APPENDIX B

Foundation Report

DRAFT FOUNDATION REPORT
REGNART CREEK TRAIL BRIDGES
CITY OF CUPERTINO, CALIFORNIA

For

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May 20, 2019 Job No. 2018-151-GEO

TABLE OF CONTENTS **PAGE**

PLATES

APPENDICES

APPENDIX A: Log of Test Borings

APPENDIX B: Laboratory Test Results

Laboratory Tests

APPENDIX C: Analyses and Calculations

Axial Pile Capacity Analyses

Lateral Soil Pressures

Slope Stability Analysis

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1.0 INTRODUCTION

This "Draft Foundation Report" presents the results of our geotechnical engineering investigation for the proposed "Regnart Creek Trail Bridges" Project for the City of Cupertino, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to HMH (Designer).

2.0 SCOPE OF WORK

The purpose of this report is to evaluate the general subsurface soil conditions and engineering properties at the project site and to provide foundation design for the proposed project. The approximate location of the project site is shown on the Project Location Map (Plate No. 1).

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the project site; site reconnaissance; obtaining representative soil samples and logging soil materials encountered in the exploratory soil borings; laboratory testing of the representative soil samples, performing engineering analyses based on the field and laboratory data, and preparation of this foundation report.

3.0 PROJECT DESCRIPTION

Envisioned as part of The Loop Cupertino and identified in the City of Cupertino 2016 Bicycle Transportation Plan and the City of Cupertino 2018 Pedestrian Plan, the Regnart Creek Trail is a planned facility which would provide a safe and convenient off-street route for bicyclists and pedestrians to access nearby destinations including Cupertino Civic Center, Cupertino Public Library, Wilson Park, Creekside Park, schools, and residential neighborhoods. Under agreement with the Satna Clara Valley Water District (SCVWD), the project would utilize an existing maintenance road along the bank of Regnart Creek in the City of Cupertino. The project would extend along the existing creek alignment from Pacifica Drive to E Estates Drive where it would connect to the existing trail to Creekside Park.

The Regnart Creek Trail Project includes the following improvements:

- From Torre Avenue to Regnart Creek, construct a Class I shared-use path along the north side of Pacifica Drive.
- From Pacifica Drive to South Blaney Avenue, construct a Class I shared-use path along the existing SCVWD maintenance access road on the west/north side of the creek.
- From South Blaney Avenue to Wilson Park and from Wilson Park to East Estates Drive, construct a Class I shared-use path along the existing SCVWD maintenance access road on the south side of the creek.
- At approximate 700 feet and 1000 feet east of Blaney Avenue, construct two pedestrian bridges over the creek and pathway improvements within Wilson Park.
- Construct trail access points at Torre Avenue, Pacifica Drive, Rodrigues Avenue, South Blaney Avenue, Wilson Park and East Estates Avenue
- Enhance the trail / roadway crossings at South Blaney Avenue and East Estates Drive.

4.0 FIELD EXPLORATION AND TESTING PROGRAM

The subsurface conditions at the site were studied by reviewing readily available geologic information and subsurface data from four exploratory borings drilled. Borings B-1 and B-2 were drilled in January 2019 by Access Drilling using three-inch diameter solid-stem augers to maximum depths of 26.5 and 31.5 feet, respectively. Borings B-3 and B-4 were drilled in March 2019 by Exploration Geoservices, Inc. using eight-inch diameter hollow-stem augers to maximum depths of 31.5 feet and 61 feet, respectively. The boring locations are shown in Plate 2.

Selected soil samples were obtained from an either 2.5-inch inside diameter (I.D.) Modified California (MC) or 1.4-inch I.D. (at shoe of the sampler) 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 required to drive the sampler were recorded for the last 12 inches.

A hammer efficiency of 60% is assumed for both rigs. When correlating standard penetration data, the blow counts for the MC Sampler may be converted to equivalent SPT blow counts by multiplying an additional conversion factor of 0.65. 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 boring and prepared the samples for subsequent laboratory testing and evaluation.

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.

5.0 LABORATORY TESTING PROGRAM

Laboratory tests were performed on the selected soil sample to evaluate the physical and engineering properties for analyses required for the project such as evaluation of liquefaction potential, pile capacity, and corrosion potential.

Laboratory tests include the following:

a) Moisture (ASTM D2216-10);
b) Density (Based on mass / volume relationships) (ASTM D7263);
c) Plastic Limit, Liquid Limit & Plastic Index (ASTM D4318-17);
d) Grain Size Distribution Analysis (ASTM D6913);
e) Unconfined Compression Test (ASTM D2166);
f) Corrosion Test (Sulfate content, chloride content, resistivity and pH) (California Test Methods 417-mod, 422-mod, and 643);
g) Hydraulic Conductivity (ASTM D5084)

The laboratory test methods and laboratory test results are presented in Appendix B.

6.0 SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

6.1 Site Geology

General geologic features pertaining to the site were evaluated by reference to the "Geologic Map of Cupertino and San Jose quadrangles, Santa Clara and Santa Cruz Counties, California" by Dibblee T.W., and Minch, J.A. dated 2007. The geologic map of the general project area is shown on Plate 3.

Based on this publication, the project site is located on the "Surficial Sediments" (Qa.1) described as "Alluvial sand, fine-grained, silt, and gravel; where differentiated represents alluvial fan deposits at base of slopes and upper fan areas" (Holocene).

A map showing Quaternary Deposits is available by Robert C. Witter, et al., "Maps of Quaternary Deposits and Liquefaction Susceptibilty in the Central San Francisco Bay Region, California", 2006. Based on this map, the site is located on Alluvial Fan deposits (Qpf) of latest Pleistocene period. The quaternary deposits map is shown on Plate 4.

6.2 Subsurface Soil Conditions

Borings B-1 and B-2, located north of the channel, generally encountered stiff to hard Lean/Fat Clays in the first 7 to 8 feet followed by dense to very dense sands with little to some gravel to the maximum depth explored.

Borings B-3 and B-4, located south of the channel, generally encountered about 14 to 18 feet of Lean/Fat Clays followed by dense to very dense sands with little to some gravel to the maximum depth explored. Boring B-4 also encountered a 6 feet thick gravel layer at about 30 feet.

No surface water was observed in the creek during investigation, and groundwater was not encountered up to 60 feet, the maximum depth explored. Depth to historical high groundwater contours on "Seismic Hazard Zone Report for the Cupertino 7.5-Minute Quadrangle" by California Geological Survey dated 2002 indicated the groundwater is deeper than 50 feet (Plate 8). For the purposes of this report, the groundwater was considered at 60 feet depth.

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

7.0 SCOUR EVALUATION

It is our understanding that the channel is partially lined with concrete. Based on our conversation with the designer, scour is not considered for design.

8.0 CORROSION EVALUATION

Chemical tests were performed on selected soil samples from the soil borings to evaluate the corrosion potential of the subsurface soil. The test results are as follows:

TABLE 1 - SUMMARY OF CORROSION TEST RESULTS

Location	Sample Depth (ft)	Minimum Resistivity (ohms-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
B-1	6	880	7.38	132.3	109.3
B-2	11	2680	6.93	19.7	9.2
B-3	6	1130	7.40	5.10	30.6
B-4	3	1310	6.66	8.50	43.8

According to Caltrans Corrosion Guidelines, March 2018 (Version 3.0), 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 1500 ppm,
- pH is 5.5 or less.

Based on the corrosion test results as shown in Table 1 above, the site is not considered corrosive to the structural elements.

9.0 SEISMIC RECOMMENDATIONS

9.1 Seismic Sources

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

Maximum magnitudes (M_{max}) of some of the closest faults in the area are based on Caltrans ARS Online Website. These maximum 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 in the table below. A Caltrans ARS Online Map showing faults in the vicinity for ARS calculation purposes is shown on Plate 5.

TABLE 2 - ARS DATA

Fault (Fault ID)	Maximum Magnitude, M_{max}	Fault Type	Approx. Site-to-Fault Distance (R_{rup})*
Silver Creek (148)	6.9	Strike-Slip	11.7 km
Cascade (153)	6.7	Reverse	0.4 km
Monte Vista-Shannon (154)	6.4	Reverse	3.3 km
San Andreas (Santa Cruz Mts) (158)	8.0	Strike-Slip	9.2 km

* The approximate distances to the fault rupture plane were estimated by Caltrans ARS Online.

9.2 Seismic Design Criteria

The design spectrum shall be designed in accordance with the 2012 Caltrans Fault Database (Version 2b) and the Acceleration Response Spectrum (ARS) Online web tool (Version 2.3.09). 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 30m/100 feet (Vs_{30m}), and other site parameters, such as fault characteristics, site-to-fault distances.

The current design methods incorporate both "Deterministic and Probabilistic Seismic Hazards" to produce the "Design Response Spectrum".

Average shear wave velocity (V_s) for the top 100 feet at the site was estimated by using established correlations and the procedure provided in the Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (November 2012). The site location and the relevant parameters are summarized as follows, and the recommended curve for the bridge design is presented on Plate 6.

1. Site Location : 37.3183ºN/-122.0204ºW
2. Estimated V_{S30m} = 315 m/s
3. Peak Ground Acceleration = ~0.7g
4. Maximum Magnitude = 7.91 (from Probabilistic Deaggregation)
5. The governing ARS case is the Caltrans Online Probabilistic ARS
6. An adjustment factor for near fault effects was applied to the calculated spectral acceleration values. The increase of 20% to the spectral acceleration values corresponds to periods longer than 1 second and linearly tapers to zero at a period of 0.5 second.
7. No adjustments were made for basin effect.

9.3 Seismic Hazards/Liquefaction Potential

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction.

9.3.1 Seismic Ground Shaking

Based on available geological and seismic data, the possibility of the site to experience strong ground shaking is considered high. PGAs of 0.7g was estimated for the site, which is discussed in Section 9.2.

9.3.2 Surface Fault Rupture

Since no known active faults pass through the site and the site is not within a mapped Alquist-Priolo Zone, the fault rupture potential at the site does not exist.

9.3.3 Liquefaction Potential

Liquefaction is a phenomenon in which saturated cohesionless 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. Clays are generally not susceptible to liquefaction.

Field exploration encountered dense to very dense sands/gravels at the site. In addition, groundwater was not encountered in the geotechnical borings.

A map showing Liquefaction susceptibility is available by Robert C. Witter, et al., "Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California", 2006. Based on this map, the site is located on “low” category for liquefaction susceptibility. The map is shown on Plate 7.

Based on the above, the liquefaction potential does not exist and was not considered for foundation design.

10.0 FOUNDATION RECOMMENDATIONS

10.1 General

This report was prepared specifically for the proposed project according to the plans provided to us. Our design criteria have been based upon the materials and subsurface soil conditions encountered in the soil borings at the project site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

10.2 Axial Pile Design

Both bridges over Regnart Creek are planned as single-span structures, and they will be supported on 30-inch diameter cast-in-drilled-hole (CIDH) piles.

Pertinent foundation design information provided by the Structural Designer (Biggs Cardosa Associates, Inc.), including Foundation Design Data and Foundation Loads, are presented in the Tables 4 and 5 located at the end of this report. The cut-off elevation is defined as the elevation of the top of the pile. Finish grade elevation is defined as the final ground surface elevation after construction.

The pile capacities of the CIDH piles were estimated in general accordance with the procedures outlined in Section 10.8.3.5 of AASHTO LRFD BDS 6th Edition (2012), which is quoted from the "Drilled Shafts: Construction Procedures and Design Methods" by O'Neill and Reese (1999). The procedure utilizes α factor for cohesive materials, where α is a function of the undrained shear strength of the clayey materials, and β factor for cohesionless materials, which is a function of the depths.

The pile capacity of the CIDH pile was derived only from frictional resistance along the pile shafts, and end bearing capacity was not included when estimating the pile capacity. Computer program "SHAFT" (by ENSOFT, Inc.) was used for calculation purpose. The analysis results are presented in Appendix C.

The foundation design recommendations and pile data tables are shown in Tables 4 and 5 located at the end of the report.

10.3 Lateral Pile Design

Lateral pile capacity analyses were performed by the structural engineer using the LPILE program.

The soil properties were estimated based on available boring data and laboratory test results. For fined-grained materials, the undrained shear strengths were estimated based on laboratory test results and correlated from the driving resistances of the soil samples (i.e., blow counts) based on NAVFAC DM 7.1. The internal friction angles of granular materials were correlated also based on the driving resistance of the samples per Meyerhof (1956), which is a function of relative density (Dr). The correlated soil properties are presented in Appendix C of the report.

As discussed in Section 6.2, permanent groundwater is relatively deep. Therefore, groundwater was not considered in the analyses.

The recommended geotechnical parameters used in LPILE analyses are provided in the table below. The parameters below apply to both bridges.

Due to the sloping ground surface in front of the piles, the full passive resistance should only be considered where the horizontal distance is 12.5 feet or greater between the center of the pile and the face of the slope .

TABLE 3A – RECOMMENDED LPILE PARAMETERS (ABUTMENT 1) BASED ON BORINGS B-3 & B-4

Elevation (ft)	Generalized Soil Profile	LPILE Soil Type	c (psf)	Phi (degrees)	Effective Unit Weight (pcf)
210 to 202	Stiff Lean/Fat Clay	Stiff Clay w/o Free Water	1,400	-	125
202 to 196	Hard Lean Clay	Stiff Clay w/o Free Water	3,500	-	125
196 to 150	Dense to V. Dense Sand	Sand (Reese)	-	37	125

Notes:
(1) Default values can be used for ε_{50} and K.
(2) P-multipliers of 0.79 and 1.00 for transverse and longitudinal directions, respectively for a pile center-to-center pile spacing of 4D.

TABLE 3B – RECOMMENDED LPILE PARAMETERS (ABUTMENT 2) BASED ON BORINGS B-1 & B-2

Elevation (ft)	Generalized Soil Profile	LPILE Soil Type	c (psf)	Phi (degrees)	Effective Unit Weight (pcf)
210 to 202	Stiff Lean/Fat Clay	Stiff Clay w/o Free Water	1,400	-	125
202 to 150	Dense to V. Dense Sand	Sand (Reese)	-	37	125

Notes:
(1) Default values can be used for ε_{50} and K.
(2) P-multipliers of 0.79 and 1.00 for transverse and longitudinal directions, respectively for center-to-center pile spacing of 4D.

10.4 Lateral Pressures on the Abutment Wall

Abutment retaining walls should be designed to resist the following Applied Lateral Earth Pressures and live load. These values assume no hydrostatic pore pressure buildup behind the wall and are based on well-drained backfill behind the walls supported in native soil.

Applied Lateral Earth Pressure

Active Condition	36 pcf Equivalent Fluid Pressure (EFP) for the structural backfill.
Seismic Pressure	36 pcf EFP (increment, in addition to static earth pressure) based on a kh of 0.35
Passive Resistance	5 ksf (ultimate) for seismic design of the abutment backwall (5.5 feet high or greater); for activated height less than 5.5 feet modify proportionally, i.e. 5×(H/5.5) ksf. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive pressure is required.

Cantilever walls which are free to rotate at least 0.004 radian may be assumed flexible for the active condition. The effect of any surcharge (dead, live, or traffic load) should be added to the preceding lateral earth pressures. A coefficient of 0.28 may be used to determine the additional earth pressure resulting from the surcharge for active condition.

10.5 Stability of Slopes at the Abutment

The impact due to the lateral pile soil reaction on the slope stability of the banks were evaluated. The analyses were performed on the typical section using SLOPE/W program with the following information and assumptions:

- Typical cross-section was based on the information shown in the "General Plan" provided by the designer. Top of the slope is about Elev. 15.6 feet after the proposed construction. Up to 3.5 feet of new fill is expected at the abutments.

- Cross-sections for both bridges are similar for slope stability analysis purposes; therefore, only Bridge 1 was evaluated. Abutment 1 (Northern) was selected and analyzed due to steeper slope (more critical).
- Slope stability was evaluated under the service (static) and seismic (pseudo-static) cases with additional loading from the abutment piles.
- The LPILE analysis from the structural engineer at Abutment 1 was used to estimate the lateral pile pressures on the slope. This analysis was modified from the original run because the passive resistance from the upper portion (where the horizontal setback is less than 12.5 feet) was neglected. The revised model considered a sloping ground condition in front the abutment. The additional pressures on the slope were estimated based on the mobilized soil reaction starting at the pile cap.
- A live load surcharge load of 250 psf was assumed for the service case, which was ignored for the seismic cases.
- A seismic loading coefficient (k_h) of 0.35g was assumed for the seismic case (pseudo-static analysis), which is one-half of the anticipated peak ground acceleration (PGA) at the project site.

The soil strength parameters used in the analyses are shown in Table 3A and 3B. Other input parameters, such as geometry, phreatic surfaces, and the factors of safety and possible critical sliding surfaces obtained from slope stability analyses are presented on the plates in Appendix C.

Based on the results of the slope stability analyses, the calculated factors of safety are 3.32 for static case (greater than 1.5) and 1.77 for the seismic condition (greater than 1.1). Based on these results, the slopes are considered stable under additional pile lateral loading for all analyzed cases.

It is our opinion that the impact of the foundation piles on the slope stability of the existing embankment/levees should be negligible because:

- The extent of the soil reaction is localized and small in comparison with the overall length of the slope. The soil reaction is resisted by the shear strength of the levee soil materials.
- The construction of the proposed CIDH piles minimizes the vibration and impact on the stability of the existing banks as opposed to driven piles.

11.0 CONSTRUCTION CONSIDERATIONS

11.1 General Considerations

To a degree, the performance of any structure is dependent upon construction procedures and quality. Hence, observation of grading operations should be carried out by the engineer-of-record or the responsible Agency. If the encountered subsurface conditions differ from those forming the basis of our recommendations, this office should be informed in order to assess the need for design changes.

