California Engineering Co. **CEICO**

GEOTECHNICAL ENGINEERS

GEOTECHNICAL SOIL REPORT

Foundation For Retaing Walls & Additions

Prepared For

Mr. Justin Carter
152 Porteous Ave.
Fairfax, CA 94930

Property Address
152 Porteous Ave., Fairfax, CA 94930

April 18 2021

Revised Feb., 06, 2022

ATTACHMENT B

California Engineering Co. **CEICO**

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Civil Structural, Geotechnical Foundation, Electrical & Mechanical Engineering CE20859, GE464, EE 23017, ME31430

April 05, 2022
Our Reference CEICO 03162021

Mr. Justin Carter
152 Porteous Ave.
Fairfax, CA 94930

Subject: Response to Second Review Letter by Miller Pacific Engineering Group dated March 18, 2022 for Soil Report dated Feb. 06, 2021 for 152 Porteous Ave., Fairfax, CA 94930

Dear Mr. Justin Carter

We thank you for forwarding yesterday the second review letter my Miller Pacific Engineering Group. We thank the reviewer for the valuable time spent to help us in your project.

We are answering each item as listed.

Items 1 and 2

Reference is made to Soil Report Section 3.1 Seismicity

"... Site Classification "D" Stiff Soil profile based on boring PA-2 (15<N<50), because boring PA-1 (N>50) was drilled at the toe of a 7' cut upslope behind present residence and if it was drilled from ground surface similar results to PA-2 would have been obtained. The site soil profile used is justified to be "D"..."

The present soil inclination of 150/80 approximated 2:1 represent the recommended allowable soil inclination. Any steeper soil inclination during construction will need shoring consideration and shoring structural design.

Item 3

The soil report can be revised through an Addendum No 1 to clarify this point.

"...The existing native soil on top of any cut may need to be excavated to the depth of 1- 2 foot, if directed by the site geotechnical engineer following soil inspection, and soil hauled away. If any, imported engineered fill or recycled concrete can be used in place of the excavated 2 foot of native soil..."

Immediately following the above statement ".. The soil observed behind the residence through the 6-7' cut is very dense and can be used without importing fill..."

The intention is not for site safety or instability but to conform to OSHA where applicable. This can be added as an Addendum No. 1 to the soil Report if found necessary.

Item 4

Soil Report Section 5.3.3 Mat foundation, will be amended through Addendum No.1 to include : ".. Stego Industries membranes underlain by a select granular fill for capillary break consisting of 3/4" drain rock compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557-78.

Item 5

Soil Report Section 5.4 Retaining walls will be amended through Addendum No.1.

The following at end of first paragraph will be deleted due to typing error: "...and / or be designed for the lateral pressure generated from the 95% relative compaction needed for the driveway and based on ASTM TD D-1557."

Item 6

There was no evidence of soil slide, erosion or soil creep at the site and behind the residence. The hill side beyond was and is still stable for years and 75' above the residence, two flat benches are present for potential new residences. There no visual indication of soil instability or reported historical adverse behavior. The soil properties as analyzed in borings PA-1 at the toe of a 7' cut in the following boring log

Drilled: 4/12/2021

BORING PA-1

DESCRIPTION	USCS Symbol	DEPTH FEET	SAMPLE NUMBER	N BLOWS PER FOOT	qu TSF	MOISTURE CONTENT %	DRY DENSITY PCF	NOTES
Tan brown silty sandy rocks, very dense	GM	1	1-1	51	4	7	115.6	This layer is 5-7'
Tan brown silty sandy rocks, very dense	GM	2	1-2	48	4	5.7	129	excavated cut
Tan brown silty sandy rocks, very dense	GM	3		69				
		4	1-3	61	4	7	119.1	Sieve Analysis
Tan brown silty sandy rocks, very dense		5		61				
	GM	6	1-4	62	4	6.3	112.6	
Practical refusal @ 5'-9"								

supports the slope stability, not to impact the planned improvements.

If the reviewer is requesting a soil slope stability analysis, which is not part of the scope of this soil report, we will request the kind services of Dr. Robert Pyke, specializing in soil slope stability assessment and calculations; we have already shared our soil report with him.

The 7' cut behind the residence has been there for few years, resisted the October 2021 severe storms. Based on the data we have on hand, our exploration and physical inspection, we do not foresee soil instability potential problem around the proposed expansion.

With due respect to the reviewer findings, this project does not have an ADU nor it is part of Manor Drive as inadvertently mentioned in the review.

Very Truly Yours,

Michel S. Khoury, MSCE, P.E.
California Geotechnical Engineering License #464

REGISTERED PROFESSIONAL ENGINEER
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April 18, 2021
Our Reference CEICO 03162021
Revised Jan 06, 2022

Mr. Justin Carter
152 Porteous Ave.
Fairfax, CA 94930

Subject: Revised Geotechnical Soil Report for Foundation For
Retaining Walls & Additions
for 152 Porteous Ave., Fairfax, CA 94930

Dear Mr. Justin Carter

We have revised herewith our Geotechnical Soil Report following the Peer Review by Miller Pacific Engineering Group dated Jan., 31 2022; we tried to respond to their valuable comments and thank them for their guidance.

In accordance with your authorization, we have conducted a Geotechnical Field Investigation for the above subject property on April 12, 2021. This report presents the results of our surface and subsurface investigations and is in conformance with the Zoning Requirements for Seismic Hazards Investigation, the State Hazards Mapping Act, California Geological Survey, 2019 California Building Code & USGS.

Based on our field, laboratory and office studies, it is our opinion that competent soil material was found to recommend the most suitable foundation for retaing walls and additions for 152 Porteous Ave., Fairfax, CA 94930. The site is stable, has very low liquefaction potential / susceptibility, is not in a slide area and has no recent history of seismic activity.

We will be happy to provide you with our additional engineering services for the foundation and expansion construction supervision. We would appreciate at least 24 hours notice for our observations during construction.

Our recommendations are contained in this report. Please do not hesitate to give us a call if you have any question.

REGISTERED PROFESSIONAL ENGINEER
MICHEL S. KHOURY
No. 464
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Very Truly Yours,

Michel S. Khoury, MSCE, P.E.
California Geotechnical Engineering License #464
California Civil Engineering License #20859 & #88815

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ATTACHMENTS

1. PURPOSE AND SCOPE OF WORK

This soil report is hereby revised to incorporate comments and answers to Peer Review by Miller Pacific Engineering group dated Jan., 31, 2022.

This Geotechnical Investigation is for the proposed foundation for retaing walls and additions at 152 Porteous Ave., Fairfax, CA 94930, performed on April 12, 2021. The main purpose for the geotechnical investigation is to provide information about the geological conditions of the site and to recommend the most optimized foundation for this project. The scope of work for structural design and construction is by a third party engineering consulting and construction companies; our firm will coordinate with the designer, the foundation design, and with the contractor all the aspects of the works including construction supervision and certification.

Based on our field, laboratory and office studies, it is our opinion that competent soil material was found to design a suitable foundation for the proposed works. This residence was built in 1948, we have observed no geological change in this site as a result of the Loma Prieta earthquake of October 1989. We are providing herewith, soil-engineering information relative to the planned development.

