APPENDIX C:
GEOTECHNICAL REPORT

Geotechnical Report

For the New Development at

22690 Stevens Creek
Cupertino, CA 95014
APN#342-14-104

Prepared for

Mr. Ali Mozaffari
Alan Enterprise LLC

By

Achievement Engineering Corp.
2455 Autumnvale Drive, Unit E
San Jose, California, 95131

Project Number: 4134
Date: March 03, 2020

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Mr. Ali Mozaffari
22690 Stevens Creek
Cupertino, CA 95014

Subject: Geotechnical Report for the New Development at
22690 Stevens Creek
Cupertino, CA 95014
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Dear Sir,

Achievement Engineering Corp. (AEC) is pleased to submit this Geotechnical Report for the above-referenced project. The purpose of this study was to evaluate the subsurface soil conditions at the proposed site and develop recommendations for the design and construction of the structure foundations.

We appreciate the opportunity to be of service to you on this project and would be happy to discuss our findings with you. We look forward to serving as your geotechnical/ environmental engineer on the future projects.

Respectfully Submitted,
Achievement Engineering Corp.

Sadaf M. Safaai, PE
Project Engineer

Copies: Mr. Ali Mozaffari, Alan Enterprise LLC

Table of Contents

1- Introduction

Achievement Engineering Corp. (AEC) has performed a geotechnical investigation at 22690 Stevens Creek, Cupertino, CA 95014. This report discusses the findings of the geotechnical investigation program, including the site soils and groundwater presence, and presents recommendations for the design and construction of the foundation of the structure.

The objective of this report is to evaluate the characteristics of the subsurface strata and to obtain geotechnical parameters for the design of the foundation.

The following report highlights the significant findings and conclusions representing our best professional judgment based on information and data available to us during the course of this investigation.

1-1- Project Description

The purpose of this study was to investigate the subsurface soil and groundwater presence at the site of 22690 Stevens Creek, Cupertino, CA 95014 and to develop foundation design recommendations for the project based on our evaluation of subsurface conditions. In addition, comments and recommendations related to foundation are provided in this report. Other geotechnical aspects of the project design, including lateral earth pressures, drainage and backfill requirements, are also discussed.

The Site is located at 22690 Stevens Creek, Cupertino, CA 95014, with coordinates of 37° 19' 18.32" N and 122° 04' 8.12" W.

The vicinity map of the project is illustrated in Exhibit III. The Site Location in Topographic Map and Landslide Map have also been presented in Exhibit III of the report showing the subject site is located on Class 0 – No Susceptibility Landslide Zone (Source USGS).

1-2- Geologic Setting and Faults

1-2-1 Regional Geology

The project site is located within the Coast Range Geomorphic Province. Local uplift of the Santa Cruz Mountains within the last 2 to 3 million years has occurred due to a restraining bend of the San Andreas Fault, producing transpressional forces across the plate boundary. Thrust faults bound the San Andreas Fault, are responsible for uplift of the range. The range is characterized by rugged hills with moderate relief, steep valleys, and locally steep hillsides abutting drainages. East-flowing drainages result in dissection of the mountain range and alluvial deposition within the San Francisco Bay structural trough.

The site is underlain by surficial sediments (Qoa/Qt), older surficial sediments (age; late Pleistocene) older alluvial terrace gravel; sand and clay, un-deformed.

The Site Location on 7.5' Series Geologic Map by USGS, has been represented in Exhibit III of the report.

1-2-2- Faults

Fault activity map of California (CGS, 2010) shows that there are some faults around the site location (Exhibit III). Among the eight faults of Monte Vista-Shannon, Berrocal, Cascade, Stanford, San Andreas San Jose, Pulgas and Butano, the nearest one to the site location is Monte Vista-Shannon fault with a distance of 0.5 mile (Exhibit III) and the most major one is San Andreas with a distance of 4.7 miles.

The project site is located on the north of Monte Vista-Shannon (0.5 mi.), northeast of Berrocal Fault (1.91 mi.), northwest of Cascade Fault (2.13), southwest of Stanford Fault (4.0 mi.), northeast of San Andreas fault (4.7 mi.), southwest of San Jose (6.32 mi.) , southeast of Pulgas (7.29 mi.) and northeast of Butano Fault (8.45 mi).

The Monte Vista Shannon Fault is a potentially active fault. It is a relatively short fault that runs between and generally parallel to the much longer San Andreas Fault and Hayward Fault Zones, trending northwest along the eastern foothills of the Santa Cruz Mountains in the Coast Range Geomorphic Province. The most recent activity has been estimated to have been approximately 700,000 years ago. It has a slip rate of 0.4 mm/year. The fault runs through the campus of the Foothill College.

The Berrocal is a late Quaternary southwest-dipping, reverse-dextral oblique slip fault zone that forms a part of what has been referred to as the Southwestern Santa Clara Valley thrust belt. The Berrocal fault zone, which is commonly associated with the Monte Vista-Shannon fault zone, offsets sediment of the Pliocene-Pleistocene Santa Clara Formation and probably deforms late Pleistocene fluvial and alluvial fan deposits. It has been concluded that the Berrocal fault zone lacks evidence of Holocene displacement. Late Quaternary slip rate is poorly constrained and the recurrence interval is not known. The amount of uplift of late Pleistocene terraces (about 250 ka) of ancestral Los Gatos Creek suggests a post-250 ka incision rate of 0.6 mm/yr.

The Cascade fault is a potentially active fault. It is a relatively short fault that stretches from City of Los Gatos to City of Los Altos in Southern Bay Area. This fault is an undifferentiated Quaternary possibly active in Late Quaternary or Holocene, reverse to reverse-dextral oblique slip fault that forms a part of what McLaughlin et al. (1996) refer to as the Southwestern Santa Clara Valley thrust belt, which is located generally along the foothills of the northeastern Santa Cruz Mountains. Slip rates for the Cascade fault is still unknown, although Hitchcock and Kelson (1999) determined a 0.2±0.05 mm/yr incision rate of Regnart Creek across the trace of the Cascade fault.

Stanford and Pulgas Faults are Quaternary fault with undifferentiated ages. The San Andreas is the best-known and largest fault system in North America. This fault trends in a northwesterly direction for nearly 780 miles through much of western California. It is a transform boundary separating two crustal plates that move very slowly. The Pacific plate located at the west, moves northwestward relative to the North America plate, causing earthquakes along the faults. The slip rate for this fault is up to 1.5 in./year.

The San Jose fault dips steeply to the north. Type of Faulting is left-lateral strike-slip; minor reverse component possible with a length of 18 km, close to Claremont, La Verne and Pomona. Its last Significant Quake was Feb. 28, 1990 (ML 5.4). Its most recent surface rupture was Late Quaternary. It has a slip rate between 0.2 and 2.0 mm/yr with probable magnitude of ML 5 to 6.

The San Andreas is the best-known and largest fault system in North America. This fault trends in a northwesterly direction for nearly 780 miles through much of western California. It is a transform boundary separating two crustal plates that move very slowly. The Pacific plate located at the west, moves northwestward relative to the North America plate, causing earthquakes along the faults. The slip rate for this fault is up to 1.5 in./year.

The Butano Fault extends for 46 km from San Gregorio to the San Andreas Fault; it exhibits right lateral motion, at slip rate of less than 0.2 mm/yr. (Quaternary Fault and Fold Database of the United States).

2- Project Investigation

A subsurface exploration program consisting of two test borings was conducted on 2 February 2020 under the supervision of AEC.

2-1- Field Investigation and Exploratory Boreholes

The test borings were drilled up to depths of 7 and 8 ft. below the ground surface. Borings were advanced using 3 $_{1/2}$” diameter hollow stem augers. Borings were terminated at these depths due to refusal. Table 1 shows the specifications of the boreholes; the boreholes location is shown in Exhibit III. Boreholes log is also presented in Exhibit I of the report.

Table 1- Specifications of the borehole

Borehole Name	Depth (ft.)	Diameter (inch)
B1	7	3 1/2”
B2	8	3 1/2”

2-1-1- Ground Water Table

According to the boreholes log, no water table has been encountered in borehole up to depth of 8 ft.

2-1-2- Standard Penetration Test (SPT) (ASTM: D1586)

Soil samples were typically recovered continuously at 1-2 ft. intervals by driving a standard split-spoon sampler ((1-3/8 in). I.D., (2 in.) O.D., a distance of 18 inches or 24 inches into the undisturbed soil under the impact of a 140 lb. hammer free-falling 30 inches. The number of blows required to advance the sampler through each 6 in. interval was recorded. The “N” value is taken as the number of blows required to advance the sampler the last 12 in. of the 18-in. sampling range. When the split-spoon sampler was advanced over 24-in. range, the “N” value is the number of blows required to drive the sampler the middle 12 in. Variations of SPT versus depth, in different boreholes, are presented in Figure 1 and Table 2.

Table 2-The value of SPT versus depth in borehole

	Depth (ft)	N_{spt}
B1	2	>50
	5	>50
B2	2	15
	5	>50

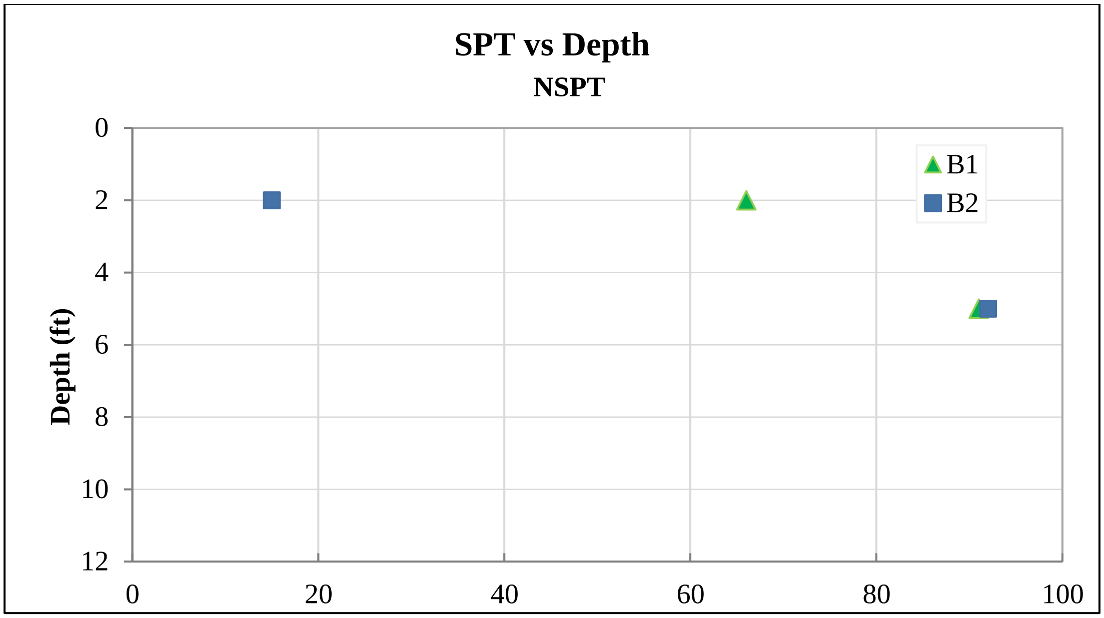

Figure 1-Variation of SPT versus depth in different boreholes

According to SPT test results, the SPT value is more than 50 in B1 which is due to a very dense layer of clayey sand, also the SPT values are 15 near the ground surface in B2 and are more than 50 in other depths, this shows existence of a firm layer of clay near ground at B2, but eventually that the consistency will change to hard.

According to the US Army Corps of Engineers, ENGINEER MANUAL ENGINEERING AND DESIGN, Geotechnical Investigations, the descriptive consistency of fine-grained soils may be classified as "very firm" to "hard" per SPT correlation and "very dense" for coarse-grained soils.

Table 3- Granular soils classification based on SPT number (US Army Corps of Engineers Manual)

Density of Coarse-Grained Soils

Descriptive Term	Blows per Foot[1,2]	Field Test
Very loose	Less than 4	----------
Loose	4-10	Easily penetrated with a 13-mm- (1/2-in.-) diam reinforcing rod pushed by hand
Medium dense	10-30	Easily penetrated with a 13-mm- (1/2-in.-) diam reinforcing rod driven with a 2.3-kg (5-lb) hammer
Dense rod	30-50	Penetrated 0.3 m (1 ft) with a 13-mm- (1/2-in.-) diam reinforcing driven with a 2.3-kg (5-lb) hammer
Very dense	Greater than 50	Penetrated only a few centimeters with a 13-mm- (1/2-in.-) diam reinforcing rod driven with a 2.3-kg (5-lb) hammer

Table 4- Strength of fine-grained soils (US Army Corps of Engineers Manual)

Descriptive Term	Blows[1] per Foot[2]	Unconfined Compressive Strength kPa	(tsf)	Field Test
Very soft	< 2	<25	(< 0.25)	Core (height twice diameter) sags under its own weight while standing on end; squeezes between fingers when fist is closed
Soft	2-4	25-50	(0.25-0.5)	Easily molded by fingers
Medium	4-8	50-100	(0.5-1.0)	Molded by strong pressure of fingers
Firm	8-15	100-190	(1.0-2.0)	Imprinted very slightly by finger pressure
Very firm	15-30	190-380	(2.0-4.0)	Cannot be imprinted with finger pressure; can be penetrated with a pencil
Hard	> 30	> 380	(> 4.0)	Imprinted only slightly by pencil point

2-2- Laboratory Test Results

A laboratory soil testing program was performed to determine soil classification and for correlation of engineering properties. Laboratory tests were performed on selected samples of the soils. Testing consisted of geotechnical index tests including water content and density determinations and grain size distributions and Atterberg Limits. The results of these tests have been used to estimate the main parameters required for designing of the foundation, such as internal friction angle and cohesion. The details of Lab tests are presented in Exhibit II.

2-2-1- Grain Size Analysis

Particle size analysis ASTM (D421-85(02)), (D422-63(02))

Atterberg limits (AASHTO T89 and T90 – ASTM D4318)

The particle size analysis is conducted on the selected soil samples in accordance with the abovementioned standards.

According to particle distribution results, soil classification is determined in compliance with the Unified Soil Classification System (USCS) (ASTM D2487 and ASTM D2488) and is recorded on the borehole log. Grain size distribution tests results are presented in Table 5. According to grain size distribution tests results, alluvial part of the site is categorized mainly as clay of low plasticity and clayey sand.

Table 5- Grain size distribution tests results

Borehole No.	Sample Depth (ft.)	Graining (%)			Atterberg Limits		Classification (USCS)
		Gravel	Sand	Clay and Silt	LL %	PI %	
B1	2	22.8	44.3	32.9	23.9	9	SC
B1	2	9.5	36.3	54.2	22.8	9.5	CL

2-3- Natural Moisture Content and Density Test

- Natural moisture content ASTM (D2216-98)

The natural moisture contents of soil samples are measured for the selected samples, the value of each is indicated in borehole logs.

- Density Tests

Density of the selected soil samples has been determined by measuring the weight and volume of the samples obtained from sample liners. Water content and dry density tests results of the soil samples are summarized in Table 6.

Table 6- Water content and dry unit weight

Borehole No.	Sample Depth (ft.)	Height of Sample	w (%)	Dry Density (pcf)
B1	1-2	-	12	-
B1	2-3.5	6”	10	122.2
B1	4-5	-	11	-
B1	5-6.5	6”	11	120.7
B2	1-2	-	9	-
B2	2-3.5	6”	10	107.4
B2	4-5	-	11	-
B2	5-6.5	6”	12	116.5

3- Description of Soil Layers

3-1- General Description of the Subsurface Soil Layers

Based on the visual observations during the drilling, in-situ test results and laboratory testing, the encountered soil is generally classified as:

- Clayey sand (SC)
- Clay of low plasticity (CL)

The soil is classified as very dense clayey sand in B1 and a very firm low plasticity clay in B2 (at surface).

3-2- Geotechnical Parameters

The SPT has been used to correlate engineering parameters such as strength, angle of internal friction (Table 7) and the stress-strain modulus (Es) as shown in Table 8.