11.2 Cast-In-Drilled-Hole (CIDH) Concrete Pile

a) Caltrans standard specifications and standard special provisions (SSP) for "Cast-in-Place Concrete Piling" should be used for the construction of CIDH concrete piles. Access tubes for acceptance testing should be provided in all CIDH concrete piles that are 24 inches in diameter or larger for construction quality control, 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 cross-hole sonic logging for verification. Gamma-Gamma Logging should be performed in accordance with California Test 233 Standard (CT233) to check the homogeneity of CIDH concrete piles.

b) Due to the presence of granular material, raveling or caving is anticipated, 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 appropriate steps can be taken to comply with the standards and maintain the integrity of the CIDH concrete pile.

c) The use of temporary casing should be expected during pile foundation construction.
d) It is recommended that the specifications set certain criteria for qualifications and previous work experience requirements to pre-qualify the potential contractors. The intent is to help select qualified contractors to reduce construction issues.
e) Relatively hard drilling could be expected due to the presence of very dense gravel/sands and intensely weathered/fractured rock at depth. During our geotechnical exploration, all holes were advanced by augers without coring.

12.0 PLAN REVIEW

This report is prepared for the proposed "Regnart Creek Trail Bridges" project. We recommend that final foundation plans for the proposed project to be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred. However, design-build elements should be reviewed only from overall compliance standpoint.

13.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

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

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

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

Very truly yours,
PARIKH CONSULTANTS, INC.

A. Emre Ortakci, P.E., G.E. 3067
Project Engineer

Frank Wang, P.E., G.E. 2862
Senior Project Engineer

TABLE 4A – FOUNDATION DESIGN DATA (BRIDGE 1)

Support No.	Design Method	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (Bottom of Footing Elevation) (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	Number of Piles per Support	Design Tip Elev for Lateral Loading (ft)
					B	L			
Abut 1	LRFD	30" Dia CIDH Pile	215.6	209.3	3	18.67	1	2	182.0
Abut 2	LRFD	30" Dia CIDH Pile	215.6	208.9	3	18.67	1	2	182.0

TABLE 4B – FOUNDATION LOADS (BRIDGE 1)

Support No.	Service-I Limit State (kips)		Strength/Construction Limit State				Extreme Event Limit State (Controlling Group, kips)			
			Compression		Tension		Compression		Tension	
	Total Load per Support	Permanent Loads per Support	Per Support	Max. per pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	122	61	197	98	0	0	97	48	0	0
Abut 2	122	61	197	98	0	0	97	48	0	0

TABLE 4C – FOUNDATION DESIGN RECOMMENDATIONS (BRIDGE 1)

Support No.	Pile Type	Cut-off Elevation (ft) (NAVD88)	Service-I Limit State Load (kips) per Support		Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev. (ft) (NAVD88)	Specified Tip Elev. (ft) (NAVD88)
						Strength Limit		Extreme Event			
			Total	Permanent		Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1.0)	Tension (φ=1.0)		
Abut 1	30" Dia CIDH Pile	209.3	122	61	1	98	N/A	48	N/A	193.0 (a-I) 199.0 (a-II) 182.0 (d) (iii)	182.0
Abut 2	30" Dia CIDH Pile	208.9	122	61	1	98	N/A	48	N/A	(a-I) 190.0 (a-II) 198.0 (d) 182.0(iii)	182.0

Notes:
(i) Design tip elevations are controlled by (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (b-I) Tension (Strength Limit), (b-II) Tension (Extreme Event), (d) Lateral Load.
(ii) Settlements under service loads do not govern the design.
(iii) Design tip elevations for lateral were provided by the structural designer (HMH).

TABLE 4D – PILE DATA TABLE (BRIDGE 1)

Support No.	Pile Type	Nominal Resistance (kips)		Design Tip Elev. (ft) (NAVD88)	Specified Tip Elev. (ft) (NAVD88)
		Compression	Tension		
Abut 1	30" Dia CIDH Pile	140	N/A	(a) 193.0 (d) 182.0	182.0
Abut 2	30" Dia CIDH Pile	140	N/A	(a) 190.0 (d) 182.0	182.0

Notes:

(1) Design tip elevations are controlled by: (a) Compression, (d) Lateral Load

(2) Settlements under service loads do not govern the design.

(3) Design tip elevations for lateral were provided by the structural designer (HMH).

TABLE 5A – FOUNDATION DESIGN DATA (BRIDGE 2)

Support No.	Design Method	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (Bottom of Footing Elevation) (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	Number of Piles per Support	Design Tip Elev for Lateral Loading (ft)
					B	L			
Abut 1	LRFD	30" Dia CIDH Pile	214.3	209.2	3	16	1	2	182.0
Abut 2	LRFD	30" Dia CIDH Pile	214.3	207.5	3	16	1	2	181.0

TABLE 5B – FOUNDATION LOADS (BRIDGE 2)

Support No.	Service-I Limit State (kips)		Strength/Construction Limit State				Extreme Event Limit State (Controlling Group, kips)			
			Compression		Tension		Compression		Tension	
	Total Load per Support	Permanent Loads per Support	Per Support	Max. per pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	118	59	190	95	0	0	94	47	0	0
Abut 2	118	59	190	95	0	0	94	47	0	0

TABLE 5C – FOUNDATION DESIGN RECOMMENDATIONS (BRIDGE 2)

Support No.	Pile Type	Cut-off Elevation (ft) (NAVD88)	Service-I Limit State Load (kips) per Support		Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elev. (ft) (NAVD88)	Specified Tip Elev. (ft) (NAVD88)
						Strength Limit		Extreme Event			
			Total	Permanent		Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1.0)	Tension (φ=1.0)		
Abut 1	30" Dia CIDH Pile	209.2	118	59	1	95	N/A	47	N/A	(a-I) 193.0 (a-II) 199.0 (d) 182.0(iii)	182.0
Abut 2	30" Dia CIDH Pile	207.5	118	59	1	95	N/A	47	N/A	(a-I) 190.0 (a-II) 198.0 (d) 181.0(iii)	181.0

Notes:
(i) Design tip elevations are controlled by (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (b-I) Tension (Strength Limit), (b-II) Tension (Extreme Event), (d) Lateral Load.
(ii) Settlements under service loads do not govern the design.
(iii) Design tip elevations for lateral were provided by the structural designer (HMH).

TABLE 5D – PILE DATA TABLE (BRIDGE 2)

Support No.	Pile Type	Nominal Resistance (kips)		Design Tip Elev. (ft) (NAVD88)	Specified Tip Elev. (ft) (NAVD88)
		Compression	Tension		
Abut 1	30" Dia CIDH Pile	140	N/A	(a) 193.0 (d) 182.0	182.0
Abut 2	30" Dia CIDH Pile	140	N/A	(a) 190.0 (d) 181.0	181.0

Notes:

(1) Design tip elevations are controlled by: (a) Compression, (d) Lateral Load

(2) Settlements under service loads do not govern the design.

(3) Design tip elevations for lateral were provided by the structural designer (HMH).

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REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO.: 2018-151-GEO | **PLATE NO.: 1**

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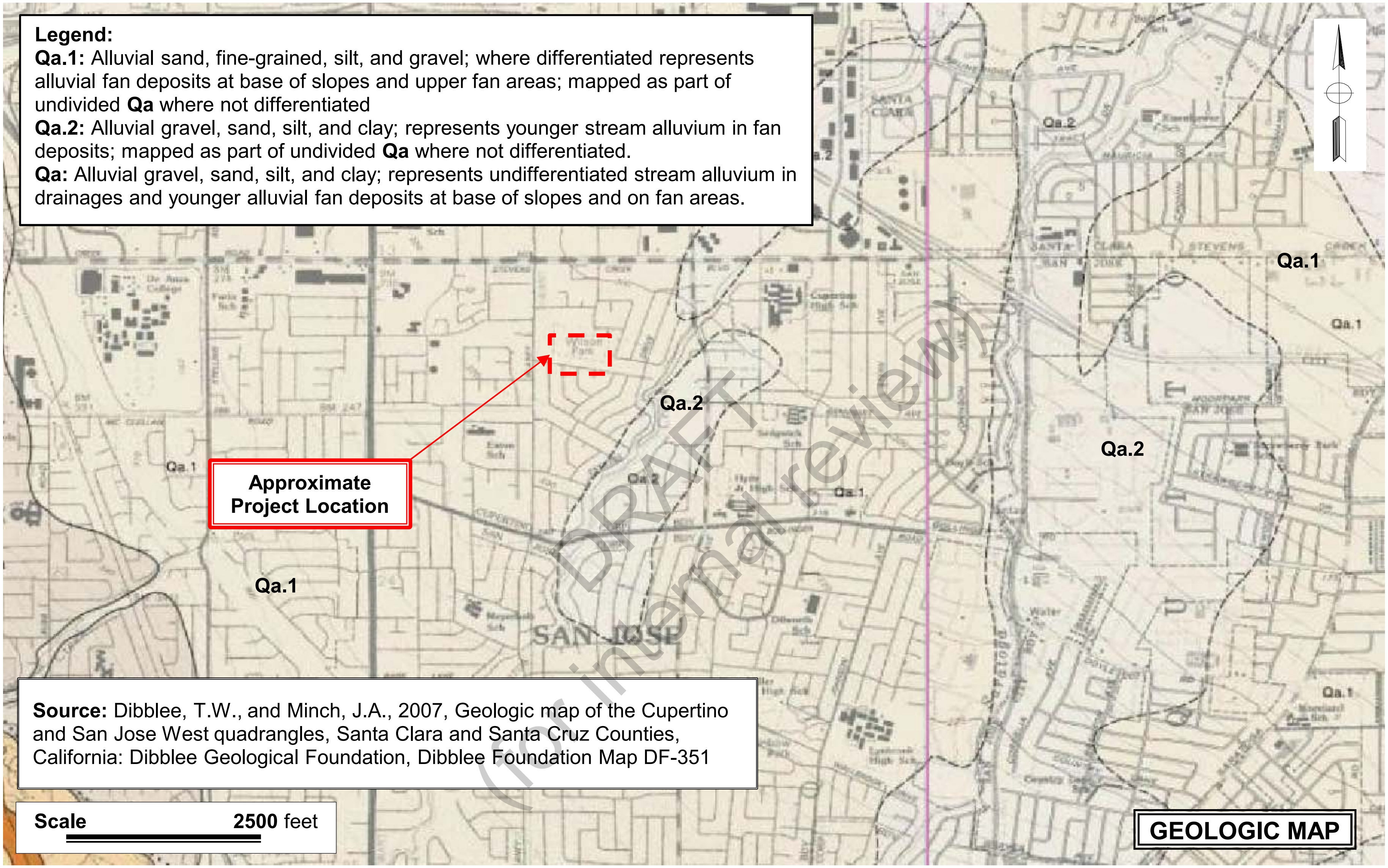
REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO.: 2018-151-GEO	PLATE NO.: 2



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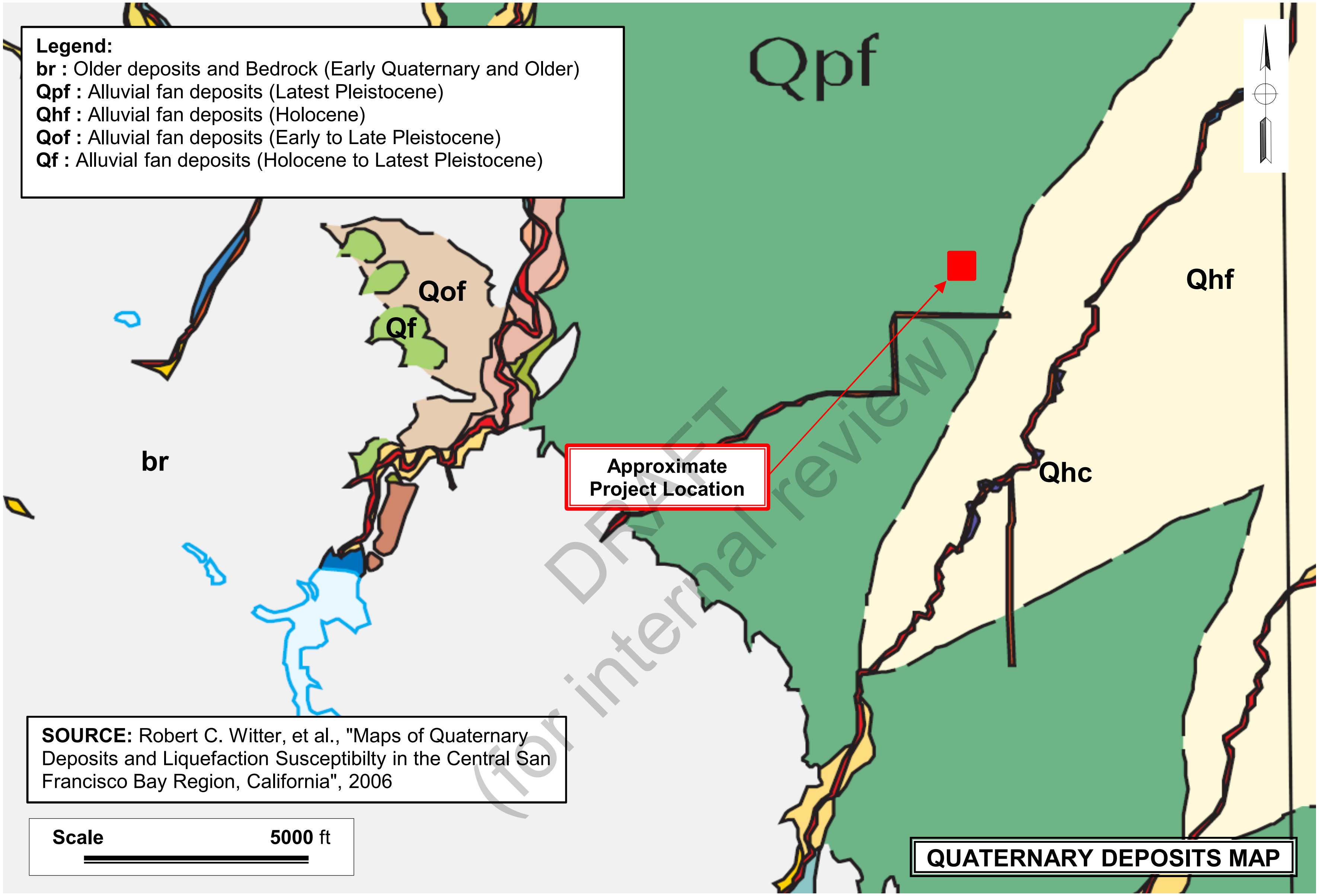
REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO.: 2018-151-GEO | **PLATE NO.: 3**



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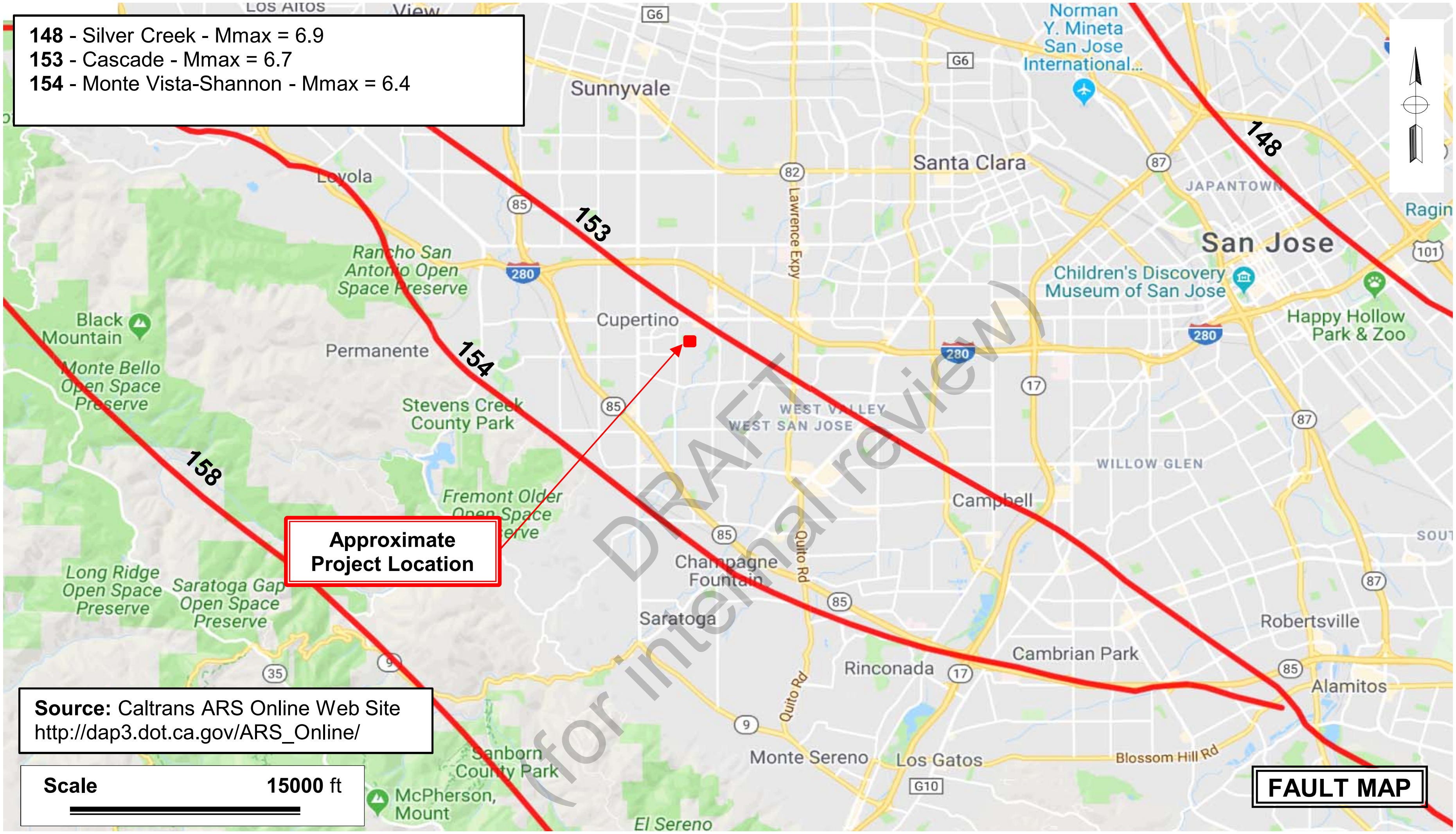
REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO.: 2018-151-GEO