The scope of our work consisted of:

1) Review, compilation, and interpretation of available seismic, hydrologic, and geologic literature including maps pertinent to the site.
2) Field investigations including surface visual inspections and exploratory subsurface boring of two bore holes drilled to depths of: CC-1: 5'-9", and CC-2: 6' below ground elevation and the extraction of 8 samples for Laboratory testing to assess the engineering properties of the soil samples.
3) Field Investigation for the proposed foundation.
4) Literature review and research including: US Geological Survey (USGS), California Division of Mines and Geology (CDMG), and Dept. of Housing and Urban Development (DHUD) at UC Berkeley Library McCone Hall.
5) Examination of soil slides, liquefaction potential, expansive and corrosive soil, seismic induced settlement and settlement records, if any.
6) Compilation of seismic factors based on the soil geology and site location from an active fault Using USGS hazard data (http://earthquake.usgs.gov/research/hazmaps), and U.S. Seismic Design Map by SEAOC/OSHPD (http://seismicmaps.org)
7) Conformance with the Zoning Requirements for Seismic Hazards Investigation, the State Hazards Mapping Act, California Geological Survey, 2019 California Building Code and USGS.
8) Development of recommendations for designs and preparation of this engineering report.

Our investigation was conducted in conformance with generally accepted geotechnical engineering principles & practices, and in accordance with the standards of practice as set by the geotechnical engineers in the area.

2. GENERAL BACKGROUND

A brief description of site location and proposed construction are given below.

2.1 SITE LOCATION

The site is at 152 Porteous Ave., Fairfax, CA 94930

2.2 PROPOSED CONSTRUCTION

Foundation for retaining walls and additions.

LITERATURE REVIEW

Available literature, maps, and miscellaneous data pertinent to the site were reviewed, compiled, and interpreted. The information was mainly obtained from the sources listed below:

a) U.S. Geological Survey (USGS), U.S. Department of Agriculture, Soil Conservation Service (SCS), and Dept. of Housing and Urban Development (DHUD)
b) State geologic agencies, such as California Division of Mines and Geology (CDMG)
c) CGS, California Geological Survey
d) U.C. Berkeley Earth Science Library, McCone Hall.

3.1 SEISMICITY

As with the rest of the San Francisco Bay Area, the site is considered to be in one of the seismically active regions of the United States. The nearest trace of active fault is the San Andreas Fault at 6.50 Miles southwest from the site (Figure 2).

Although research on earthquake prediction has greatly increased in recent years, seismologists have not yet reached the point where they can accurately predict when and where an earthquake will occur. Nevertheless, on the basis of current technology, and U.S. Geological Survey compiled data, it is reasonable to assume that the proposed structures will be subjected to at least one moderate or severe earthquake. The building or addition to the existing building should be structurally designed to withstand such earthquake.

Five earthquakes with energies exceeding Richter magnitude 7 have occurred in the last 150 years within the San Francisco Bay Area. The 1906 and 1989 San Andreas Fault earthquakes were the last of these major events. Moderate earth tremors and a slow creeping movement on some active Bay Area faults evidence present day seismicity. The maximum probable seismic shock to be expected from the Hayward Fault is a 7.2 magnitude event on the Richter scale. The site and general area could experience bedrock accelerations of 0.4 to 0.5 g. The site may be considered to be a firm site with a Characteristic Site Period (Ts) equal to 0.5 seconds and shock duration of 0.5 minutes.

It should be clearly understood that California and especially the greater San Francisco Bay

Area is an area of higher seismic risk. It should also be realized that, in general, it is not economically feasible to build totally earthquake-resistant structures that would be resistant to any and all earthquakes. Therefore it is possible that if a large or close earthquake to this site occurred, the site and structure could be damaged and there is an irreducible risk associated with living in a seismically active area such as California with many active faults.

Reference is made to the California Building Code CBC 2019, USGS, SEAOC/OSHPD, OSHPD Seismic Design Maps (http://seismicmaps.org), and Third Party Graphical User Interface GUI's. Based on the latitude & longitude of the site at 152 Porteous Ave., Fairfax, CA 94930 and the site characteristics, the seismic factors are determined as follows: Latitude 37.9764608 degree North, Longitude -122.5909348 degree West. Site Classification "D" Stiff Soil profile based on boring PA-2 (15>N>50), because boring PA-1 (N>50) was drilled at the toe of a 7' cut upslope behind present residence and if it was drilled from ground surface similar results to PA-2 would have been obtained. The site soil profile used is justified to be "D". The soil factors including maximum accelerations and design accelerations are compiled as function of the spectral acceleration and they are:

Site Coefficients Fa=1.2, Fv=null

Spectral acceleration for 0.2 sec. period (short)	Ss=1.5g
Spectral acceleration for 1 sec. period	S1=0.6g
Maximum spectral response acceleration (short)	SMs=FaSs=1.8g
Maximum spectral response acceleration	SM1=FvS1=null
Design spectral response acceleration (short)	SDs=2/3SMs=1.2g
Design spectral response acceleration	SD1=2/3SM1=null

3.2 GEOLOGY

Published data does not indicate the presence of any significant geological problems associated with the site. The site is mapped by the USGS, Geologic Map of Marin County, Google Earth, most of the site is moderately to steeply- sloping, and being underlain by map unit "fsr"; the site is also mapped by CGS, Geologic Map of California, as "kJfm" Franciscan Complex metamorphic rocks, mélange of fragmented and sheared Franciscan Complex rocks (Figures 5 and 5-1).

3.3 HYDROLOGY

Published data does not indicate the presence of any significant hydrological problems associated with the site.

Rainfall at the site is about 39-41 inches per year (Figure 6), with about 80% of the rain falling between the months of November and April. This amount of rainfall is about average for the Bay Area, which receives from 14 inches per year along the Bay Shore, to a maximum of 40 inches in the hills.

3.4 LIQUEFACTION POTENTIAL

Soil liquefaction describes a phenomenon whereby a saturated or partially saturated soil temporarily loses strength and acts as a fluid in response to an applied cyclic stress. The phenomenon is most often observed in saturated, loose low density or un-compacted, sandy soil. Shaking experienced at the subject site depends strongly on the type of deposits found near the surface. Generally there are three factors that need to take place for liquefaction to occur.

1. Loose, granular sediment
2. Saturation of the sediment
3. Strong shaking

Liquefaction is a relatively near-surface phenomenon in flat terrain, often with no visible effects if it occurs under more than approximately 4-5 meters of cover (Ishihara, 1985). Kishida (1970) observed the grain size distribution of boils ejected at Nanaehama Beach, Japan during the Tokachioki earthquake of 1968. Figueroa et al. (1995) examined the grain size distribution of soil samples collected from liquefaction related sand boils generated at the Lower San Fernando Dam, California during the Northridge earthquake of 1994. The grain size distribution indicates that the soil liquefying was very silty sand with clay content less than 10 percent. General consensus from studies indicate Clay Content <15 percent has a tendency to liquefy provided there is a relatively high ground water table and the cyclic stresses induced in the soil have sufficient intensity. The soil mechanics community generally agree based upon analytical study and instrumental records, that earthquake magnitude, distance from the hypocenter and local subsurface conditions are the three major factors that affect the seismic intensity at a site.