Table 7- Typical values of soil friction angle for different soils according to USCS

Description	USCS	Soil friction angle [°]		Reference
		min	max	
Inorganic clays, silty clays, sandy clays of low plasticity	CL	27	35	[1]
Silty clay	OL, CL, OH, CH	18	32	[2]
Clay	CL, CH, OH, OL	18	28	[2]

1. Swiss Standard SN 670 010b, Characteristic Coefficients of soils, Association of Swiss Road and Traffic Engineers
2. Minnesota Department of Transportation, Pavement Design, 2007

Table 8-Equations for stress-strain modulus E_s by several test methods (Bowles, 2002)

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf.

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ — — — $‡E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ $*E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	$¶E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	$†E_s = 40\,000 + 1050N$ $E_{s(OCR)} \approx E_{s,nc}\sqrt{OCR}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$	$E_s = (1 \text{ to } 2)q_c$
	If $q_c < 2500$ kPa use $\quad {}^{\$}E_s' = 2.5q_c$ $2500 < q_c < 5000$ use $\quad E_s' = 4q_c + 5000$ where $E_s' = \text{constrained modulus} = \frac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \frac{1}{m_v}$	
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

Final values of geotechnical parameters for the subject site using the field observations, in-situ and laboratory tests are summarized in Table 9.

Table 9- Geotechnical Parameters Estimates

Material	γ_{wet} (pcf)	γ_{sat} (pcf)	c (ksf)	φ (degrees)	E_s (ksf)	ν	K_0	K_a	K_p
SC	134	138	0.3	30	600	0.3	0.5	0.33	3.0
CL	124	132	0.35	27	350	0.4	0.55	0.38	2.66

γ_{wet} : wet unit weight in the field.
γ_{sat} : saturated unit weight.
c : cohesion.
φ : angle of internal friction
E_s : elasticity modulus
ν : poisson ratio
K_0 : at rest earth pressure
K_a , K_p: active and passive earth pressure

4- Foundation Design Recommendations

Recommendations presented herein are based on the proposed building layout and site development plan as understood at this time. The development is muli-family residential and commercial structures of three story. However, at the time of preparation of this report, structural column loads were not available and no construction document is available. As further information is developed by the architect and/or structural engineer concerning these items, the design criteria should be reviewed by AEC for continued applicability. As a general recommendation, foundation and below-grade elements of the building should be designed in accordance with the building code selected for design. The following sections provide specific geotechnical design recommendations for the foundation and below-grade structure, if any.

The foundation bearing soils are typically very firm low plasticity clay and very dense clayey sand. It is necessary to build up the subgrade to achieve the proposed footing subgrade level, for this it is recommended that compacted structural fill be used. The compacted structural fill should be graded in accordance with the recommendations in Section 7.2.1.

4-1- Recommended Foundation

Based on the loading conditions assumed by us and subsurface conditions as observed in the field investigations it is our opinion that direct soil bearing foundations such as reinforced concrete **strip foundation** will likely provide the most technically-feasible and cost-effective foundation system for the proposed structure.

4-2- Allowable Bearing Capacity

As noted above, the foundation bearing soils at the site consist of very firm low plasticity clay and very dense clayey sand. The recommended maximum allowable gross bearing pressure for design of **strip footing** in these soils in undisturbed condition is **3.3 ksf** for **18 in. width** and **3.2 ksf** for **15 in.** widths. This bearing pressure value applies to the total dead load plus permanently and/or frequently applied live loads including the weight of the foundation elements. This bearing pressure may however, be increased by one-third when considering transient loads such as earthquake forces.

The least lateral dimension of continuous footings should be 18 in., for the structures. Exterior footings and footings in unheated areas should bear a minimum of 12 in. below the adjacent ground surface. The bottom of footings should be established below a 2 horizontal to 1 vertical (2H:1V) slope line drawn upward and outward from the bottom of any adjacent utility or structure.

The outputs of foundation bearing capacity are presented in Exhibit V and can be consulted for other footing widths, in case of existence of detached parkings in the development for example.

4-3- Total Settlement

Settlement considerations, rather than bearing capacity, generally control spread footing or reinforced concrete mat foundation selection and design at these depths in these soils. It is our opinion that for the maximum allowable bearing pressure recommended above, soil bearing foundations should experience a maximum post-construction settlement of approximately 1 in. We anticipate that the majority of the settlements will occur during or soon after construction with the largest settlements occurring at the center of the structure. As noted above, the anticipated bearing pressure is more than the existing pressure of the overburden soils at the proposed bearing elevation so settlement will control foundation selection.

4-4- Differential Settlement

Differential settlements are generally caused by variations in soil profile (including layer thickness), compressibility characteristics, applied load, bearing pressures, foundation dimension, and foundation stiffness. At this time, it is expected that the differential settlement should be on the order of ½ inch. However, when the design documents are ready, this value should be re-evaluated.

4-5- Modulus of Subgrade Reaction

If a reinforced concrete strip is selected as the preferred option, the structural design of reinforced concrete strip foundations typically requires a modulus of subgrade reaction (Winkler spring) or a similar elastic analysis method to determine thickness and reinforcing requirements for the **strip** foundation. We recommend that a modulus of subgrade reaction (k_s) of **116 kips per cubic foot (kcf)** be used.

4-6- Ground Floor Slabs

It is recommended that the ground floor slabs of buildings and structures, if any, be designed as soil-supported slabs-on-grade, bearing on a minimum 6-inch thick layer of crushed stone that is graded in accordance with the recommendations in Section 7.2. We also recommend that a 10 mil-thick polyethylene vapor barrier be placed on top of the aggregate layer to reduce moisture condensation on the underside of the slab-on-grade.

4-7- Lateral Resistance

Shallow foundations bearing on a reinforced subgrade or on compacted structural fill may be designed to resist lateral forces using a friction coefficient of 0.4 along the bottom of the foundations and a passive resistance of 365 pounds per square foot per foot (pcf) of depth on the vertical sides of the foundations. This value does not include a safety factor; a safety factor of 1.5 should be used against sliding in the design. The frictional and passive pressure components of lateral resistance may be combined, provided that passive resistance does not exceed two-thirds of the total. The top 24-in of soil should be neglected when calculating passive lateral earth pressures unless the area around the foundation is covered with pavement.

Retaining wall, if any, will be subjected to lateral earth pressures. A soil wet unit weight and coefficient of active lateral earth pressure (k_a) of 129 pcf and 0.42, respectively, should be utilized for design of walls.

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

4-8- Site and Foundation Drainage

As previously discussed, and as shown on the test boring logs, groundwater was not encountered in any of the other explorations. However, during periods of significant precipitation, or during the spring thaw, there is a possibility that water could become trapped on the outside face of the walls, with no way to relieve the pressure head from the accumulated water, the water could exert excess pressure on the walls and leak into the finished below grade spaces.

To drain such water, it is recommended that a perimeter wall drain be provided along the outside of the wall. The perimeter drain should consist of a 0.1 m (4-inch) diameter perforated pipe surrounded by 0.15 m (6 inches) of crushed stone, graded in accordance with the recommendations in Section 7.2.2, placed inside a non-woven geotextile filter fabric to limit silting. The perimeter drain trench should be backfilled with compacted structural fill. Pipe invert elevations should be kept below the bottom of the adjacent slab but above the footing bearing elevation. The perimeter drain should be pitched to drain by gravity to the site storm drain system.

All grades must provide effective drainage away from the structures, during and after construction. Water ponding next to the structures can result in greater than calculated soil movement and differential floor slab settlement, cracked slab and wall movement or leaked roof. Effective drainage should be maintained during life time of the building.

Exposed ground should be sloped at a minimum 5 percent away from the structure for the at least 10 ft. beyond the perimeter of the structure. After the construction (building and landscape), we recommend final grades to be inspected for effective drainage. Grades of the around of the building should also be inspected periodically during life time of the building.

Planters located within 10 ft. of the structure should be self-contained to prevent water accessing the building and pavement subgrade soil (if any). Sprinkler main and spray heads should be located a minimum 5 ft. away from the building lines. Low volume, drip styled landscaped irrigation should not be used near the building. Roof run off should be located in the drains or gutters. Roof drain and downspouts should discharge onto pavements that slope away from building/structures or the downspouts should extend a minimum of 10 ft. away from the structures.

4-9- Utility Trenches

Utility trenches should be properly backfilled. The pipes should be bedded on clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by a representative from our firm. The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained as below:

Utility trenches should be backfilled with fill placed in lifts not exceeding 8 inches in uncompacted thickness. Native backfill materials should be compacted to at least 90 percent relative compaction and granular import material should be compacted to at least 95 percent relative compaction. These compaction recommendations assume a reasonable “cushion” layer around the pipes.

If imported granular soil is used, sufficient water should be added during the trench backfilling operations to prevent the soil from “bulking” during compaction.

5- Liquefaction Consideration

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The project location on liquefaction map (Source CGS) site is not within liquefaction hazard zone, thus further study was not within the scope of services for this report.

6- Seismic Design Considerations

The details of USGS seismic design are presented in Exhibit IV.

7- Construction Considerations

7-1- General

The primary purpose of this section of the report is to comment on items related to excavation, earthwork and related geotechnical aspects of the proposed foundation design. It is written primarily for the engineer having responsibility of preparation for the plans and specifications of the foundation, but it may also aid personnel who monitor the construction. Prospective contractors for this project must evaluate construction problems on the basis of their experience on similar projects, taking into account their own construction methods and procedures.

7-2- Fill Materials

7-2-1- Compacted Structural Fill

The structural fill should be a well-graded granular material. Caltrans AB Class II is recommended to be used for this purpose with the following specifications.

Table 10-CALTRANS AB Class II recommended parameters

Material	γ_d (pcf)	γ_{sat} (pcf)	c (ksf)	φ (degrees)
CALTRANS AB CLASS II (92% compacted)	125	130	0.1	38

Minimum 5 feet of the compacted backfill behind any wall is required for wall of 10’ tall, shorter wall can have narrower backfill zone.

Imported structural fill should be used if the on-site excavated soils cannot meet the gradation requirements indicated above.

In addition to the above requirements, structural fill to be placed in the upper 3 ft. of filled areas during periods of wet and/or freezing weather should contain less than 5 percent passing the No. 200

sieve. Material proposed as structural fill should be tested and approved by a qualified geotechnical engineer prior to its use.

To evaluate the suitability and the quality of the fill source, we recommend that the laboratory testing of fill material be performed in accordance with the ASTM Test Methods indicated below.

Table 11- Summary of ASTM Test Methods

Summary of ASTM Test Methods	
Test	ASTM Designation
Moisture Content	D 2216
Modified Proctor	D 1557
Sieve Analysis	D 422
Atterberg Limits	D 4318

Structural fill in unconfined areas should be placed in horizontal lifts not exceeding 9-in. in loose thickness and compacted to at least 95 percent of the laboratory maximum dry density, as determined by ASTM Test D 1557 (Modified Proctor). Structural fill should be moisture conditioned to within ±2 percentage points of the optimum moisture content.

Structural fill should be compacted by self-propelled vibratory rollers or other approved compaction equipment. Where compaction occurs in confined areas, the loose lift thickness should be reduced to a maximum of 6 in. and compaction performed by hand-guided vibratory compactors or tampers.

Before placing fill materials, the exposed natural soil should be observed and proof rolled to identify any soft compressible layers. At the end of each day's operations, the last lift should be rolled by a smooth-wheel roller to eliminate ridges of un-compacted soil to aid runoff and drainage. No layer of fill should be placed until the underlying materials have been approved.

7-2-2- Common Fill

Common (non-structural) fill should consist of sandy or gravelly soil with a maximum particle size of 3 inches, with less than 35 percent passing the No. 200 sieve, and with a plasticity index of 20 or less.

7-3- Quality Control

Placement and compaction of all fill materials should be monitored and tested by a qualified technician under supervision of a professional geotechnical engineer. We recommend that all structural fill placements be tested in accordance with ASTM D2922 and D3017 (Nuclear Density Method) to verify the density, degree of compaction, and moisture content of the fill. The specifications should call for frequent testing on each lift. In the event where any portion of the fill fails to meet the compaction requirements, the area should be reworked, re-compacted, and retested until the specified compaction is achieved.

8- Summary of Design recommendation

The site soil parameters need to be chosen from Table 9.

The in-fill soil back of any wall in contact with geogrid in general needs to be in compliance of section 7.2.

All the design methods and parameters including factor of safeties need to be followed per requirements of the engineer designing the structure. All the construction details are required to be per direction of the engineer designing the structure.

Drainage is required per detail and specs of footings.

All the deviations from this report needs to be brought to the attention of AEC as will be discussed in section 9.

Section 7.3 of this report and all the special inspection requirements mentioned in the report are required to be performed by AEC and needs to be identified on the cover sheet of the construction documents before being submitted to the authority having jurisdiction. The plans are required to be reviewed by AEC and be verified to be in compliance with the requirements of this report before being submitted to jurisdiction having authority.

9- Limitations

This Report was prepared pursuant to an Agreement dated 01/24/2020 between Mr. Ali Mozaffari (Alan Enterprise LLC) and AEC. All uses of this Report are subject to, and deemed acceptance of, the conditions and restrictions contained in the Agreement. The observations and conclusions described in this Report are based solely on the Scope of Services provided pursuant to the Agreement. AEC has not performed any additional observations, investigations, studies or other testing not specified in the Agreement and the Report. AEC shall not be liable for the existence of any condition the discovery of which would have required the performance of services not authorized under the Agreement.

This Report is prepared for the exclusive use Alan Enterprise LLC in connection with the design and construction of the mentioned development. There are no intended beneficiaries other than Alan Enterprise LLC AEC shall owe no duty, whatsoever, to any other person or entity on account of the

Agreement or the Report. Use of this Report by any person or entity other than Alan Enterprise LLC for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from Alan Enterprise LLC and from AEC. Use of this Report by such other person or entity without the written authorization of Alan Enterprise LLC and AEC shall be at such other person's or entities sole risk, and shall be without legal exposure or liability to AEC.

Use of this Report by any person or entity, including by Alan Enterprise LLC for a purpose other than for the design and construction of the proposed development is expressly prohibited unless such person or entity obtains written authorization from AEC indicating that the Report is adequate for such other use. Use of this Report by any person or entity for such other purpose without written authorization by AEC shall be at such person's or entities sole risk and shall be without legal exposure or liability to AEC.

This report reflects site conditions observed and described by records available to AEC as of the date of report preparation. The passage of time may result in significant changes in site conditions, technology, or economic conditions which could alter the findings and/or recommendations of the report. Accordingly, Alan Enterprise LLC and any other party to whom the report is provided recognize and agree that AEC shall bear no liability for deviations from observed conditions or available records after the time of report preparation.

Use of this Report by any person or entity in violation of the restrictions expressed in this Report shall be deemed and accepted by the user as conclusive evidence that such use and the reliance placed on this Report, or any portions thereof, is unreasonable, and that the user accepts full and exclusive responsibility and liability for any losses, damages or other liability which may result.

10- References

1- 7.5' Series Geologic Map, USGS.

2- 7.5' Series Topographic Map, USGS.

3- ASTM test methods.

4- Department of the U.S. army corps of engineers Washington, DC 20314-1000, Engineering and design geotechnical investigations, Manual No. 1110-1-1804.

5- Fault activity map of California, CGS, 2010.

6- Foundation analysis and design, Joseph E. Bowles, McGraw-Hill, fifth edition.

7- Liquefaction map, USGS.

8- Landslide map, USGS.

9- Minnesota Department of Transportation, Pavement Design, 2007.

10- Swiss Standard SN 670 010b, Characteristic Coefficients of soils, Association of Swiss Road and Traffic Engineers.

11- Quaternary Fault and Fold Database of the United States.