PLATE NO.: 4

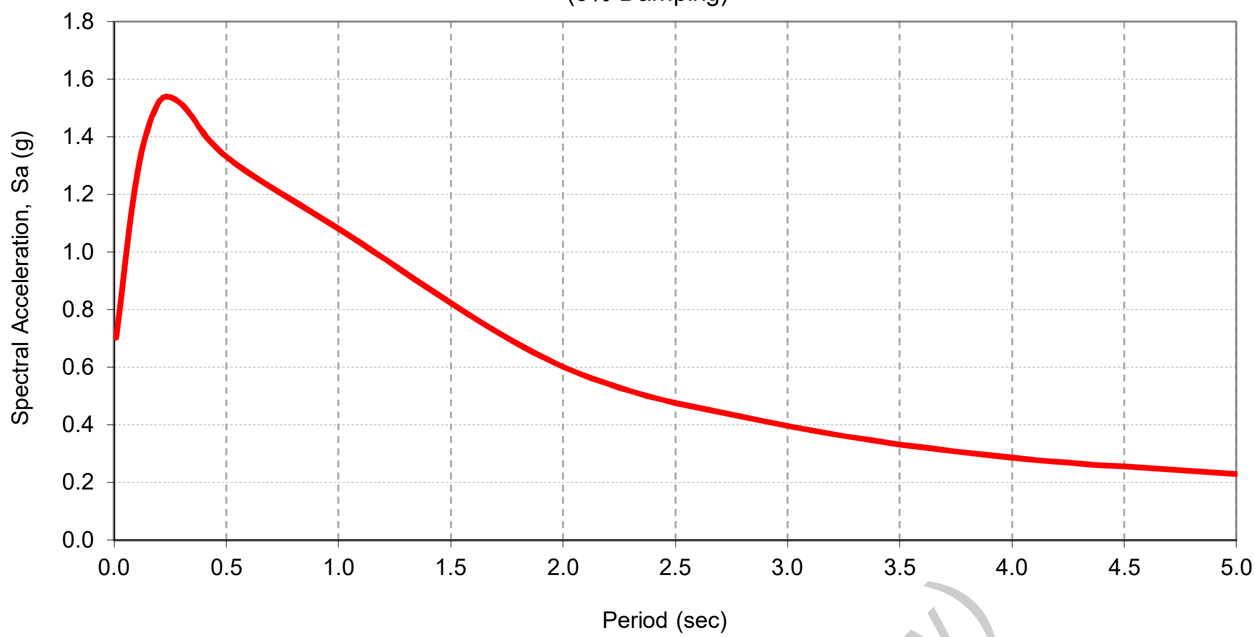


REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO.: 2018-151-GEO	PLATE NO.: 5

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

(5% Damping)



Site Information

Latitude:	37.3183
Longitude	-122.0204
V_{S30} (m/s) =	315
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Unified Hazard Tool. Dist (km) =	9.46

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	0.703	1	1	0.703
0.1	1.26	1	1	1.260
0.2	1.521	1	1	1.521
0.3	1.514	1	1	1.514
0.5	1.332	1	1	1.332
1.0	0.901	1.2	1	1.081
2.0	0.502	1.2	1	0.602
3.0	0.331	1.2	1	0.397
4.0	0.239	1.2	1	0.287
5.0	0.192	1.2	1	0.230

Source:

1. Caltrans ARS Online tool (V.2.3.09, http://dap3.dot.ca.gov/ARS_Online/)

2. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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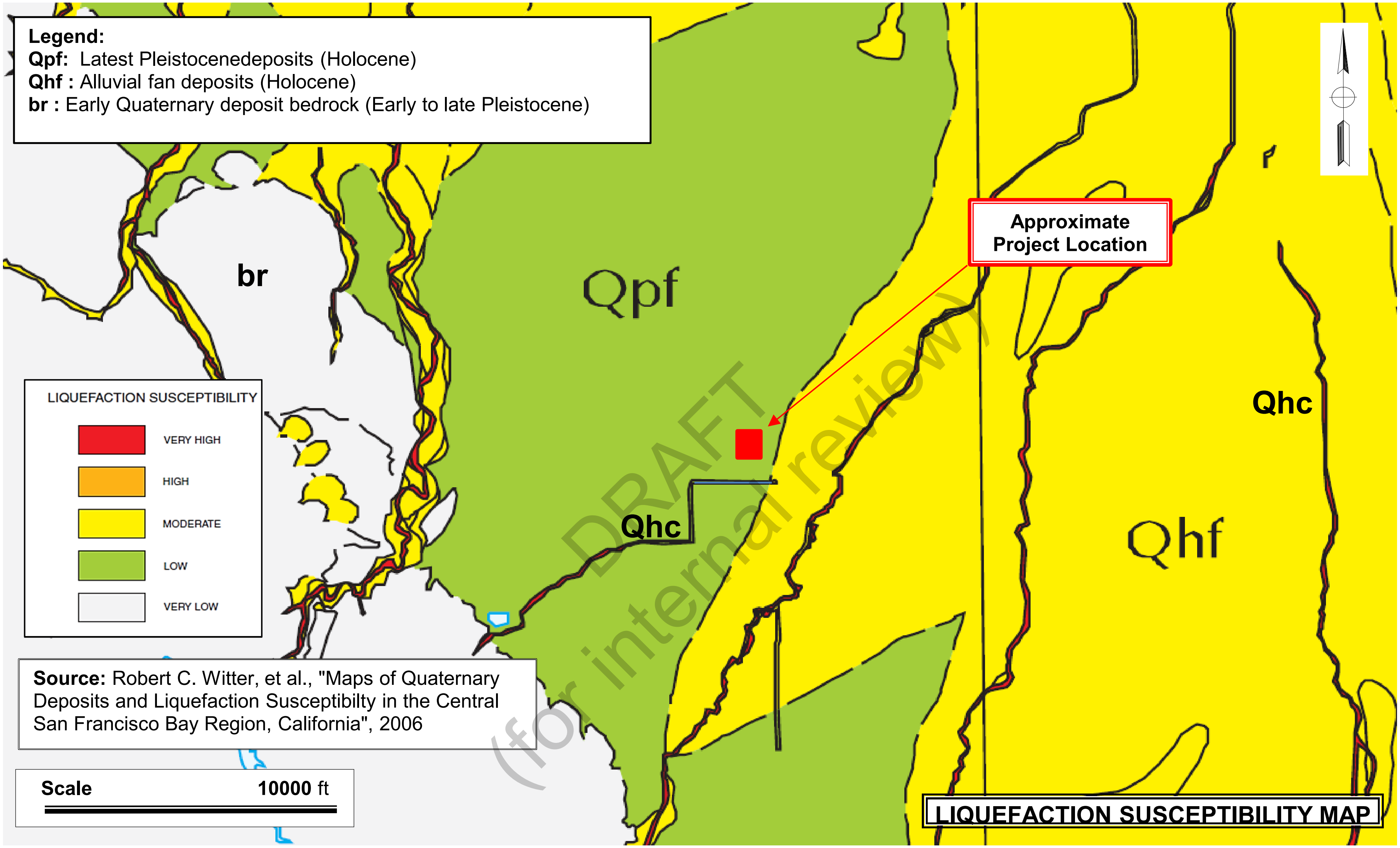
REGNART CREEK TRAIL BRIDGES CUPERTINO, CALIFORNIA	
JOB NO.: 2018-151-GEO	PLATE NO.: 6



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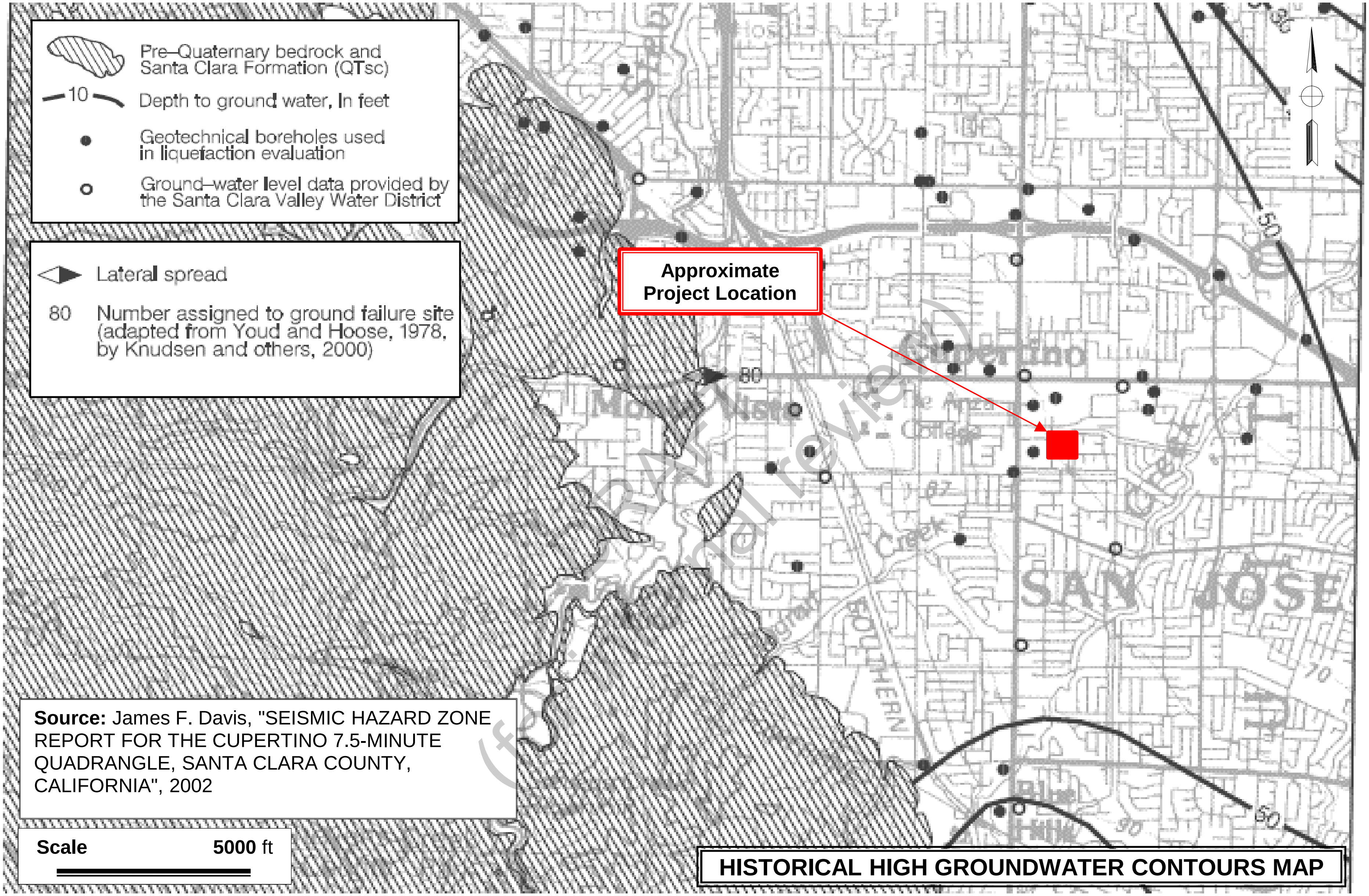
REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO.: 2018-151-GEO | PLATE NO.: 7



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REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO.: 2018-151-GEO	**PLATE NO.: 8**

APPENDIX A

GROUP SYMBOLS AND NAMES

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

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

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

BORING RECORD LEGEND

PARIKH Practicing in the Geosciences

REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 5/3/2019 | Job No.: 2018-151-GEO

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-0A

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

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

PERCENT OR PROPORTION OF SOILS

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

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

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

CEMENTATION

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

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only.

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

BORING RECORD LEGEND

PARIKH
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REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 5/3/2019 | Job No.: 2018-151-GEO

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Plate: A-0B

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
Virgil S.	**1-14-18**	**1-14-18**	**37° 19' 6.02" / 122° 1' 10.99"**	**B-1**
DRILLING CONTRACTOR **Access Soil Drilling**			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION **~211.0 ft**
DRILLING METHOD **Solid-Stem Auger**			DRILL RIG **Minuteman**	BOREHOLE DIAMETER **4 in**
SAMPLER TYPE(S) AND SIZE(S) ID **MC (2.5"), SPT (1.4")**			SPT HAMMER TYPE **140 lbs Manual Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **60%**
BOREHOLE BACKFILL AND COMPLETION **Neat Cement Grout**			GROUNDWATER READINGS DURING DRILLING **Not encountered** AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **26.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Remarks
	0	Fat CLAY (CH); very stiff; brownish GRAY; moist; w/ chunk of wood; (PP=2.5 tsf). (LL=54, PI=34).	1	5 8	8/6	23	96		100		PI
209.00	2										
	3	SANDY lean CLAY (CL); hard; grayish brown; moist; (PP>4.5 tsf).									
207.00	4										
205.00	5–6		2	15 21 30	51	12	120		100		CR
	7	SILTY SAND with GRAVEL (SM); dense; yellowish brown; moist; fine SAND; [weathered Conglomerate].									
203.00	8										
201.00	10		3	26 50 50/4"	100/10	11	110		100		
199.00	12										
197.00	14										
195.00	15–16	Very dense; grayish brown; [weathered Sandstone and Siltstone]; (+#4=16.9%, -#200=29.6%).	4	18 53 50/3.5"	103/10	5			77		PA
193.00	18	SILTY SAND (SM); dense; grayish brown; moist; [weathered Sandstone].									
191.00	20–21	(+#4=13.8%, -#200=17.1%).	5	14 17 16	33	4			89		PA
189.00	22										
	23	Poorly graded SAND with SILT and GRAVEL (SP-SM); very dense; grayish brown; moist; [weathered Sandstone].									
187.00	24										
	25										

(continued)

LOG OF TEST BORING

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REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018 | Boring ID: B-1 | Job No.: 2018-151-GEO

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
185.00	25 26		Poorly graded SAND with SILT and GRAVEL (SP-SM).		6	42 36 25	61	4			100				
	27		Bottom of borehole at 26.5 ft bgs/Elev. 184.5 ft												
183.00	28														
	29														
181.00	30														
	31														
179.00	32														
	33														
177.00	34														
	35														
175.00	36														
	37														
173.00	38														
	39														
171.00	40														
	41														
169.00	42														
	43														
167.00	44														
	45														
165.00	46														
	47														
163.00	48														
	49														
161.00	50														
	51														
159.00	52														
	53														
157.00	54														
	55														

LOG OF TEST BORING

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REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018 | Boring ID: B-1 | Job No.: 2018-151-GEO

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Plate: A-1B

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
Virgil S.	**1-15-18**	**1-15-18**	**37° 19' 6.35" / 122° 1' 14.08"**	**B-2**
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Access Soil Drilling				**~209.0 ft**
DRILLING METHOD			DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger			**Minuteman**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) ID			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
MC (2.5"), SPT (1.4")			**140 lbs Manual Hammer with 30" Drop**	**60%**
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat Cement Grout			**Not encountered**	**31.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Remarks
	0	SANDY lean CLAY (CL); very stiff; dark gray; moist; trace GRAVEL; medium to fine SAND; (PP=1.5 tsf).	1	3 10 9	19				100		
207.00	2	Lean CLAY (CL); stiff; brown; moist; trace fine SAND.									
203.00	5–6	(UC= 1.38 tsf).	2	12 12 12	24	17	43	0.69	100		UC
201.00	8	SILTY SAND with GRAVEL (SM); very dense; yellowish brown; moist; [weathered Conglomerate].									
199.00	10–11	(+#4=32.4%, -#200=18.9%).	3	21 44 50/4"	94/10	9	64		100		CR, PA
193.00	15–16		4	26 30 29	59	9			72		
191.00	18	Poorly graded SAND with GRAVEL (SP); dense; gray; moist; weathered.									
189.00	20–21		5	22 16 21	37	5			72		
	23	SILTY SAND with GRAVEL (SM); very dense; gray and yellowish brown; moist; weathered.									
185.00	24										
	25										

(continued)

LOG OF TEST BORING

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REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018 | Boring ID: B-2 | Job No.: 2018-151-GEO

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Plate: A-2A

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	25		SILTY SAND with GRAVEL (SM).		6	36 42 35	77	6			78				
183.00	26														
	27														
181.00	28														
	29														
179.00	30				7	23 15 24	39	8			33				
	31		Dense. (+#4=37.2%, -#200=18.1%).												PA
177.00	32		Bottom of borehole at 31.5 ft bgs/Elev. 177.5 ft												
	33														
175.00	34														
	35														
173.00	36														
	37														
171.00	38														
	39														
169.00	40														
	41														
167.00	42														
	43														
165.00	44														
	45														
163.00	46														
	47														
161.00	48														
	49														
159.00	50														
	51														
157.00	52														
	53														
155.00	54														
	55														

LOG OF TEST BORING

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REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018 | Boring ID: B-2 | Job No.: 2018-151-GEO

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Plate: A-2B

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
Jackson Z. & Do N.	**3-13-19**	**3-13-19**	**37° 19' 5.21" / 122° 1' 11.03"**	**B-3**
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Exploration Geoservices				**~208.0 ft**
DRILLING METHOD			DRILL RIG	BOREHOLE DIAMETER
Hollow-Stem Auger			**Mobile B53**	**8 in**
SAMPLER TYPE(S) AND SIZE(S) ID			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
MC (2.5")			**140 lbs Semi-Automatic Hammer with 30" Drop**	**63%**
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS DURING DRILLING / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat Cement Grout			DURING DRILLING: **Not encountered**	**31.3 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Remarks
	0	Fat CLAY (CH); very stiff; brown; moist; trace medium to fine SAND; medium plasticity fines; trace root (PP=3.0 tsf).									
206.00	2		1	3 6 8	14	15			56		PI
204.00	4	Lean CLAY (CL); very stiff; yellowish brown; moist; low plasticity fines; Claystone (PP>4.5 tsf).									
202.00	5–6		2	17 33 37	70	13	105		72		CR
200.00	8										
198.00	10		3	26 50/6"	50/6	19			100		
196.00	12										
194.00	14										
192.00	15–16		4	22 26 35	61	9			94		PI
190.00	18	Well-graded SAND with SILT and GRAVEL (SW-SM); very dense; brown; moist; fine GRAVEL, max. 1/2" in. dia.; fine SAND.									
188.00	20		5	28 50/5"	50/5	5			100		PA
186.00	22										
184.00	23–24	CLAYEY SAND with GRAVEL (SC); very dense; brown; moist; fine GRAVEL, max. 1/2" in. dia.; medium to fine SAND.									
	25										