The USGS show the site to be in very low Liquefaction Susceptibility, Reference: Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine County San Francisco Bay Region, California by Keith Knudsen et al 2000. The site is 153 feet away from a site of moderate Liquefaction Susceptibility. Reference: USGS, Geologic Map of Marin County, Google Earth (References Figure 5A & 5B)

3.5 SEISMIC INDUCED SETTLEMENT

A) Seismic consideration

San Andreas Fault: The northern leg of the fault runs from Hollister, through the Santa Cruz Mountains, epicenter of the 1989 Loma Prieta earthquake, then up the San Francisco Peninsula, where it was first identified by Professor Lawson in 1895, then offshore at Daly City near Mussel Rock. This is the approximate location of the epicenter of the 1906 San Francisco earthquake. The fault returns onshore at Bolinas Lagoon just north of Stinson Beach in Marin County. It returns underwater through the linear trough of Tomales Bay which separates the Point Reyes Peninsula from the mainland, runs just east of Bodega Head through Bodega Bay and back underwater, returning onshore at Fort Ross. (In this region around the San Francisco Bay Area several significant "sister faults" run more-or-less parallel, and each of these can create significantly destructive earthquakes.) From Fort Ross, the northern segment continues overland, forming in part a linear valley through which the Gualala River flows. It goes back offshore at Point Arena. After that, it runs underwater along the coast until it nears Cape Mendocino, where it begins to bend to the west, terminating at the Mendocino Triple Junction

B) Settlement Consideration

Settlement is broadly classified as total settlement and differential (uneven) settlement. Total settlement refers to the uniform settlement of the entire structure and occurs due to weight of the structure and imposed loads. Differential or uneven settlement can occur if the loads on the structure are unevenly distributed, variations in the soil properties or due to construction related variations.

3.6 EXPANSIVE AND CORROSIVE SOIL

The Atterberg limits were only tested on sample PA-2-1@2' with the results: LL 0.00 PL 0.00 and PI 0.00., Sample PA-2-3 @4' is the only sample among 8 with some clay identified as " silty sandy clay"; it is noted that soil in the area to be slightly to moderately plastic. Creep design for drilled pier is included based on possibility of other silty sandy clay soil may exist on the site, for safety.

The soil in the development area for 152 Porteous Ave., Fairfax, CA 94930 is not included to be a corrosive soil as surveyed by Reconnaissance Soil Survey of the San Francisco Bay Area Region and Contra Costa County, California Corrosive Soil Survey by USDA, E.J Carpenter and S.W. Cobby University of California 1938.

For additional safety and certainty, we recommend soil corrosion testing be performed before installing steel piping that go through the native soil. In the event the soil has traces of corrosion, Portland cement Type II with moderate sulfate resistance or Portland Cement Type V high sulfate resistance must be used depending on the sample testing results.

4. SITE INVESTIGATIONS

Our site investigations, performed by our engineers on April 12, 2021, consisted of surface site reconnaissance, inspection and subsurface exploration, drilling and sampling to very dense soil strata.

4.1 SURFACE FEATURES AND CONDITIONS

A surface reconnaissance of the site was performed to evaluate the surface site conditions and observe if any obvious indications of geotechnical or drainage problems were present. In addition, slopes on or adjacent to the site were also examined to provide supplemental information on the character of exposed soil materials including bedrock outcrops if any.

The site appears not to have been significantly affected by the wet winters, and the El Nino of 1998.

4.2 SUBSURFACE INVESTIGATION

We have conducted on April 12, 2021 a subsurface investigation of this site at 152 Porteous Ave., Fairfax, CA 94930. The investigation consisted of two boreholes: PA-1, and PA-2. The logs of the boring are attached to this report (Figures 12, and 13). The test boring PA-1 (Lat 37.9766397 N, Lon -122.590997 W) was terminated at the depth of 5'-9", the soil encountered at the bottom of the hole was, very dense silty sandy rocks. The test boring PA-2 (Lat 37.976575 N, Lon -122.590614 W) was terminated at the depth of 6', the soil encountered at the bottom of the hole was, very dense tan brown silty sand. (Unified Classification ASTM D-2487).

Sieve Analysis is made to aid in the classification of the soil, it was performed on Sample PA-1-3@4 foot with the following results:

<u>Sieve Analysis % of Sample Passing PA-1-3@4'</u>

Passing No. 30	62.21%
Passing No. 50	68.04%
Passing No. 100	74.75%
Passing No. 200	79.43%

<u>Sieve Analysis % of Soil Passing PA-1-3@4'</u>

Passing No. 30	37.89%
Passing No. 50	31.96%
Passing No. 100	25.25%
Passing No. 200	20.57%

The Atterberg Plastic Limit / Liquid Limit / Plasticity Index test is made to evaluate the soil expansive potential and it was performed on sample PA-2-1@2 foot with the following results:

<u>Atterberg Limits PA-2-1@2'</u>
Non- Plastic

The results encountered in all the referenced logs are fundamentally similar in nature. Our site reconnaissance and subsurface exploration confirmed that the materials shown on published geological maps were present. Based on the consistency of the soil materials and the results obtained from the boring at the site, **<u>it is our professional opinion that no additional exploration, boring or laboratory testing</u>** is necessary at this time for all practical purpose.

We wish to point out that the attached test logs and related information depict subsurface conditions only at the approximate locations shown on the Boring Location Figure and on the dates designated on the logs. Subsurface conditions at other locations and times will differ somewhat from the conditions occurring at our test locations.

5. CONCLUSIONS, RECOMMENDATIONS, AND DESIGN GUIDELINES

Based on our field and office studies, it is our opinion that from a soil and foundation engineering standpoint, foundation for retaining walls and additions to the residence can be performed using the parameters developed in this report. The proposed foundation types are designed to resist earthquake forces utilizing seismic factors derived from USGS Earthquake Hazards Programs and SEAOC/OSHPD (http://seismicmaps.org)

Detailed soil and foundation engineering for use in the design and construction of the proposed works are presented in the subsequent sections of this report. It is recommended that a geotechnical engineer be retained to:

(1) Review the soil engineering aspects of foundation plans prior to construction
(2) Observe the general excavation
(3) Observe the placement of concrete
(4) Observe the sub grade conditions in the foundation excavations and/or pier drilling
(5) Observe the surface drainage facilities installed.

5.1 SITE PREPARATION AND EARTHWORK OPERATIONS

All sub-grade surfaces that will receive structural or engineered fill should be scarified, moisture-conditioned wet of optimum and compacted to the requirements given below. The proposed structural fill to be used should be tested for compaction characteristics.

A soil vertical cut upslope of about 6-7 feet was made behind the residence and the soil profile is visible, portable "Torvane" Tests yielded to over 4 TSF, and the cut is standing stable through time without the need of any retention as the blow counts at the bottom of the cut was N=51 for the first foot drilled. Native sandy soil, medium dense, was present on the top of the cut. The existing native soil on top of any cut may need to be excavated to the depth of 1- 2 foot, if directed by the site geotechnical engineer following soil inspection, and soil hauled away. If any, imported engineered fill or recycled concrete can be used in place of the excavated 2 foot of native soil. The soil observed behind the residence through the 6-7' cut is very dense and can be used without importing fill, but if soil is imported, the fill used at the site should be a non-expansive soil with a plasticity index of 12 or less. All fill and backfill materials placed at the site should not contain rocks for lumps greater than 6 inches in greatest dimension with no more than 15 percent larger than 2.5 inches.

