concrete type)
- Water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to concrete.

Based on the results of the laboratory testing performed, the soil sample tested had values for Sulfate and Chloride that do not meet ACI criteria and is considered non-corrosive to concrete.

Corrosion results are to be considered preliminary and are an indicator of potential soil corrosivity for the sample tested. Other soils found onsite may be more, less, or of similar corrosive nature. Our scope of services does not include corrosion engineering; therefore, a detailed analysis of the corrosion tests is not included.

5.5 GEOTECHNICAL CONSIDERATIONS

Significant geotechnical issues that will affect the design and construction of the proposed retaining wall and roadway are as follows:

- **Retaining Wall** –Based on the site geometry, depth to competent materials, and proposed retaining wall layout, the anticipated wall heights are achievable through conventional cantilever wall design, or through a tieback wall approach. Advantages to using a cantilever wall include simpler design and fewer steps in construction, whereas disadvantages include the need for a larger steel section, longer steel, and deeper pier hole excavation. Advantages to a tieback wall include shorter steel and smaller steel section, whereas disadvantages include and more complex design, more steps in the construction and the need to install tiebacks. Recommendations have been provided in Section 6 for the retaining wall.

- **Underpinning System** – The exposed foundation of the bridge abutment wingwall will be underpinned to prevent further damage to the to the bridge system. Underpinning is to consist of drilled piers structurally attached to the wing wall to transfer loads to the piers and underlying bedrock.

- **Surface Water Drainage** – Surface water runoff should be collected from the roadway above and discharged in an appropriate energy dissipater away from the slide area below the proposed repair. Surface drainage improvements should be designed to adequately collect and accommodate the volume of water that reach these drainages.

- **Drillability** – Subsurface exploration was complete using hollow stem augers and did not encounter auger refusal to the maximum depths explored of 31.5 feet. Based on the subsurface exploration, we anticipate than an appropriately sized drill rig equipped with rock bits will be able to drill through the soil and bedrock underlying the project site.

6.0 RECOMMENDATIONS

6.1 EARTHWORK

Grading required for the project will include excavation to develop temporary site access and create a bench for drilling and construction of the soldier piles and tiebacks. This bench will also be needed for placement of excavated material as engineered fill in order to approximately restore original grade at the site. Minor grading could also be required to modify or construct drainage facilities and to distribute excess excavated material onsite, as appropriate.

Prior to commencement of the grading operation, the site should be cleared and grubbed of existing vegetation. The conduit conveying AT&T utility lines is exposed and overhead utility lines go across the bridge and are close to the slope repair area. Care should be taken not to damage any utilities present. This should be done in coordination with utility providers. All existing structures and debris should be removed from the site, including but not limited to: existing pavement, foundations systems, buried pipes, etc. Prior to placement of engineered fill, loose soil and vegetation should be removed from the areas to receive fill. All depressions created by tree and stump removal and demolition of structures should be excavated to firm soil or bedrock prior to placement of fill.

6.1.1 Engineered Fill Placement and Compaction

On-site soils that will likely be excavated as part of the retaining wall construction may be used as general engineered fill. Fill should be placed and compacted to a relative compaction of 90 percent ASTM D-1557 latest edition. Fill materials shall be spread evenly and compacted in uniform lifts not exceeding 8 inches in uncompacted thickness. Fill materials which do not meet the specified relative compaction shall be ripped, moisture conditioned, and re-worked until the required relative compaction and moisture content are attained.

6.1.2 Select Import Backfill

All imported fill must be reviewed and approved by the geotechnical engineer prior to importation to the site. A minimum of five days will be required to evaluate and test the suitability of all proposed imported materials. All select import backfill materials should meet the following criteria:

The import materials shall be non-expansive and have a Plasticity Index less than 12 percent and a Liquid Limit of 30 percent or less with a minimum friction angle of 34 degrees. The import material shall not contain rocks or lumps larger than 6 inches in greatest dimension and should not contain more than 15 percent of the material larger than 3 inches. These materials shall be free of organic debris or contaminated materials.

Imported fill materials should be placed and compacted to a minimum of 90 percent relative compaction at a moisture content of at least 2 percent over optimum as determined by the ASTM D-1557 (latest revision) test procedure. Fill material in the upper 24 inches of the pavement subgrade shall be compacted to a minimum of 95 percent relative compaction.

6.1.3 Wet Weather Construction

We recommend that earthwork not be performed during wet weather seasons. If site grading and construction is to be performed during the rainy periods, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms could cause unstable excavations, delays to construction and damage to previously completed work by saturating compacted fills or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The contractor should be responsible to protect his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

6.2 SEISMIC DESIGN PARAMETERS

As a result of earthquake shaking, the soil or bedrock behind the retaining walls will exert an additional horizontal force on the walls. We recommend using an additional seismic equivalent fluid pressure (EFP) of 13 pcf to model the earthquake-induced force on the walls The seismic equivalent fluid pressure was selected based on the design response spectrum peak ground acceleration (PGA), which is 2/3 of the Maximum Considered Earthquake (MCE) PGA, determined using the ASCE-7-10 Hazard Tool Program for Site Class D type soils. Using methods published by Sitar and Mikola 2013 in their paper for *Seismic Earth Pressures on Retaining Structures in Cohesionless Soils* a seismic equivalent fluid pressure equal to the following was used.

$$EFP = 0.25\gamma(PGA)$$

Where γ is the total unit weight of the retaining wall backfill soil. Based on load factors of 1.6 used for earth pressures and 1.0 for seismic the seismic EFP was reduced by 1.6.

The following seismic design parameters in Table 6.1 below are from Chapter 16 of the 2016 California Building Code for Site Class C type soils (California Building Code, 2016). The design parameters utilize a Maximum Considered Earthquake (MCE) PGA of 0.63g.

Table 6.1. Seismic Design Parameters

Item	Factor or Coefficient	Value	CBC 2016 Table/Figure
Site Class Definition	Site Class	C	Table 1613.5.2
0.2 Second Spectral Response Acceleration	S_S	1.687g	Figure 1613.5(3)
1.0 Second Spectral Response Acceleration	S_1	0.578g	Figure 1613.5(4)
Values of Site Coefficient	F_a	1.0	Table 1613.5.3(1)
Value of Site Coefficient	F_v	1.3	Table 1613.5.3(2)
Designed Spectral Response Acceleration for Short Periods	S_{DS}	1.125g	Equation 16-38 (S_{DS}=2/3(Fa S_S)
Designed Spectral Response Acceleration for 1-Second Periods	S_{D1}	0.501g	Equation 16-40 (S_{DS}=2/3(Fv S_1)

6.3 RETAINING WALLS

To stabilize the existing slope, we recommend constructing a soldier pile and concrete lagging retaining wall with tieback anchors. Based on the results of our subsurface exploration, the depth to the top of bedrock along the retaining wall alignment extends to approximately 20 feet below the anticipated top of wall and the existing slope below the wall is generally 1H:1V in inclination. Retaining walls with heights of 20 feet and with the geometry that exists along the proposed alignment typically require deep piers with larger structural elements for a cantilever wall or require the use of tiebacks in order to scale down the size and depth of the soldier piles. The end piers for the wall may be designed as cantilever structural elements and the interior piers should be designed with tiebacks incorporated into the structure. Recommendations for this wall type are included in the following sections. The planned wall alignment and cross section are shown in Appendix D.

It should be noted that the suitability for repair using a geogrid-reinforced slope or a geogrid-reinforced wall was also considered. These approaches were eliminated as likely economically infeasible due to the difficult construction access on the steep site slopes for an earthwork solution and the long distance down the slope that would be required to start a fill slope as well as the large volume of required excavation and the space required for soil stockpiling and processing, as well as the likely need to work within the creek channel.

6.3.1 Tieback Retaining Wall

The tieback soldier pile retaining wall should be designed for unrestrained (active) conditions using the following:

- The planned wall alignment is not yet known. However, based on the topography and soil conditions, we estimate the wall height to be approximately 20 feet. For walls over about 12 feet in height, tiebacks will likely be needed.
- The first design loading condition should be for a cantilever wall extending to the excavation limits for the tiebacks. For design of the cantilever section, we recommend an active equivalent fluid pressure of 36 pcf and a passive equivalent fluid pressure of 300 pcf. The passive pressure should be taken on two pile diameters and begin 5 feet below the bench excavation for installation of the tiebacks. The active equivalent fluid pressure should be assumed to act continuously along the wall. This loading condition is temporary for construction purposes and should have a target factor of safety of 1.2.
- A passive equivalent fluid pressure of 300 psf/ft starting 3 feet below the bottom of the creek bed acting over two pier diameters;
- A seismic equivalent fluid pressure of 13 pcf acting over the full height of the retaining wall. Seismic loading should be applied in addition to the above active equivalent fluid pressure ignoring traffic live load.
- For the tieback loading condition a trapezoidal shaped load distribution based upon the apparent earth pressure diagram on page 51 of the FHWA manual (Figure 24 – included in Appendix C). The FHWA diagram results in distribution of the load to the anchors based on the anchor locations.
- Determination of the tieback force and soldier pile maximum moment should be based upon comparison of the requirements from both temporary cantilever loading and final tieback loading conditions. This requirement is necessary since the requirements will vary at each stage and the pile and tieback must be designed to handle both cases.
- Tieback rods should be a minimum of 1-inch diameter, ASTM A722, Grade 150, Class I double corrosion protected, or equivalent;
- The tieback should be locked off at 100 percent of the design load;

- For preliminary tieback design, tiebacks should be drilled at an inclination of 15 to 20 degrees below the horizontal and have an unbounded zone of 10 feet;
- Ultimate ground-grout bond strength of 110 psi is recommended for preliminary tieback design. This bond strength assumes the tieback is pressure grouted;
- The tieback should be designed with a post-grout tube in the event secondary grouting is determined to be necessary;
- Since the construction methods used to install tiebacks can dramatically influence the capacity of the anchors, the final tieback design and length of the bonded zone shall be the responsibility of the contractor to achieve the design capacity. Anchors may use secondary grouting techniques;
- Proof and performance testing should be performed to a maximum load of 1.33 times the dead load. At least one anchor shall be performance tested. Anchor acceptance should conform to the criteria included in the FHWA manual for creep and apparent free length;
- Minimum pile diameter of 30 inches;
- Minimum pile spacing of three diameters on center;
- Minimum pile depth of 10 feet below the creek bottom;
- Minimum tieback anchor diameter of 6 inches;

The Active and seismic equivalent fluid pressures assume the retaining wall will be backfilled using on-site materials excavated during soldier pile drilling operations or select import backfill with a minimum friction angle of 34 degrees and as outlined in Section 6.1.