(continued)

LOG OF TEST BORING

PARIKH Practicing in the Geosciences

REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018	Boring ID: B-3	Job No.: 2018-151-GEO

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-3A

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	25		CLAYEY SAND with GRAVEL (SC).		6	50/4"	REF	5			100				
182.00	26														
	27														
180.00	28														
	29		Well-graded SAND with SILT and GRAVEL (SW-SM); dense; brown; moist; fine GRAVEL, max. 1/2" in. dia.; fine SAND.												
178.00	30				7	21 25 50/4"	75/10	7			100				PA
	31		Bottom of borehole at 31.3 ft bgs/Elev. 176.7 ft												
176.00	32														
	33														
174.00	34														
	35														
172.00	36														
	37														
170.00	38														
	39														
168.00	40														
	41														
166.00	42														
	43														
164.00	44														
	45														
162.00	46														
	47														
160.00	48														
	49														
158.00	50														
	51														
156.00	52														
	53														
154.00	54														
	55														

LOG OF TEST BORING

PARIKH
Practicing in the Geosciences

REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018 | Boring ID: B-3 | Job No.: 2018-151-GEO

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-3B

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
Jackson Z. & Do N.	**3-13-19**	**3-13-19**	**37° 19' 5.77" / 122° 1' 15.36"**	**B-4**

DRILLING CONTRACTOR **Exploration Geoservices**	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION **~209.0 ft**
DRILLING METHOD **Hollow-Stem Auger**	DRILL RIG **Mobile B53**	BOREHOLE DIAMETER **8 in**
SAMPLER TYPE(S) AND SIZE(S) ID **MC (2.5")**	SPT HAMMER TYPE **140 lbs Semi-Automatic Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **63%**
BOREHOLE BACKFILL AND COMPLETION **Neat Cement Grout**	GROUNDWATER READINGS — DURING DRILLING: **Not encountered**; AFTER DRILLING (DATE):	TOTAL DEPTH OF BORING **61.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		Lean CLAY (CL); stiff; dark brown; moist; trace fine GRAVEL; medium to fine SAND; low to medium plasticity fines; (PP=1.25 tsf).												
	1														
207.00	2				1	2 8 11	19	16			39				
	3														CR
205.00	4														
	5		Very stiff; light brown; low plasticity fines; with root (PP=3.5 tsf).		2	8 10 18	28	11	116		83				
203.00	6														PI
	7														
201.00	8														
	9														
199.00	10		Very stiff to hard; yellowish brown; dry; with Claystone (PP>4.5 tsf).		3	27 50/6"	50/6	9			100				
	11														
197.00	12														
	13														
195.00	14		Poorly graded SAND with SILT and GRAVEL (SP-SM).												
	15				4	20 30 36	66	5			83				
193.00	16														PA
	17														
191.00	18														
	19														
189.00	20		Wet.		5	18 21 25	46	6			78				
	21														
187.00	22														
	23														
185.00	24														
	25														

(continued)

LOG OF TEST BORING

PARIKH Practicing in the Geosciences

REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018 | Boring ID: B-4 | Job No.: 2018-151-GEO

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	25		Poorly graded SAND with SILT and GRAVEL (SP-SM); dense; brown; moist; fine GRAVEL, max. 1 1/2" in. dia.; medium to fine SAND.		6	5 26 30	56	8			78				
183.00	26														
	27														
181.00	28		Well-graded GRAVEL with SAND (GW); very dense; yellowish brown; wet; coarse to fine SAND.												
	29														
179.00	30				7	50/5"	REF	5			100				PA
	31														
177.00	32														
	33														
175.00	34		Poorly graded SAND with SILT and GRAVEL (SP-SM); very dense; yellowish brown; wet; medium to fine SAND; with brown Claystone.												
	35				8	31 50/6"	50/6	6			100				
173.00	36														
	37														
171.00	38														
	39														
169.00	40		Dense; dark yellowish brown.		9	27 23 25	48	9			94				
	41														
167.00	42														
	43														
165.00	44														
	45		Moist.		10	28 40 41	81	7			94				
163.00	46														
	47														
161.00	48														
	49														
159.00	50				11	35 50/6"	50/6	11			100				PA
	51														
157.00	52														
	53														
155.00	54														
	55														

(continued)

LOG OF TEST BORING

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Practicing in the Geosciences

REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018 | Boring ID: B-4 | Job No.: 2018-151-GEO

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-4B

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
			Poorly graded SAND with SILT and GRAVEL (SP-SM). Very dense; yellowish brown; wet.		12	50/5"	REF	10			100				
153.00	56														
	57														
151.00	58														
	59														
149.00	60		Dark yellowish brown.		13	35 50/6"	50/6	7			92				
	61		Bottom of borehole at 61.0 ft bgs/Elev. 148.0 ft												
147.00	62														
	63														
145.00	64														
	65														
143.00	66														
	67														
141.00	68														
	69														
139.00	70														
	71														
137.00	72														
	73														
135.00	74														
	75														
133.00	76														
	77														
131.00	78														
	79														
129.00	80														
	81														
127.00	82														
	83														
125.00	84														
	85														

LOG OF TEST BORING

PARIKH Practicing in the Geosciences

REGNART CREEK TRAIL

CUPERTINO, CALIFORNIA

Date: 1/14/2018 | Boring ID: B-4 | Job No.: 2018-151-GEO

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-4C

PCI-CT 5 BR 2018-151-GEO.GPJ TEMPLATE 7-22-11.GDT 5/3/19

APPENDIX B

APPENDIX B
LABORATORY TESTS

Classification Tests

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

Moisture-Density

The natural moisture contents were determined for selected undisturbed samples of the soils in general accordance with ASTM D2216-10 and dry unit weights based on mass/volume relationships. This information was used to classify and correlate the soils. The results are presented on Plate B-1 "Summary of Laboratory Test Results", Appendix B.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM D4318-17. The results of the test are presented on Plate B-2, "Plasticity Chart", Appendix B.

Grain Size Classification

Grain size classification tests (ASTM Test Method D 6913) were performed on selected samples to aid in the classification. The results are presented on Plate B-3, "Grain Size Distribution Curves", Appendix B.

Corrosion Tests

A corrosion test was performed by Sunland Analytical on selected sample to determine the corrosion potential of the soils. The pH and minimum resistivity tests (California Test Method 643), Sulfate (California Test Method 417-mod) and Chloride (California Test Method 422mod) tests were performed by Sunland Analytical. The test results are presented on Plates B-4A to B-4D, Appendix B.

Unconfined Compression Tests

Unconfined Compression Tests were performed in general accordance with ASTM D2166 to determine the shear strength of the soils under undrained condition. The test results are presented on plate B-5, Appendix B.

Hydraulic Conductivity Tests

Hydraulic Conductivity Tests were performed by Cooper Testing Labs in general accordance with ASTM D5084 to determine the permeability of porous materials. The test results are presented on Plate B-6, Appendix B.

Borehole	Sample Number	Depth	Classi-fication	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Unconfined Shear Strength (tsf)
B-1	1	0.5	CH	23.0	95.6	54	20	34			
B-1	2	6.0	CL	12.1	119.6						
B-1	3	11.0	SM	10.5	110.2						
B-1	4	16.0	SM	4.9	-				16.9	29.6	
B-1	5	21.0	SM	3.7	-				13.8	17.1	
B-1	6	26.0	SP-SM	4.1	-						
B-2	1	1.0	CL	-	-						
B-2	2	6.0	CL	16.7	43.5						0.69
B-2	3	11.0	SM	9.4	64.2				32.4	18.9	
B-2	4	16.0	SM	9.3	-						
B-2	5	21.0	SP	5.1	-						
B-2	6	26.0	SM	6.3	-						
B-2	7	31.0	SM	8.2	-				37.2	18.1	
B-3	1	3.0	CH	15.4	-	54	20	34			
B-3	2	6.0	CL	12.6	105.2						
B-3	3	10.5	CL	19.4	-						
B-3	4	16.0	CL	9.3	-	38	17	21			
B-3	5	20.5	SW-SM	5.3	-				32.6	10.7	
B-3	6	25.0	SC	4.8	-						
B-3	7	31.0	SW-SM	7.1	-				32.9	7.5	
B-4	1	3.0	CL	16.0	-						
B-4	2	6.0	CL	10.9	116.1	30	15	15			
B-4	3	10.5	CL	9.1	-						
B-4	4	16.0	SP-SM	5.1	-				26.2	8.1	
B-4	5	21.0	SP-SM	6.4	-						
B-4	6	26.0	SP-SM	7.8	-						
B-4	7	30.0	GW	5.1	-				63.8	4.8	
B-4	8	35.5	SP-SM	5.8	-						
B-4	9	41.0	SP-SM	9.0	-						
B-4	10	46.0	SP-SM	7.1	-						
B-4	11	50.5	SP-SM	11.2	-				27.3	10.1	
B-4	12	55.0	SP-SM	9.7	-						
B-4	13	60.5	SP-SM	6.7	-						

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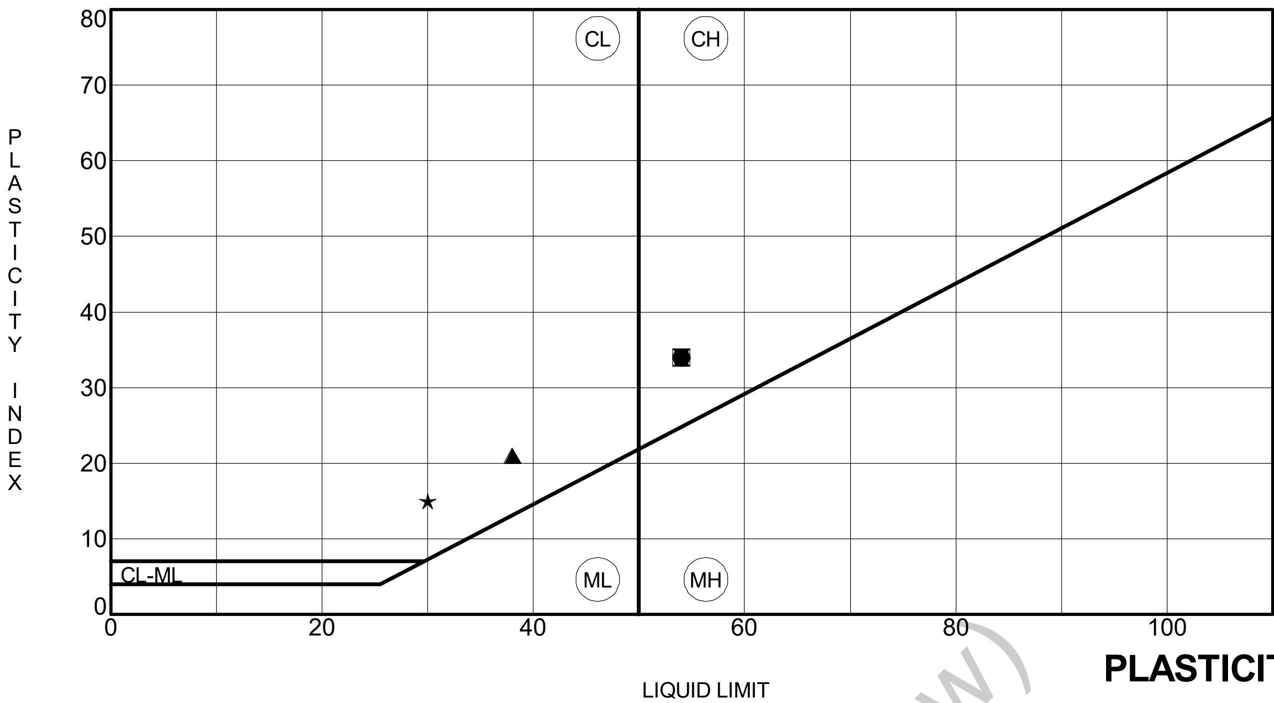
REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO: 2018-151-GEO **PLATE NO:** B-1



PLASTICITY

	BOREHOLE	SAMPLE #	DEPTH	LL	PL	PI	Fines	Classification
●	**B-1**	**1**	**0.5**	**54**	**20**	**34**		**Fat CLAY**
⧗	**B-3**	**1**	**3.0**	**54**	**20**	**34**		**Fat CLAY**
▲	**B-3**	**4**	**16.0**	**38**	**17**	**21**		**Lean CLAY**
★	**B-4**	**2**	**6.0**	**30**	**15**	**15**		**Lean CLAY**

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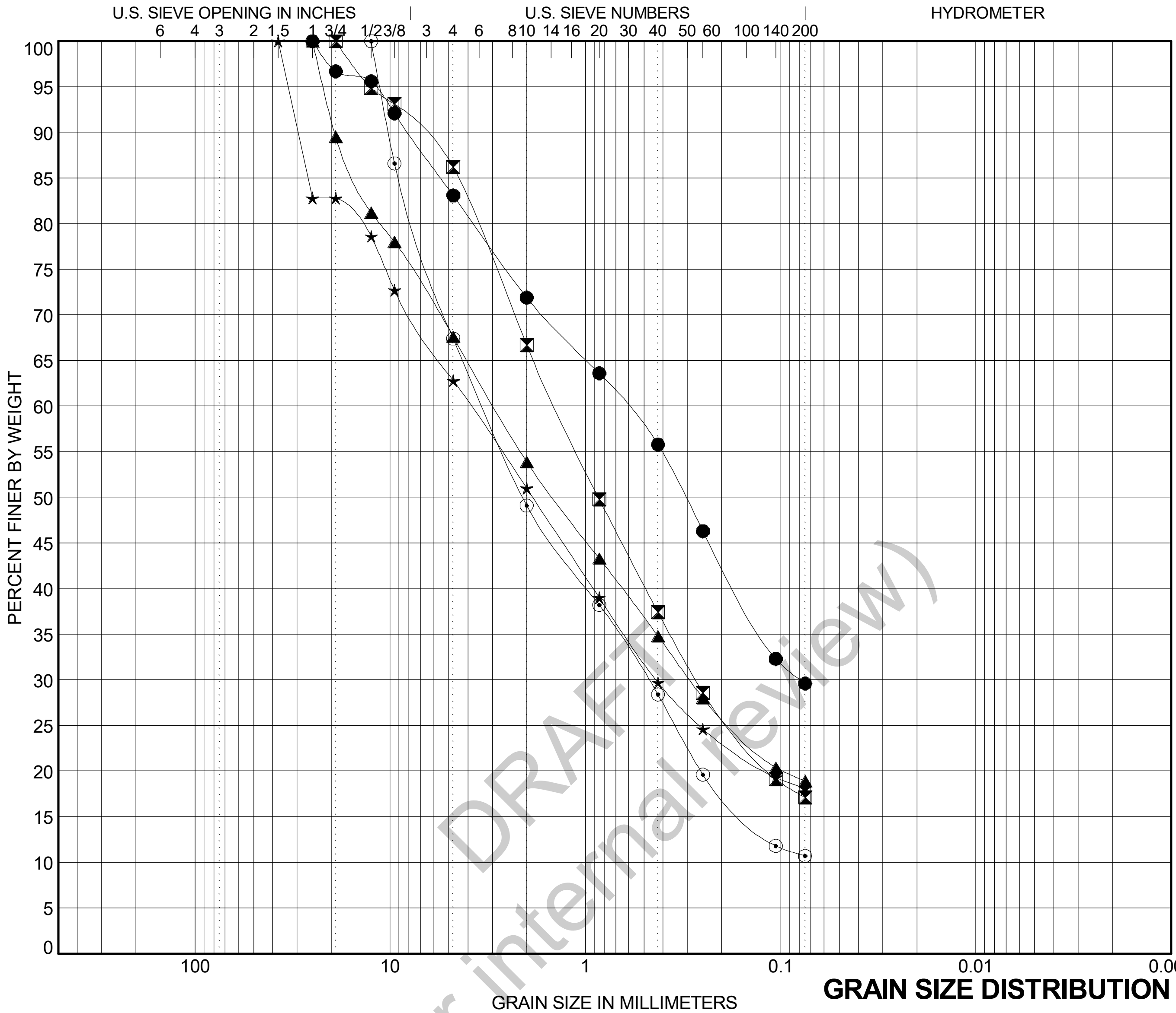
CUPERTINO, CALIFORNIA

JOB NO: 2018-151-GEO | **PLATE NO:** B-2



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**4**	**16.0**	**SILTY SAND with GRAVEL**					
⧗	**B-1**	**5**	**21.0**	**SILTY SAND**					
▲	**B-2**	**3**	**11.0**	**SILTY SAND with GRAVEL**					
★	**B-2**	**7**	**31.0**	**SILTY SAND with GRAVEL**					
⊙	**B-3**	**5**	**20.5**	**Well-graded SAND with SILT and GRAVEL**				**1.12**	**55.63**

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**4**	**16.0**	**25**	**0.617**	**0.079**		**16.9**	**53.5**	**29.6**	
⧗	**B-1**	**5**	**21.0**	**19**	**1.425**	**0.272**		**13.8**	**69.1**	**17.1**	
▲	**B-2**	**3**	**11.0**	**25**	**2.94**	**0.292**		**32.4**	**48.7**	**18.9**	
★	**B-2**	**7**	**31.0**	**37.5**	**3.869**	**0.435**		**37.2**	**44.7**	**18.1**	
⊙	**B-3**	**5**	**20.5**	**12.5**	**3.348**	**0.476**		**32.6**	**56.7**	**10.7**	