If applicable, all structural fill and backfill materials placed at the site should be compacted to at least 90 percent relative compaction by mechanical means only as determined by ASTM Test Designation D1557-78. The upper six inches of sub grade should be compacted to 95% density. The fill and backfill materials should be spread and compacted in lifts not exceeding 6 inches in thickness.

The exposed sub-grade soils under grade beams shall be pre-wetted 24 and 12 hours prior to pouring concrete. The pre-wetting of the soils is intended to reduce the expansive potential of the soils by increasing the moisture content to depths of 2 foot or greater.

5.2 DRAINAGE

It should be realized that considerable amount of runoff water from prolonged and intense rainfall flows along the surface of the ground and down the slope. A significant amount of water may percolate through the upper portions of the top soil materials, then flows along the surface of impervious soil layers or along the surface of the bedrock because the bedrock is much more dense and compact than the above soil materials. Improvement of both the surface and subsurface drainage conditions is necessary to insure the stability of the site and to improve hydrologic conditions.

5.2.1 SUBSURFACE DRAINAGE

In general, positive surface drainage should be provided adjacent to the structures as to direct surface water away from foundations to suitable discharge facilities. Water should not be allowed to flow over the tops of any slopes and pooling of surface water should not be allowed adjacent to the structures. Water seeps underground at various depths depending on the nature of the soil and its geological formation.

5.3 FOUNDATIONS

The foundation design must include the following:

1. For the retaining walls, drilled-in cast-in place reinforced concrete friction piers, drilled into the very dense tan brown sandy rocks or silty sand strata and connected together on top with grade beams forming a grid to provide collectively confinement and better resistance to earthquake forces

2. For the additions, a mat foundation resting on grade beams supported by drilled-in cast-in place reinforced concrete friction piers, drilled into the very dense tan brown sandy rocks soil strata. The grade beams can be integrated inside the mat forming a moment frame to provide collectively confinement and better resistance to earthquake forces.

5.3.1 DRILLED PIER FOUNDATION

The foundation must be supported on drilled cast-in-place straight-shaft friction piers that are designed to develop their load-carrying capacity through friction between the sides of the piers and the surrounding subsurface materials and through end bearing.

Friction piers should have a minimum diameter of 12 inch The depth of the piers must extend to a minimum depth of 11 foot or deeper below grade depending on the site condition and geotechnical engineer observation during the drilling operations.

The actual diameter and depth of the piers can be determined using an allowable skin friction value of 500 pounds per square foot for dead plus live loads with a one-third increase for all loads including wind or seismic, the upper 2 foot around the pier must be neglected. Up to 1/2 of the downward pier load capacity may be used for uplift. Pier lengths should be checked under lateral, vertical, and uplift loading conditions. Soil creep will develop on slopes as a result of moisture fluctuations, which cause the soil to swell and shrink. Although the predominate soil on site is silty sand and silty sandy rocks without creep properties, we are specifying creep pressure due to one sample had clay and for safety. Creep pressures on piers and foundation grade beams, may be approximated by a uniform lateral pressure of 150 pounds per square foot acting in the top two feet of soil.

We recommend that all the piers be reinforced with at least No. 5 bars that extend to the depth of the pier holes, with #3 ties at 10-inch centers or spiral ties . We also recommend that all piers be tied together with tie beams or grade beams between piers. Grade beams should be at least 12 inch wide, 12 inch deep and contain 2#5 bars top and bottom with #3 ties at 10-inch centers. Good foundation continuity should be provided with 30-inch bends at all grade beam corners and intersections. We also recommend that the steel from the piers extend sufficient distance into tie beams and grade beams to develop its full strength in bond. In addition, we recommend that all grade beams be designed to span between the piers in accordance with structural requirements. We recommend that the steel from the piers extend sufficient distance into tie beams and grade beams to develop its full strength in bond. Structural wood joist floors should be used to support the residence and should derive all their support from the pier and grade beam foundation.

The above Minimum pier and grade beam sizes and reinforcements are based upon geotechnical engineering considerations and should not be reduced by the structural engineer, without concurrence with the geotechnical engineer. The foundation should also be designed to resist the minimum loads as required by the 2016/2019 California Building Code.

Pier holes should be drilled plumb and cleaned of loose soil and standing water. We judge that pier foundations can possibly be drilled using conventional heavy-auger drilling equipment. However, several zones of relatively hard sandstone / siltstone layers could be encountered. The foundation contractor should be prepared to utilize suitable hard rock drilling techniques (such as core barrels), if necessary. Even though the piers will be designed to develop their capacity through friction, their bottoms should be dry and reasonably free of loose cuttings prior to installing reinforcing steel and placing concrete. We wish to point out that some of the pier holes may encounter refusal, with the drilling equipment, short of their design depths; our engineers will evaluate such piers on an individual basis at the time of drilling. It is very essential that we monitor/supervise the pier drilling operation to insure that sufficient penetration to bedrock is properly achieved. Concrete in uncased holes shall be vibrated continuously as the concrete is poured, using a standard internal vibrator. Adequate vibration is necessary for the development of skin friction stress on the pier shaft. If pier shafts will not stand open, temporary casing may be necessary to support the sides of the pier shafts until concrete is placed. Drilling to achieve the required depth into bedrock may require an increase in time and effort because of variable hardness. The exposed sub grade soils under grade beams shall be pre-wetted 24 and 12 hours prior to pouring concrete. The pre-wetting of the soils is intended to reduce the expansive potential of the soils by increasing the moisture content to depths of 2' or greater.

5.3.2 GRADE BEAMS FOUNDATION

Grade beams connecting the Concrete Piers should be used for additional stability against lateral earthquake forces and sliding. The grade beams must form a grid if practical, within the footprint. The grade beams should be at least 12 inch wide, 12 inch deep and contain 2#5 bars top and bottom with #3 ties at 10-inch centers. Good foundation continuity should be provided with interfacing the existing piers. Depending on the structural engineer design the re bars should extend sufficient distance into existing piers.

The above **Minimum Grade Beam** sizes and reinforcements are based upon geotechnical engineering considerations and can be reduced by the structural engineer with coordination with the geotechnical engineer. The foundation should also be designed to resist the minimum loads as required by the California Building Code 2016/2019.

5.3.3 MAT FOUNDATION

Mat foundation should be supported on a minimum of 18 inch of imported engineering fill unless the excavation shows good native soil as approved by the geotechnical engineer. The Mat shall be a minimum of 10 inch thick with a minimum reinforcing steel bars #5@12" both ways top and bottom and an allowable soil bearing pressure of 2500 pounds per square foot. The upper 6 inches of the soil materials should be compacted to at least 95 percent relative compaction as determined by ASTM Test Designation D1557-78. In any mat area where mat wetness would be undesirable, we recommend that an impermeable membrane be placed over the free draining gravel and that the membrane be covered with 2 inches of sand to protect it during construction. The membrane must be a minimum of 12 mils similar to Stego Industries membranes underlain by a select granular fill for capillary break consisting of Class 2 aggregates baserock 6-8" thick.

The grade beams connecting the piers together on top can be integrated within the mat foundation with proper steel rebars distribution.