Exhibit I
Boring Log

EXPLORATORY BORING LOG	Project No : 4134	Borehole No. : B1
Address: Cupertino	Test Date : 2/4/2020	Logged By : Nami
Company Drilling: AEC Drilling Corp.	BORING DIA.:	BORING ELEV.: ---
LOACATION OF BOREHOLE : Specified on Plan	SAMPLER: SPT SPT; C2 :cal. 2"; C2.5 :cal. 2.5"	Weather: Sunny

Notes:

Description:	USCS SOIL TYPE	DEPTH (feet)	SAMPLE	BLOWS PER FOOT	N-SPT	POCKET PEN. (tsf)	DRY DENSITY (pcf)	MOIST. CONT. (%)	FINES (%)	SANDS (%)	GRAVELS (%)	LIQUID LIMIT	PLASTICITY INDEX	
0.0 - 7 ft **Brown clayey sand with gravel (SC) - Damp - Particle is angular with elongated shape and gravel maximum size - None reaction with HCL - Hard to very hard consistency - Strong cementation - Blocky structure - None dilatancy - Low to none dry strength - High toughness - Non plastic - Very dense.**		1						12.0						
		2	C2.5	40 49 53	66	>4.5	122.2	10	32.9	44.3	22.8	23.9	9.0	
		3												
		4						11.0						
		5	C2.5	58 67 73	91	>4.5	120.7	11.0						
		6												
		7												
Refusal depth at 7 ft **No underground water encountered**		8												
		9												
		10												
		11												
		12												
		13												
		14												
		15												
		16												
		17												
		18												
		19												
		20												
		21												
		22												
		23												
		24												
		25												
		26												

EXPLORATORY BORING LOG	Project No : 4134	Borehole No. : B2
Address: Cupertino	Test Date : 2/4/2020	Logged By : Nami
Company Drilling: AEC Drilling Corp.	BORING DIA.:	BORING ELEV.: ---
LOACATION OF BOREHOLE : Specified on Plan	SAMPLER: SPT SPT C2 :cal. 2" C2.5 :cal. 2.5"	Weather: Sunny

Notes:

Description:	USCS SOIL TYPE	DEPTH (feet)	SAMPLE	BLOWS PER FOOT	N-SPT	POCKET PEN. (tsf)	DRY DENSITY (pcf)	MOIST. CONT. (%)	FINES (%)	SANDS (%)	GRAVELS (%)	LIQUID LIMIT	PLASTICITY INDEX	
0.0 - 4 ft **Brown to light brown sandy lean clay (CL)** **Wet - Maximum particle size is cobble and angular with elongated shape - No odor - None reaction with HCL-Hard consistency Strong cementation - Blocky structure - None dilatancy - None dry strength - High Toughness - None plastic - Stiff.**		1												
								9.0						
		2		7										
			C2.5	11	15	>4.5	107.4	10	54.2	36.3	9.5	22.8	9.5	
		3		12										
		4												
								11.0						
4.0 - 8.0 ft		5		44										
Same as above but hard.			C2.5	68	92	>4.5	116.5	12.0						
		6		73										
		7												
		8												
Refusal depth at 8 ft **No underground water encountered**		9												
		10												
		11												
		12												
		13												
		14												
		15												
		16												
		17												
		18												
		19												
		20												
		21												
		22												
		23												
		24												
		25												
		26												

Exhibit II
Lab Results

ACHIEVEMENT ENGINEERING CORP.

Moisture Density
(AASHTO T265 - ASTM D2216)

Report Date: 2/17/2020
Project No: 4134
Project Name: Alan
Project Address: Cupertino
Technician: Nami

Type of Material:	Soil	Sample Description:	
Source:	Field		
Sampled by:	Nami	Sample Date:	2/4/2020

Sample No:	B1 1'- 2'	B1 2' - 3.5'	B1 4' - 5'	B1 5' - 6.5'			
Ht. of Sample:	Disturbed	6.00	Disturbed	6.00			
Tare No:	CA - 17	CA - 10	CA - 5	CA - 14			
Gross Wet Wt:	1057.69	1255.72	1270.78	1241.74			
Gross Dry Wt:	977.95	1167.76	1174.42	1152.29			
Tare Wt:	309.93	315.16	313.25	310.13			
Net Dry Wt:	668.02	852.60	861.17	842.16			
Wt. of Water:	79.74	87.96	96.36	89.45			
% Moisture	12%	10%	11%	11%			
Liners Dia		2.5"		2.5"			
Density Factors		0.860		0.860			
Dry Density		122.21		120.71			

Tested By: Nami

Reviewed E A.F

Signature:

Signature:

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Moisture Density
(AASHTO T265 - ASTM D2216)

Report Date:	2/17/2020
Project No:	4134
Project Name:	Alan
Project Address:	Cupertino
Technician:	Nami

Type of Material:	Soil	Sample Description:	
Source:	Field		
Sampled by:	Nami	Sample Date:	2/4/2020

Sample No:	B2 1'- 2'	B2 2' - 3.5'	B2 4' - 5'	B2 5' - 6.5'			
Ht. of Sample:	Disturbed	6.00	Disturbed	6.00			
Tare No:	CA - 2	CA - 3	CA - 6	CA - 15			
Gross Wet Wt:	1221.66	1139.35	1328.28	1217.07			
Gross Dry Wt:	1144.92	1060.98	1224.34	1123.01			
Tare Wt:	311.43	311.66	311.80	309.96			
Net Dry Wt:	833.49	749.32	912.54	813.05			
Wt. of Water:	76.74	78.37	103.94	94.06			
% Moisture	9%	10%	11%	12%			
Liners Dia		2.5"		2.5"			
Density Factors		0.860		0.860			
Dry Density		107.40		116.54			

Tested By: Nami   Reviewed E A.F

Signature: ____________   Signature: ____________

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SIEVE ANALYSIS SHEET
(AASHTO T27-ASTM C136 and D6913)

ACHIEVEMENT ENGINEERING CORP.

Date:	2/20/2020
Project No.:	4134
Project Name:	Alan
Project Address:	Cupertino
Tested By:	Nami
Material:	Soil

Borehole Number and Depth: **B1 2' - 3.5'**
Nominal Max. Size in sample = ½”
Min. Test Sample size in kg [lb] = 2 [4]
Nominal Dimension of seive = 8"

	Sieve Size	Sieve Size	Wt. Ret. (gr)	% Ret.	% Passing	Retained Limit (kg)
4	100mm	4”	0	0.00%	100.00%	
3	75mm	3”	0	0.00%	100.00%	
2	50mm	2”	0	0.00%	100.00%	
1	37.5mm	1 ½”	0	0.00%	100.00%	
1	25mm	1”	0	0.00%	100.00%	
1	19mm	¾”	0	0.00%	100.00%	
0	12.5mm	½”	23.15	7.87%	92.13%	
0	9.5mm	3/8”	8.56	2.91%	89.21%	
0	4.75mm	#4	35.44	12.06%	77.16%	
0	2.36mm	#8	27.6	9.39%	67.77%	
0	1.18mm	#16	18.9	6.43%	61.34%	
0	600μm	#30	16.07	5.47%	55.87%	
0	300μm	#50	22.47	7.64%	48.23%	
0	150μm	#100	24.18	8.23%	40.00%	
0	75μm	#200	21.01	7.15%	32.86%	

Pan #: CA - 10
Pan weight (gr): 315.16
Mass of pan & dried sample before wash (gr): 609.13
Original mass before wash (gr): 293.97
Min. readability of scale (gr) = 0.29
Mass of pan & dried sample after wash (gr): 512.6
Mass of sample after wash & being dried (gr): 197.44
Mass after mechanical shake (gr): 197.38

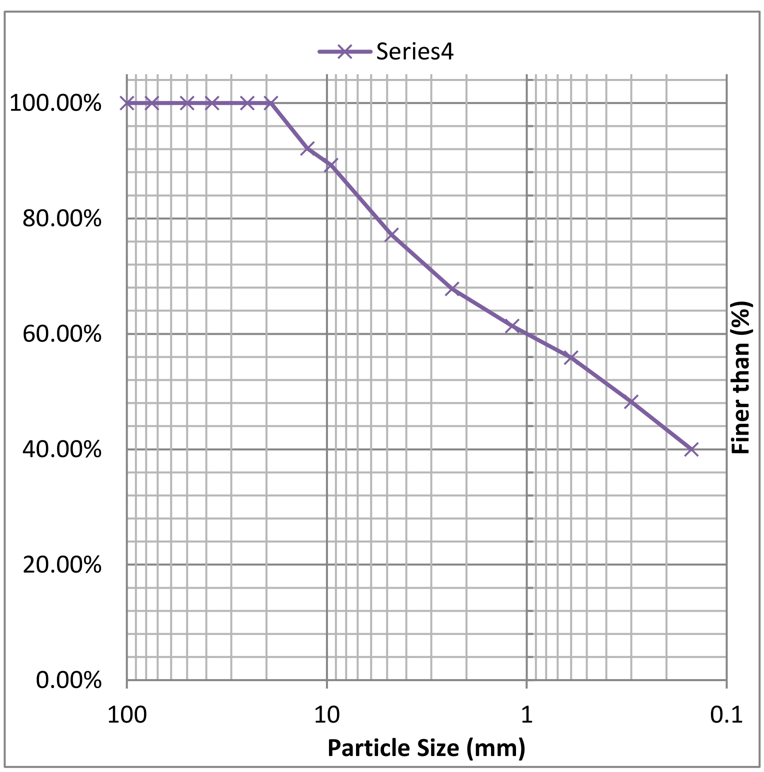
Percent of Gravel = 22.84%
Fine Content = 32.86%
Percent of Sand = 44.30%

D_{10} (mm)= 0.0750
D_{30} (mm)= 0.0750
D_{60} (mm)= 1.0379
D_{50} (mm)= 0.3695

Cc= 13.8
Cu= 0.1

Check for waste limt (0.3%) : 0.03%

100



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SIEVE ANALYSIS SHEET
(AASHTO T27-ASTM C136 and D6913)

Date:	2/20/2020
Project No.:	4134
Project Name:	Alan
Project Address:	Cupertino
Tested By:	Nami
Material:	Soil

Borehole Number and Depth: **B2 2' - 3.5'**
Nominal Max. Size in sample = ½”
Min. Test Sample size in kg [lb] = 2 [4]
Nominal Dimension of seive = 8"

	Sieve Size	Sieve Size	Wt. Ret. (gr)	% Ret.	% Passing	Retained Limit (kg)
4	100mm	4”	0	0.00%	100.00%	
3	75mm	3”	0	0.00%	100.00%	
2	50mm	2”	0	0.00%	100.00%	
1	37.5mm	1 ½”	0	0.00%	100.00%	
1	25mm	1”	0	0.00%	100.00%	
1	19mm	¾”	0	0.00%	100.00%	
0	12.5mm	½”	7.43	2.49%	97.51%	
0	9.5mm	3/8”	5.59	1.87%	95.64%	
0	4.75mm	#4	15.17	5.09%	90.55%	
0	2.36mm	#8	12.91	4.33%	86.22%	
0	1.18mm	#16	13.15	4.41%	81.81%	
0	600µm	#30	14.83	4.97%	76.84%	
0	300µm	#50	22.92	7.68%	69.16%	
0	150µm	#100	23.68	7.94%	61.22%	
0	75µm	#200	20.87	7.00%	54.23%	

Pan #: CA - 3
Pan weight (gr): 311.66
Mass of pan & dried sample before wash (gr): 609.98
Original mass before wash (gr): 298.32
Min. readability of scale (gr) = 0.30
Mass of pan & dried sample after wash (gr): 448.3
Mass of sample after wash & being dried (gr): 136.64
Mass after mechanical shake (gr): 136.55

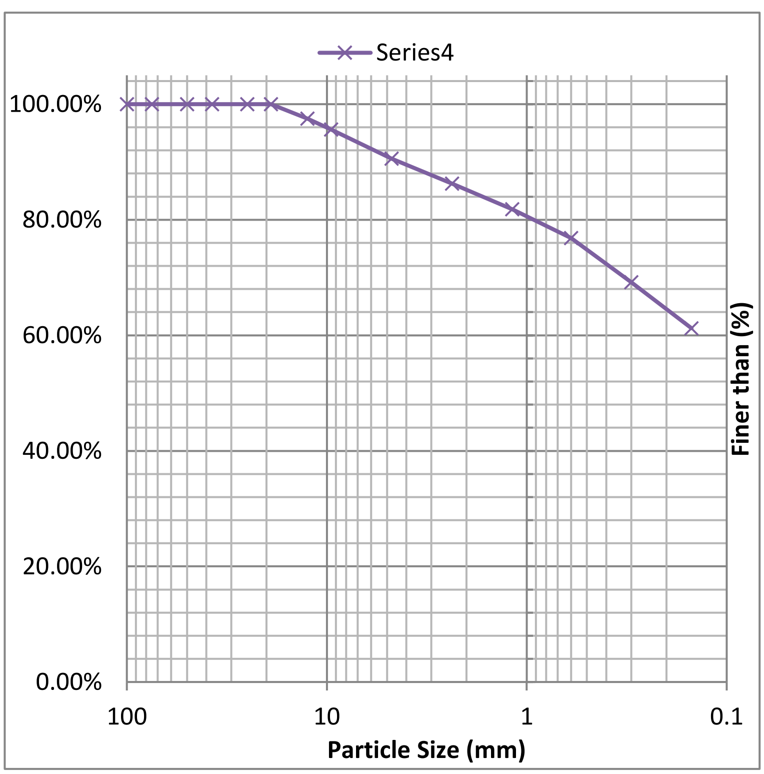
Percent of Gravel = 9.45%
Fine Content = 54.23%
Percent of Sand = 36.32%

D_{10} (mm)= 0.0750
D_{30} (mm)= 0.0750
D_{60} (mm)= 0.1369
D_{50} (mm)= 0.0750

Cc= 1.8
Cu= 0.5

Check for waste limt (0.3%) : 0.07%

100

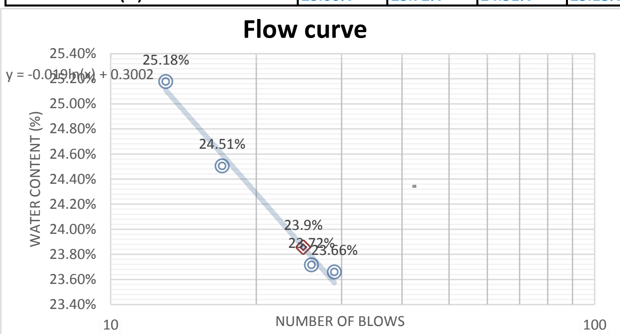


Atterberg Limits
(AASHTO T89 and T90 - ASTM D4318)

ACHIEVEMENT ENGINEERING CORP.

Sample Description:	SOIL	Report Date:	2/28/2020
Boring No:	B1	Project No:	4134
Sample ID:	228204134	Project Name:	Alan
Sample Depth:	2' - 3.5'	Project Address:	Cupertino
Material:	SOIL	Technician:	Nami

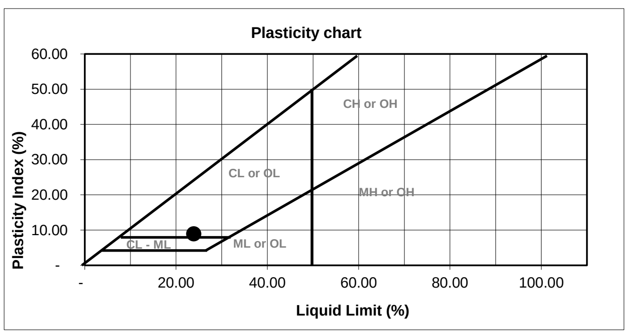
	Liquid Limit				Plastic Limit		
	1	2	3	4	1	2	3
No. of blows	29	26	17	13			
Tare No.	AB - 2	LB - 6	AE - 2	AD - 1	LB - 8	AC - 2	AD - 2
Gross Wet Weight (gr)	20.63	23.33	23.90	21.82	5.42	5.21	5.38
Gross Dry Weight (gr)	18.82	21.02	21.42	19.69	5.29	5.08	5.25
Tare Weight (gr)	11.17	11.28	11.30	11.23	4.35	4.27	4.37
Net Dry Weight (gr)	7.65	9.74	10.12	8.46	0.94	0.81	0.88
Weight of Water (gr)	1.81	2.31	2.48	2.13	0.13	0.13	0.13
Water Content (%)	23.66%	23.72%	24.51%	25.18%	13.83%	16.05%	14.77%



Group Symbol CL

Shrinkage Limit Results

Liquid Limit %	23.86
Plastic Limit %	14.88
Plasticity Index	8.97
Shrinkage Limit %	
B – Value	
Toughness Index	



Tested By: Nami　　　Reviewed By: A.F

Signature: ______________　　　Signature: ______________

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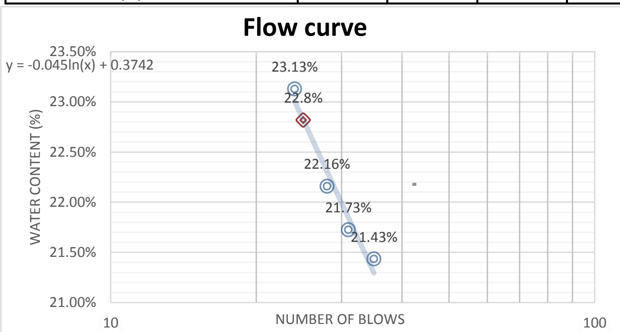
ACHIEVEMENT ENGINEERING CORP.