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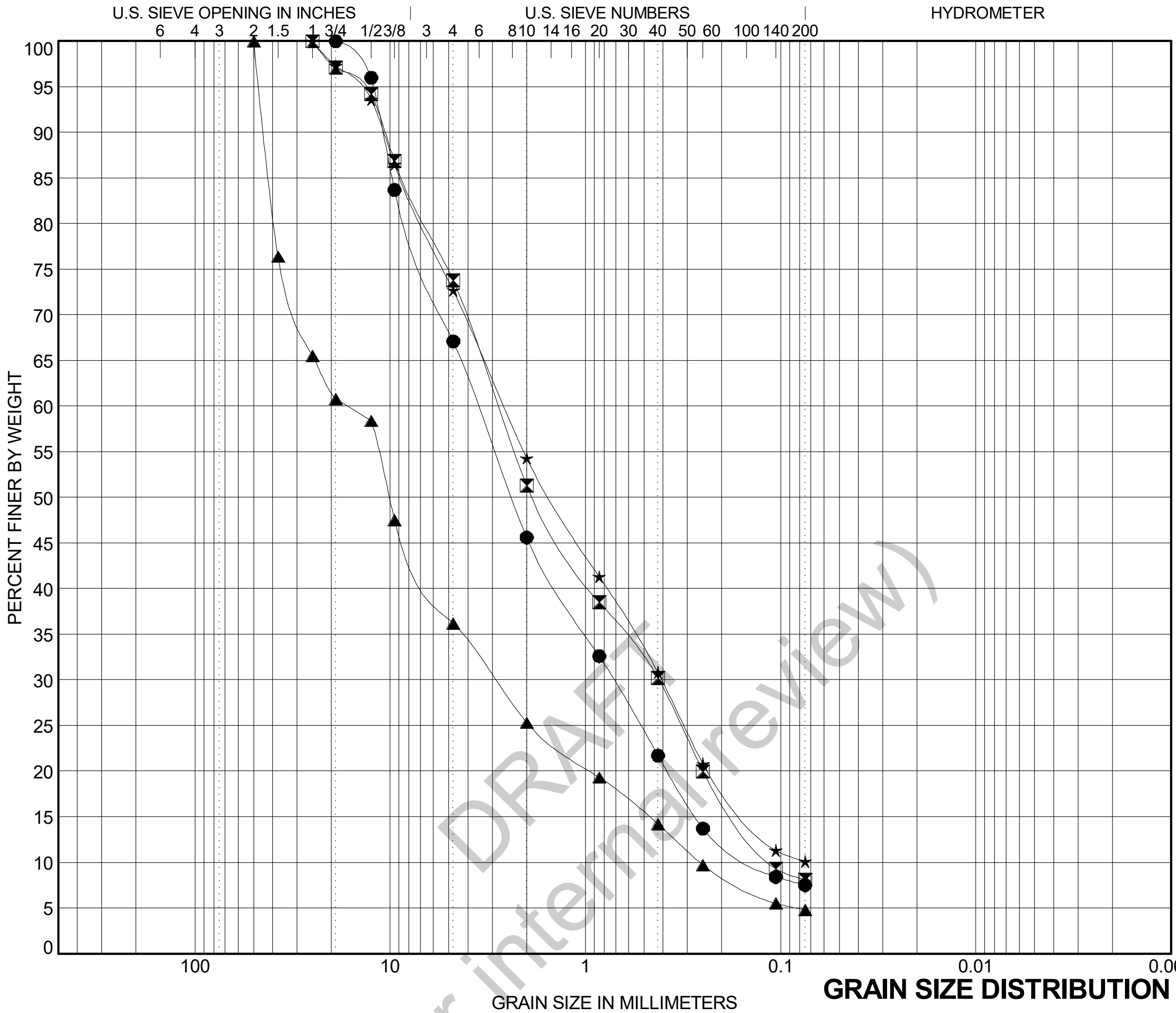
REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO: 2018-151-GEO | **PLATE NO: B-3A**



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-3**	**7**	**31.0**	**Well-graded SAND with SILT and GRAVEL**				**1.06**	**25.99**
⧗	**B-4**	**4**	**16.0**	**Poorly graded SAND with SILT and GRAVEL**				**0.56**	**24.92**
▲	**B-4**	**7**	**30.0**	**Well graded gravel with sand**				**1.97**	**63.80**
★	**B-4**	**11**	**50.5**	**Poorly graded SAND with SILT and GRAVEL**				**0.87**	**35.88**

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt / %Clay
●	**B-3**	**7**	**31.0**	**19**	**3.57**	**0.72**	**0.137**	**32.9**	**59.6**	**7.5**
⧗	**B-4**	**4**	**16.0**	**25**	**2.794**	**0.421**	**0.112**	**26.2**	**65.7**	**8.1**
▲	**B-4**	**7**	**30.0**	**50**	**16.525**	**2.904**	**0.259**	**63.8**	**31.4**	**4.8**
★	**B-4**	**11**	**50.5**	**25**	**2.615**	**0.407**		**27.3**	**62.6**	**10.1**

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Practicing in the Geosciences

REGNART CREEK TRAIL
CUPERTINO, CALIFORNIA

JOB NO: 2018-151-GEO
PLATE NO: B-3B

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

Date Reported 02/06/2019
Date Submitted 02/01/2019

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 : 2018-151-GEO Site ID : B-1 #2@6FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 78915-164978.

EVALUATION FOR SOIL CORROSION

Soil pH	7.38		
Minimum Resistivity	0.88 ohm-cm (x1000)		
Chloride	132.3 ppm	00.01323	%
Sulfate	109.3 ppm	00.01093	%

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

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

Date Reported 02/06/2019
Date Submitted 02/01/2019

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 : 2018-151-GEO Site ID : B-2 #3@11FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 78915-164979.

EVALUATION FOR SOIL CORROSION

Soil pH	6.93	
Minimum Resistivity	2.68 ohm-cm (x1000)	
Chloride	19.7 ppm	00.00197 %
Sulfate	9.2 ppm	00.00092 %

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

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

Date Reported 04/12/2019
Date Submitted 04/09/2019

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 : 2018-151-GEO Site ID : B-3 2@6.
Thank you for your business.

* For future reference to this analysis please use SUN # 79310-165635.

EVALUATION FOR SOIL CORROSION

Soil pH	7.40	
Minimum Resistivity	1.13 ohm-cm (x1000)	
Chloride	5.1 ppm	00.00051 %
Sulfate	30.6 ppm	00.00306 %

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

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

Date Reported 04/12/2019
Date Submitted 04/09/2019

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 : 2018-151-GEO Site ID : B-4 1@3.
Thank you for your business.

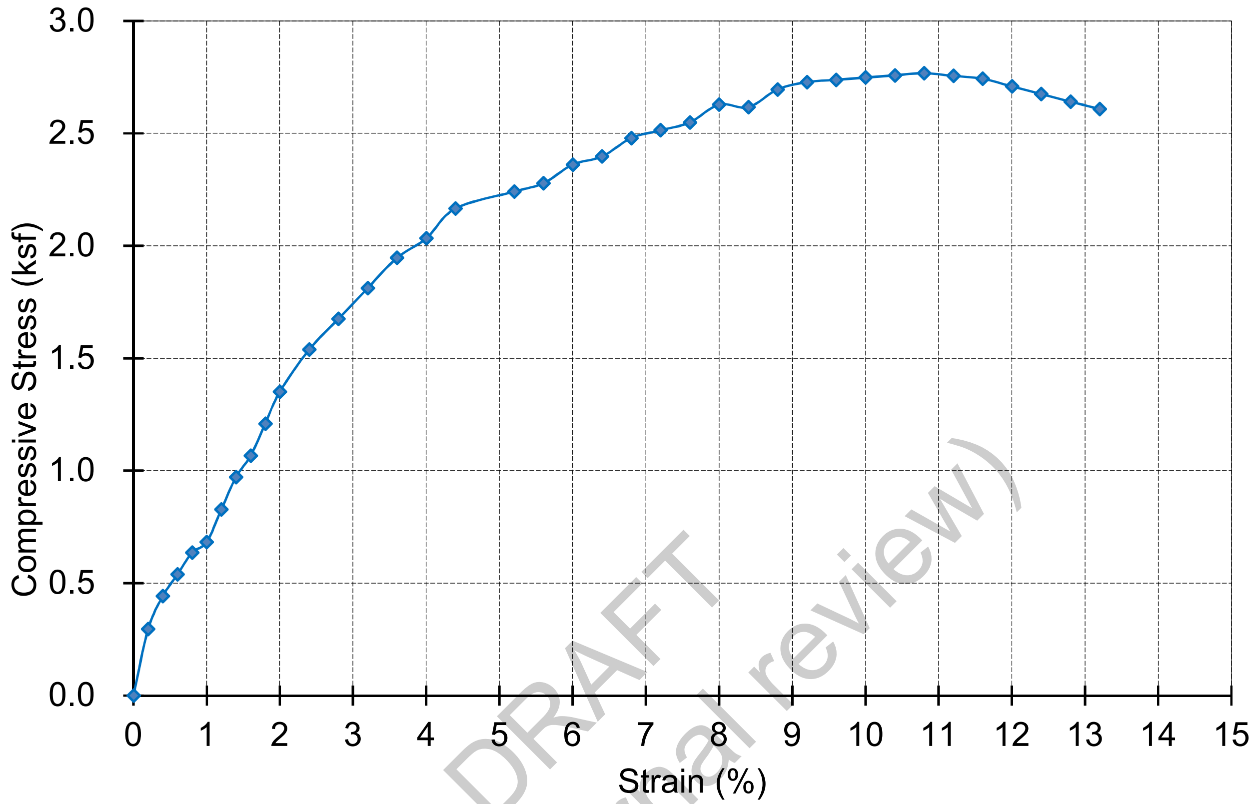
* For future reference to this analysis please use SUN # 79310-165636.

EVALUATION FOR SOIL CORROSION

Soil pH	6.66		
Minimum Resistivity	1.31 ohm-cm (x1000)		
Chloride	8.5 ppm	00.00085	%
Sulfate	43.8 ppm	00.00438	%

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

UNCONFINED COMPRESSION TEST



Boring No.:	B-2	**Unconfined Compressive Strength (ksf):**	2.77
Sample No. :	2	**Shear Strength (ksf)**	1.38
Depth (feet):	6	**Strain @ Failure (%):**	10.8
Sample Type:	MC - 2.416 inch dia.	**Initial Dry Density (pcf):**	217
Test Method	ASTM D2166	**Water Content (%):**	16.74
Material Type:	CL		
Material Description:	Lean Clay		

Initial Height (inch): 5.00
Initial Diameter (inch) 2.42
Initial Area (ft^2): 0.032
Strain Rate (inch/min) 0.1

Remarks:

PARIKH
Practicing in the Geosciences

CUPERTINO, CALIFORNIA
REGNART CREEK TRAIL

JOB NO.: 2018-151-GEO | **PLATE NO.: B-5**

Hydraulic Conductivity
ASTM D 5084
Method C: Falling Head Rising Tailwater

COOPER TESTING LABORATORY

Job No:	157-362	**Boring:**	B-2	**Date:**	02/11/19
Client:	Parikh Consultants	**Sample:**	1	**By:**	MD/PJ
Project:	Regnart Creek Trail - 2018-151-GEO	**Depth, ft.:**	1	**Remolded:**	

Visual Classification: Grayish Brown Sandy CLAY

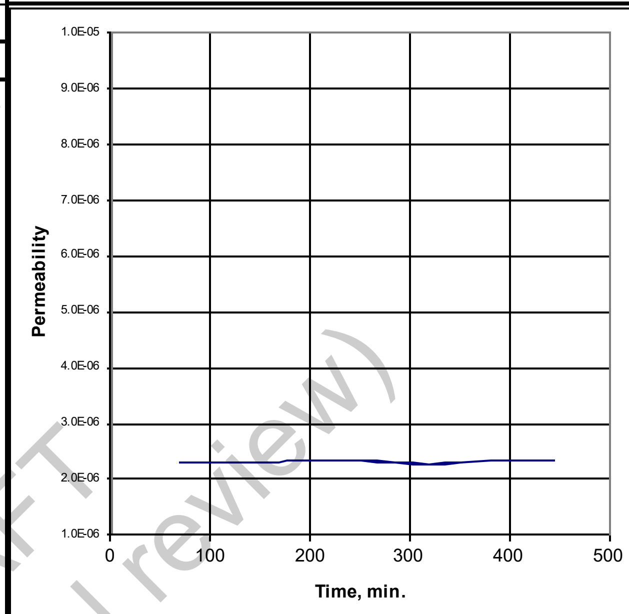
Max Sample Pressures, psi:

Cell:	Bottom	Top	Avg. Sigma3
53.5	49	48	5

B: = >0.95 **("B" is an indication of saturation)**

Max Hydraulic Gradient: = 17

Date	Minutes	Head, (in)	K,cm/sec
2/6/2019	0.00	51.69	Start of Test
2/6/2019	69.00	46.79	2.3E-06
2/6/2019	160.00	40.99	2.3E-06
2/6/2019	190.00	39.09	2.3E-06
2/6/2019	251.00	35.79	2.3E-06
2/6/2019	319.00	32.79	2.3E-06
2/6/2019	382.00	29.59	2.3E-06
2/6/2019	445.00	26.94	2.3E-06



Average Hydraulic Conductivity: **2.E-06** **cm/sec**

Sample Data:	**Initial (As-Received)**		**Final (At-Test)**
Height, in	3.02		2.98
Diameter, in	2.41		2.39
Area, in2	4.55		4.49
Volume in3	13.72		13.37
Total Volume, cc	224.8		219.1
Volume Solids, cc	129.2		129.2
Volume Voids, cc	95.6		89.9
Void Ratio	0.7		0.7
Total Porosity, %	42.5		41.0
Air-Filled Porosity (θa),%	14.1		1.7
Water-Filled Porosity (θw),%	28.5		39.3
Saturation, %	66.9		95.8
Specific Gravity	2.70	Assumed	2.70
Wet Weight, gm	412.7		434.9
Dry Weight, gm	348.7		348.7
Tare, gm	0.00		0.00
Moisture, %	18.3		24.7
Wet Bulk Density, pcf	114.6		123.9
Dry Bulk Density, pcf	96.8		99.3
Wet Bulk Dens.ρb, (g/cm^3)	1.84		1.98
Dry Bulk Dens.ρb, (g/cm^3)	1.55		1.59

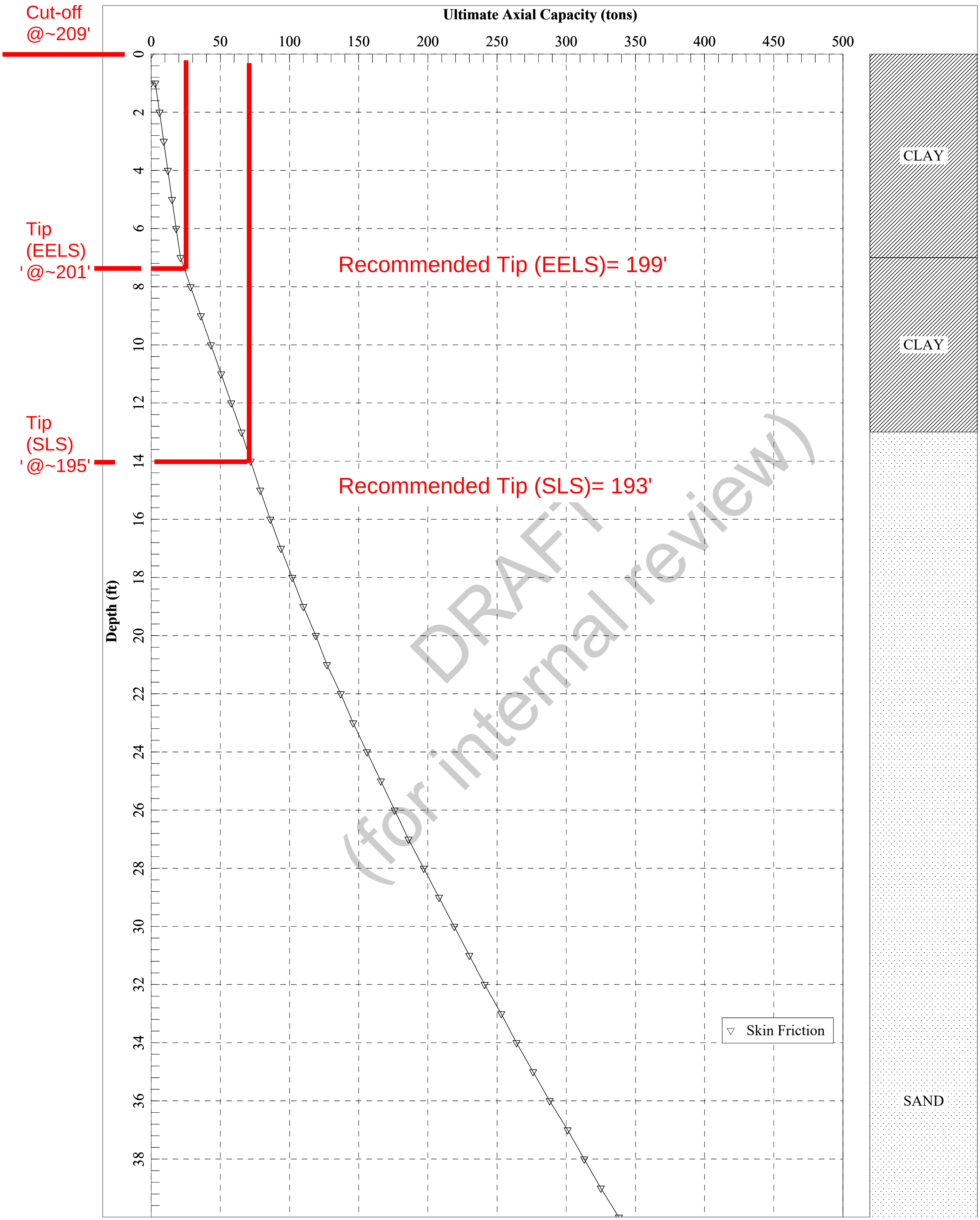
Remarks:

PLATE NO. B-6

APPENDIX C

Axial Pile Capacity Analyses

Required Nominal Resistance for Bridge 1: 98/0.7= 140 kips = 70 tons (SLS)
Required Nominal Resistance for Bridge 2: 95/0.7= 136 kips = 68 tons = ~70tons (SLS)
Required Nominal Resistance for Bridge 1: 48 kips ~= 25 tons (EELS)
Required Nominal Resistance for Bridge 2: 47 kips ~= 25 tons (EELS)