5.4 RETAINING WALLS

Retaining walls, must be supported on reinforced concrete drilled-in pier foundation. We recommend that **unrestrained walls** with a level surface or with a sloping surface flatter than 4:1 are designed to resist an equivalent pressure of 50 pounds per cubic foot. Where the sloping surface is at an inclination or 2:1 or slightly steeper, the unrestrained walls should be designed to resist an equivalent fluid pressure of 70 pounds per cubic foot. For walls with a sloping surface at an inclination of between 4:1 and 2:1, a straight-line interpolation between the 50 and 70 pounds per cubic foot may be used, and / or be designed for the lateral pressure generated from the 95% relative compaction needed for the driveway and based on ASTM TD D-1557.

We recommend that **restrained walls** (non-yielding / rigid) constrained against movement subjected to at-rest lateral earth pressures equivalent to a fluid weighing 70 pounds per cubic foot where the back slope is level; 80 pounds per cubic foot, where the back slope is inclined at 3:1; or 90 pounds per cubic foot, where the back slope is inclined at 2:1. For walls with a sloping surface at an inclination of between 4:1 and 2:1, a straight-line interpolation between the 70 and 90 pounds per cubic foot may be used.

If the structural engineer determines that they are any additional surcharge loads on the walls, the unrestrained walls should also be designed to resist an additional uniform pressure equivalent to one-third the maximum anticipated surcharge load applied at the surface behind the walls. The structural engineer must determine in his calculation the safest method in his design using the above generated loads or the "Lateral Load Resistance" presented below.

The above pressures assume that sufficient drainage will be provided behind the walls to prevent the build-up of hydrostatic pressures from the surface / subsurface water infiltration.

5.5 LATERAL LOAD RESISTANCE

Lateral loads may be resisted by passive pressures acting against the sides of the piers / grade beams / walls. We recommend a passive pressure equal to an equivalent fluid weighing 300 pounds per square foot per foot of depth to a maximum of 1500 pounds per square foot. This value can be acting against 1.5 times the diameter of the individual pile starting at a depth of 2 feet below the bottom of the grade beams.

Lateral loads on piers / anchors/ footings / grade beams may be resisted by (1) Friction between the foundation bottoms and the supporting sub grade materials.(2) Passive pressures acting against the sides of the footings. We recommend the following coefficient of friction and passive pressures:

Depth Below Existing Site Grades (feet)	Coefficient of Friction	Passive Pressures (Pounds Per Cubic Foot)
2 to 3	0.25	300
3 to 6	0.30	400
6 to 9	0.35	500
Below 9	0.40	600

We wish to note that the passive pressures presented above can be increased to a maximum value of 1500 pounds per square foot.

We recommend that the factor of safety against sliding and over-turning for all retaining walls be at least 1.5. We wish to note that when calculating the weight of soil on the portion of the protruding wall footing where backfill will be placed, (i.e. the weight of soil that will resist over-turning forces), an imaginary line at an inclination of 10 degrees from vertical can be used starting at the top of the wall footing. In addition, the backfill materials can be assumed to have a unit weight of at least 110 pounds per cubic foot.

Design Example:

The retaining wall design with active and seismic pressures shall be compared with a retaining wall design of equivalent height using only active pressure resulting in a uniform lateral pressure composed of two triangular distributions with one inverted triangular distribution.

Seismic Pressure:
The following guidelines shall be used when designing a retaining wall with Seismic pressure:
Combined effect of static and seismic lateral force:
$P_{AE} = F_1 + F_2$
F_1 = 1/2*A*H^2 Resultant acting at a distance of H/3 from base of wall
F_2 = 3/8*K_h* γ*H^2 Resultant acting at a distance of (0.6*H) from base of wall
Where:
F_1 = Static Force (plf) based on active pressure
F_2 = Seismic Lateral Force (plf) based on seismic pressure
F_2 = Seismic Lateral Force (plf) based on seismic pressure
120= γ pcf
$K_h = S_{DS}/2.5$
A = Active Pressure (pcf)
H = Height of retained soil (ft)
(Reference: 1807.2 Building Code Manual, County of Los Angeles, Dept. of Public Works, Safety Division)

5.6 SEISMIC FACTORS

Reference is made to the California Building Code CBC 2019, USGS, SEAOC/OSHPD, OSHPD Seismic Design Maps (http://seismicmaps.org), and Third Party Graphical User Interface GUI's. Based on the latitude & longitude of the site at 152 Porteous Ave., Fairfax, CA 94930 and the site characteristics, the seismic factors are determined as follows: Latitude 37.9764608 degree North, Longitude -122.5909348 degree West. Site Classification "D" Stiff Soil profile based on boring PA-2 (15>N>50), because boring PA-1 (N>50) was drilled at the toe of a 7' cut upslope behind present residence and if it was drilled from ground surface similar results to PA-2 would have been obtained. The site soil profile used is justified to be "D". The soil factors including maximum accelerations and design accelerations are compiled as function of the spectral acceleration and they are:

Site Coefficients Fa=1.2, Fv=null	
Spectral acceleration for 0.2 sec. period (short)	Ss=1.5g
Spectral acceleration for 1 sec. period	S1=0.6g
Maximum spectral response acceleration (short)	SMs=FaSs=1.8g
Maximum spectral response acceleration	SM1=FvS1=null
Design spectral response acceleration (short)	SDs=2/3SMs=1.2g
Design spectral response acceleration	SD1=2/3SM1=null

5.7 SETTLEMENTS

It is estimated that the post-construction settlement of the foundations under design loads should not exceed approximately 1-2 inch (approximated based on our experience but not calculated). It must be noted that the foundation of drilled-in friction piers and grade beams is more efficient than other types of foundation.

5.8 UTILITY TRENCHES

If required and needed, shallow utility trenches shall be designed to withstand minimal bracing. The California Safety Orders requires more substantial bracing or shoring for trenches deeper than 5 feet. Utility trenches should be designed to minimize the transmission of water into the sub grade soils beneath pavements, slabs on grade or structures. We suggest plugging the full depth of the trench with on-site, clayey soil, for a distance of two feet on either side of such structures.

For the balance of this section of this report, "bedding" is described as that material placed around the pipe, such as sand, concrete, and "backfill".

Unless concrete bedding is required, we recommend that imported free-draining sand be used as bedding and compacted to 85% relative density. The sand proposed for use, as bedding should be tested to determine its suitability prior to its delivery to the site.

On-site, inorganic soils may be used as trench backfill. Such soils should be placed in 8 inch layers and compacted to achieve a density equivalent to at least 85% of the maximum dry density of the soil according to ASTM Test D1557-78. Contractors may use other compaction techniques, so long as the required density is achieved. Beneath pavements, foundations and concrete slabs on grade, trench backfill should be compacted to 90%. Beneath pavements, the surficial 6 inches of trench backfill should be compacted to 95%.

5.9 PLANTING

We recommend landscape installation of plants that require minimum watering. Do not plant shallow-rooted trees so close to structures or pavement that root heaver can occur.

5.10 MAINTENANCE

Annual flushing with a garden hose of all under drains and catch basins is recommended. If any pipes become clogged, they should be cleared so that hydrostatic pressures do not reduce the shear strength of the soils.