6.3.2 Retaining Wall Drainage

The above equivalent fluid pressures for both wall options assume fully drained conditions behind the soldier pile wall. Therefore, the wall should be provided with a full height back wall drainage consisting of a 12-inch-wide layer of Caltrans Class 2 permeable material that stops 12 inches below the ground surface. Native clayey soil or aggregate base and asphalt pavement should be used for the upper foot of wall backfill and should cap the drainage material. As an alternative to the Class 2 Permeable drainage material, a clean coarse gravel or drain rock may be used. If coarse gravel or drain rock is selected as a drainage material it should be separated from all adjacent soil by an engineering filter fabric such as Mirafi 140N, or a similar geotextile. Enough space should be provided between the laggings to allow seepage through the face of the wall.

In lieu of the above mentioned drain rock, a prefabricated drainage composite such as "CCW MiraDRAIN 6000XL" or equivalent may be used for drainage behind the retaining walls. This drainage composite should be installed in accordance with manufacturers recommendations on the back of the tieback wall at least 1 foot below the ground surface and should be wrapped around a drainage pipe at the base of the wall.

6.3.3 Construction Considerations

The bottoms of soldier piles should be dry and free of loose cuttings and debris prior to installation of the steel beams and concrete. This shall be done to the satisfaction of the engineer or geologist from Cal Engineering & Geology, Inc. who observes the drilling operations. The concrete should be placed carefully in the drilled holes so that over pouring of the piles (mushrooming at the top) does not occur and the concrete does not have a free fall drop in excess of 4 feet.

Free groundwater was encountered to the depths 23 to 25 feet during the exploratory drilling. The drilling contractor should be prepared to drill and place steel and concrete for the piles on the same day. Under no circumstances shall water be allowed to remain in a drilled pile hole overnight. Should this occur, it will be necessary for the contractor to enlarge the hole to a wider diameter and/or a greater depth to the satisfaction of the engineer or geologist from our office who is observing the drilling operation.

6.4 UNDERPINNING SYSTEM OF WING WALL

To restore foundation support for the exposed wingwall, an underpinning system is recommended. Underpinning is to consist of drilled piers designed to transfer the load from the wingwall to the bedrock below. A grade beam may be needed to facilitate a structural connection between the grade beam and the piers.

The recommended design parameters for a pier and grade beam foundation system are as follows.

- Minimum pier diameter of 18 inches.
- Allowable skin friction of 500 psf in competent soil below a depth of 3 feet below the finished grade.
- Minimum pile spacing of three diameters on center.
- Minimum pier depths of 12 feet.
- We recommend as a minimum, that the piers be at least 18 inches in diameter and be reinforced with at least four #5 vertical bars with horizontal stirrup's extending

the entire depth of the pier. The project structural engineer should make the final determination of the pier dimensions and reinforcement.

- Underpinning piers should be drilled near vertically and structurally connected to the existing wingwall foundation (or to a new perimeter grade beam where necessary) so that all foundation loads are transferred to the piers.

Final design pier depths and spacing should be based on structural design considerations. The grade beams and tie beams should be designed by the project structural engineer. Grade beam and tie beam dimensions and steel reinforcing requirements should be determined based on the design structural loads.

The bottoms of the foundation pier holes should be dry and free of loose cuttings and debris prior to installation of the reinforcing steel and concrete. This shall be done to the satisfaction of the engineer or geologist from Cal Engineering & Geology, Inc. observing the drilling operations. The concrete should be placed carefully in the pier holes so that over pouring of the piers (mushrooming at the top) does not occur and the concrete does not have a free fall drop in excess of 6 feet. If groundwater is encountered during drilling, it should either be sumped from the holes prior to pouring concrete or the concrete should be placed using a tremie.

Groundwater was encountered in the borings at depths ranging from 23 to 25 feet below the top of the creek bank. The contractor must be prepared to drill and place the steel and concrete for the foundation piers on the same day, should adverse groundwater conditions be encountered during construction. Water should not be allowed to remain in a drilled pier hole overnight. Should this occur, it will be necessary for the contractor to enlarge the hole to a wider diameter and/or a greater depth to the satisfaction of the engineer or geologist from our office who is observing the drilling operation.

Our representative should observe the foundation excavations to determine if the excavations extend into suitable bearing materials and that they are cleaned of all soil and debris prior to pouring concrete

6.5 ROCK SLOPE PROTECTION (RSP)

Unprotected steep slopes are prone to future localized slumping and shallow slope failures. Remediation to reduce the potential for future slope erosion and shallow failures include flattening and/or armoring the existing slopes. We recommend that the toe of the creek bank slopes be armored with riprap or similar form of erosion protection to minimize erosion. Slope armoring, such as placing riprap slope protection at the toe of the slopes, will reduce the potential for toe erosion and progressive slope instability

6.6 SURFACE DRAINAGE

Surface drainage along the roadway is to be considered by the Design Team and incorporated into the project plans where appropriate. Collected surface water from the roadway should be conveyed by a pipe to a discharge point below active sliding or gullying, and appropriate energy dissipaters should be constructed at the outlet points to reduce the potential for future slope instability or erosion/gullying.

6.7 SOIL OR BEDROCK CORROSION POTENTIAL

The corrosion potential of the onsite soil and bedrock materials were tested as part of this investigation. Samples at 6 feet deep from boring B-2 was chosen to have resistivity, and chloride and sulfate tests at Cooper Testing Laboratory. Following the standard stated in California Test 643, the resistivity of the sample was determined, and the sample is classified as one with moderate corrosion potential. As a result, for design purposes the County should use a coating for all steel beams, and use Class 1 corrosion protection for tiebacks. If the County has previous experience on the corrosivity of the onsite soils and import material, and/or additional corrosion testing is completed, these recommendations can be modified accordingly.

7.0 LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the planned construction, and the results of the geologic mapping, subsurface exploration, and testing, combined with interpolation of the subsurface conditions between boring locations. Site conditions described in the text of this report are those existing at the time of our last field reconnaissance and are not necessarily representative of the site conditions at other times or locations. This information notwithstanding, the nature and extent of subsurface variations between borings may not become evident until construction. If variations are encountered during construction, Cal Engineering & Geology, Inc. should be notified promptly so that conditions can be reviewed and recommendations reconsidered, as appropriate.

It is the County's responsibility to ensure that recommendations contained in this report are carried out during the construction phases of the project. This report was prepared based on preliminary design information provided which is subject to change during the design process. At approximately the 90 percent design level, Cal Engineering & Geology, Inc. should review the design assumptions made in this report and prepare addenda or memoranda as appropriate. Any modifications included in these addenda or memoranda should be carefully reviewed by the project designers to make sure that any conclusions or recommendations that are modified are accounted for in the final design of the project.

The findings of this report should be considered valid for a period of three years unless the conditions of the site change. After a period of three years, CE&G should be contacted to review the site conditions and prepare a letter regarding the applicability of this report.

This report presents the results of a geotechnical and geologic investigation only and should not be construed as an environmental audit or study. The evaluation or identification of the potential presence of hazardous materials at the site was not requested and was beyond the scope of this investigation and report.

The conclusions and recommendations contained in this report are valid only for the project described in this report. We have employed accepted geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

8.0 REFERENCES

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Figures

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M:\2019\190360-SanMateoCounty-HigginsCynRdRepair\GIS\ArcGIS\190360-Fig1-Site-Location.mxd; 8/7/2019; kdrozynska

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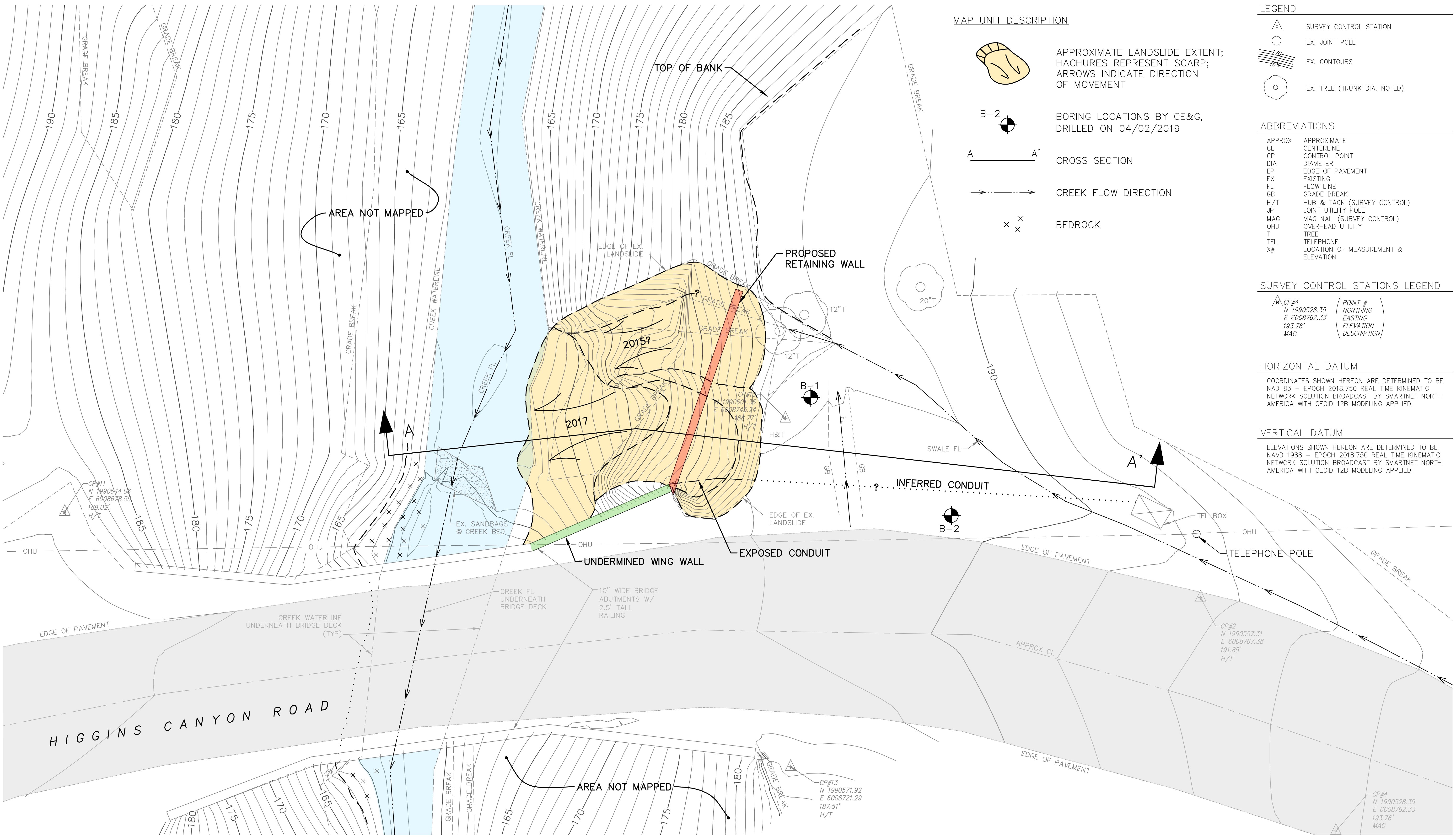
HIGGINS CANYON RD LANDSLIDE REPAIR PROJECT
HIGGINS CANYON ROAD
SAN MATEO COUNTY, CALIFORNIA

SITE LOCATION MAP

190360	AUGUST 2019	FIGURE 1



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1. TOPOGRAPHIC BASEMAP FROM RUGGERI–JENSEN–AZAR (RJA), TITLED "TOPOGRAPHIC MAP HIGGINS CANYON ROAD BRIDGE, SAN MATEO COUNTY, CALIFORNIA", DATED APRIL 22, 2019.