Regnart Creek Bridges - South Abutments (Abutment 1) - 30" CIDH

```
=========================================================================

                 SHAFT for Windows, Version 2017.8.9

                  Serial Number :  291911540

                VERTICALLY LOADED DRILLED SHAFT ANALYSIS
                 (c) Copyright ENSOFT, Inc., 1987-2017
                       All Rights Reserved

=========================================================================


     Path to file locations      : C:\Users\eortakci\Parikh Consultants
Inc\Projects - Ongoing_Projects\2018\2018-151 Regnart Creek Trail
Bridges\Calculations\Shaft\
     Name of input data file     : Regnart Creek_South Abutments.sf8d
     Name of output file         : Regnart Creek_South Abutments.sf8o
     Name of plot output file    : Regnart Creek_South Abutments.sf8p
     Name of runtime file        : Regnart Creek_South Abutments.sf8r


     -------------------------------------------------------------------------
                    Time and Date of Analysis
     -------------------------------------------------------------------------

               Date:  April 26, 2019     Time:  15:34:18

     New Pile


     PROPOSED DEPTH =      40.0 FT
     ----------------


     NUMBER OF LAYERS =    3
     ------------------


     WATER TABLE DEPTH =      60.0 FT.
     -------------------



     FACTOR OF SAFETY APPLIED TO THE ULTIMATE SIDE FRICTION CAPACITY = 2.50
     -------------------------------------------------------
     FACTOR OF SAFETY APPLIED TO THE ULTIMATE BASE CAPACITY = 3.00
     ------------------------------------------------------
```



```
SOIL INFORMATION
---------------

LAYER NO  1----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.600E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.140E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.000E+00


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.140E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.700E+01


LAYER NO  2----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.535E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.350E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.700E+01


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                   = 0.535E+00  (*)
  END BEARING COEFFICIENT-Nc                        = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                = 0.350E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
```

```
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.130E+02


LAYER NO  3----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                     = 0.101E+01  (*)
  INTERNAL FRICTION ANGLE, DEG.                       = 0.370E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.130E+02

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                     = 0.463E+00  (*)
  INTERNAL FRICTION ANGLE, DEG.                       = 0.370E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST       = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                          = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT            = 0.100E+11
  DEPTH, FT                                           = 0.590E+02


(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS



INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    2.500  FT.
 MAXIMUM SHAFT DIAMETER    =    2.500  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =    0.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.290E+07 LB/SQ IN
```



```
COMPUTATION RESULTS
-------------------



- CASE ANALYZED      :     1
  VARIATION LENGTH   :     1
  VARIATION DIAMETER :     1



DRILLED SHAFT INFORMATION

-------------------------

 DIAMETER OF STEM          =    2.500  FT.
 DIAMETER OF BASE          =    2.500  FT.
 END OF STEM TO BASE       =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =    0.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 AREA OF ONE PERCENT STEEL =    7.069  SQ.IN.
 ELASTIC MODULUS, Ec       = 0.290E+07 LB/SQ IN
 VOLUME OF UNDERREAM       =    0.000  CU.YDS.
 SHAFT LENGTH              =   40.000  FT.



PREDICTED RESULTS
-----------------

QS     = ULTIMATE SIDE RESISTANCE;
QB     = ULTIMATE BASE RESISTANCE;
WT     = WEIGHT OF DRILLED SHAFT (FOR UPLIFT CAPACITY ONLY);
QU     = TOTAL ULTIMATE RESISTANCE;
QBD    = TOTAL ALLOWABLE LOAD USING A FACTOR OF SAFETY
         APPLIED TO THE ULTIMATE BASE RESISTANCE;
QDN    = TOTAL ALLOWABLE LOAD USING FACTORS OF SAFETY
         APPLIED TO THE ULTIMATE SIDE RESISTANCE AND
         THE ULTIMATE BASE RESISTANCE.
```

LENGTH (FT)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	QBD (TONS)	QDN (TONS)	QU/VOLUME (TONS/CU.YDS)
1.0	0.18	3.02	28.42	31.44	12.50	10.68	172.92
2.0	0.36	6.05	29.73	35.78	15.96	12.33	98.38
3.0	0.55	9.07	43.19	52.26	23.47	18.03	95.81
4.0	0.73	12.10	58.29	70.38	31.53	24.27	96.77
5.0	0.91	15.12	71.00	86.12	38.79	29.71	94.72

6.0	1.09	18.15	77.32	95.47	43.92	33.03	87.51
7.0	1.27	21.17	77.32	98.49	46.94	34.24	77.38
8.0	1.45	28.52	77.32	105.84	54.29	37.18	72.76
9.0	1.64	35.87	70.33	106.20	59.31	37.79	64.90
10.0	1.82	43.22	63.18	106.40	64.28	38.35	58.52
11.0	2.00	50.57	58.37	108.94	70.02	39.68	54.47
12.0	2.18	57.92	58.06	115.98	77.27	42.52	53.15
13.0	2.36	65.27	61.75	127.02	85.85	46.69	53.74
14.0	2.55	71.86	65.45	137.31	93.68	50.56	53.94
15.0	2.73	78.82	69.14	147.95	101.86	54.57	54.25
16.0	2.91	86.12	72.83	158.95	110.40	58.73	54.64
17.0	3.09	93.76	76.52	170.28	119.27	63.01	55.09
18.0	3.27	101.73	80.21	181.94	128.47	67.43	55.59
19.0	3.45	110.01	83.91	193.91	137.98	71.97	56.13
20.0	3.64	118.59	87.60	206.19	147.79	76.63	56.70
21.0	3.82	127.46	90.28	217.74	157.55	81.08	57.02
22.0	4.00	136.61	91.79	228.40	167.21	85.24	57.10
23.0	4.18	146.03	92.30	238.32	176.79	89.18	56.99
24.0	4.36	155.70	92.30	248.00	186.47	93.05	56.83
25.0	4.55	165.62	92.30	257.92	196.39	97.01	56.74
26.0	4.73	175.78	92.30	268.08	206.55	101.08	56.71
27.0	4.91	186.17	92.30	278.47	216.94	105.23	56.72
28.0	5.09	196.78	92.30	289.08	227.55	109.48	56.78
29.0	5.27	207.60	92.30	299.89	238.36	113.80	56.87
30.0	5.45	218.61	92.30	310.91	249.38	118.21	57.00
31.0	5.64	229.82	92.30	322.11	260.58	122.69	57.15
32.0	5.82	241.20	92.30	333.50	271.97	127.25	57.32
33.0	6.00	252.76	92.30	345.06	283.53	131.87	57.51
34.0	6.18	264.49	92.30	356.78	295.25	136.56	57.71
35.0	6.36	276.36	92.30	368.66	307.13	141.31	57.93
36.0	6.55	288.39	92.30	380.69	319.16	146.12	58.16
37.0	6.73	300.55	92.30	392.85	331.32	150.99	58.39
38.0	6.91	312.85	92.30	405.14	343.61	155.90	58.64
39.0	7.09	325.27	92.30	417.56	356.03	160.87	58.88
40.0	7.27	337.80	92.30	430.09	368.56	165.88	59.13

AXIAL LOAD VS SETTLEMENT CURVES

RESULT FROM TREND (AVERAGED) LINE

TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.5521E-01	0.2321E-04	0.1077E-02	0.1000E-04

0.2760E+00	0.1160E-03	0.5384E-02	0.5000E-04
0.5521E+00	0.2321E-03	0.1077E-01	0.1000E-03
0.2793E+02	0.1166E-01	0.5384E+00	0.5000E-02
0.4190E+02	0.1749E-01	0.8076E+00	0.7500E-02
0.5587E+02	0.2332E-01	0.1077E+01	0.1000E-01
0.1280E+03	0.5718E-01	0.2692E+01	0.2500E-01
0.2074E+03	0.1048E+00	0.5384E+01	0.5000E-01
0.2496E+03	0.1437E+00	0.8076E+01	0.7500E-01
0.2740E+03	0.1771E+00	0.1077E+02	0.1000E+00
0.3386E+03	0.3509E+00	0.2661E+02	0.2500E+00
0.3552E+03	0.6106E+00	0.4715E+02	0.5000E+00
0.3602E+03	0.7383E+00	0.5311E+02	0.6250E+00
0.3660E+03	0.8660E+00	0.5907E+02	0.7500E+00
0.3998E+03	0.1632E+01	0.9368E+02	0.1500E+01

RESULT FROM UPPER-BOUND LINE

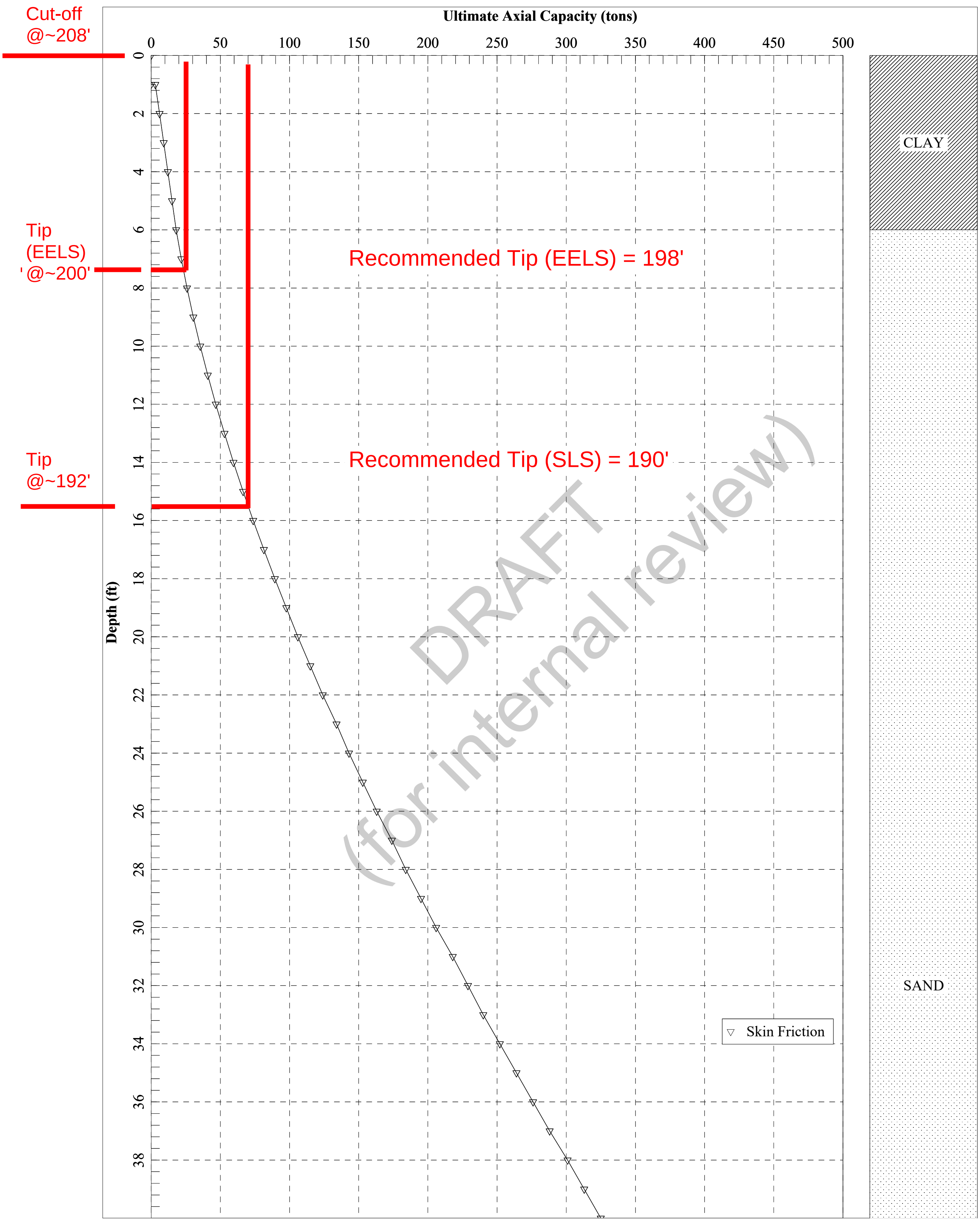
TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.8400E-01	0.2901E-04	0.1538E-02	0.1000E-04
0.4200E+00	0.1451E-03	0.7691E-02	0.5000E-04
0.8400E+00	0.2901E-03	0.1538E-01	0.1000E-03
0.4269E+02	0.1462E-01	0.7691E+00	0.5000E-02
0.6403E+02	0.2193E-01	0.1154E+01	0.7500E-02
0.8538E+02	0.2925E-01	0.1538E+01	0.1000E-01
0.1796E+03	0.6981E-01	0.3846E+01	0.2500E-01
0.2657E+03	0.1223E+00	0.7691E+01	0.5000E-01
0.3073E+03	0.1618E+00	0.1154E+02	0.7500E-01
0.3270E+03	0.1940E+00	0.1538E+02	0.1000E+00
0.3661E+03	0.3604E+00	0.3723E+02	0.2500E+00
0.3869E+03	0.6221E+00	0.6322E+02	0.5000E+00
0.3900E+03	0.7487E+00	0.6668E+02	0.6250E+00
0.3935E+03	0.8753E+00	0.7015E+02	0.7500E+00
0.4230E+03	0.1639E+01	0.9968E+02	0.1500E+01

RESULT FROM LOWER-BOUND LINE

TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.3138E-01	0.1802E-04	0.6153E-03	0.1000E-04
0.1569E+00	0.9012E-04	0.3077E-02	0.5000E-04
0.3138E+00	0.1802E-03	0.6153E-02	0.1000E-03
0.1581E+02	0.9034E-02	0.3077E+00	0.5000E-02
0.2371E+02	0.1355E-01	0.4615E+00	0.7500E-02
0.3162E+02	0.1807E-01	0.6153E+00	0.1000E-01
0.7784E+02	0.4506E-01	0.1538E+01	0.2500E-01

0.1397E+03	0.8659E-01	0.3077E+01	0.5000E-01
0.1851E+03	0.1246E+00	0.4615E+01	0.7500E-01
0.2178E+03	0.1595E+00	0.6153E+01	0.1000E+00
0.3111E+03	0.3414E+00	0.1600E+02	0.2500E+00
0.3230E+03	0.5991E+00	0.3107E+02	0.5000E+00
0.3304E+03	0.7278E+00	0.3953E+02	0.6250E+00
0.3386E+03	0.8567E+00	0.4799E+02	0.7500E+00
0.3767E+03	0.1625E+01	0.8768E+02	0.1500E+01

Required Nominal Resistance for Bridge 1: 98/0.7= 140 kips = 70 tons (SLS)
Required Nominal Resistance for Bridge 2: 95/0.7= 136 kips = 68 tons = ~70tons (SLS)
Required Nominal Resistance for Bridge 1: 48 kips ~= 25 tons (EELS)
Required Nominal Resistance for Bridge 2: 47 kips ~= 25 tons (EELS)



Regnart Creek Bridges - North Abutments (Abutment 2) - 30" CIDH

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=========================================================================

                SHAFT for Windows, Version 2017.8.9

                  Serial Number :  291911540

              VERTICALLY LOADED DRILLED SHAFT ANALYSIS
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                        All Rights Reserved

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Path to file locations      : C:\Users\eortakci\Parikh Consultants
Inc\Projects - Ongoing_Projects\2018\2018-151 Regnart Creek Trail
Bridges\Calculations\Shaft\
Name of input data file     : Regnart Creek_North Abutments.sf8d
Name of output file         : Regnart Creek_North Abutments.sf8o
Name of plot output file    : Regnart Creek_North Abutments.sf8p
Name of runtime file        : Regnart Creek_North Abutments.sf8r


-------------------------------------------------------------------------
                    Time and Date of Analysis
-------------------------------------------------------------------------

          Date:  April 26, 2019     Time:  15:39:26

New Pile


PROPOSED DEPTH =      40.0 FT
----------------


NUMBER OF LAYERS =    2
------------------


WATER TABLE DEPTH =      60.0 FT.
-------------------



FACTOR OF SAFETY APPLIED TO THE ULTIMATE SIDE FRICTION CAPACITY = 2.50
-----------------------------------------------------
FACTOR OF SAFETY APPLIED TO THE ULTIMATE BASE CAPACITY = 3.00
------------------------------------------------------
```



```
SOIL INFORMATION
---------------

LAYER NO  1----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.600E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.140E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.000E+00


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.550E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.888E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.140E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.600E+01


LAYER NO  2----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.117E+01  (*)
  INTERNAL FRICTION ANGLE, DEG.                      = 0.370E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.600E+01