Atterberg Limits
(AASHTO T89 and T90 - ASTM D4318)

Sample Description:	SOIL	Report Date:	2/28/2020
Boring No:	B2	Project No:	4134
Sample ID:	22820 -4134	Project Name:	Alan
Sample Depth:	2' - 3.5'	Project Address:	Cupertino
Material:	SOIL	Technician:	Nami

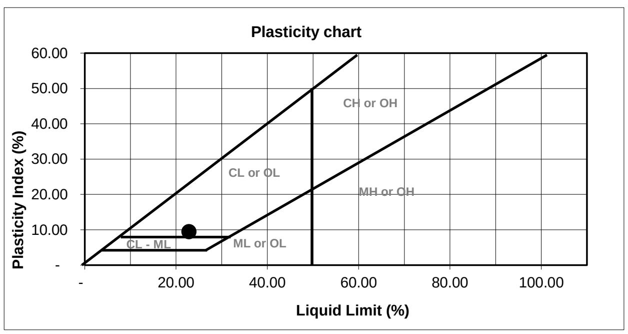
	Liquid Limit				Plastic Limit		
	1	2	3	4	1	2	3
No. of blows	35	31	28	24			
Tare No.	AL - 2	AE - 3	AD - 2	LB - 1	L - 3	AC - 3	AE - 6
Gross Wet Weight (gr)	24.86	22.92	24.38	23.76	5.45	5.48	5.82
Gross Dry Weight (gr)	22.44	20.83	22.00	21.41	5.32	5.34	5.66
Tare Weight (gr)	11.15	11.21	11.26	11.25	4.32	4.36	4.39
Net Dry Weight (gr)	11.29	9.62	10.74	10.16	1	0.98	1.27
Weight of Water (gr)	2.42	2.09	2.38	2.35	0.13	0.14	0.16
Water Content (%)	21.43%	21.73%	22.16%	23.13%	13.00%	14.29%	12.60%



Group Symbol CL

Shrinkage Limit Results

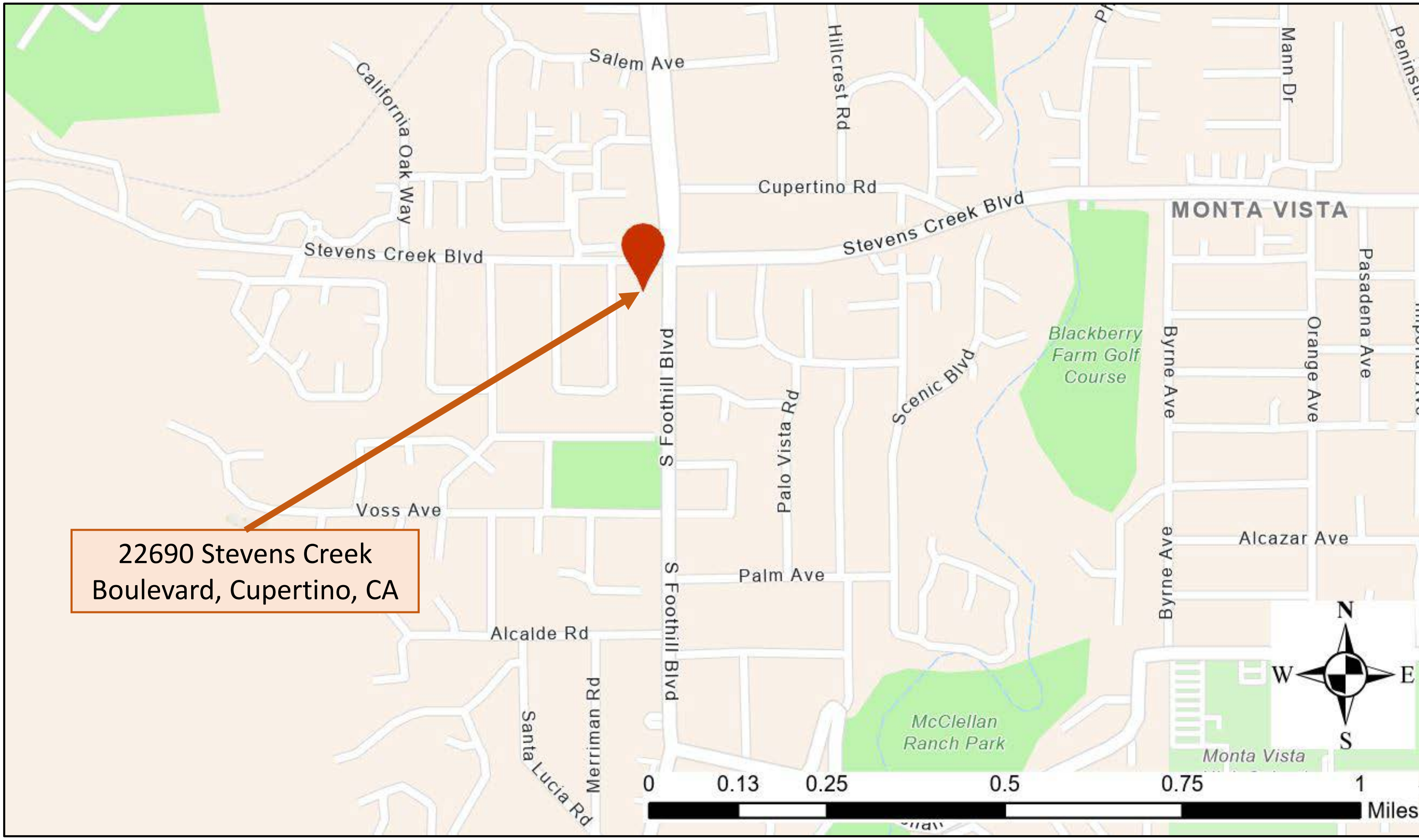
Liquid Limit %	22.82
Plastic Limit %	13.29
Plasticity Index	9.53
Shrinkage Limit %	
B – Value	
Toughness Index	



Tested By: Nami Reviewed By: A.F

Signature: Signature:

Exhibit III
Maps



ACHIEVEMENT ENGINEERING CORP.

Project Number:
4134

Project Title:
Ali Mozaffari - 22690 Stevens Creek Blv – Exhibit III

Vicinity Map

REVISIONS		
	MM/DD/YYYY	REMARKS
0	02/29/2020	
1	__/__/__	
2	__/__/__	
3	__/__/__	
4	__/__/__	

M

01

ACHIEVEMENT ENGINEERING CORP.

Project Number:
4134

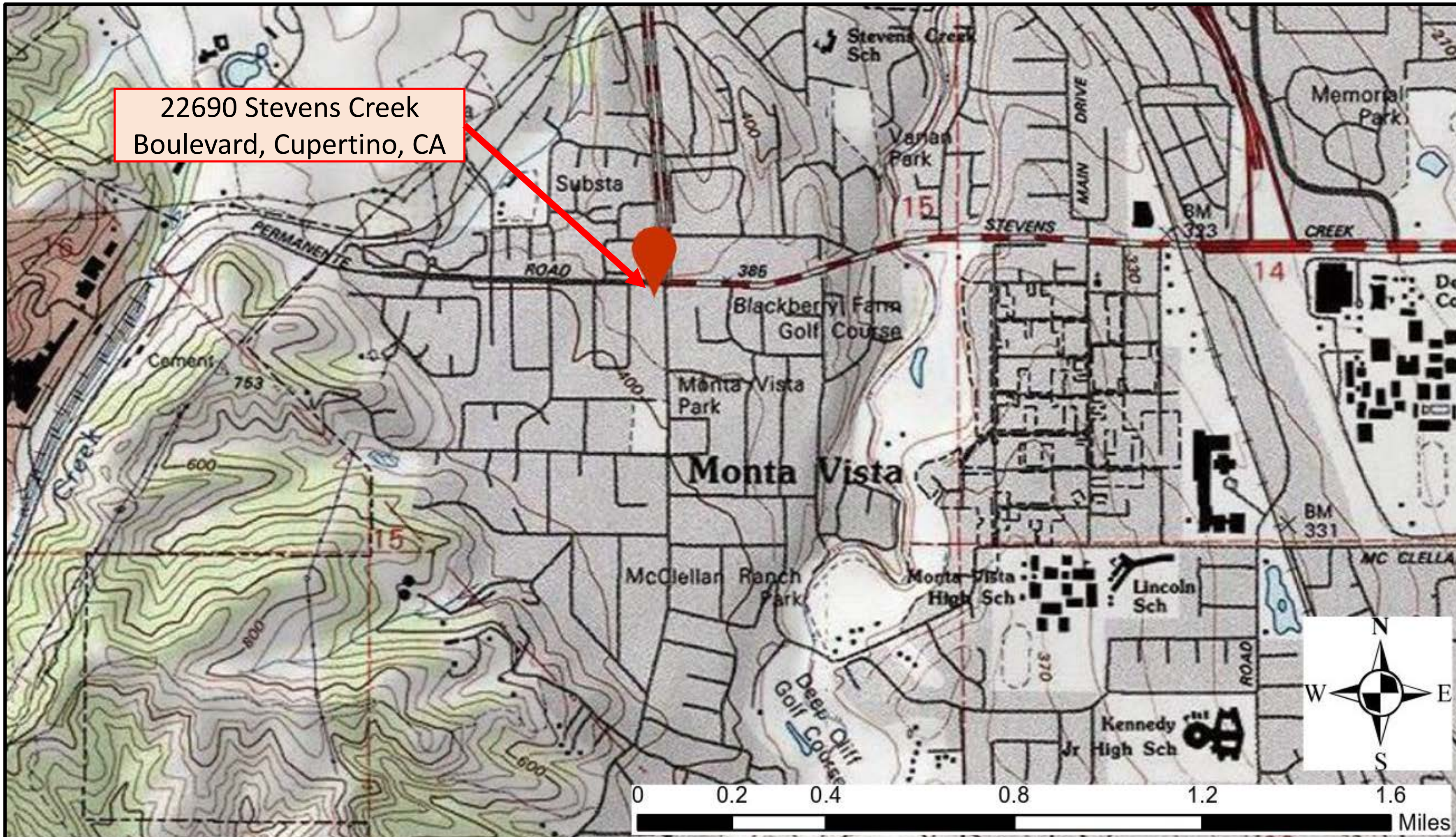
Project Title:
Ali Mozaffari - 22690 Stevens Creek Blv – Exhibit III

Boring Location Map

REVISIONS

	MM/DD/YYYY	REMARKS
0	02/29/2020	
1	__/__/__	
2	__/__/__	
3	__/__/__	
4	__/__/__	

M
02



ACHIEVEMENT ENGINEERING CORP.

Project Number: 4134

Project Title: Ali Mozaffari - 22690 Stevens Creek Blv – Exhibit III

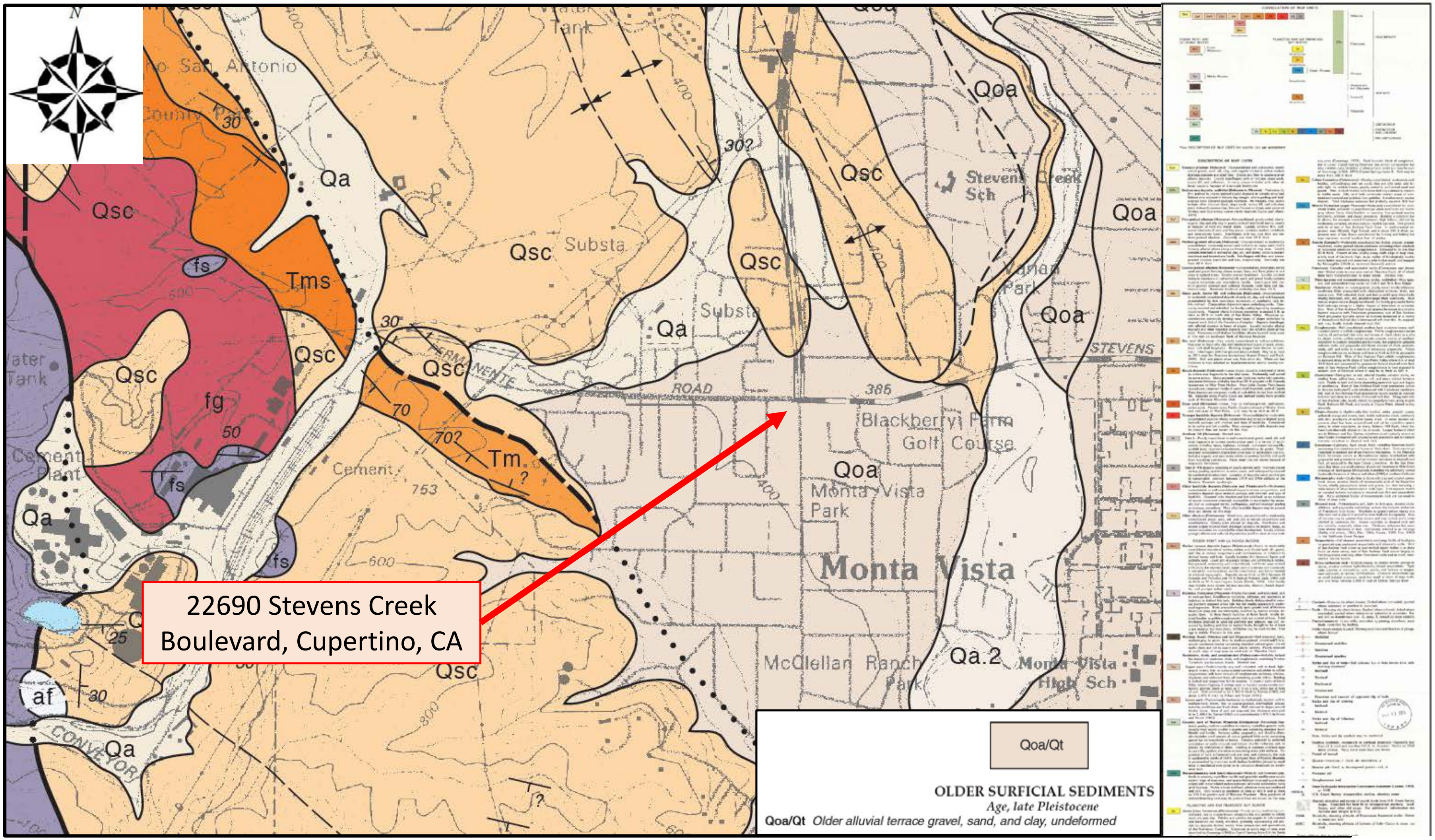
Site Location on 7.5' quadrangle Series Topographical Map by USGS