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SAN MATEO COUNTY, CALIFORNIA

SITE PLAN

190360	AUGUST 2019	FIGURE 2

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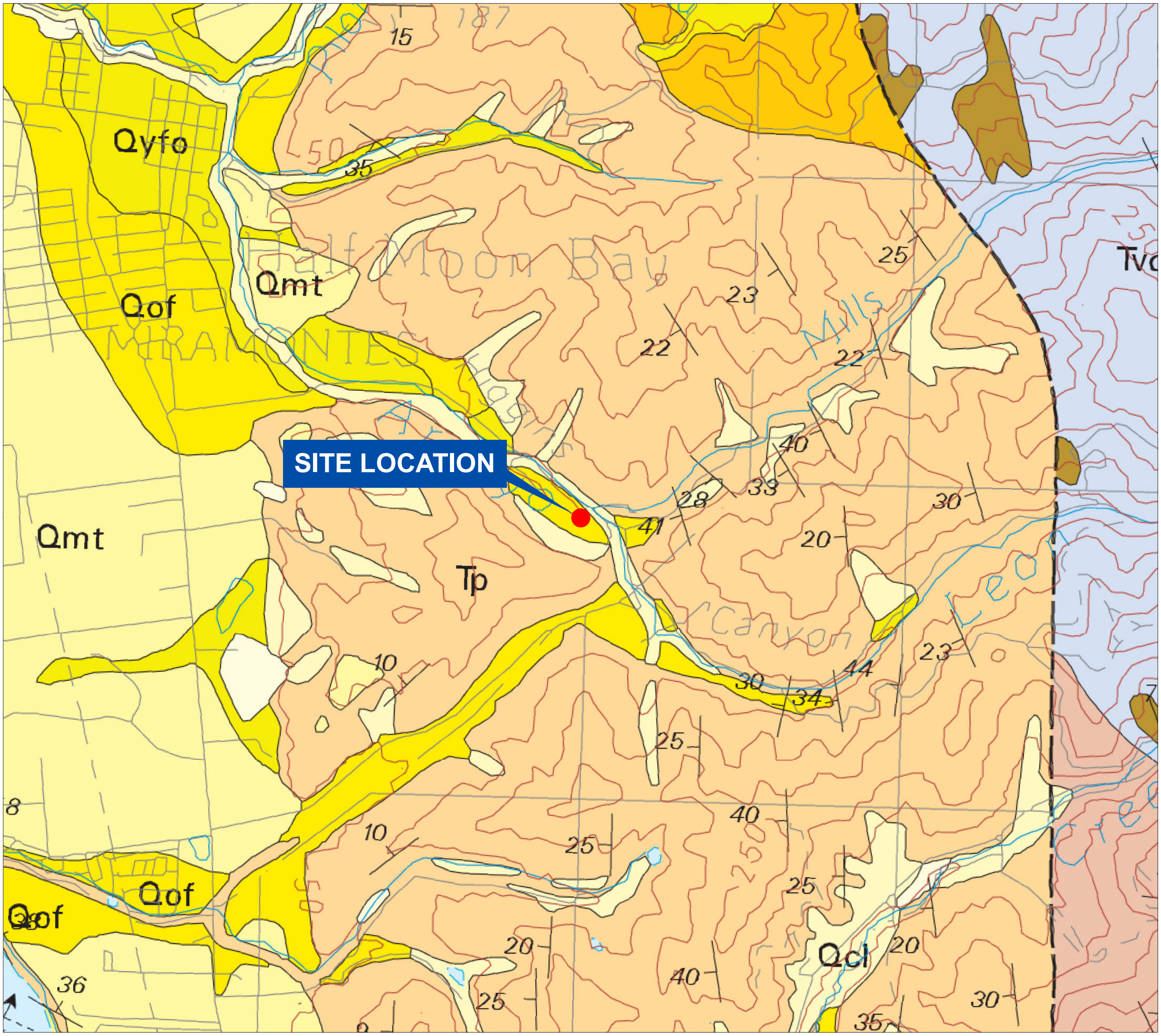

BASEMAP REFERENCE

1. REGIONAL GEOLOGY MAP FROM BRABB ET AL. (1998)

0 0.25 0.5 1
MILES

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MAP UNIT DESCRIPTION

Qyf	YOUNGER (INNER) ALLUVIAL FAN DEPOSITS (HOLOCENE)	Qmt	MARINE TERRACE DEPOSITS (PLEISTOCENE)
Qyfo	YOUNGER (OUTER) ALLUVIAL FAN DEPOSITS (HOLOCENE)	Tp	PURISIMA FORMATION (PLIOCENE AND UPPER MIOCENE)
Qcl	COLLUVIUM (HOLOCENE)	Tm	MONTEREY FORMATION (MIDDLE MIOCENE)
Qs	SAND DUNE AND BEACH DEPOSITS (HOLOCENE)	Tla	LAMBERT SHALE (OLIGOCENE AND LOWER MIOCENE)
Qal	ALLUVIUM (HOLOCENE)	Tmb	MINDEGO BASALT AND RELATED VOLCANIC ROCKS (MIOCENE AND/OR OLIGOCENE)
Qof	COARSE-GRAINED OLDER ALLUVIAL FAN AND STREAM TERRACE DEPOSITS (PLEISTOCENE)	Tvq	VAQUEROS SANDSTONE (LOWER MIOCENE AND OLIGOCENE)

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REGIONAL GEOLOGY

190360	AUGUST 2019	FIGURE 3

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0 1.5 3 6
MILES

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MAP UNIT DESCRIPTION

- Historical (<150 years), Well Constrained Location
- Historical (<150 years), Moderately Constrained Location
- Historical (<150 years), Inferred Location
- Latest Quaternary (<15,000 years), Well Constrained Location
- Latest Quaternary (<15,000 years), Moderately Constrained Location
- Latest Quaternary (<15,000 years), Inferred Location
- Late Quaternary (<130,000 years), Well Constrained Location
- Late Quaternary (<130,000 years), Moderately Constrained Location
- Late Quaternary (<130,000 years), Inferred Location
- Undifferentiated Quaternary (<1.6 million years), Well Constrained Location
- Undifferentiated Quaternary (<1.6 million years), Moderately Constrained Location
- Undifferentiated Quaternary (<1.6 million years), Inferred Location

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FAULT ACTIVITY MAP

190360	AUGUST 2019	FIGURE 4

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BASEMAP REFERENCE

1. BASEMAP FROM DIGITALGLOBE (2017).
2. LANDSLIDE DATA FROM CALIFORNIA GEOLOGICAL SURVEY LANDSLIDE INVENTORIES ACCESSED ONLINE ON 13 MARCH 2019.

0 0.25 0.5 1
MILES

N

MAP UNIT DESCRIPTION

ACTIVITY:
- HISTORIC
- DORMANT YOUNG
- DORMANT MATURE
- DORMANT OLD/RELICT
- DORMANT AGE NOT SPECIFIED

CONFIDENCE:
- DEFINITE
- PROBABLE
- QUESTIONABLE

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LANDSLIDE ACTIVITY MAP

190360	AUGUST 2019	FIGURE 5

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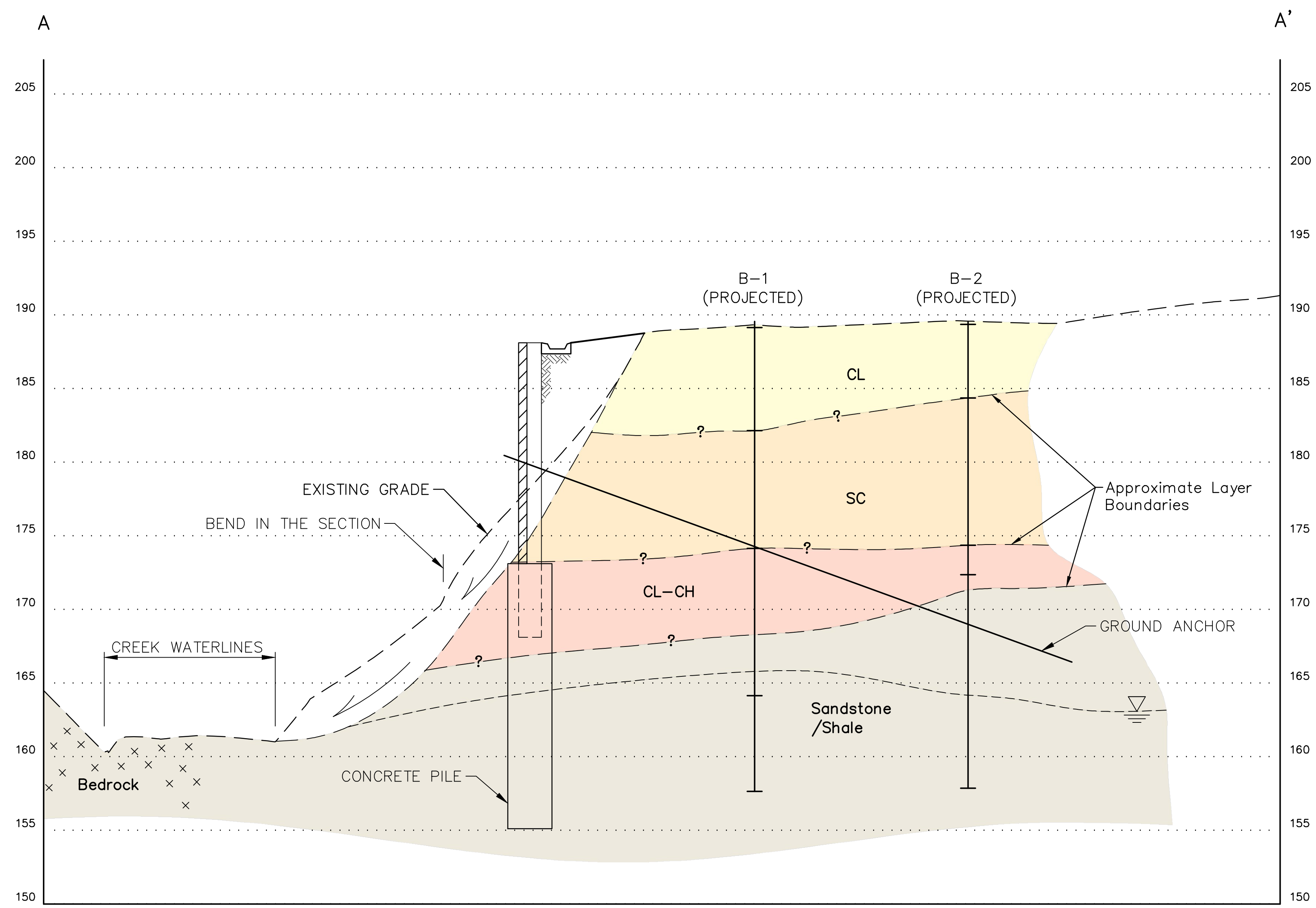