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                    = 0.472E+00  (*)
  INTERNAL FRICTION ANGLE, DEG.                      = 0.370E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.125E+03
```

MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT = 0.100E+11
DEPTH, FT = 0.580E+02

(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS

INPUT DRILLED SHAFT INFORMATION

MINIMUM SHAFT DIAMETER	=	2.500	FT.
MAXIMUM SHAFT DIAMETER	=	2.500	FT.
RATIO BASE/SHAFT DIAMETER	=	0.000	FT.
ANGLE OF BELL	=	0.000	DEG.
IGNORED TOP PORTION	=	0.000	FT.
IGNORED BOTTOM PORTION	=	0.000	FT.
ELASTIC MODULUS, Ec	=	0.290E+07	LB/SQ IN

COMPUTATION RESULTS

- CASE ANALYZED : 1
  VARIATION LENGTH : 1
  VARIATION DIAMETER : 1

DRILLED SHAFT INFORMATION

DIAMETER OF STEM	=	2.500	FT.
DIAMETER OF BASE	=	2.500	FT.
END OF STEM TO BASE	=	0.000	FT.
ANGLE OF BELL	=	0.000	DEG.
IGNORED TOP PORTION	=	0.000	FT.
IGNORED BOTTOM PORTION	=	0.000	FT.
AREA OF ONE PERCENT STEEL	=	7.069	SQ.IN.
ELASTIC MODULUS, Ec	=	0.290E+07	LB/SQ IN
VOLUME OF UNDERREAM	=	0.000	CU.YDS.

SHAFT LENGTH = 40.000 FT.

PREDICTED RESULTS

QS = ULTIMATE SIDE RESISTANCE;
QB = ULTIMATE BASE RESISTANCE;
WT = WEIGHT OF DRILLED SHAFT (FOR UPLIFT CAPACITY ONLY);
QU = TOTAL ULTIMATE RESISTANCE;
QBD = TOTAL ALLOWABLE LOAD USING A FACTOR OF SAFETY APPLIED TO THE ULTIMATE BASE RESISTANCE;
QDN = TOTAL ALLOWABLE LOAD USING FACTORS OF SAFETY APPLIED TO THE ULTIMATE SIDE RESISTANCE AND THE ULTIMATE BASE RESISTANCE.

LENGTH (FT)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	QBD (TONS)	QDN (TONS)	QU/VOLUME (TONS/CU.YDS)
1.0	0.18	3.02	28.42	31.44	12.50	10.68	172.92
2.0	0.36	6.05	28.34	34.39	15.50	11.87	94.57
3.0	0.55	9.07	28.54	37.61	18.58	13.14	68.95
4.0	0.73	12.10	29.67	41.77	21.99	14.73	57.42
5.0	0.91	15.12	32.22	47.34	25.86	16.79	52.07
6.0	1.09	18.15	35.91	54.06	30.12	19.23	49.55
7.0	1.27	21.79	39.60	61.40	34.99	21.92	48.24
8.0	1.45	25.91	43.30	69.20	40.34	24.80	47.58
9.0	1.64	30.48	46.99	77.47	46.14	27.85	47.34
10.0	1.82	35.48	50.68	86.16	52.38	31.09	47.39
11.0	2.00	40.91	54.37	95.28	59.03	34.49	47.64
12.0	2.18	46.74	58.06	104.80	66.09	38.05	48.03
13.0	2.36	52.95	61.75	114.71	73.54	41.77	48.53
14.0	2.55	59.55	65.45	124.99	81.36	45.63	49.10
15.0	2.73	66.50	69.14	135.64	89.55	49.65	49.73
16.0	2.91	73.81	72.83	146.64	98.09	53.80	50.40
17.0	3.09	81.45	76.52	157.97	106.96	58.09	51.11
18.0	3.27	89.42	80.21	169.63	116.15	62.50	51.83
19.0	3.45	97.70	83.91	181.60	125.66	67.05	52.57
20.0	3.64	106.28	87.60	193.87	135.48	71.71	53.31
21.0	3.82	115.15	90.28	205.43	145.24	76.15	53.80
22.0	4.00	124.30	91.79	216.09	154.89	80.32	54.02
23.0	4.18	133.71	92.30	226.01	164.48	84.25	54.04
24.0	4.36	143.39	92.30	235.68	174.15	88.12	54.01
25.0	4.55	153.31	92.30	245.61	184.08	92.09	54.03
26.0	4.73	163.47	92.30	255.77	194.24	96.15	54.10
27.0	4.91	173.86	92.30	266.16	204.63	100.31	54.21
28.0	5.09	184.47	92.30	276.76	215.23	104.55	54.36
29.0	5.27	195.28	92.30	287.58	226.05	108.88	54.54

30.0	5.45	206.30	92.30	298.59	237.06	113.28	54.74
31.0	5.64	217.50	92.30	309.80	248.27	117.77	54.96
32.0	5.82	228.89	92.30	321.19	259.66	122.32	55.20
33.0	6.00	240.45	92.30	332.75	271.22	126.95	55.45
34.0	6.18	252.17	92.30	344.47	282.94	131.63	55.72
35.0	6.36	264.05	92.30	356.35	294.82	136.39	55.99
36.0	6.55	276.08	92.30	368.37	306.84	141.20	56.28
37.0	6.73	288.24	92.30	380.54	319.01	146.06	56.56
38.0	6.91	300.54	92.30	392.83	331.30	150.98	56.85
39.0	7.09	312.95	92.30	405.25	343.72	155.95	57.15
40.0	7.27	325.48	92.30	417.78	356.25	160.96	57.44

AXIAL LOAD VS SETTLEMENT CURVES

RESULT FROM TREND (AVERAGED) LINE

TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.4745E-01	0.2225E-04	0.1077E-02	0.1000E-04
0.2373E+00	0.1112E-03	0.5384E-02	0.5000E-04
0.4745E+00	0.2225E-03	0.1077E-01	0.1000E-03
0.2398E+02	0.1117E-01	0.5384E+00	0.5000E-02
0.3597E+02	0.1676E-01	0.8076E+00	0.7500E-02
0.4796E+02	0.2234E-01	0.1077E+01	0.1000E-01
0.1136E+03	0.5533E-01	0.2692E+01	0.2500E-01
0.1890E+03	0.1024E+00	0.5384E+01	0.5000E-01
0.2315E+03	0.1412E+00	0.8076E+01	0.7500E-01
0.2575E+03	0.1748E+00	0.1077E+02	0.1000E+00
0.3250E+03	0.3489E+00	0.2661E+02	0.2500E+00
0.3456E+03	0.6091E+00	0.4715E+02	0.5000E+00
0.3513E+03	0.7368E+00	0.5311E+02	0.6250E+00
0.3571E+03	0.8645E+00	0.5907E+02	0.7500E+00
0.3909E+03	0.1630E+01	0.9368E+02	0.1500E+01

RESULT FROM UPPER-BOUND LINE

TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.6999E-01	0.2732E-04	0.1538E-02	0.1000E-04
0.3499E+00	0.1366E-03	0.7691E-02	0.5000E-04
0.6999E+00	0.2732E-03	0.1538E-01	0.1000E-03
0.3550E+02	0.1375E-01	0.7691E+00	0.5000E-02

0.5325E+02	0.2063E-01	0.1154E+01	0.7500E-02
0.7101E+02	0.2751E-01	0.1538E+01	0.1000E-01
0.1599E+03	0.6726E-01	0.3846E+01	0.2500E-01
0.2487E+03	0.1199E+00	0.7691E+01	0.5000E-01
0.2907E+03	0.1594E+00	0.1154E+02	0.7500E-01
0.3107E+03	0.1917E+00	0.1538E+02	0.1000E+00
0.3505E+03	0.3581E+00	0.3723E+02	0.2500E+00
0.3750E+03	0.6203E+00	0.6322E+02	0.5000E+00
0.3785E+03	0.7469E+00	0.6668E+02	0.6250E+00
0.3819E+03	0.8735E+00	0.7015E+02	0.7500E+00
0.4115E+03	0.1637E+01	0.9968E+02	0.1500E+01

RESULT FROM LOWER-BOUND LINE

TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.2814E-01	0.1761E-04	0.6153E-03	0.1000E-04
0.1407E+00	0.8804E-04	0.3077E-02	0.5000E-04
0.2814E+00	0.1761E-03	0.6153E-02	0.1000E-03
0.1416E+02	0.8823E-02	0.3077E+00	0.5000E-02
0.2125E+02	0.1323E-01	0.4615E+00	0.7500E-02
0.2833E+02	0.1765E-01	0.6153E+00	0.1000E-01
0.6999E+02	0.4404E-01	0.1538E+01	0.2500E-01