REVISIONS

	MM/DD/YYYY	REMARKS
0	02/29/2020	
1	__/__/__	
2	__/__/__	
3	__/__/__	
4	__/__/__	

M

03



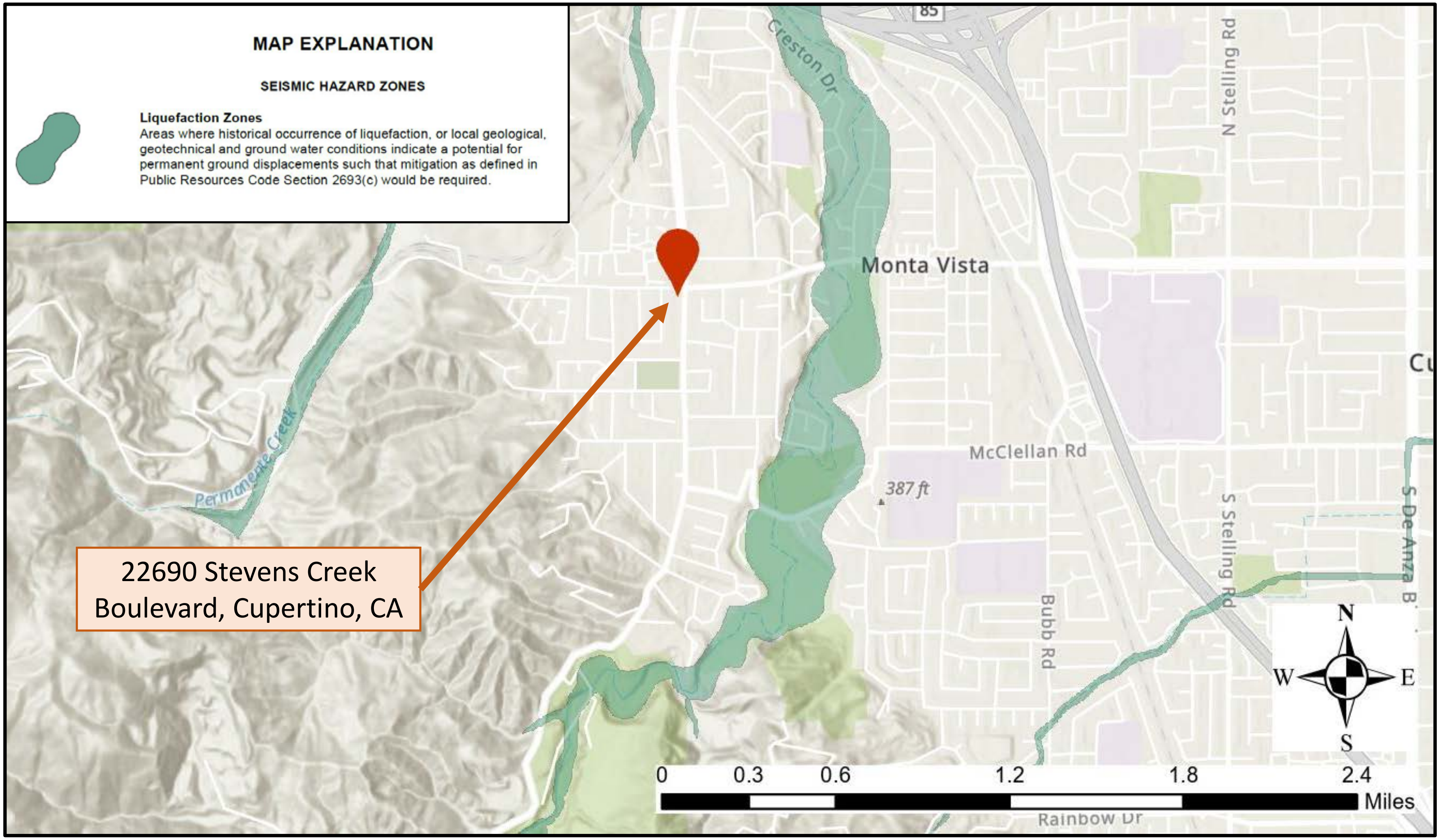
ACHIEVEMENT ENGINEERING CORP.

Project Number: 4134	Project Title: Ali Mozaffari - 22690 Stevens Creek Blv – Exhibit III
Site Location on 7.5' quadrangle Series Geological Map by USGS	

REVISIONS		
	MM/DD/YYYY	REMARKS
0	02/29/2020	
1	___/___/___	
2	___/___/___	
3	___/___/___	
4	___/___/___	

M

04



ACHIEVEMENT ENGINEERING CORP.

Project Number:
4134

Project Title:
Ali Mozaffari - 22690 Stevens Creek Blv – Exhibit III

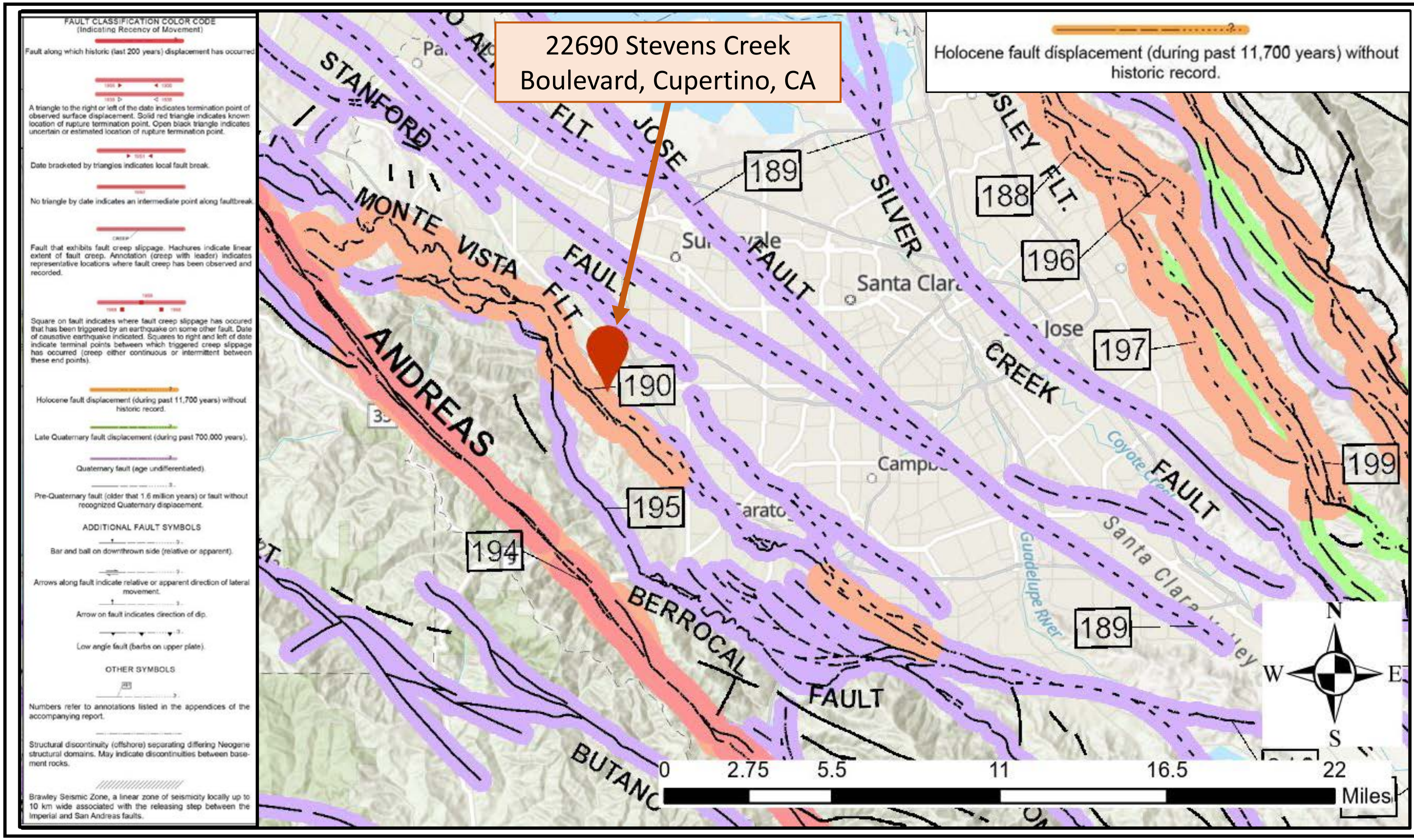
Site Location on State Map for Earthquake Zone of required investigation by CGS (site is **NOT** located within hazard zone)

REVISIONS

	MM/DD/YYYY	REMARKS
0	02/29/2020	
1	__/__/__	
2	__/__/__	
3	__/__/__	
4	__/__/__	

M

05



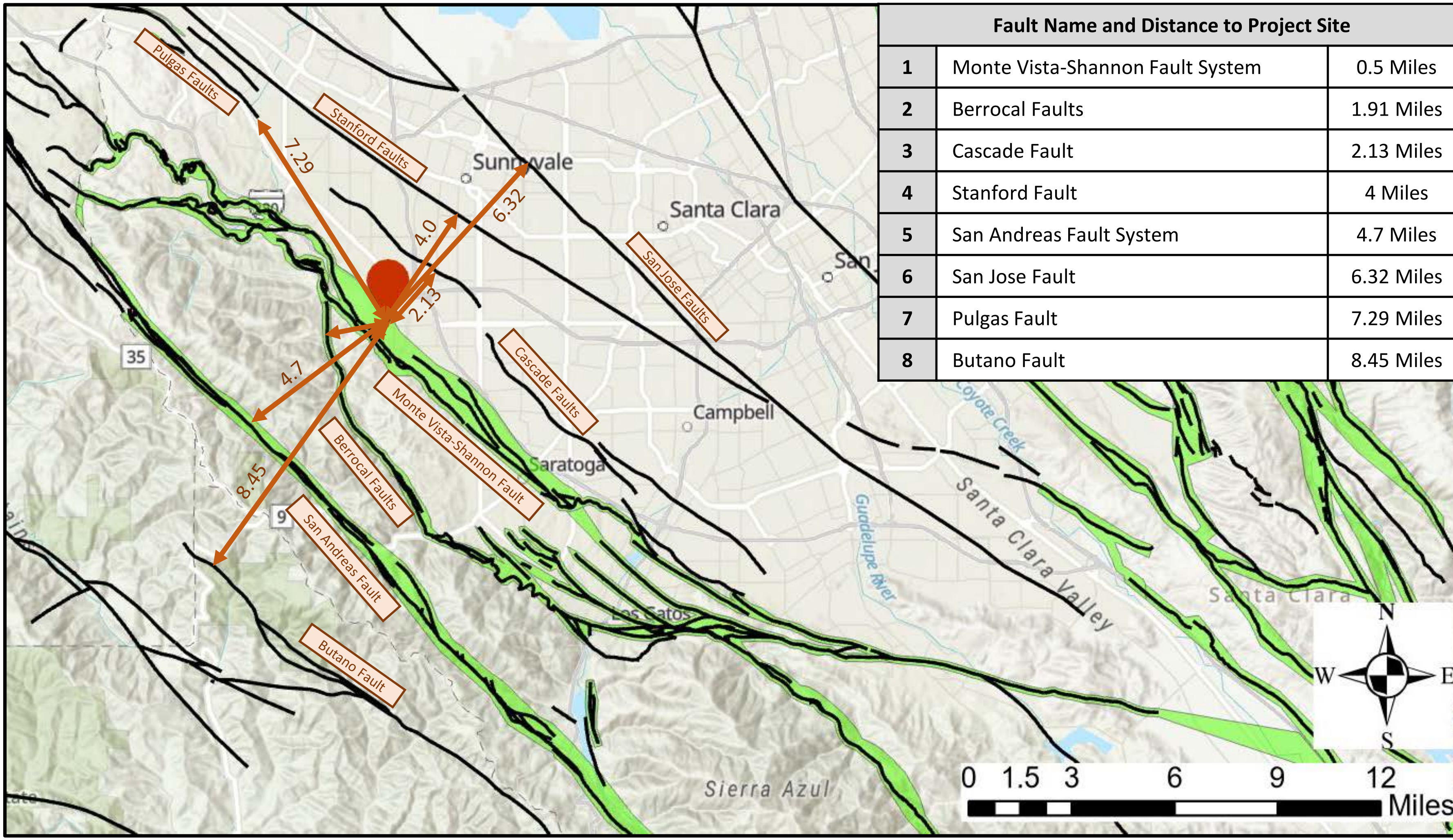
ACHIEVEMENT ENGINEERING CORP.

Project Number: 4134	Project Title: Ali Mozaffari - 22690 Stevens Creek Blv – Exhibit III
Site Location on Fault Activity Map of California (2010) by CGS	

REVISIONS		
	MM/DD/YYYY	REMARKS
0	02/29/2020	
1	___/___/___	
2	___/___/___	
3	___/___/___	
4	___/___/___	

M

06



Fault Name and Distance to Project Site		
1	Monte Vista-Shannon Fault System	0.5 Miles
2	Berrocal Faults	1.91 Miles
3	Cascade Fault	2.13 Miles
4	Stanford Fault	4 Miles
5	San Andreas Fault System	4.7 Miles
6	San Jose Fault	6.32 Miles
7	Pulgas Fault	7.29 Miles
8	Butano Fault	8.45 Miles

ACHIEVEMENT ENGINEERING CORP.

Project Number:
4134

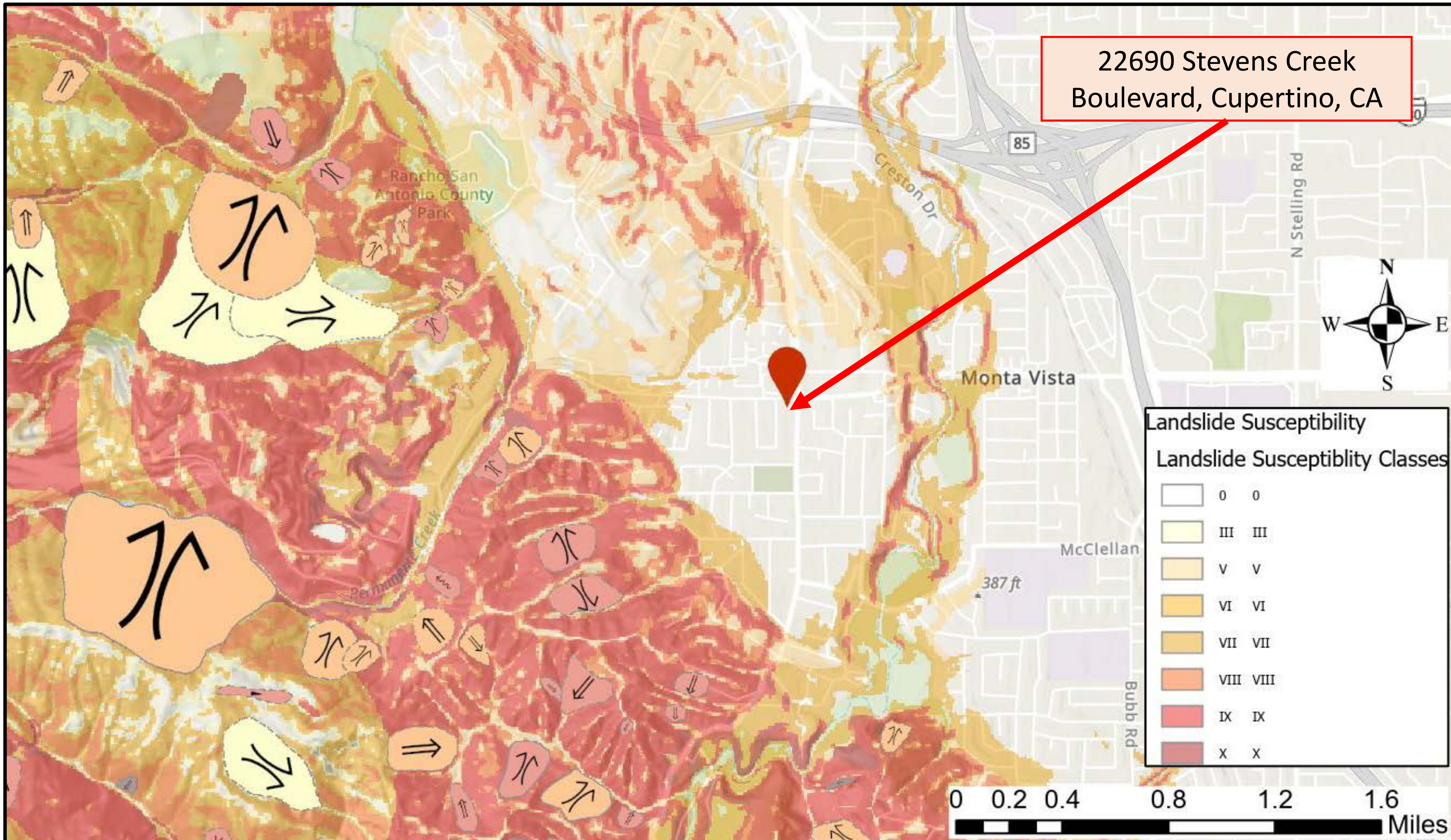
Project Title:
Ali Mozaffari - 22690 Stevens Creek Blv – Exhibit III