5.11 Cal/OSHA TEMPORARY CUT SLOPE

The soil-type classifications and recommendation for temporary cut slopes for retaining walls constructed and backfilled shall be Type B with maximum allowable inclination of height depth ratio of 1:1

6. CONCLUSIONS AND LIMITATIONS

The following conclusions are based on the results of our study of the subject site.

3. It is our opinion, that a foundation for the retaining walls and additions is feasible from a Geotechnical Engineering standpoint provided the recommendations contained in the Soil Report are followed.
4. Seismically induced ground shaking with some structural damage may occur within the economic life of the development. Sub-grade preparation and foundation type as: piers & grade beams to very dense soil strata, are intended to minimize the effects of ground shaking below the structure.
5. The sub-soil profile and properties are considered to be relatively uniform containing silty sand and silty sandy rocks so the earthquake induced differential settlement is considered to be low. Overall cumulative settlement from a seismic event is estimated and not calculated to be on the order of 1 to 2 inch.
6. A very low liquefaction potential/susceptibility exists at the subject site and the site is 153 feet away from a moderate liquefaction potential/susceptibility area. This opinion is based on predominantly the soil, groundwater/ Bay mud conditions, the CGS and USGS maps.
7. Based on the soil encountered at the site, the following foundation can be used for the retaining walls and addition in the back of residence:
   - For the retaining walls, drilled-in cast-in place reinforced concrete friction piers, drilled into the very dense tan brown sandy rocks or silty sand strata and connected together on top with grade beams forming a grid to provide collectively confinement and better resistance to earthquake forces
   - For the additions, a mat foundation resting on grade beams supported by drilled-in cast-in place reinforced concrete friction piers, drilled into the very dense tan brown sandy rocks or silty sand soil strata. The grade beams can be integrated by design inside the mat forming a moment frame to provide collectively confinement and better resistance to earthquake forces.
8. Risk of geotechnical hazards will always exist due to uncertainties of geologic conditions and the unpredictability of seismic activity in the Bay Area. However, in our opinion, based on available data, there are no indications of geotechnical hazards that

would preclude use of the site for the proposed foundation settlement mitigation.

Our services consist of professional opinions, conclusions, and recommendations were performed in accordance with generally accepted geotechnical engineering principles and practices and in accordance with the standards of practice as set by the geotechnical engineers in the area. This warranty is in lieu of all other warranties either expressed or implied. Our liability is limited to the amount charged for this report.

This report assumes that a qualified geotechnical engineer will be retained to provide geotechnical engineering and construction supervision services during foundation work including drilling for the piers, steel bars reinforcing and concrete casting for piers and grade beams. It must be thoroughly understood that the recommendations that are presented in this report should not be construed to be any type of long-term guarantee or insurance against future geotechnical problems that may occur at the site. We feel that the recommendations contained in this report will greatly reduce the risks of any future geotechnical problems, but any risk that still remains must be borne by the Owners of the dwellings.

If you have any questions regarding this report, please call us at (510) 524-1494. We will be happy to provide you with the additional services our company can perform.

Very Truly Yours,
***CAL ENGINEERING INT CO*, Geotechnical Engineers**

Mike Khoury, MSCE, P.E.
California Geotechnical Engineering License #464
California Civil Engineering License #20859

STATE OF CALIFORNIA GEOTECHNICAL No. 464 REGISTERED PROFESSIONAL ENGINEER MICHEL S. KHOURY

Attachments: - one original hard copy report, signed and wet stamped
- one PDF copy emailed

REGISTERED PROFESSIONAL ENGINEER MICHEL S. KHOURY No. 20859 CIVIL STATE OF CALIFORNIA

REFERENCES

1. USGS, Misc. Field Studies Map MF-574, Preliminary Geologic Map of Marin and San Francisco Counties and Parts of Alameda and Contra Costa Counties By M. C. Blake Jr. et all 1974.
1A. CGS California Geological Survey, Geological Map Of California https://maps.conservation.ca.gov/cgs/gmc/
2. USGS, Liquefaction Susceptibility, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region by Robert Witter et al 2006
2A USGS, Geologic Map of Marin County, Google Earth
3. USGS, Earthquake Hazards Program, Seismic Design Maps and Tools for Buildings and Bridges http://earthquake.usgs.gov/research/hazmaps
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5.CDMG, Special Publication 42, Fault Hazard Zones in California, 1976.
5.American Society for Testing Materials (ASTM), Annual Standards.
7.ASTM, Volume 4.08, Designation D420 - 69, Standard Recommended Practice for Investigating and Sampling Soil and Rock for Engineering Purposes.
8.County of Contra Costa, Corrosive Soil Survey by USDA, E.J Carpenter and S.W. Cobby University of California 1938
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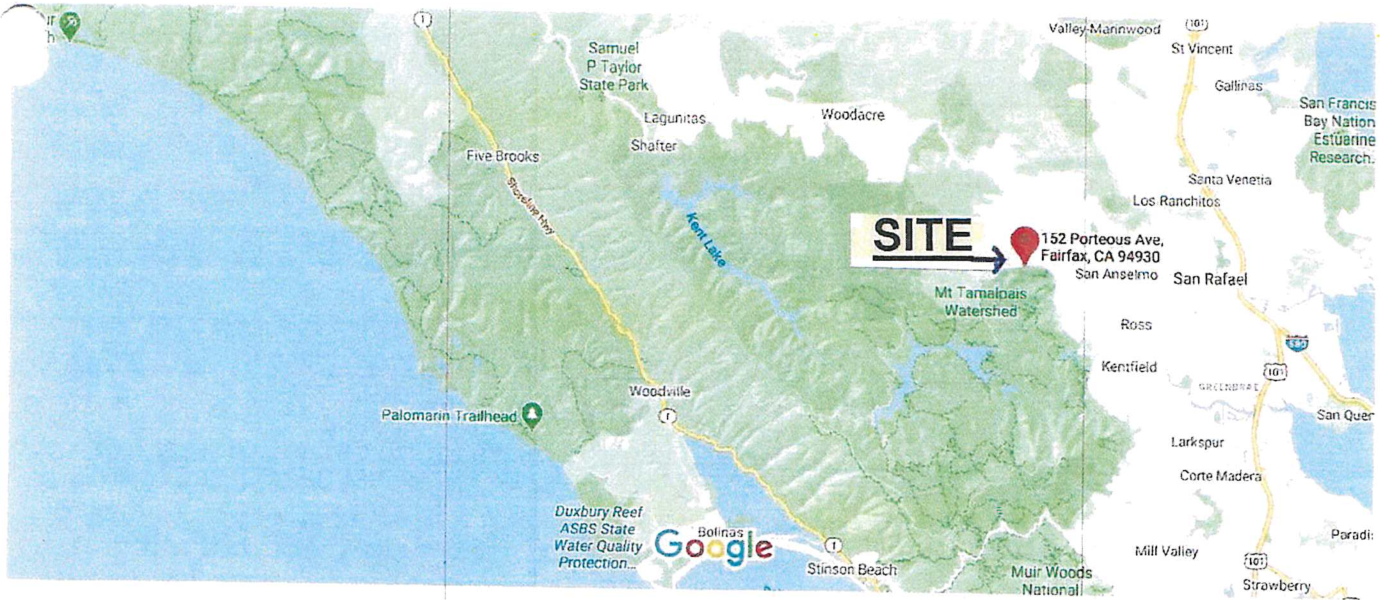
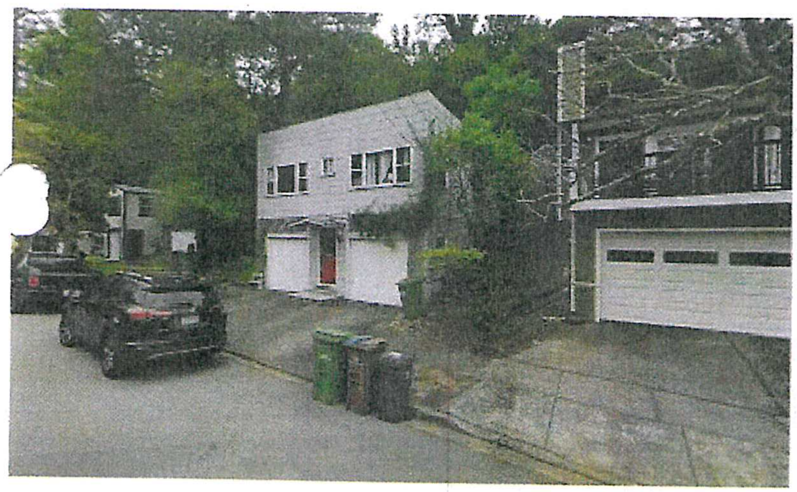
11.Soil Mechanics, Foundations, And Earth Structures", by G. P. Tschebotarioff
12.ASTM, volume 4.08, Designation D420 - 69, Standard Recommended Practice for Investigating and Sampling Soil and Rock for Engineering Purposes.
19. ASCE, Manual No. 56, Subsurface Investigation for Design and Construction of Foundations of Buildings, 1976.
20. Building Code Manual, County of Los Angeles, Dept. of Public Works, Safety Division