0.1264E+03	0.8485E-01	0.3077E+01	0.5000E-01
0.1692E+03	0.1225E+00	0.4615E+01	0.7500E-01
0.2018E+03	0.1574E+00	0.6153E+01	0.1000E+00
0.2993E+03	0.3396E+00	0.1600E+02	0.2500E+00
0.3162E+03	0.5979E+00	0.3107E+02	0.5000E+00
0.3242E+03	0.7267E+00	0.3953E+02	0.6250E+00
0.3324E+03	0.8556E+00	0.4799E+02	0.7500E+00
0.3703E+03	0.1624E+01	0.8768E+02	0.1500E+01

Lateral Soil Pressures

Rankine Active Lateral Pressure Coefficient (K_a)

Project Name/Number: Regnart Creek **By:** EO
Structure Name/Number: Abutments **Date:** 4/17/2019

Parameters	Angle in degrees	Angle in radians	
ϕ	34	0.593	(Friction Angle of Soil)
β	0	0.000	(Backfill angle with horizontal)

K_a	0.283

$$K_a = \frac{\cos\beta - \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}{\cos\beta + \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}$$

M-O Seismic Active Lateral Pressure Coefficient (K_{AE})

Project Name/Number: Regnart Creek **By:** EO
Structure Name/Number: Abutments **Date:** 4/17/2019

Parameters	Angle in degrees	Angle in Radians	
ϕ	34	0.593	(Friction Angle of Soil)
i	0	0.000	(Backfill angle with horizontal)
β	0	0.000	(Wall backface angle **with vertical**)
δ	22.78	0.398	(Friction Angle between Soil and the backface of the wall)
kh (no unit)	0.35		
kv (no unit)	0		
θ_{MO} (rad)		0.337	

ΔKae=0.57-0.283 = 0.287
=0.287*125~=36 pcf EFP

K_{ae}	0.57

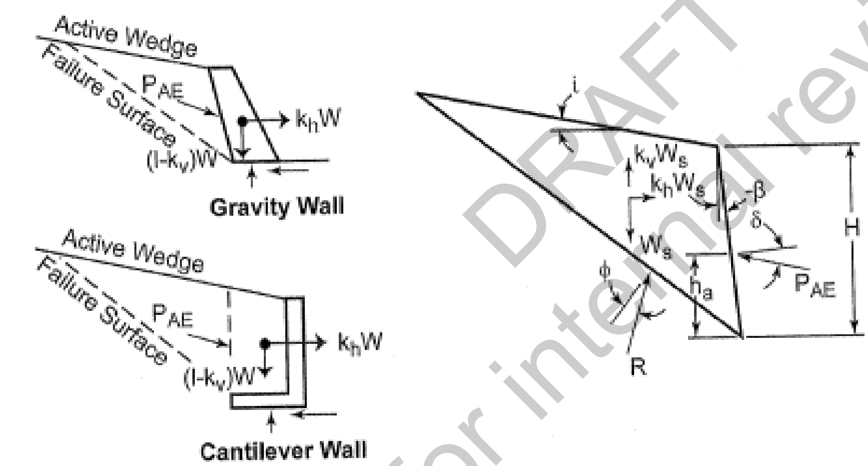


Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams

$$K_{AE} = \frac{\cos^2(\phi - \theta_{MO} - \beta)}{\cos\theta_{MO}\cos^2\beta\cos(\delta + \beta + \theta_{MO})} \times \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta_{MO} - i)}{\cos(\delta + \beta + \theta_{MO})\cos(i - \beta)}}\right]^{-2} \quad \text{(A11.3.1-1)}$$

where:

K_{AE} = seismic active earth pressure coefficient (dim)
γ = unit weight of soil (kcf)
H = height of wall (ft)
h = height of wall at back of wall heel considering height of sloping surcharge, if present (ft)
ϕ_f = friction angle of soil (degrees)
θ_{MO} = arc tan $[k_h/(1 - k_v)]$ (degrees)
δ = wall backfill interface friction angle (degrees)
k_h = horizontal seismic acceleration coefficient (dim.)
k_v = vertical seismic acceleration coefficient (dim.)
i = backfill slope angle (degrees)
β = slope of wall to the vertical, negative as shown (degrees)

Rankine Active Lateral Pressure Coefficient (K_a)

Project Name/Number: Regnart Creek **By:** EO
Structure Name/Number: Retaining Wall and Railing **Date:** 4/17/2019

Parameters	Angle in degrees	Angle in radians	
ϕ	28	0.489	(Friction Angle of Soil)
β	0	0.000	(Backfill angle with horizontal)

K_a	0.361

$$K_a = \frac{\cos\beta - \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}{\cos\beta + \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}$$

M-O Seismic Active Lateral Pressure Coefficient (K_{AE})

Project Name/Number: Regnart Creek **By:** EO
Structure Name/Number: Retaining Wall and Railing **Date:** 4/17/2019

Parameters	Angle in degrees	Angle in Radians	
ϕ	28	0.489	(Friction Angle of Soil)
i	0	0.000	(Backfill angle with horizontal)
β	0	0.000	(Wall backface angle **with vertical**)
δ	18.76	0.327	(Friction Angle between Soil and the backface of the wall)
kh (no unit)	0.35		
kv (no unit)	0		
θ_{MO} (rad)		0.337	

ΔKae=0.70-0.361 = 0.339
=0.339*125~=43 pcf EFP

K_{ae}	0.70

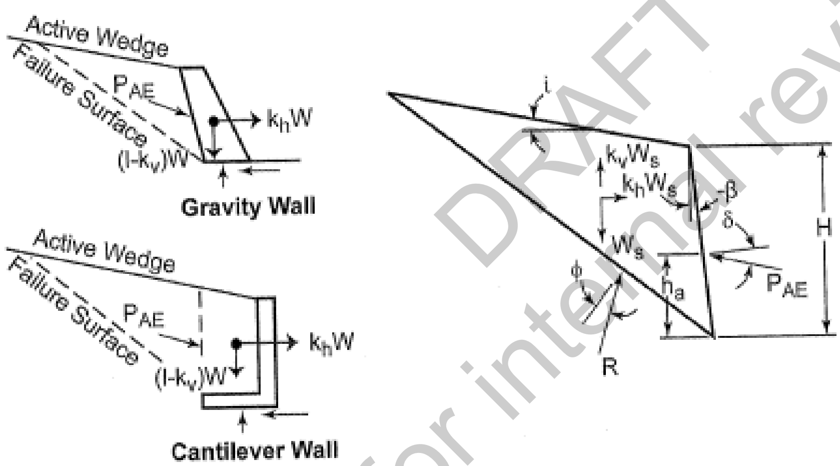


Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams

$$K_{AE} = \frac{\cos^2(\phi-\theta_{MO}-B)}{\cos\theta_{MO}\cos^2\beta\cos(\delta+\beta+\theta_{MO})}\times\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\theta_{MO}-i)}{\cos(\delta+\beta+\theta_{MO})\cos(i-\beta)}}\right]^{-2} \quad \text{(A11.3.1-1)}$$

where:

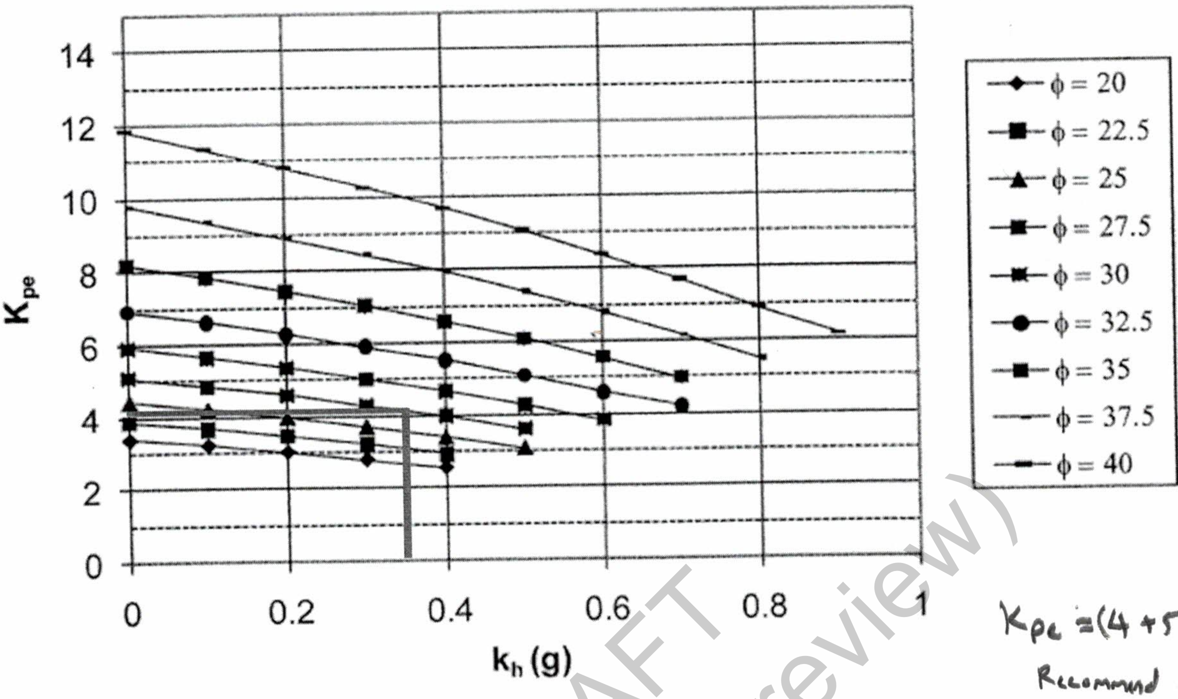
K_{AE} = seismic active earth pressure coefficient (dim)
γ = unit weight of soil (kcf)
H = height of wall (ft)
h = height of wall at back of wall heel considering height of sloping surcharge, if present (ft)
ϕ_f = friction angle of soil (degrees)
θ_{MO} = arc tan $[k_h/(1-k_v)]$ (degrees)
δ = wall backfill interface friction angle (degrees)
k_h = horizontal seismic acceleration coefficient (dim.)
k_v = vertical seismic acceleration coefficient (dim.)
i = backfill slope angle (degrees)
β = slope of wall to the vertical, negative as shown (degrees)

5/7/19

kh = 0.35, For c = 150 psf, H = 8', γ = 125 pcf

c/γH = 0.15, φ = 28°

c/γH = 0.1

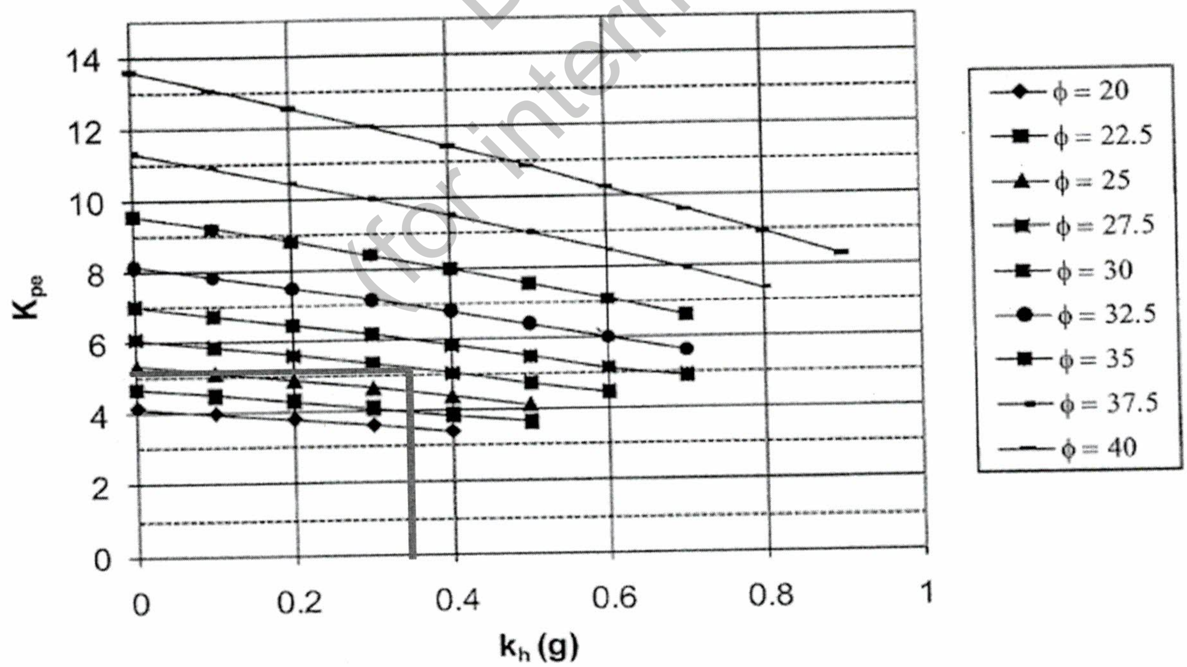


$K_{pe} = (4+5)/2 = 4.5$

Recommend 4.0 //

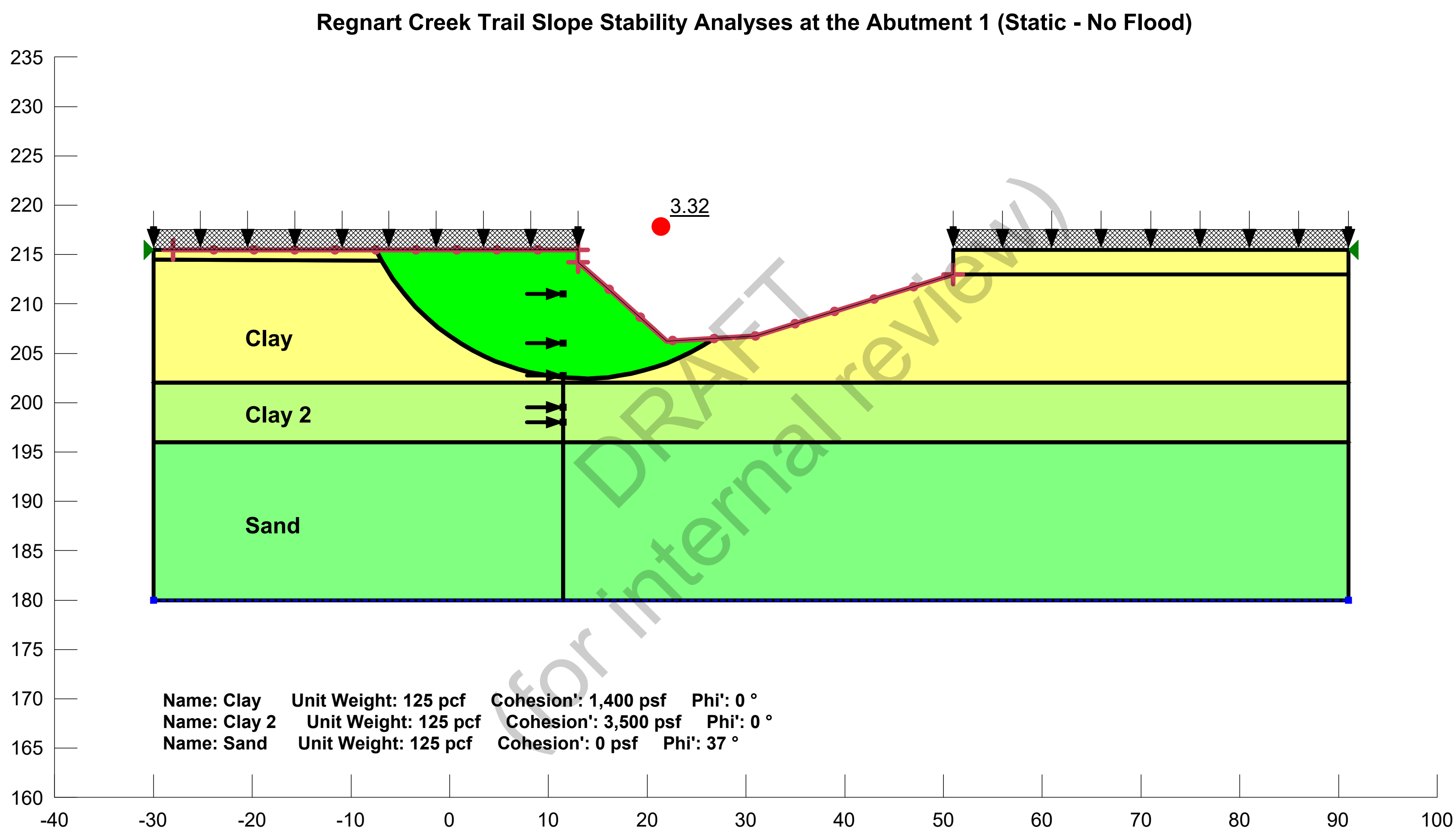
$K_p (k_h = 0) = (5+6)/2 = 5.5$ //

c/γH = 0.2



Regnart Creek - Passive pressure for wall and railing

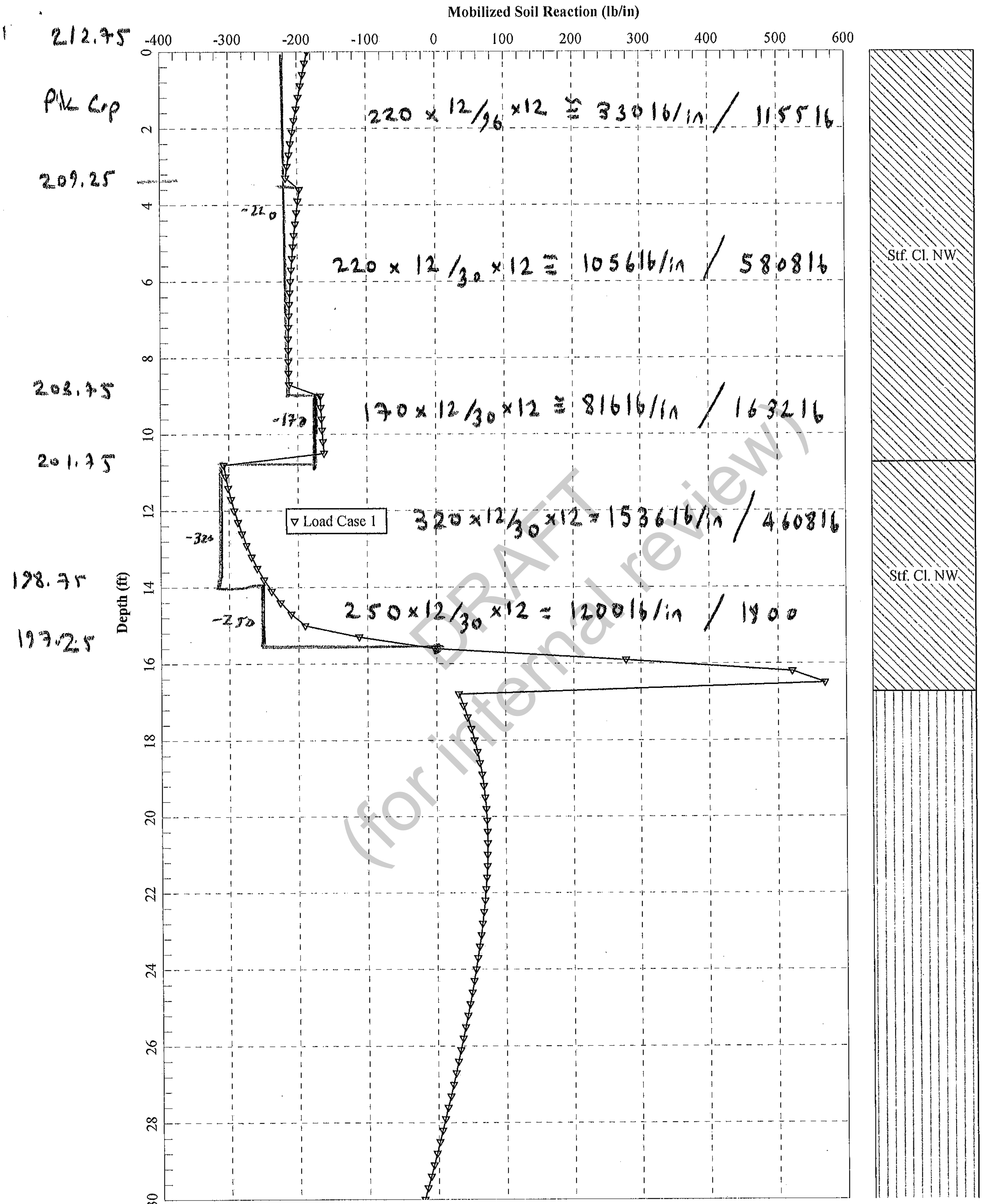
Slope Stability Analysis



Regnart Creek Trail Slope Stability Analyses at the Abutment 1 (Static - No Flood)
3.32
Clay
Clay 2
Sand
Name: Clay Unit Weight: 125 pcf Cohesion': 1,400 psf Phi': 0 °
Name: Clay 2 Unit Weight: 125 pcf Cohesion': 3,500 psf Phi': 0 °
Name: Sand Unit Weight: 125 pcf Cohesion': 0 psf Phi': 37 °
235
230
225
220
215
210
205
200
195
190
185
180
175
170
165
160
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80
90
100

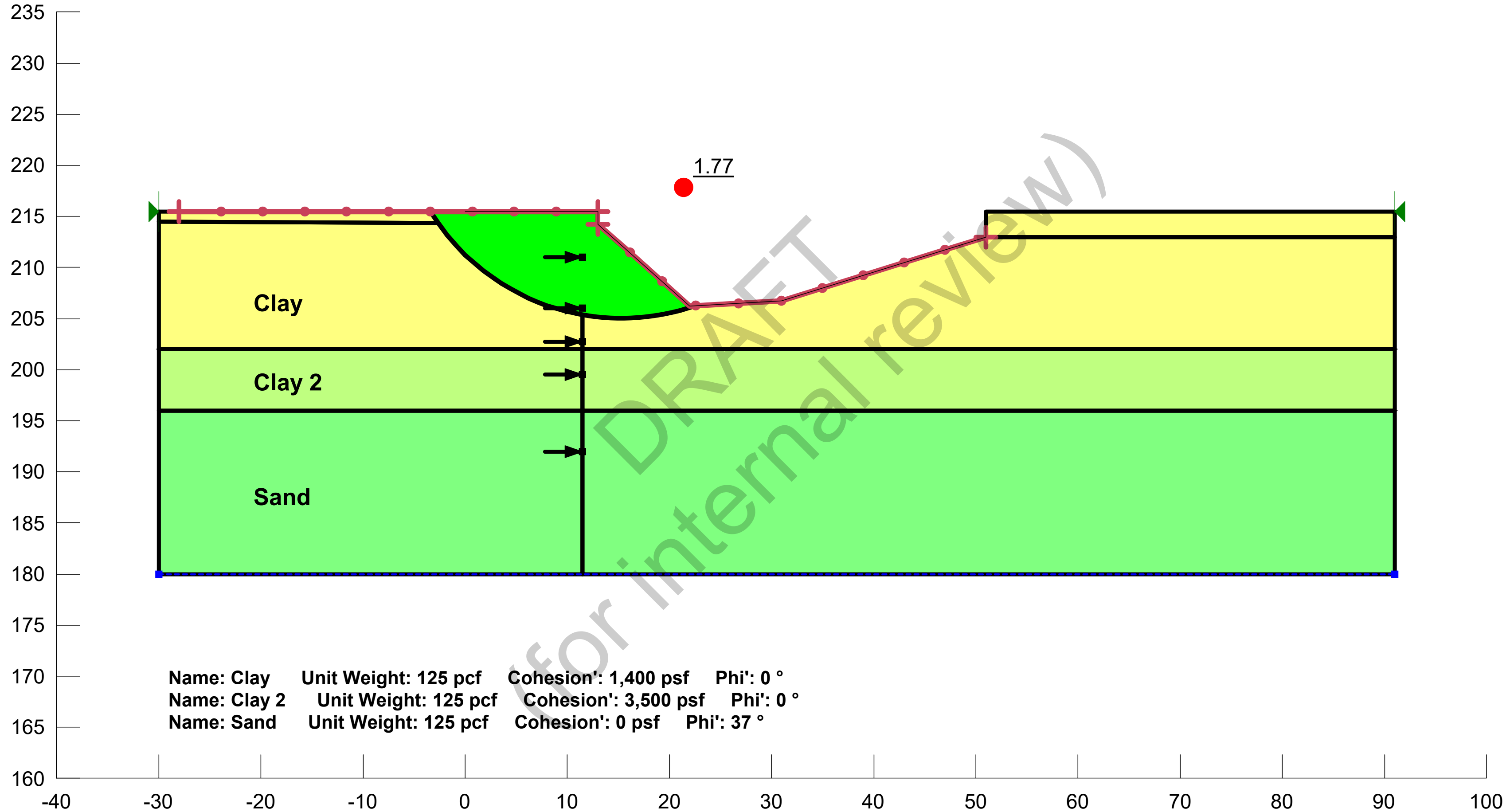
Static - Sloped Ground - 36°

5/6/19

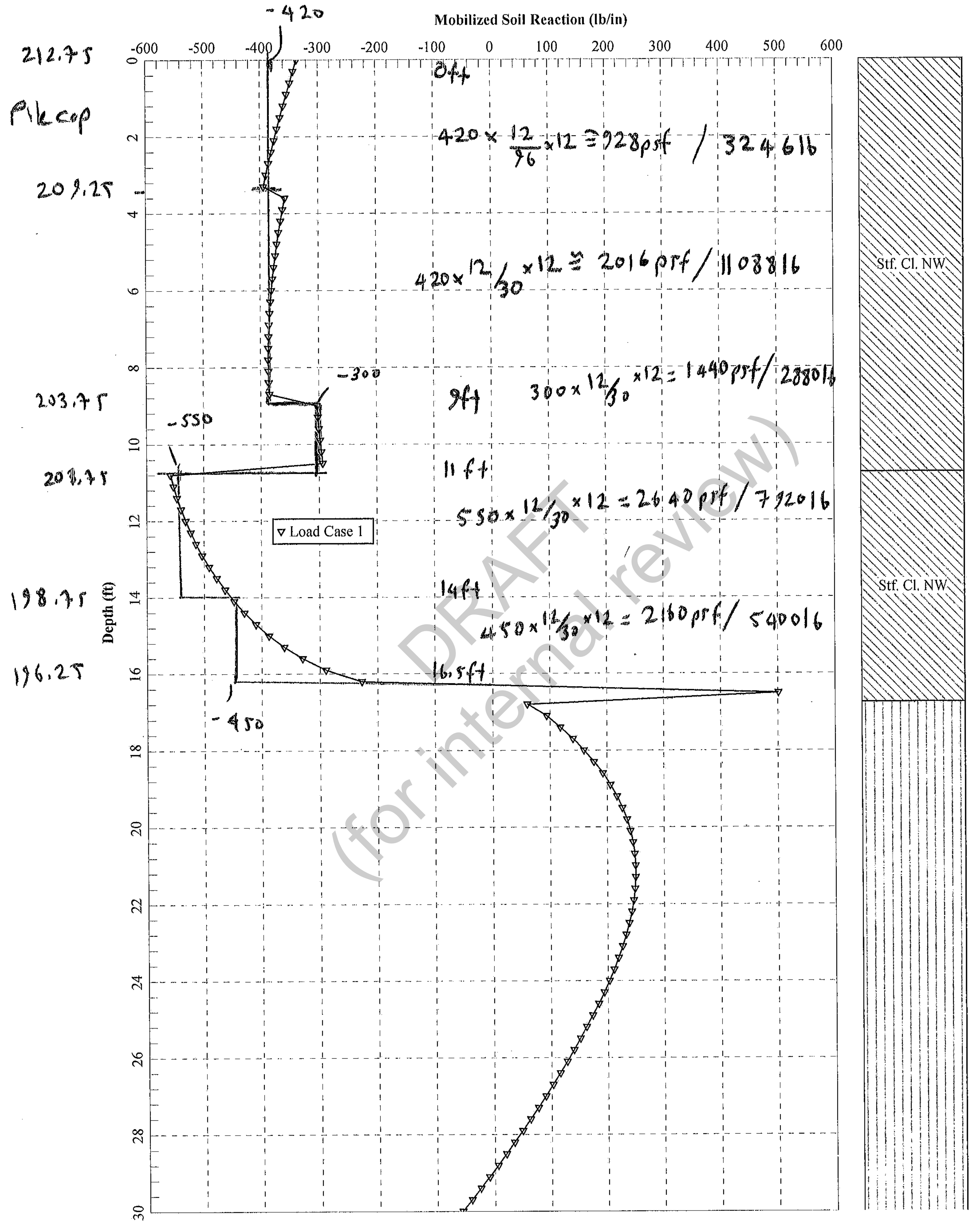


Reynart Creek - 30" CIDH - Abut 1

Regnart Creek Trail Slope Stability Analyses at the Abutment 1 (Pseudo-static kh=0.35)



Seismic - Sloped Ground - 36°



Reynort creek - 30" CIDH - Abut 1