Site Location distance to Nears Faults
(10-mile Radius)

REVISIONS		
	MM/DD/YYYY	REMARKS
0	02/29/2020	
1	___/___/___	
2	___/___/___	
3	___/___/___	
4	___/___/___	

M

07



ACHIEVEMENT ENGINEERING CORP.	Project Number: 4134	Project Title: Ali Mozaffari - 22690 Stevens Creek Blv – Exhibit III
	Project Location on Landslide Susceptibility Map (Source USGS) site is within **Class 0 – No Susceptibility**	

REVISIONS

	MM/DD/YYYY	REMARKS
0	02/29/2020	
1	__/__/__	
2	__/__/__	
3	__/__/__	
4	__/__/__	

M

08

Exhibit IV
USGS Seismic Design

4134

22690 Stevens Creek Blvd, Cupertino, CA 95014, USA

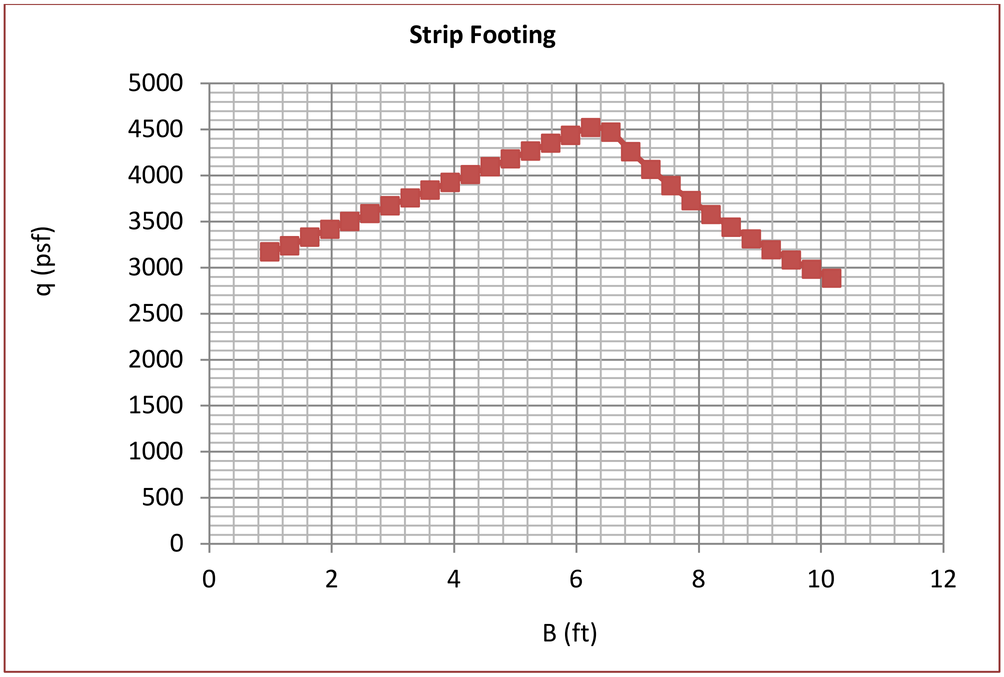
Latitude, Longitude: 37.3217554, -122.068922

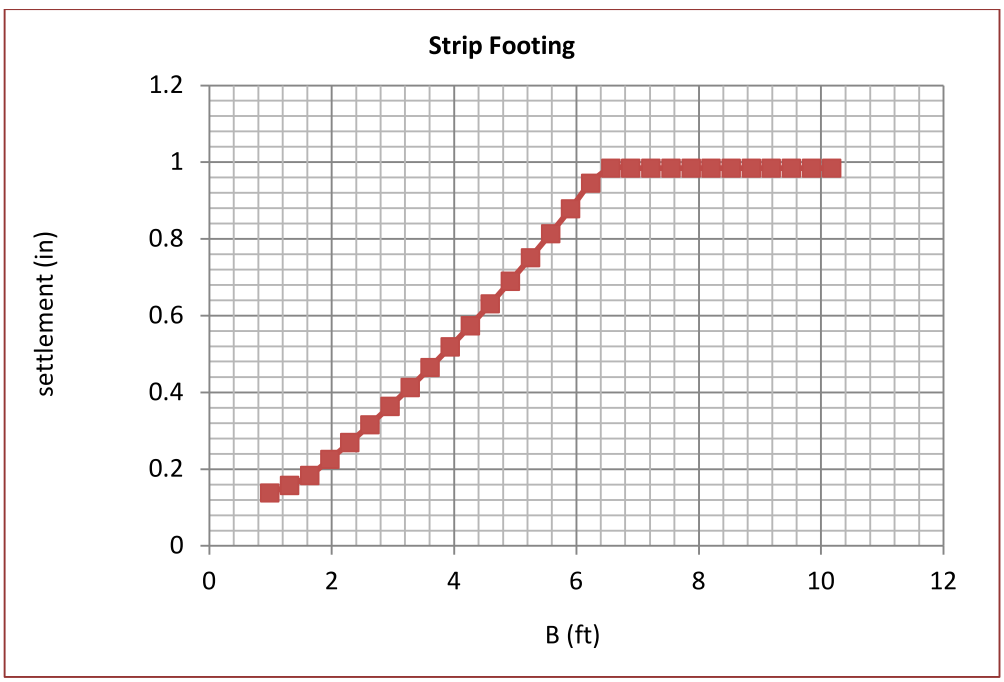
Date	2/13/2020, 2:11:29 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.281	MCE_R ground motion. (for 0.2 second period)
S_1	0.821	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.281	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.521	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.943	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	1.037	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	2.335	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.573	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.281	Factored deterministic acceleration value. (0.2 second)
S1RT	0.941	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.054	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.821	Factored deterministic acceleration value. (1.0 second)
PGAd	0.943	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.907	Mapped value of the risk coefficient at short periods
C_{R1}	0.893	Mapped value of the risk coefficient at a period of 1 s

Exhibit V
Shallow Footing Design


Strip Footing
q (psf)
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0
0
2
4
6
8
10
12
B (ft)


Strip Footing
settlement (in)
1.2
1
0.8
0.6
0.4
0.2
0
0
2
4
6
8
10
12
B (ft)

Project Number: 4134
Date: March 25, 2020

Mr. Ali Mozaffari
22690 Stevens Creek
Cupertino, CA 95014

Subject: Addendum to Geotechnical Report (Pavement Design) for the New Development at
22690 Stevens Creek
Cupertino, CA 95014
APN#342-14-104

Dear Sir,

In response to your inquiry, and your authorization, the following professional services were provided:

- Two sets of CBR test (ASTM D1383) have been performed on selected samples collected from the site (Please refer to the attached test results).
- Recommendations for pavement design and pedestrian concrete sidewalk.

Pavement Design and Pedestrian Rigid Concrete Sidewalk Recommendations

As previously discussed, two sets of CBR tests (per ASTM D1383) have been performed for two surficial samples collected from the Site. California Bearing Ratio (CBR) test results indicated that near surface soils have an average CBR value of approximately 1.4 that is classified as poor subgrade per Reference 1. Based on the correlations between CBR and MR (Resilient Modulus) per Reference 2, the corresponding MR and California R-Value for the surficial soil at the Site are 2025 psi and 3, respectively. The soil classification test shows the surface soil of the site is SC /CL, in Unified Soil Classification System.

Subgrade Preparation

Remove all debris, large rocks, vegetation and topsoil from the area to be paved. These items either do not compact well or cause non-uniform compaction and mat thickness.

It is recommended that the poor soil undergo subgrade treatment or replacement before placing aggregate and asphalt. For more information on subgrade treatments refer to Chapter 4.0 of Reference 1.

The subgrade should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557 to ensure the compacted subgrade is able to support construction traffic. If the subgrade ruts excessively under construction traffic, it should be repaired before being paved over. Left unrepaired subgrade ruts may reflectively cause premature pavement rutting.

It is recommended that a representative from our firm be present at the site and observe the integrity of the subgrade during the construction. In case the poor soil is present or unsuitable materials are encountered as predicted, the subgrade may require stabilization (such as lime treatment), over-excavation (and replacing the unsuitable soil with gravel borrows) and adding a base course and perhaps a subbase course over the subgrade, that proper methods will be recommended if needed during construction observation.

After final grading (often called fine-grading), the subgrade elevation should generally conform closely to the construction plan subgrade elevation. Large elevation discrepancies should not be compensated for by varying pavement or base thickness because hot mix asphalt (HMA) and aggregates are more expensive than subgrade.

Rigid Concrete Pavement Recommendation for a TI (Traffic Index) below 9

Utilizing the Reference 3 rigid pavement catalog decision tree, the site surface soil is classified as Type II of subgrade and the Site is located in Caltrans Pavement Climate Region of Central Coast. Thus, the recommended rigid pavement structural depth for TI ≤ 9, with lateral support is 0.70 ft. doweled JPCP (Jointed Plain Concrete Pavement) or 1.00 ft. AB (Class 2 Aggregate Base) and for the case without lateral support is 0.75 ft. doweled JPCP or 1.00 ft. AB and the (Table 623.1E, Reference 3).

AC (Asphalt Concrete) pavement recommendations for a TI of 5, 6 and 7

Considerations regarding worker safety, short construction windows, or the amount of area to be paved may make it desirable to reduce the total thickness of the pavement by placing full depth hot mix asphalt (HMA). Also, full depth hot mix asphalt is less affected by moisture or frost, does not allow moisture build up in the subgrade, provides no permeable layers that entrap water, and has a more uniform pavement structure. In this step of design, assuming a full depth HMA for pavement and using the Reference 3, the recommendation for AC pavement structural depth has been summarized in the table below.

Table 1- Recommended AC pavement structural depth

TI	GE [1](ft.)	HMA Thickness (ft.)
5	1.6	0.60
6	2.0	0.75
7	2.3	0.90

1) Gravel Equivalent in ft.

Proper mix of AC with Performance Grade for climate region of Central Coast can be used for full depth pavement (please refer to Table 632.1 of Reference 3).

Please note that the thicknesses determined and outlined in this section, are not intended to preclude other combinations and thicknesses of materials. Adjustments to the thickness of the various materials may be made to accommodate construction restrictions or practices, and minimize costs, provided the minimum thicknesses, maximum thicknesses, and minimum GE requirements (including safety factors) of the entire pavement structure and each layer are as specified and the contractor can modify them based on credible references as the project progresses and more data will be available (Per Reference 3).

It is our pleasure to provide you our professional services. If you have any question or need any additional information, please do not hesitate to call us at your convenience.

Sincerely Yours,

Sadaf M. Safaai P.E.
State of California Licensed Civil Engineer

Reference

1- Asphalt Paving Design Guide, Asphalt Pavement Association of Oregon, Revised October 2003.

2- FHWA, Geotechnical Aspects of Pavements Reference Manual / Participant Workbook, Publication No. FHWA NHI-05-037, May 2006.

3- California Department of Transportation, Highway Design Manual, Sixth Edition, 2017.

CBR

Test Result

Alan

Enterprise LLc

4134

C B R 1

California Bearing Ratio (CBR)

ASTM: D 1883 - AASHTO: T193-99

ACHIEVEMENT ENGINEERING CORP.

Report Date:	3/23/2020	AEC Project#:	4134		
*Client:	Alan Enterprise	Sample No.:	CBR1-1		
Report ID:	**032320 - 4134 - L50 - NH**		Page #:	1	
*Project Address:	Cupertino				

Density Data		
Condition of specimen	**Before Soaking**	**After Soaking**
Wt. of Compacted Sample, Mold and Base Plate, (Lb)	25.848	25.951
Wt. of Mold and Base Plate, (Lb)	15.83	
Wt. of Sample, (Lb)	10.018	
Height of Speciment, (in3)	4.59	
Vol. of Specimen, (in3)	6	
Moisture Content, (%)	12	
Dry Density, (Lbs/C.F)	126.72	

Expansion Ratio Determination		
Surcharge Weight, (Lb)	10	Expansion Ratio (ER):
Initial Height of Specimen, (in)	4.59	
Initial Dial Gauge Reading, (in)	0	
Final Dial Gauge Reading, (in)	0.022	
Difference, (in)	0.022	

$$ER = \left(\frac{0.022}{4.59}\right) = 0.479$$

Water Content Data		
Sample Condition:	**before Soaking**	**After Soaking**
Sample No	**CBR1-1**	
Tare No	CA - 19	
Gross wet weight	589.15	
Gross Dry Weight	562.13	
Tare Weight	336.93	
Net Dry weight	225.2	
Weight of Water	27.02	
Moisture (%)	12	

Remark:

Tested by (Name / Initial):	Nami	NH	**Signature:**	
Form#: L-50	Date Prepared: 02/11/2019		Revision No: 1	Revised: 04/06/2019

California Bearing Ratio (CBR)

ASTM: D 1883 - AASHTO: T193-99

ACHIEVEMENT ENGINEERING CORP.

Report Date:	3/23/2020	AEC Project#:	4134		
*Client:	Alan Enterprise	Sample No.:	CBR1-2		
Report ID:	**032320 - 4134 - L50 - NH**		Page #:	2	
*Project Address:	Cupertino				

Density Data		
Condition of specimen	**Before Soaking**	**After Soaking**
Wt. of Compacted Sample, Mold and Base Plate, (Lb)	26.019	25.951
Wt. of Mold and Base Plate, (Lb)	15.866	
Wt. of Sample, (Lb)	10.153	
Height of Speciment, (in3)	4.59	
Vol. of Specimen, (in3)	6	
Moisture Content, (%)	12	
Dry Density, (Lbs/C.F)	126.72	

Expansion Ratio Determination		
Surcharge Weight, (Lb)	10	Expansion Ratio (ER):
Initial Height of Specimen, (in)	4.59	$ER = \left(\frac{0.022}{4.59}\right) = 0.479$
Initial Dial Gauge Reading, (in)	0	
Final Dial Gauge Reading, (in)	0.022	
Difference, (in)	0.022	

Water Content Data		
Sample Condition:	**before Soaking**	**After Soaking**
Sample No	**CBR1-2**	
Tare No	CA - 22	
Gross wet weight	535.63	
Gross Dry Weight	514.41	
Tare Weight	336.73	
Net Dry weight	177.68	
Weight of Water	21.22	
Moisture (%)	12	

Remark:

Tested by (Name / Initial):	Nami	NH	**Signature:**	
Form#: L-50	Date Prepared: 02/11/2019		Revision No: 1	Revised: 04/06/2019

California Bearing Ratio (CBR)

ASTM: D 1883 - AASHTO: T193-99

ACHIEVEMENT ENGINEERING CORP.