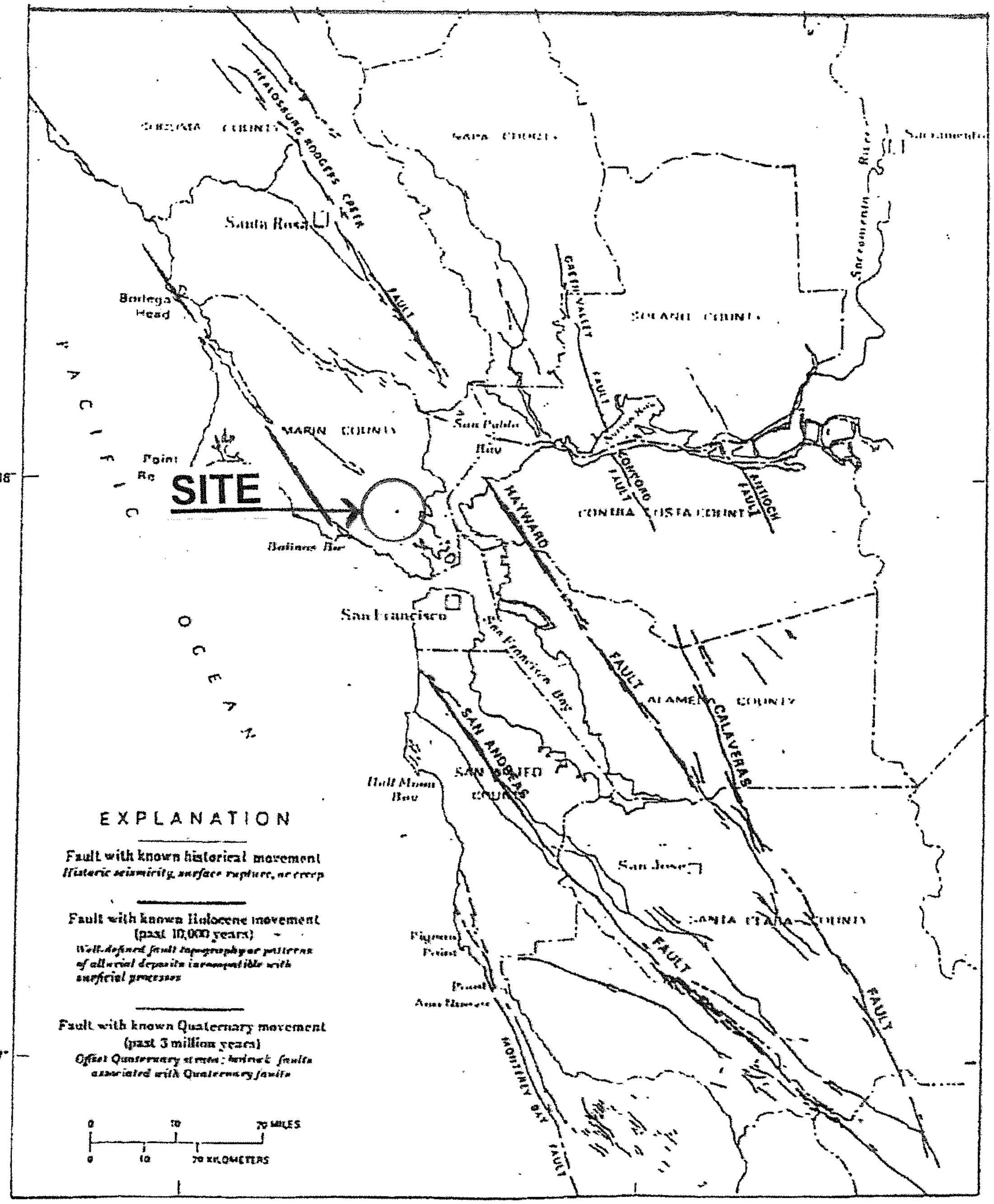
Google Maps 152 Porteous Ave



Map data ©2021 Google 2 mi



CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
LOCATION MAP	DATE:	4/18/2021
	FIGURE:	1



CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
SEISMICITY MAP	DATE:	4/18/2021
	FIGURE:	2