SECTION A–A'

CE&G
CAL ENGINEERING & GEOLOGY
6455 Almaden Expwy.
Suite 100
San Jose, CA 95120
Phone: (408) 440-4542

HIGGINS CANYON RD LANDSLIDE REPAIR PROJECT
HIGGINS CANYON ROAD
SAN MATEO COUNTY, CALIFORNIA

CONCEPTUAL REPAIR CROSS SECTION

190360	AUGUST 2019	FIGURE 6

M:\2019\190360-SanMateoCounty-HigginsCynRdRepair\AutoCAD-Figures\Figures\FIG2-SITE-PLAN.dwg 8-07-19 12:40:06 PM kdrozynska

Appendix A. Boring Logs

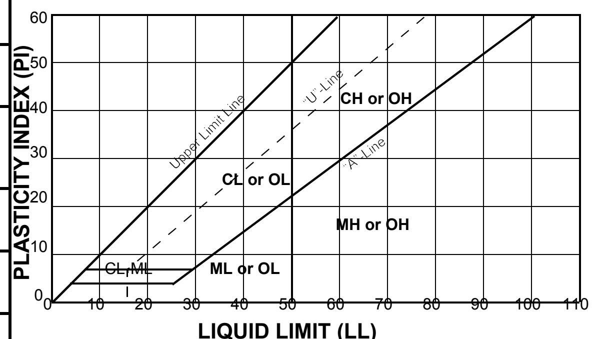
UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Field Identification			Group Symbols	Typical Names	Laboratory Classification Criteria			
Coarse-Grained Soils More than 50% of material is retained on the No. 200 sieve.	**Gravels** More than 50% coarse fraction retained on the No. 4 sieve	**Clean Gravels** < 5% Fines	**GW**	Well-graded gravels, gravel-sand mixtures, little or no fines	**CLASSIFICATION OF GRAVELS & SANDS WITH 5% TO 12% FINES REQUIRES DUAL SYMBOLS**	Gravel/Silty Gravel Gravel/Clayey Gravel Sand/Silty Sand Sand/Clayey Sand GW/GM or GP/GM: GW/GC or GP/GC: SW/SM or SP/SM: SW/SC or SP/SC:	$C_U = D_{60} \div D_{10} \geq 4$ and $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1$ & ≤ 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines			$C_U = D_{60} \div D_{10} < 4$ and/or $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) < 1$ & > 3	
		Gravels with Fines >12% Fines	**GM**	Silty gravels, poorly graded gravel-sand-silt mixtures			Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol GC/GM
			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures			Fines classify as CL or CH	
	Sands More than 50% coarse fraction passes the No. 4 sieve	**Clean Sands** < 5% Fines	**SW**	Well-graded sands, gravelly sands, little or no fines			$C_U = D_{60} \div D_{10} \geq 6$ and $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1$ & ≤ 3	
			SP	Poorly graded sands, gravelly sands, little or no fines			$C_U = D_{60} \div D_{10} < 6$ and/or $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) < 1$ & > 3	
		Sands with Fines >12% Fines	**SM**	Silty sands, poorly graded sand-silt mixtures			Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol SC/SM
			SC	Clayey sands, poorly graded sand-clay mixtures			Fines classify as CL or CH	
Fine-Grained Soils More than 50% of material passes the No. 200 sieve.	**Identification Procedures on Percentage Passing the No. 40 Sieve**							
	Silts & Clays Liquid Limit less than 50%		**ML**	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands with slight plasticity				
			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and/or silty clays, lean clays				
			OL	Organic silts, organic silty clays of low plasticity				
	Silts & Clays Liquid Limit greater than 50%		**MH**	Inorganic silts, micaceous or diatomaceous fine sandy/-silty soil, elastic silts				
			CH	Inorganic clays of high plasticity, fat clays				
			OH	Organic clays of medium to high plasticity				
HIGHLY ORGANIC SOILS			**PT**	Peat and other highly organic soils				

PLASTICITY CHART

For Classification of Fine-Grained Soils and Fine-Grained Fraction of Coarse-Grained Soils

Equation of "A"-Line: PI = 4 @ LL = 4 to 25.5, then PI = 0.73 × (LL − 20)
Equation of "U"-Line: LL = 16 @ PI = 0 to 7, then PI = 0.9 × (LL − 8)



KEY TO SAMPLER TYPES AND OTHER LOG SYMBOLS

CS	California Standard Sampler	▽	Depth at which Groundwater was Encountered During Drilling
CM	California Modified Sampler	▼	Depth at which Groundwater was Measured After Drilling
SPT	Standard Penetration Test Sampler	**PP**	Pocket Penetrometer Test
SHL	Shelby Tube Sampler	**PTV**	Pocket Torvane Test
BU	Bulk Sample	**-#200**	% of Material Passing the No. 200 Sieve Test (ASTM D-1140)
LL	Liquid Limit of Sample (ASTM D-4318)	**PSA**	Particle-Size Analysis (ASTM D-422 & D-1140)
PI	Plasticity Index of Sample (ASTM D-4318)	**C**	Consolidation Test (ASTM D-2435)
Q_U	Unconfined Compression Test (ASTM D-2166)	**TXUU**	Unconsolidated Undrained Compression Test (ASTM D-2850)

KEY TO SAMPLE INTERVALS

CS	Length of Sampler Interval with a CS Sampler	BU	Bulk Sample Recovered for Interval Shown (i.e., cuttings)
	Length of Sampler Interval with a CM Sampler		Length of Coring Run with Core Barrel Type Sampler
	Length of Sampler Interval with a SPT Sampler	NR	No Sample Recovered for Interval Shown
SHL	Length of Sampler Interval with a SHL Sampler		

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UNIFIED SOIL CLASSIFICATION SYSTEM AND KEY TO BORING LOG

Rock Hardness Descriptions

Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimen requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves to 1/4-inch deep can be excavated by hard blow of geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16-inch deep by firm pressure of knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small tin pieces can be broken by finger pressure.
Very Soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Bedding Thickness & Joint/Fracture Spacing Descriptions

Centimeters	Inches	Bedding	Joints/Fractures
< 2	< ¾	Laminated	Extremely Close
2-5	¾-2	Very Thin	Very Close
5-30	2-12	Thin	Close
30-90	12-36	Medium	Moderate
90-300	36-120	Thick	Wide
> 300	> 120	Very Thick	Very Wide

Rock Weathering Descriptions

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very Slight	Rock generally fresh, joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dulled and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately Severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very Severe	All rock except quartz discolored or stained. Rock "fabric" discernible. But mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

The above Bedrock Characteristics are based on the ASCE Manual No. 56, "Subsrface Investigation For Design And Construction Of Foundations Of Buildings," 1976.

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KEY TO SYMBOLS

CLIENT San Mateo County
PROJECT NAME Higgins Canyon Road Repair
PROJECT NUMBER 190360
PROJECT LOCATION San Mateo County, CA

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

ASPHALT: Asphalt

BEDROCK: Bedrock

FILL: Fill (made ground)

GW: USCS Well-graded Gravel

SM: USCS Silty Sand

SAMPLER SYMBOLS

California Modified Sampler

Standard Penetration Test

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
Water Level at Time Drilling, or as Shown
Water Level at End of Drilling, or as Shown
Water Level After 24 Hours, or as Shown

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BORING NUMBER B-1

CLIENT San Mateo County Department of Public Works
PROJECT NAME Higgins Canyon Road Repair
PROJECT NUMBER 190360
PROJECT LOCATION Half Moon Bay, CA
DATE STARTED 4/2/2019 **COMPLETED** 4/2/2019
GROUND ELEVATION 188.5 ft **DATUM** NAVD88 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR Britton Exploration
COORDINATES: LATITUDE 37.44629 **LONGITUDE** -122.40483
DRILLING RIG/METHOD CME-55/8-in. Hollowstem Auger
∇ **GROUNDWATER AT TIME OF DRILLING** 23.0 ft / Elev 165.5 ft
LOGGED BY A.Johns **CHECKED BY** K.Feng
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		Sandy Lean CLAY (CL): dark olive brown, moist, soft to medium stiff [Alluvium] Hand augered to 5 ft.									
5		Pocket Torvane: 3 tsf	CM	2-3-3	0.5 0.5	96	25	31	21	10	63
		Clayey SAND (SC): Dark olive brown, moist, loose to medium dense Pocket Torvane: 1.5 to 2 tsf	CM	2-3-3		103	22				44
10		Clayey SAND with Gravel (SC): reddish brown, moist, loose to medium dense, fine sand, rounded gravel Pocket Torvane: 2 tsf	CM	3-5-5		101	20				
15		Sandy Lean CLAY (CL): dark reddish brown, stiff, low to high plasticity Pocket Torvane: 2 tsf	CM	3-4-7	0.5 1	108	21				58
20		olive brown, hard	CM	35-50/4"	1.5 1.5						63
	∇										
25		SANDSTONE: grayish green, highly weathered, closely fractured, friable to moderately strong [Purisima Formation] Drill rig chattering in harder drilling material at 28 ft.	CM	30-50/5"							
30		SHALE: gray, slightly weathered, weak	SPT	40-50/5"							

Bottom of borehole at 31.5 ft. Borehole backfilled with cement grout.

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BORING NUMBER B-2

CLIENT San Mateo County Department of Public Works
PROJECT NAME Higgins Canyon Road Repair
PROJECT NUMBER 190360
PROJECT LOCATION Half Moon Bay, CA
DATE STARTED 4/2/2019 **COMPLETED** 4/2/2019
GROUND ELEVATION 189.5 ft **DATUM** NAVD88 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR Britton Exploration
COORDINATES: LATITUDE 37.44627 **LONGITUDE** -122.40479
DRILLING RIG/METHOD CME-55/8-in. Hollowstem Auger
∇ **GROUNDWATER AT TIME OF DRILLING** 25.0 ft / Elev 164.5 ft
LOGGED BY A.Johns **CHECKED BY** K.Feng
GROUNDWATER AT END OF DRILLING --- N/A
HAMMER TYPE 140 lb hammer with 30 in. autotrip
GROUNDWATER AFTER DRILLING --- N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0		Sandy Lean CLAY (CL): dark yellowish brown, moist, soft to medium stiff [Alluvium] Hand augered to 3 ft.									
		Medium stiff	CM	4-5-5	1 1	106	20	26	18	8	59
5		Clayey SAND with Gravel (SC): reddish brown, moist, dense	CM	5-11-29							
			CM	11-9-16		109	17				33
10			SPT	6-10-13			17				36
15		CLAY with sand (CL): Olive brown, moist, low to high plasticity	SPT	6-17-15			24				83
		SHALE: highly to completely weathered, plastic/friable [Purisima Formation]									
		SANDSTONE interbedded with SHALE: highly to completely weathered, friable to plastic harder drilling/ more competent rock at 17 ft.									
20			SPT	9-11-17							
25		∇	SPT	18-30-50/5"							
		SPT refusal at 27 ft.									
30		some meta shale	SPT	20-40-41							

Bottom of borehole at 31.5 ft. Borehole backfilled with cement grout.