Report Date:	3/23/2020	AEC Project#:	4134
*Client:	Alan Enterprise	Sample No.:	CBR1-3
Report ID:	**032320 - 4134 - L50 - NH**	Page #:	3
*Project Address:	Cupertino		

Density Data		
Condition of specimen	**Before Soaking**	**After Soaking**
Wt. of Compacted Sample, Mold and Base Plate, (Lb)	25.949	26.028
Wt. of Mold and Base Plate, (Lb)	15.792	
Wt. of Sample, (Lb)	10.157	
Height of Speciment, (in3)	4.59	
Vol. of Specimen, (in3)	6	
Moisture Content, (%)	12	
Dry Density, (Lbs/C.F)	126.72	

Expansion Ratio Determination		
Surcharge Weight, (Lb)	10	Expansion Ratio (ER):
Initial Height of Specimen, (in)	4.59	$$ER = \left(\frac{0.022}{4.59}\right) = 0.479$$
Initial Dial Gauge Reading, (in)	0	
Final Dial Gauge Reading, (in)	0.022	
Difference, (in)	0.022	

Water Content Data		
Sample Condition:	**before Soaking**	**After Soaking**
Sample No	**CBR1-3**	
Tare No	CA - 21	
Gross wet weight	526.28	
Gross Dry Weight	505.99	
Tare Weight	337.93	
Net Dry weight	168.06	
Weight of Water	20.29	
Moisture (%)	12	

Remark:

Tested by (Name / Initial):	Nami	NH	**Signature:**	

Form#: L-50	Date Prepared: 02/11/2019	Revision No: 1	Revised: 04/06/2019

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR1-1	**Sample**	CBR1-1

Test Details	
Standard	ASTM D1883-99 / AASHTO T193-98
Sample Type	Bulk disturbed sample
Sample Description	Silty Sand
Location	Cupertino
Variations from Procedure	None

Specimen & Equipment Details			
Specimen Reference	A	**Method of Sample Preparation**	ASTM D 1883
Diameter	6.0000 in		
Height	4.5900 in		
Dry Density before Soak	1.91 lb/ft3	**Dry Density after Soak**	1.91 lb/ft3
Surcharge Weight	10.0000 lb	**Comments**	
Moisture Content			
Before Compaction	1.00 %	**After Compaction**	12.00 %
Top 1" Layer after penetration	0.00 %	**Average after soak**	13.15 %

Soaking Details	
Soaking Time	96.00 hrs
Sample Weight after Soaking	10.1210 lb
Soaking Travel	0.0220 in
Swell	0.48 %

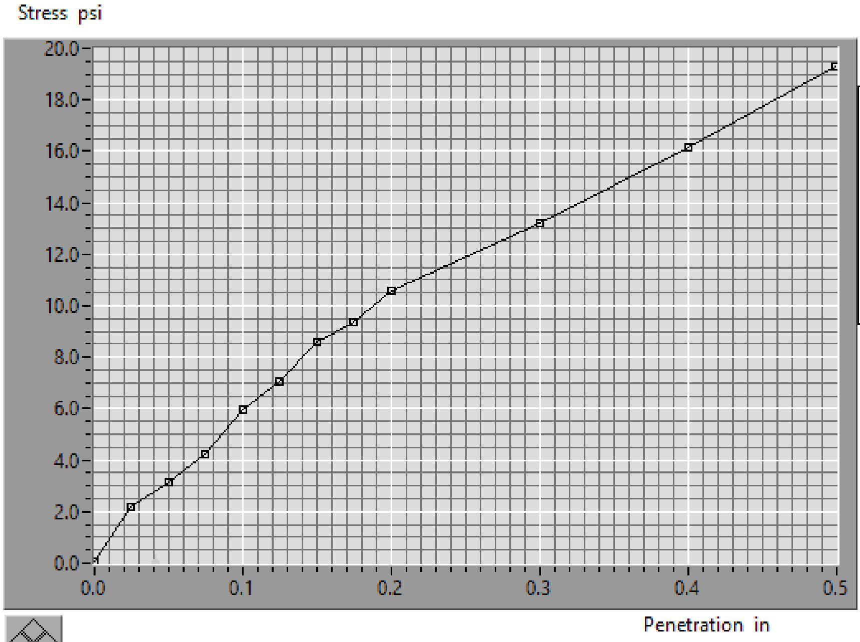
Tested By and Date:	Nami 03/23/20
Checked By and Date:	A.F 03/23/20
Approved By and Date:	S.H 03/236/20

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR1-1	**Sample**	CBR1-1

ASTM-D1883-99 / AASHTO-T193-98

Penetration Stage



Penetration	0.10 in	0.20 in
Stress	6.0 psi	10.6 psi
Standard Stress	1000.0 psi	1492.8 psi
CBR	0.6 %	0.7 %

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR1	**Sample**	CBR1-2

Test Details	
Standard	ASTM D1883-99 / AASHTO T193-98
Sample Type	Bulk disturbed sample
Sample Description	Silty Sand
Location	Cupertino
Variations from Procedure	None

Specimen & Equipment Details			
Specimen Reference	B	**Method of Sample Preparation**	ASTM D 1883
Diameter	6.0000 in		
Height	4.5900 in		
Dry Density before Soak	1.93 lb/ft3	**Dry Density after Soak**	1.93 lb/ft3
Surcharge Weight	1.0000 lb	**Comments**	
Moisture Content			
Before Compaction	1.00 %	**After Compaction**	12.00 %
Top 1” Layer after penetration	0.00 %	**Average after soak**	12.88 %

Soaking Details	
Soaking Time	96.00 hrs
Sample Weight after Soaking	10.2330 lb
Soaking Travel	0.0220 in
Swell	0.48 %

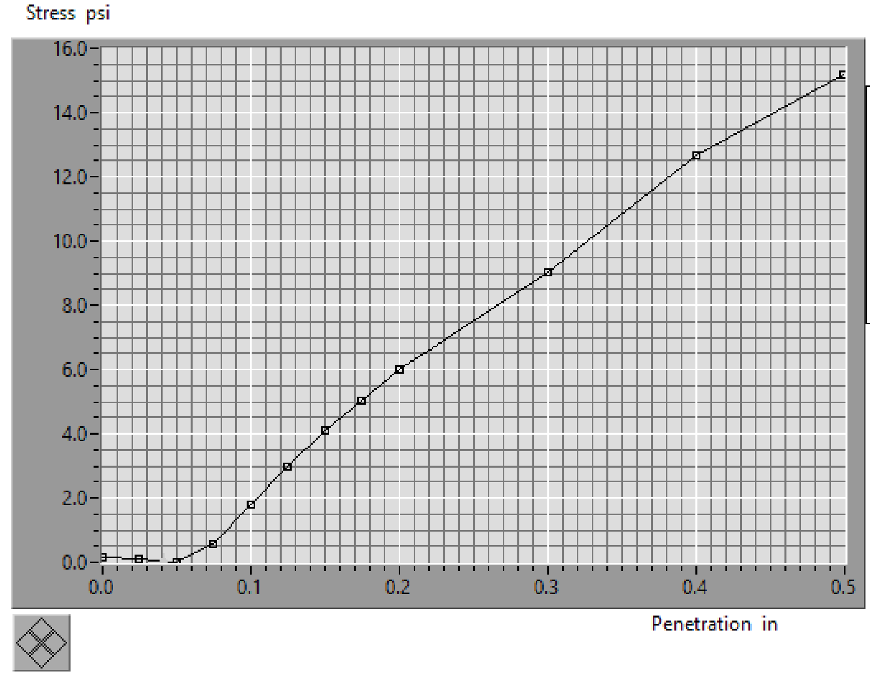
Tested By and Date:	Nami 03/23/2020
Checked By and Date:	A.F 03/23/2020
Approved By and Date:	S.H 03/23/2020

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR1	**Sample**	CBR1-2

ASTM-D1883-99 / AASHTO-T193-98

Penetration Stage



Penetration	0.10 in	0.20 in
Stress	1.8 psi	6.0 psi
Standard Stress	1000.0 psi	1492.8 psi
CBR	0.2 %	0.4 %

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR1	**Sample**	CBR1-3

Test Details	
Standard	ASTM D1883-99 / AASHTO T193-98
Sample Type	Bulk disturbed sample
Sample Description	Silty Sand
Location	Cupertino
Variations from Procedure	None

Specimen & Equipment Details			
Specimen Reference	C	**Method of Sample Preparation**	ASTM D 1883
Diameter	6.0000 in		
Height	4.5900 in		
Dry Density before Soak	1.93 lb/ft3	**Dry Density after Soak**	1.93 lb/ft3
Surcharge Weight	10.0000 lb	**Comments**	
Moisture Content			
Before Compaction	1.00 %	**After Compaction**	12.00 %
Top 1” Layer after penetration	0.00 %	**Average after soak**	12.87 %

Soaking Details	
Soaking Time	96.00 hrs
Sample Weight after Soaking	10.2360 lb
Soaking Travel	0.0220 in
Swell	0.48 %

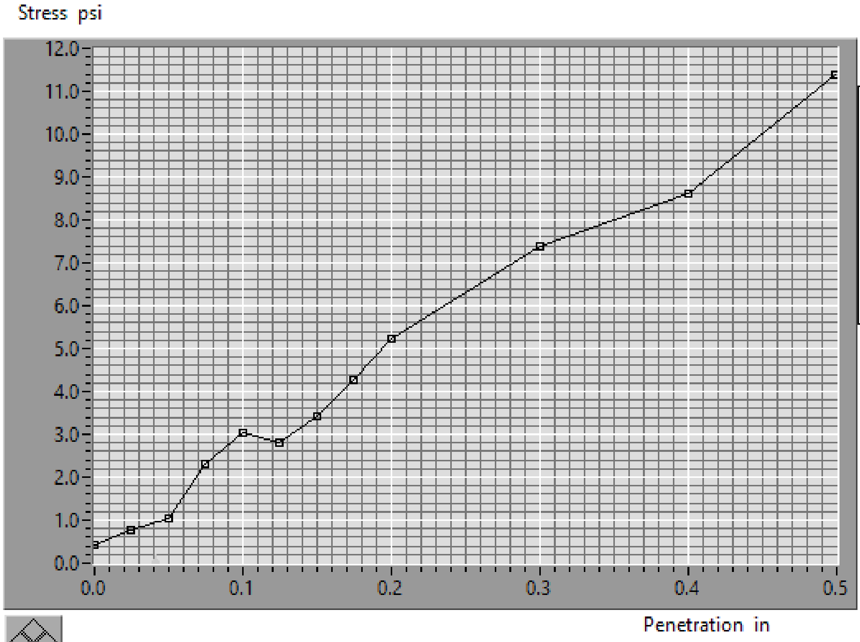
Tested By and Date:	Nami 03/23/2020
Checked By and Date:	A.F 03/23/2020
Approved By and Date:	S.H 03/23/2020

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR1	**Sample**	CBR1-3

ASTM-D1883-99 / AASHTO-T193-98

Penetration Stage



Penetration	0.10 in	0.20 in
Stress	3.0 psi	5.2 psi
Standard Stress	1000.0 psi	1492.8 psi
CBR	0.3 %	0.3 %

LABORATORY COMPACTION (Modified)
(ASTM D1557)

Report Date:	3/10/2020
Project No:	4134
Project Name:	Alan Enterprise LLC
Project Address:	22690 Stevens Creek BLVD, Cupertino
Technician	GIVO
Page #:	Page 1 of 2

Sample #:	CBR1
Sample ID:	03102020 - 4134 - Soil - CBR1
Curve No.	1

Type of Material:	Soil	Material Description:	Silty Sand
Source:	FIELD	Sample Date:	3/10/2020
Sampled by:	MOBIN		

Soil Compaction Data:

Compaction Sample No.	1	2	3	4
Weight of Wet Soil & Mold (gr)	10163.1	10431.5	10709.8	10474.9
Weight of Mold (gr)	5905.7	5905.7	5905.7	5905.7
Wet Unit Weight (pcf)	125.31	133.21	141.40	134.49
Tare Number	H-19	AE-9	H-25	H-30
Weight of Tare (gr)	126	125.6	127.5	127.3
Weight of Wet Soil & Tare (gr)	457.8	442.3	389.9	456.2
Weight of Dry Soil & Tare (gr)	438.12	416.95	362.65	413.46
Weight of Water (gr)	19.68	25.35	27.25	42.74
Weight of Dry Soil (gr)	312.12	291.35	235.15	286.16
Moisture Content %	6.31	8.70	11.59	14.94
Dry Unit Weight (pcf)	117.88	122.55	126.72	117.01

Maximum Dry Density (Lbs/C.F):	**126.72**
Optimum Mositure (%):	**12%**

	#4	3/8"	3/4"	
Weight Class	A	B	C	D
Weight (lbs)	687.9	54.3	36.6	236.5
Percentage (%)	0.678	0.053	0.036	0.233

Bulk Mass (lbs):	1015.3

List of Methods	
Method A:	No
Method A w/ Correction OR Method B:	No
Method B:	No
Method B w/ Correction OR Method C:	No
Method C:	YES
Not Applicable:	No

Rammer Type:	Mechanical

Tested By: GIVO

Reviewed By: #REF!

Signature: ____________

Signature: ____________

2455 Autumnvale Drive, Unit E | San Jose, CA 95131| Tel: (408) 217-9174 | Fax: (408) 217-9632
www.achieveng.com

ACHIEVEMENT ENGINEERING CORP.

Moisture Density
(AASHTO T265 - ASTM D2216)

Report Date: 3/23/2020
Project No: 4134
Project Name: Alan Enterprise LLC
Project Address: Cupertino
Technician: Nami

Type of Material:	Soil	Sample Description:	
Source:	Field		
Sampled by:	Nami	Sample Date:	3/8/2020

Sample No:	CBR1-1	CBR1-2	CBR1-3				
Ht. of Sample:	Disturbed	Disturbed	Disturbed				
Tare No:	CA - 19	CA - 22	CA - 21				
Gross Wet Wt:	589.15	535.63	526.28				
Gross Dry Wt:	562.13	514.41	505.99				
Tare Wt:	336.93	336.73	337.93				
Net Dry Wt:	225.20	177.68	168.06				
Wt. of Water:	27.02	21.22	20.29				
% Moisture	12%	12%	12%				
Liners Dia							
Density Factors							
Dry Density							

Tested By: Nami

Reviewed E A.F

Signature: ____________

Signature: ____________

C B R 2

California Bearing Ratio (CBR)

ASTM: D 1883 - AASHTO: T193-99

ACHIEVEMENT ENGINEERING CORP.

Report Date:	3/23/2020	AEC Project#:	4134
*Client:	Alan Enterprise	Sample No.:	CBR2-1
Report ID:	**032320 - 4134 - L50 - NH**	Page #:	4
*Project Address:	Cupertino		

Density Data

Condition of specimen	**Before Soaking**	**After Soaking**
Wt. of Compacted Sample, Mold and Base Plate, (Lb)	24.947	25.509
Wt. of Mold and Base Plate, (Lb)	15.822	
Wt. of Sample, (Lb)	9.125	
Height of Speciment, (in3)	4.59	
Vol. of Specimen, (in3)	6	
Moisture Content, (%)	11	
Dry Density, (Lbs/C.F)	123.99	

Expansion Ratio Determination

Surcharge Weight, (Lb)	10	Expansion Ratio (ER):
Initial Height of Specimen, (in)	4.59	$$ER = \left(\frac{0.048}{4.59}\right) = 1.045$$
Initial Dial Gauge Reading, (in)	0	
Final Dial Gauge Reading, (in)	0.048	
Difference, (in)	0.048	

Water Content Data

Sample Condition:	**before Soaking**	**After Soaking**
Sample No	**CBR2-1**	
Tare No	CA - 3	
Gross wet weight	861.22	
Gross Dry Weight	811.47	
Tare Weight	342.2	
Net Dry weight	469.27	
Weight of Water	49.75	
Moisture (%)	11	

Remark:

Tested by (Name / Initial):	Nami	NH	**Signature:**	

2455 Autumnvale Drive, Unit E, San Jose, CA 95131, Tel: (408) 217-9174, Fax: (408) 217-9632 / www.achieveng.com

California Bearing Ratio (CBR)

ASTM: D 1883 - AASHTO: T193-99

ACHIEVEMENT ENGINEERING CORP.