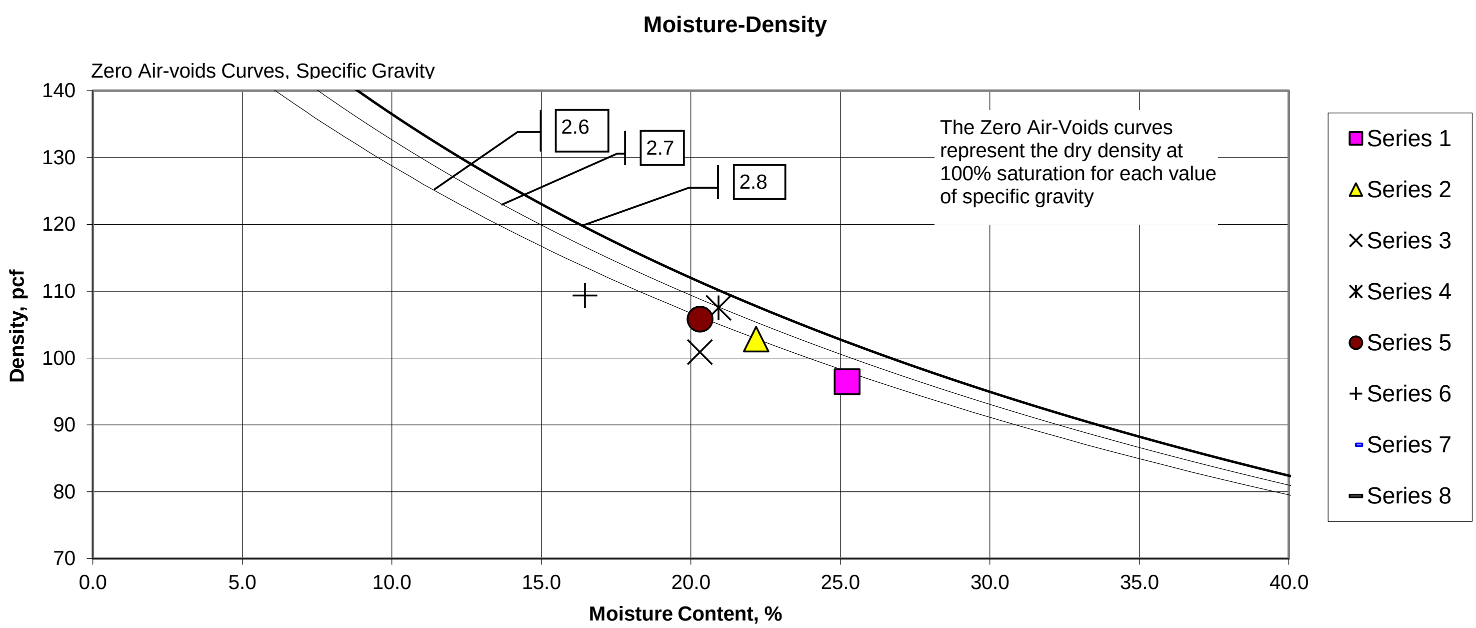
Appendix B. Laboratory Testing

Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

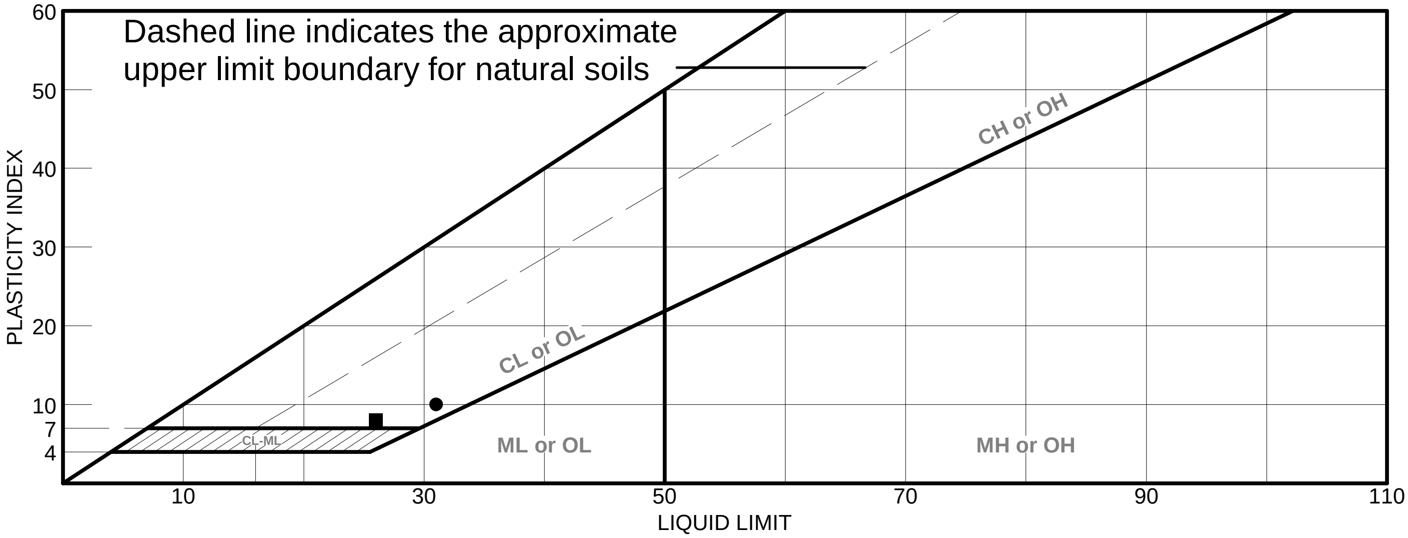
CTL Job No:	471-258	**Project No.**	190360	**By:**	RU
Client:	Cal Engineering & Geology	**Date:**	04/30/19		
Project Name:	Higgins Canyon Rd	**Remarks:**			

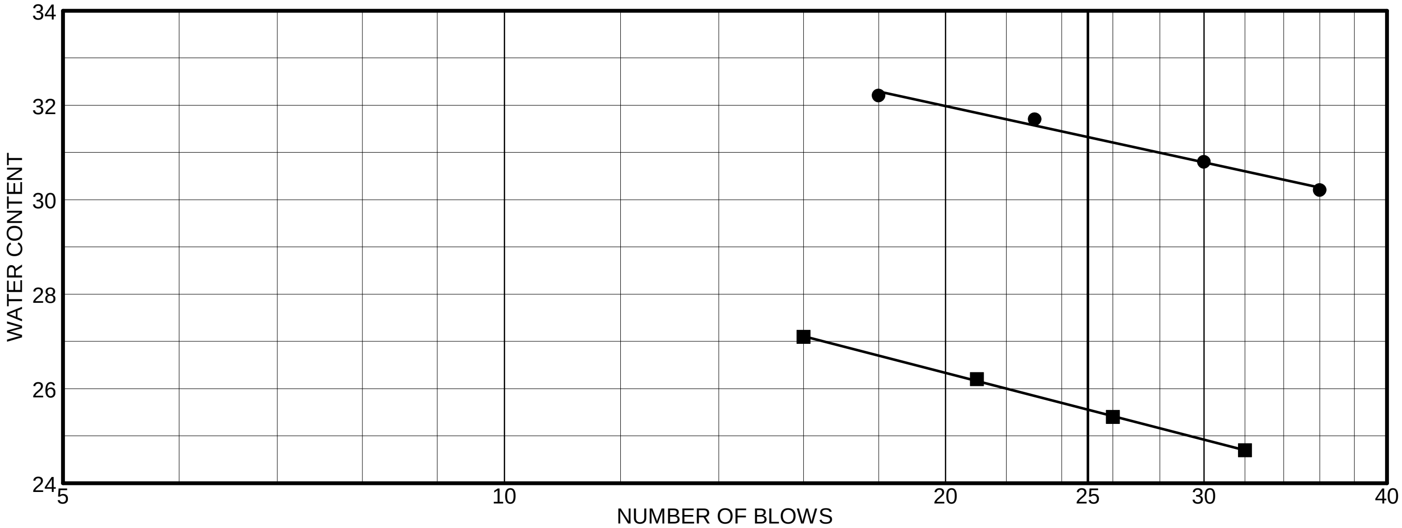
Boring:	B-1	B-1	B-1	B-1	B-2	B-2	B-2	B-2
Sample:	1c	2c	3c	4b	1c	3c	4a	5a
Depth, ft:	6	8	11	15.5	4	8	10	15
Visual Description:	Dark Olive Brown Sandy Lean CLAY	Dark Olive Brown Sandy CLAY	Dark Reddish Brown Clayey SAND	Dark Reddish Brown Sandy CLAY	Dark Yellowish Brown Sandy Lean CLAY	Reddish Brown Clayey SAND w/ Gravel	Reddish Brown Clayey SAND w/ Gravel	Olive Brown CLAY w/ Sand
Actual G_s								
Assumed G_s	2.70	2.70	2.70	2.70	2.70	2.70		
Moisture, %	25.2	22.2	20.3	20.9	20.3	16.5	16.7	23.6
Wet Unit wt, pcf	120.8	125.6	121.4	130.0	127.3	127.3		
Dry Unit wt, pcf	96.5	102.8	100.9	107.5	105.8	109.3		
Dry Bulk Dens.pb, (g/cc)	1.55	1.65	1.62	1.72	1.69	1.75		
Saturation, %	91.0	93.5	81.7	99.4	92.2	81.9		
Total Porosity, %	42.8	39.0	40.2	36.3	37.3	35.2		
Volumetric Water Cont, Өw,%	39.0	36.5	32.8	36.0	34.4	28.8		
Volumetric Air Cont., Өa,%	3.9	2.5	7.4	0.2	2.9	6.4		
Void Ratio	0.75	0.64	0.67	0.57	0.59	0.54		
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.



LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Olive Brown Sandy Lean CLAY	31	21	10			
■	Dark Yellowish Brown Sandy Lean CLAY	26	18	8			

Project No. 471-258 **Client:** Cal Engineering & Geology

Project: Higgins Canyon Rd - 190360

● **Source:** B-1 **Sample No.:** 1c **Elev./Depth:** 6'

■ **Source:** B-2 **Sample No.:** 1c **Elev./Depth:** 4'

Remarks:

●

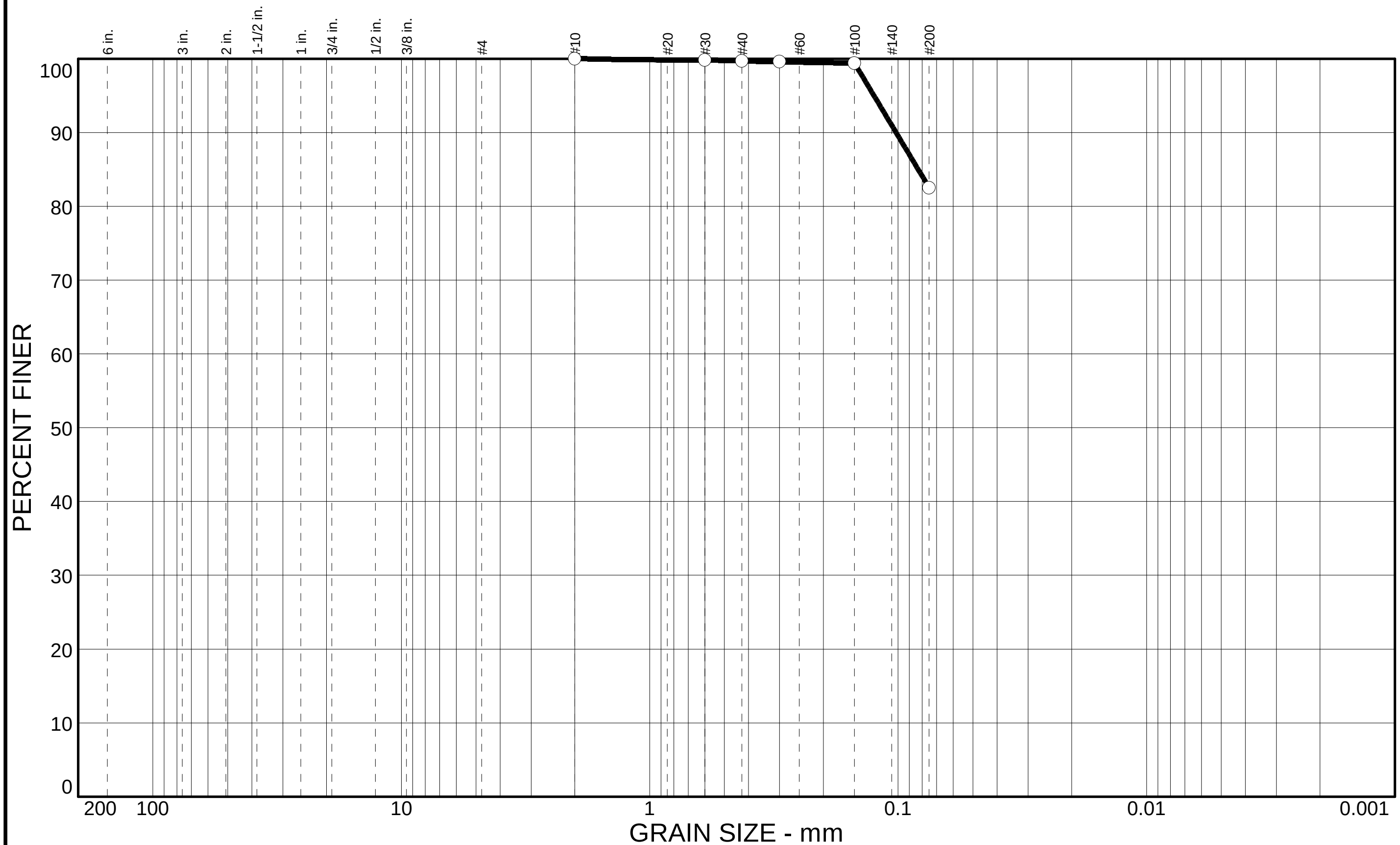
■

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○			17.5	82.5					

SIEVE inches size	PERCENT FINER ○		
	GRAIN SIZE		
D_{60}			
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○		
#10	100.0		
#30	99.8		
#40	99.7		
#50	99.6		
#100	99.4		
#200	82.5		

SOIL DESCRIPTION

○ Olive Brown CLAY w/ Sand

REMARKS:

○

○ Source: B-2 Sample No.: 5a Elev./Depth: 15'

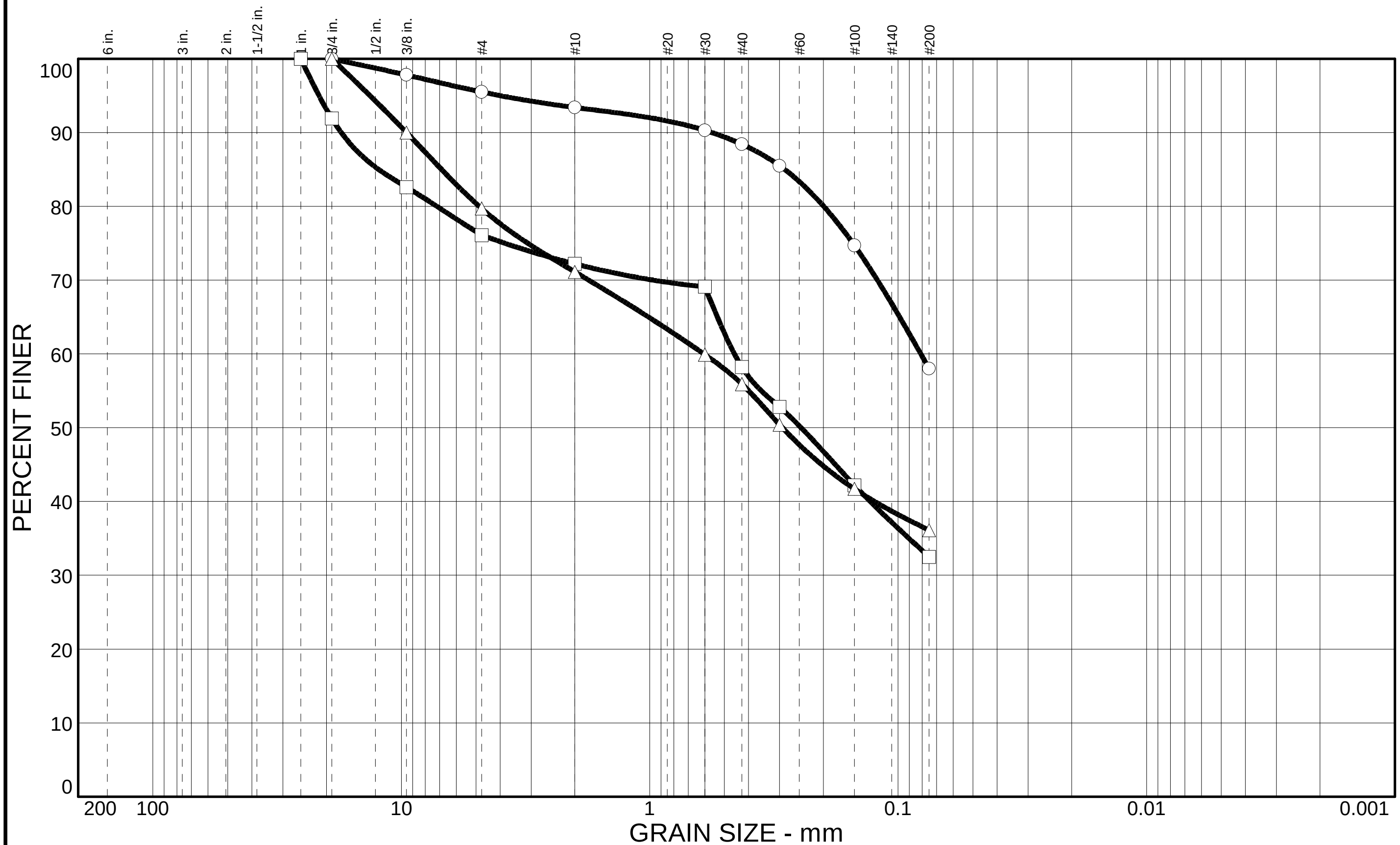
COOPER TESTING LABORATORY

Client: Cal Engineering & Geology

Project: Higgins Canyon Rd - 190360

Project No.: 471-258 Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		4.5	37.5	58.0					
□		23.9	43.6	32.5					
△		20.3	43.6	36.1					

SIEVE inches size	PERCENT FINER ○	□	△
1"		100.0	
3/4"	100.0	91.9	100.0
3/8"	97.8	82.6	90.0
	GRAIN SIZE		
D_{60}	0.0811	0.456	0.606
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	95.5	76.1	79.7
#10	93.4	72.2	71.1
#30	90.3	69.1	59.9
#40	88.4	58.2	55.9
#50	85.5	52.8	50.4
#100	74.7	42.2	41.7
#200	58.0	32.5	36.1

SOIL DESCRIPTION
○ Dark Reddish Brown Sandy CLAY
□ Reddish Brown Clayey SAND w/ Gravel
△ Reddish Brown Clayey SAND w/ Gravel

REMARKS:
○
□ Due to the small sample size, relative to the largest particle size, this data should be considered to be approximate.
△

○ Source: B-1 Sample No.: 4b Elev./Depth: 15.5'
□ Source: B-2 Sample No.: 3c Elev./Depth: 8'
△ Source: B-2 Sample No.: 4a Elev./Depth: 10'

COOPER TESTING LABORATORY

Client: Cal Engineering & Geology
Project: Higgins Canyon Rd - 190360
Project No.: 471-258 Figure