Report Date:	3/23/2020	AEC Project#:	4134
*Client:	Alan Enterprise	Sample No.:	CBR2-1
Report ID:	**032320 - 4134 - L50 - NH**	Page #:	5
*Project Address:	Cupertino		

Density Data		
Condition of specimen	**Before Soaking**	**After Soaking**
Wt. of Compacted Sample, Mold and Base Plate, (Lb)	27.418	27.697
Wt. of Mold and Base Plate, (Lb)	17.558	
Wt. of Sample, (Lb)	9.86	
Height of Speciment, (in3)	4.59	
Vol. of Specimen, (in3)	6	
Moisture Content, (%)	11	
Dry Density, (Lbs/C.F)	123.99	

Expansion Ratio Determination		
Surcharge Weight, (Lb)	10	Expansion Ratio (ER):
Initial Height of Specimen, (in)	4.59	
Initial Dial Gauge Reading, (in)	0	
Final Dial Gauge Reading, (in)	0.048	
Difference, (in)	0.048	

$$ER = \left(\frac{0.048}{4.59}\right) = 1.045$$

Water Content Data		
Sample Condition:	**before Soaking**	**After Soaking**
Sample No	**CBR2-2**	
Tare No	CA - 4	
Gross wet weight	525.94	
Gross Dry Weight	506.92	
Tare Weight	336.3	
Net Dry weight	170.62	
Weight of Water	19.02	
Moisture (%)	11	

Remark:

Tested by (Name / Initial):	Nami	NH	**Signature:**	

Form#: L-50	Date Prepared: 02/11/2019	Revision No: 1	Revised: 04/06/2019

California Bearing Ratio (CBR)

ASTM: D 1883 - AASHTO: T193-99

ACHIEVEMENT ENGINEERING CORP.

Report Date:	3/23/2020	AEC Project#:	4134
*Client:	Alan Enterprise	Sample No.:	CBR2-1
Report ID:	**032320 - 4134 - L50 - NH**	Page #:	6
*Project Address:	Cupertino		

Density Data		
Condition of specimen	**Before Soaking**	**After Soaking**
Wt. of Compacted Sample, Mold and Base Plate, (Lb)	27.798	27.969
Wt. of Mold and Base Plate, (Lb)	17.696	
Wt. of Sample, (Lb)	10.102	
Height of Speciment, (in3)	4.59	
Vol. of Specimen, (in3)	6	
Moisture Content, (%)	11	
Dry Density, (Lbs/C.F)	123.99	

Expansion Ratio Determination		
Surcharge Weight, (Lb)	10	Expansion Ratio (ER):
Initial Height of Specimen, (in)	4.59	
Initial Dial Gauge Reading, (in)	0	
Final Dial Gauge Reading, (in)	0.048	
Difference, (in)	0.048	

$$ER = \left(\frac{0.048}{4.59}\right) = 1.045$$

Water Content Data		
Sample Condition:	**before Soaking**	**After Soaking**
Sample No	**CBR2-3**	
Tare No	CA - 5	
Gross wet weight	416.63	
Gross Dry Weight	408.8	
Tare Weight	337.02	
Net Dry weight	71.78	
Weight of Water	7.83	
Moisture (%)	11	

Remark:

Tested by (Name / Initial):	Nami	NH	**Signature:**	
Form#: L-50	Date Prepared: 02/11/2019		Revision No: 1	Revised: 04/06/2019

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR2	**Sample**	CBR2-1

Test Details	
Standard	ASTM D1883-99 / AASHTO T193-98
Sample Type	Bulk disturbed sample
Sample Description	Silty Sand
Location	Cupertino
Variations from Procedure	None

Specimen & Equipment Details			
Specimen Reference	A	**Method of Sample Preparation**	ASTM D 1883
Diameter	6.0000 in		
Height	4.5900 in		
Dry Density before Soak	1.75 lb/ft3	**Dry Density after Soak**	1.75 lb/ft3
Surcharge Weight	10.0000 lb	**Comments**	
Moisture Content			
Before Compaction	1.00 %	**After Compaction**	11.00 %
Top 1" Layer after penetration	0.00 %	**Average after soak**	17.84 %

Soaking Details	
Soaking Time	96.00 hrs
Sample Weight after Soaking	9.6870 lb
Soaking Travel	0.0480 in
Swell	1.05 %

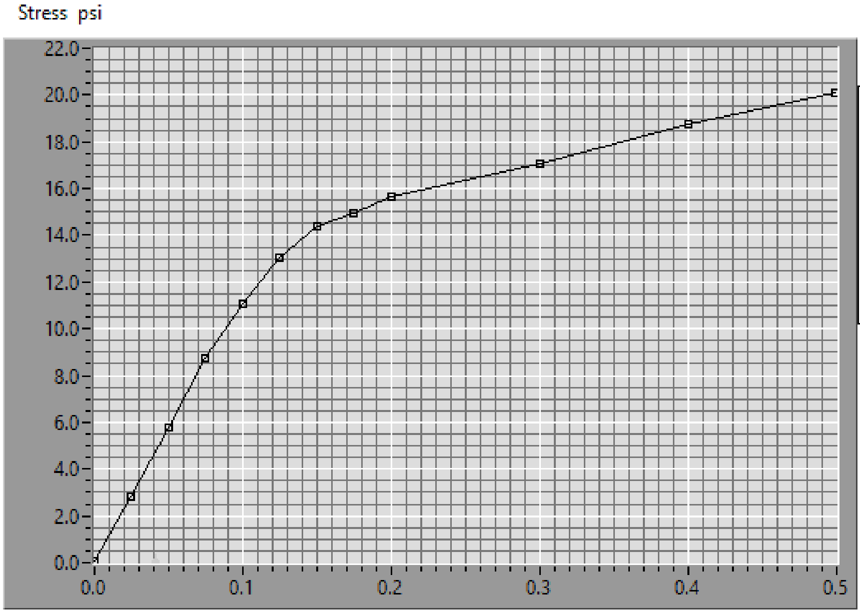
Tested By and Date:	Nami 03/23/2020
Checked By and Date:	A.F 03/23/2020
Approved By and Date:	S.H 03/23/2020

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR2	**Sample**	CBR2-1

ASTM-D1883-99 / AASHTO-T193-98

Penetration Stage



Penetration	0.10 in	0.20 in
Stress	11.1 psi	15.7 psi
Standard Stress	1000.0 psi	1492.8 psi
CBR	1.1 %	1.1 %

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR2	**Sample**	CBR2-2

Test Details	
Standard	ASTM D1883-99 / AASHTO T193-98
Sample Type	Bulk disturbed sample
Sample Description	Silty Sand
Location	Cupertino
Variations from Procedure	None

Specimen & Equipment Details			
Specimen Reference	B	**Method of Sample Preparation**	ASTM D 1883
Diameter	6.0000 in		
Height	4.5900 in		
Dry Density before Soak	1.89 lb/ft3	**Dry Density after Soak**	1.89 lb/ft3
Surcharge Weight	10.0000 lb	**Comments**	
Moisture Content			
Before Compaction	1.00 %	**After Compaction**	11.00 %
Top 1" Layer after penetration	0.00 %	**Average after soak**	14.14 %

Soaking Details	
Soaking Time	96.00 hrs
Sample Weight after Soaking	10.1390 lb
Soaking Travel	0.0480 in
Swell	1.05 %

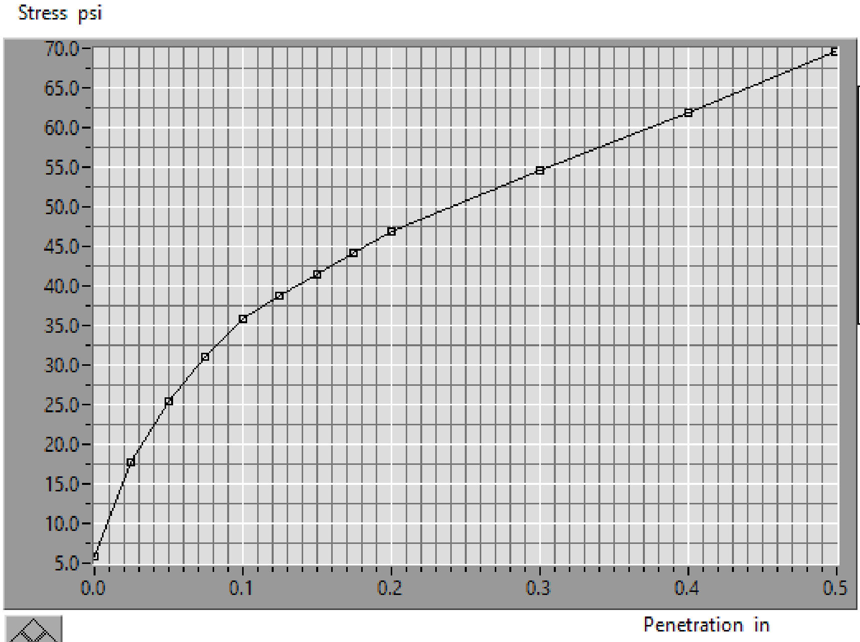
Tested By and Date:	Nami 03/23/2020
Checked By and Date:	A.F 03/23/2020
Approved By and Date:	S.H 03/23/2020

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	Lab Ref	
Project	Alan Enterprise LLC	Job	4134
Borehole	CBR2	Sample	CBR2-2

ASTM-D1883-99 / AASHTO-T193-98

Penetration Stage



Penetration	0.10 in	0.20 in
Stress	35.8 psi	46.8 psi
Standard Stress	1000.0 psi	1492.8 psi
CBR	3.6 %	3.1 %

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR2	**Sample**	CBR2-3

Test Details	
Standard	ASTM D1883-99 / AASHTO T193-98
Sample Type	Bulk disturbed sample
Sample Description	Silty Sand
Location	Cupertino
Variations from Procedure	None

Specimen & Equipment Details			
Specimen Reference	C	**Method of Sample Preparation**	ASTM D 1883
Diameter	6.0000 in		
Height	4.5900 in		
Dry Density before Soak	1.94 lb/ft3	**Dry Density after Soak**	1.94 lb/ft3
Surcharge Weight	10.0000 lb	**Comments**	
Moisture Content			
Before Compaction	1.00 %	**After Compaction**	11.00 %
Top 1” Layer after penetration	0.00 %	**Average after soak**	12.88 %

Soaking Details	
Soaking Time	96.00 hrs
Sample Weight after Soaking	10.2730 lb
Soaking Travel	0.0480 in
Swell	1.05 %

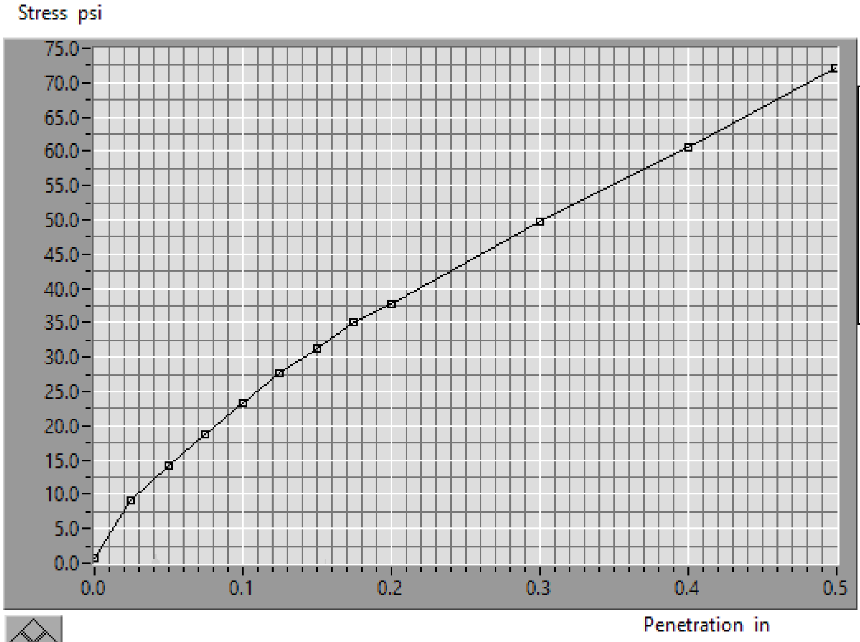
Tested By and Date:	Nami 03/23/2020
Checked By and Date:	A.F 03/23/2020
Approved By and Date:	S.H 03/23/2020

California Bearing Ratio of Laboratory Compacted Soils (CBR)

Client	AEC	**Lab Ref**	
Project	Alan Enterprise LLC	**Job**	4134
Borehole	CBR2	**Sample**	CBR2-3

ASTM-D1883-99 / AASHTO-T193-98

Penetration Stage



Penetration	0.10 in	0.20 in
Stress	23.3 psi	37.7 psi
Standard Stress	1000.0 psi	1492.8 psi
CBR	2.3 %	2.5 %

ACHIEVEMENT ENGINEERING CORP.

LABORATORY COMPACTION (Modified)
(ASTM D1557)

Report Date:	3/10/2020
Project No:	4134
Project Name:	Alan Enterprise LLC
Project Address:	22690 Stevens Creek BLVD, Cupertino
Technician	GIVO
Page #:	Page 1 of 2

Sample #:	CBR2
Sample ID:	03102020 - 4134 - Soil - CBR2
Curve No.	2

Type of Material:	Soil	Material Description:	Silty Sand
Source:	FIELD	Sample Date:	3/10/2020
Sampled by:	MOBIN		

Soil Compaction Data:

Compaction Sample No.	1	2	3	4
Weight of Wet Soil & Mold (gr)	10206.8	10503.3	10591.2	10310.9
Weight of Mold (gr)	5905.7	5905.7	5905.7	5905.7
Wet Unit Weight (pcf)	126.60	135.32	137.91	129.66
Tare Number	A-7	AE-21	H-31	H-8
Weight of Tare (gr)	134.2	126.3	127.9	129.5
Weight of Wet Soil & Tare (gr)	451.4	453.8	433.5	444.4
Weight of Dry Soil & Tare (gr)	431.75	425.35	400.95	402.1
Weight of Water (gr)	19.65	28.45	32.55	42.3
Weight of Dry Soil (gr)	297.55	299.05	273.05	272.6
Moisture Content %	6.60	9.51	11.92	15.52
Dry Unit Weight (pcf)	118.75	123.57	123.22	112.24

Maximum Dry Density (Lbs/C.F):	**123.99**
Optimum Mositure (%):	**11%**

	#4	3/8"	3/4"	
Weight Class	A	B	C	D
Weight (lbs)	756.3	37.5	45.6	219.3
Percentage (%)	0.714	0.035	0.043	0.207

Bulk Mass (lbs):	1058.7

List of Methods	
Method A:	No
Method A w/ Correction OR Method B:	No
Method B:	No
Method B w/ Correction OR Method C:	No
Method C:	YES
Not Applicable:	No

Rammer Type:	Mechanical

Tested By: GIVO

Reviewed By: #REF!

Signature:

Signature:

2455 Autumnvale Drive, Unit E | San Jose, CA 95131| Tel: (408) 217-9174 | Fax: (408) 217-9632
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Moisture Density
(AASHTO T265 - ASTM D2216)

Report Date:	3/23/2020
Project No:	4134
Project Name:	Alan Enterprise LLC
Project Address:	Cupertino
Technician:	Nami

Type of Material:	Soil	Sample Description:	
Source:	Field		
Sampled by:	Nami	Sample Date:	3/8/2020

Sample No:	CBR2 - 1	CBR2 - 2	CBR2 - 3				
Ht. of Sample:	Disturbed	Disturbed	Disturbed				
Tare No:	CA - 3	CA - 4	CA - 5				
Gross Wet Wt:	861.22	525.94	416.63				
Gross Dry Wt:	811.47	506.92	408.80				
Tare Wt:	342.20	336.30	337.02				
Net Dry Wt:	469.27	170.62	71.78				
Wt. of Water:	49.75	19.02	7.83				
% Moisture	11%	11%	11%				
Liners Dia							
Density Factors							
Dry Density							

Tested By: Nami   Reviewed E A.F

Signature: ____________   Signature: ____________

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