COOPER TESTING LABORATORY

#200 Sieve Wash Analysis
ASTM D 1140

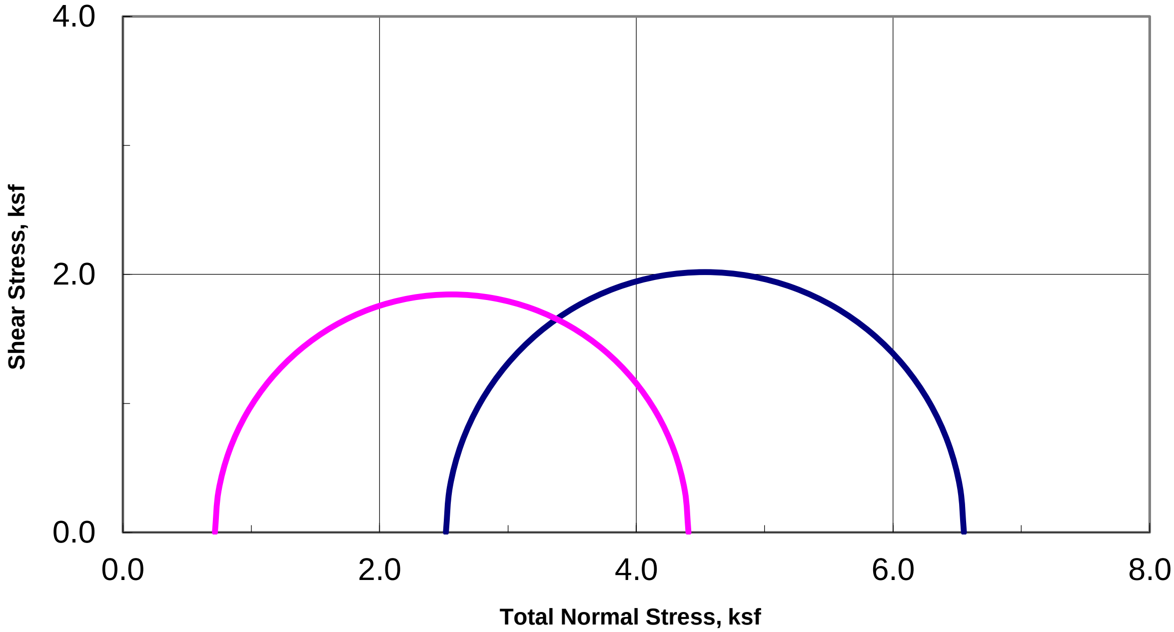
Job No.:	471-258	**Project No.:**	190360	**Run By:**	MD
Client:	Cal Engineering & Geology	**Date:**	5/10/2019	**Checked By:**	DC
Project:	Higgins Canyon Rd				

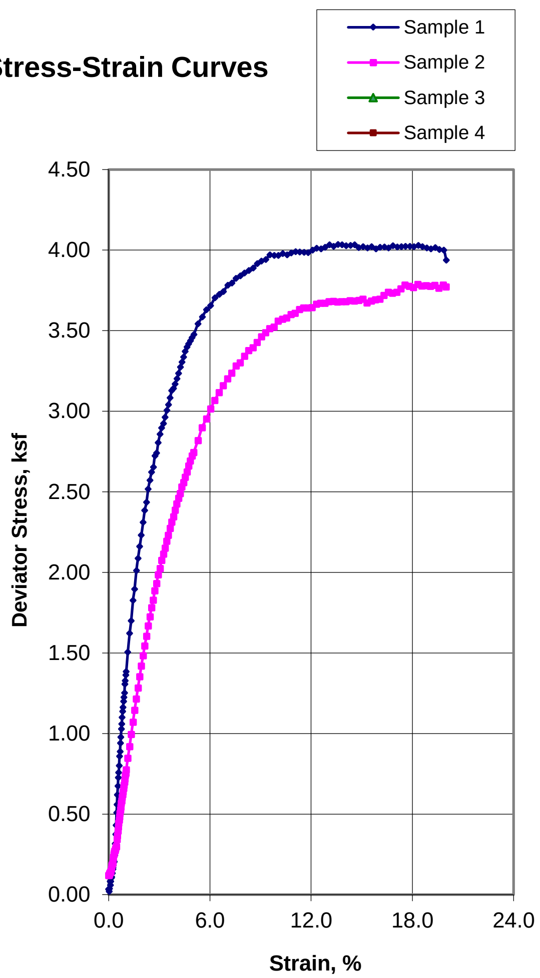
Boring:	B-1	B-1	B-1	B-2				
Sample:	1c	2c	5c	1c				
Depth, ft.:	6	8	21	4				
Soil Type:	Dark Olive Brown Sandy Lean CLAY	Dark Olive Brown Clayey SAND	Olive Brown Sandy CLAY	Dark Yellowish Brown Sandy Lean CLAY				
Wt of Dish & Dry Soil, gm	495.9	500.2	531.8	548.2				
Weight of Dish, gm	176.4	176.6	173.8	174.4				
Weight of Dry Soil, gm	319.6	323.6	358.0	373.9				
Wt. Ret. on #4 Sieve, gm	4.6	8.3	0.5	6.0				
Wt. Ret. on #200 Sieve, gm	118.5	180.1	131.5	154.1				
% Gravel	**1.4**	**2.6**	**0.1**	**1.6**				
% Sand	**35.7**	**53.1**	**36.6**	**39.6**				
% Silt & Clay	**62.9**	**44.3**	**63.3**	**58.8**				

Remarks: As an added benefit to our clients, the gravel fraction may be included in this report. Whether or not it is included is dependent upon both the technician's time available and if there is a significant enough amount of gravel. The gravel is always included in the percent retained on the #200 sieve but may not be weighed separately to determine the percentage, especially if there is only a trace amount, (5% or less).

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data	1	2	3	4
Moisture %	25.6	19.4		
Dry Den,pcf	97.5	110.6		
Void Ratio	0.728	0.524		
Saturation %	94.9	99.8		
Height in	5.00	5.04		
Diameter in	2.41	2.42		
Cell psi	17.5	5.0		
Strain %	13.59	15.00		
Deviator, ksf	4.036	3.689		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	471-258			
Client:	Cal Engineering & Geology			
Project:	190360			
Boring:	B-1	B-2		
Sample:	5C	2C		
Depth ft:	21	6		

Visual Soil Description

Sample #	
1	Olive Brown Sandy CLAY
2	Yellowish Brown Sandy CLAY w/ Gravel
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

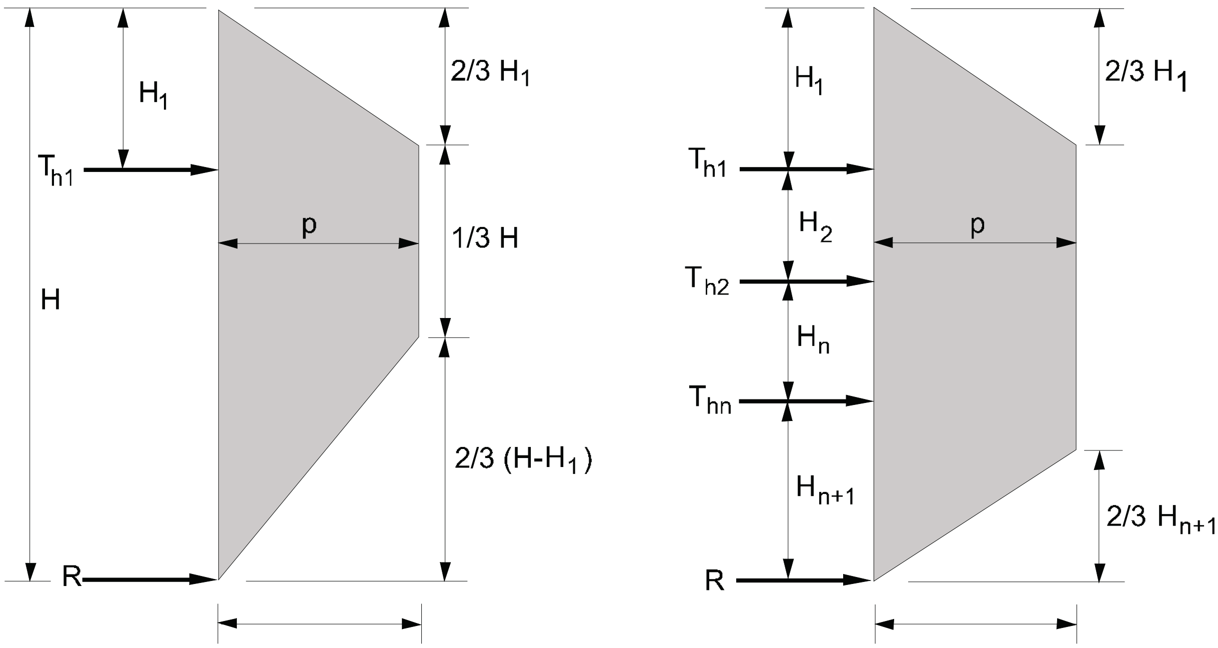
COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL #	471-258	Date:	5/9/2019	Tested By:	PJ	Checked:	PJ
Client:	Cal Engineering & Geology	Project:	Higgins Canyon RD			Proj. No:	190360
Remarks:							

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-2	2c	6	-	2593	-	29	131	0.0131	7.5	-	3.0	Yellowish Brown Sandy CLAY w/ Gravel

Appendix C. FHWA Loading Diagram



$p \approx 0.2\gamma H - 0.4\gamma H$ $\qquad$ $p \approx 0.2\gamma H - 0.4\gamma H$

(a) Walls with one level of ground anchors

(b) Walls with multiple levels of ground anchors

H_1 = Distance from ground surface to uppermost ground anchor

H_{n+1} = Distance from base of excavation to lowermost ground anchor

T_{hi} = Horizontal load in ground anchor i

R = Reaction force to be resisted by subgrade (i.e., below base of excavation)

p = Maximum ordinate of diagram

TOTAL LOAD (kN/m/meter of wall) = $3H^2 - 6H^2$ (H in meters)

Figure 27. Recommended apparent earth pressure envelope for stiff to hard clays.

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HIGGINS CANYON RD LANDSLIDE REPAIR PROJECT
HIGGINS CANYON ROAD
SAN MATEO COUNTY, CALIFORNIA

FHWA LOADING DIAGRAM

JOB NO. 190360	AUGUST 2019	FIGURE 4

Appendix D. Concept Design Plans

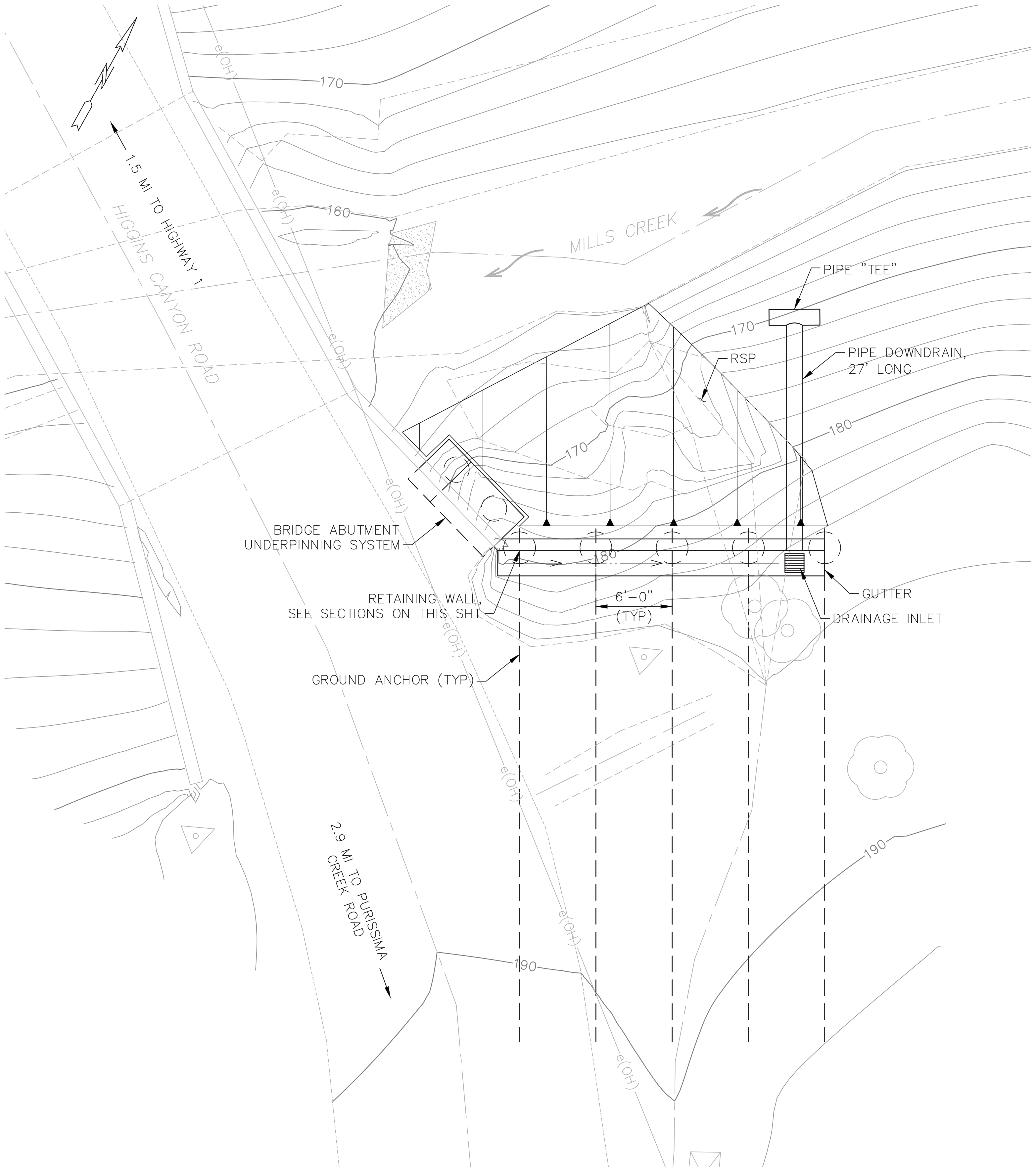
M:\2019\190360-SanMateoCounty-HigginsCynRdRepair\AutoCAD\Sheets\190360-03-PLAN-PROFILE-SECTION.dwg 7-05-19 12:45:10 PM Evaccaro

APPROVED:

DATE:

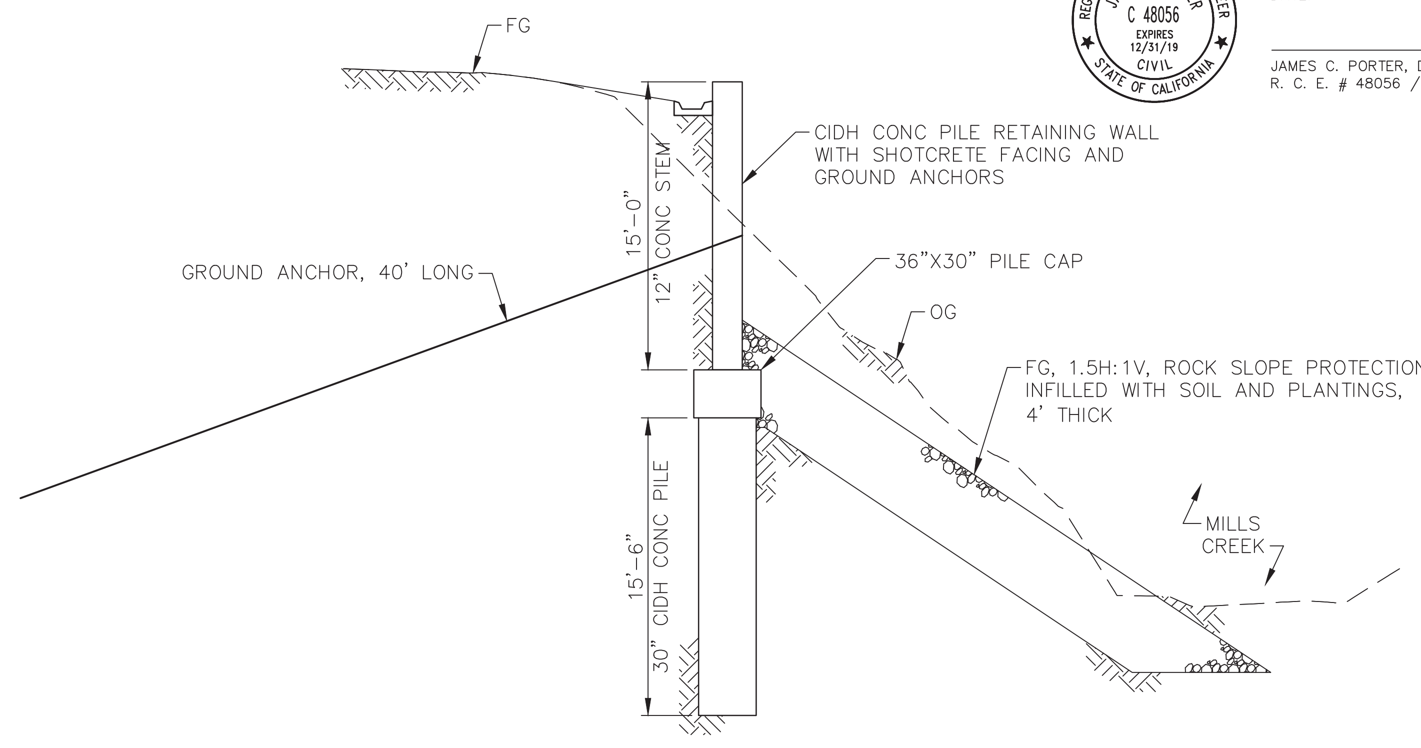
JAMES C. PORTER, DIRECTOR OF PUBLIC WORKS
R. C. E. # 48056 / EXPIRES 12-31-2019

REGISTERED PROFESSIONAL ENGINEER JAMES C. PORTER C 48056 EXPIRES 12/31/19 CIVIL STATE OF CALIFORNIA

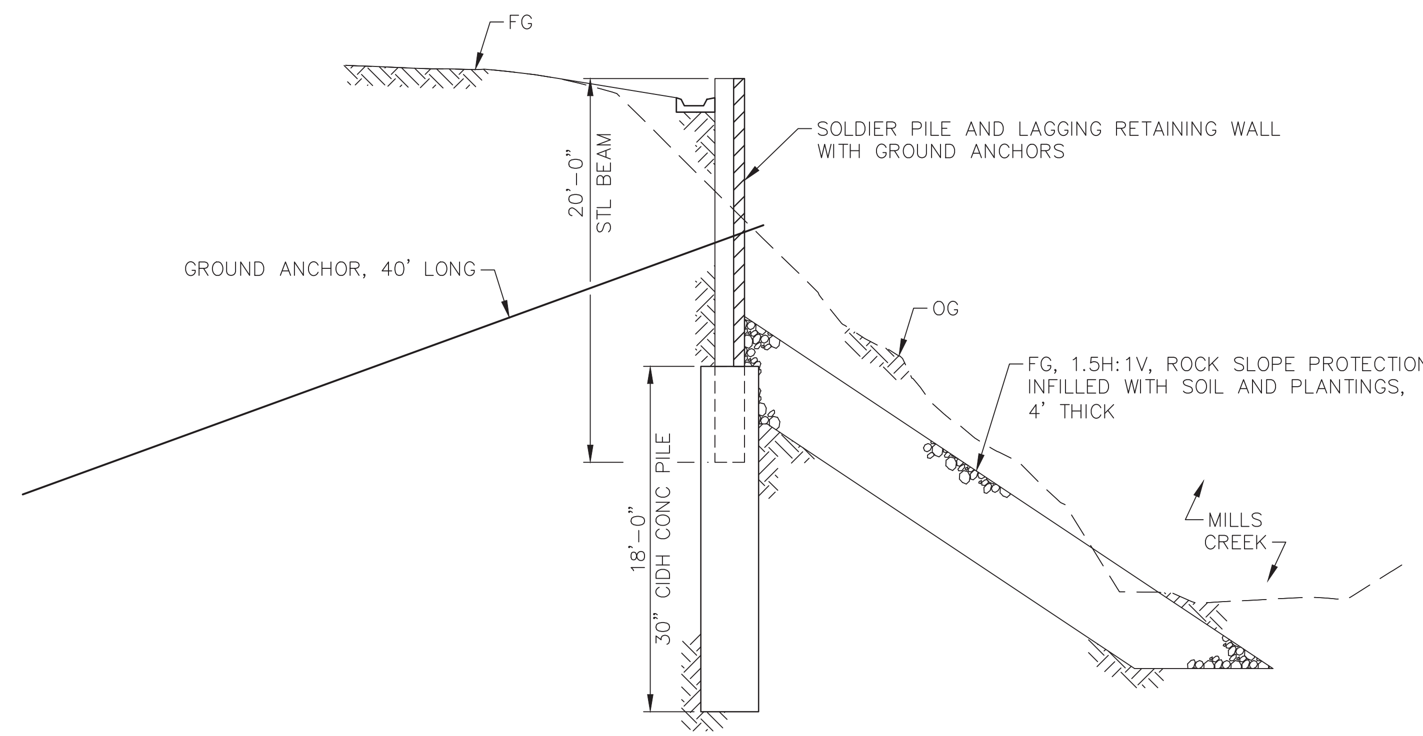


PLAN

SCALE 5 0 5 10 FEET



CIDH CONC PILE RETAINING WALL



SOLDIER PILE AND LAGGING RETAINING WALL

15% SUBMITTAL FOR REVIEW
NOT FOR CONSTRUCTION
5 JULY 2019

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APPROVED DATE: DD-MM-YYYY	
CHRIS HOCKETT, P.E., G.E., PRINCIPAL ENGINEER	
CAL ENGINEERING & GEOLOGY, INC.	
R.C.E. # 71938, R.G.E. # 2928 / EXP 12-31-19	

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COUNTY OF SAN MATEO PUBLIC WORKS

REVISION	DATE			
		DESIGNED BY: KF	HIGGINS CANYON ROAD BRIDGE AT MILLS CREEK EMBANKMENT AND WINGWALL REPAIRS	SCALE: 1" = 5'
		DRAWN BY: EV		DATE: JULY 2019
		CHECKED BY: CH	PLAN AND TYPICAL SECTIONS	FILE NO.: 190360
		JAMES C. PORTER, DIRECTOR OF PUBLIC WORKS SAN MATEO COUNTY	555 COUNTY CENTER, 5th FLOOR REDWOOD CITY, CALIFORNIA 94063	

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES 0 1 2 3 4