OSHPD

52 Porteous Ave, Fairfax, CA 94930, USA

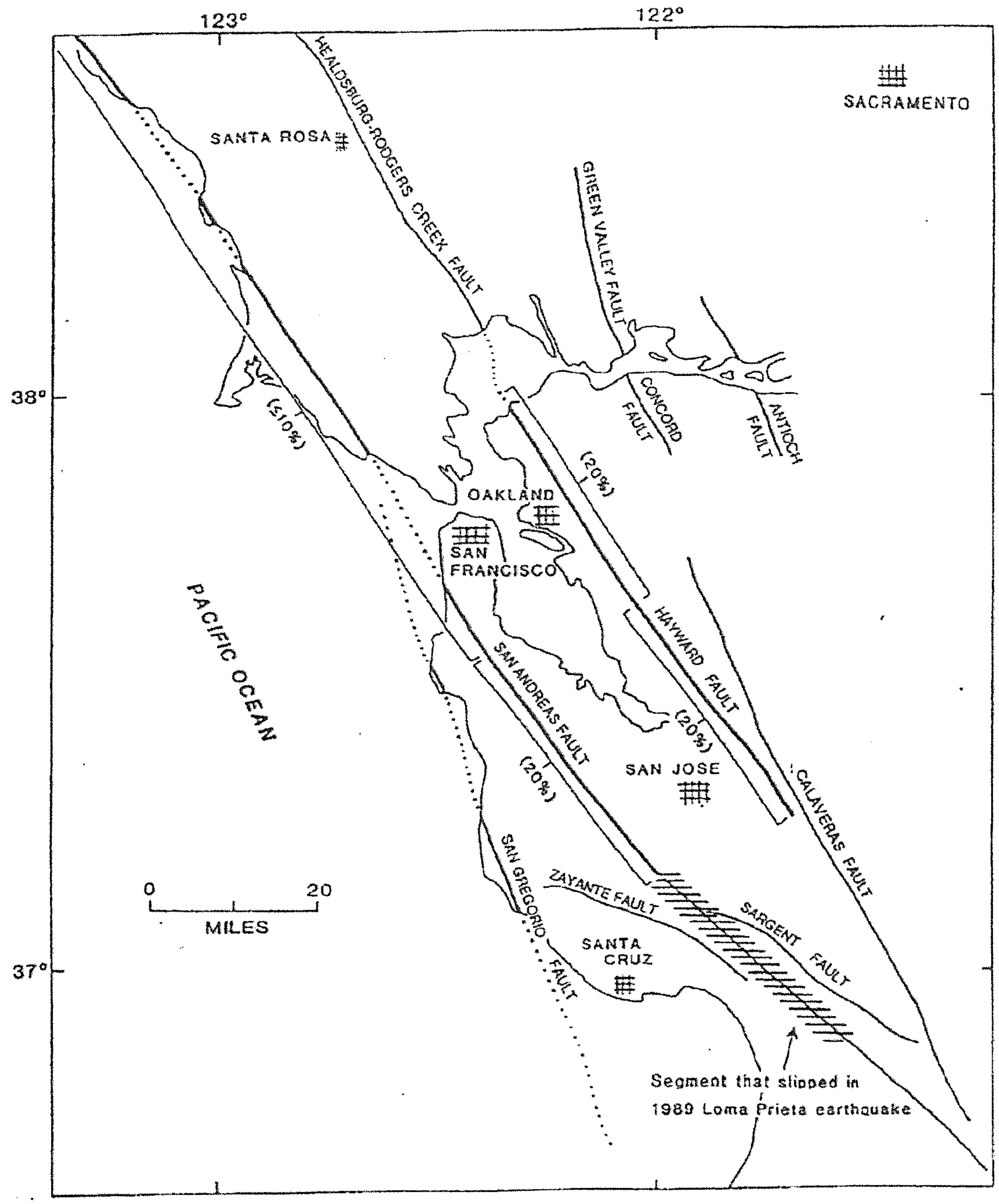
Latitude, Longitude: 37.9764608, -122.5909348

Date	4/15/2021, 6:43:05 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.8	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.2	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	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.622	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.746	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	1.839	Probabilistic risk-targeted ground motion. (0.2 second)

CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
SEISMIC DESIGN SPECTRUM	DATE:	4/18/2021
	FIGURE:	2A

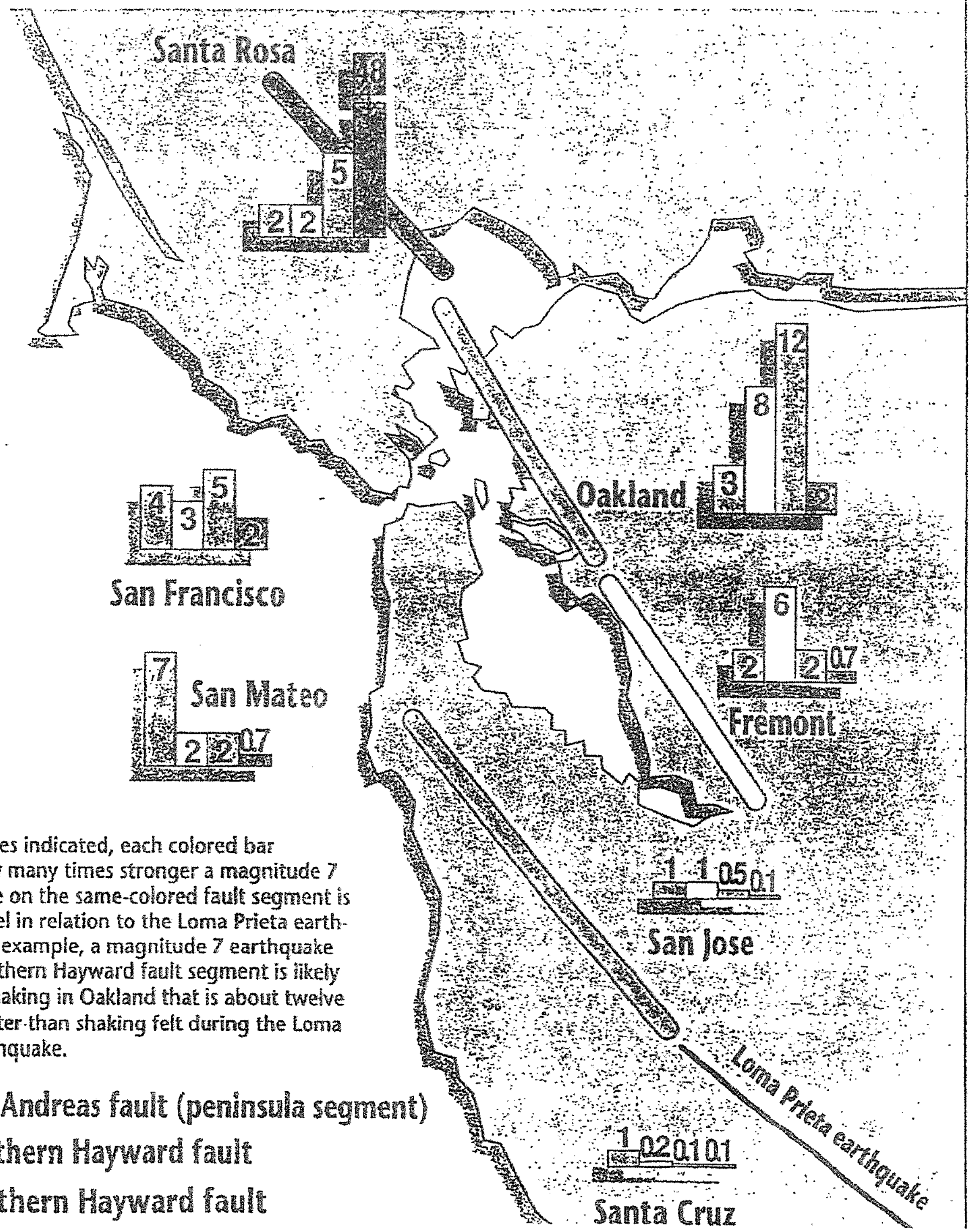


Segments of San Andreas and Hayward faults (heavy lines) showing chance of occurrence of an earthquake in the next 30 years (U.S. Geological Survey, 1988). Faults dotted where concealed.

CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
SEISMICITY MAP, % OF OCCURRENCE	DATE:	4/18/2021
	FIGURE:	3

HOW MUCH THE GROUND WILL SHAKE

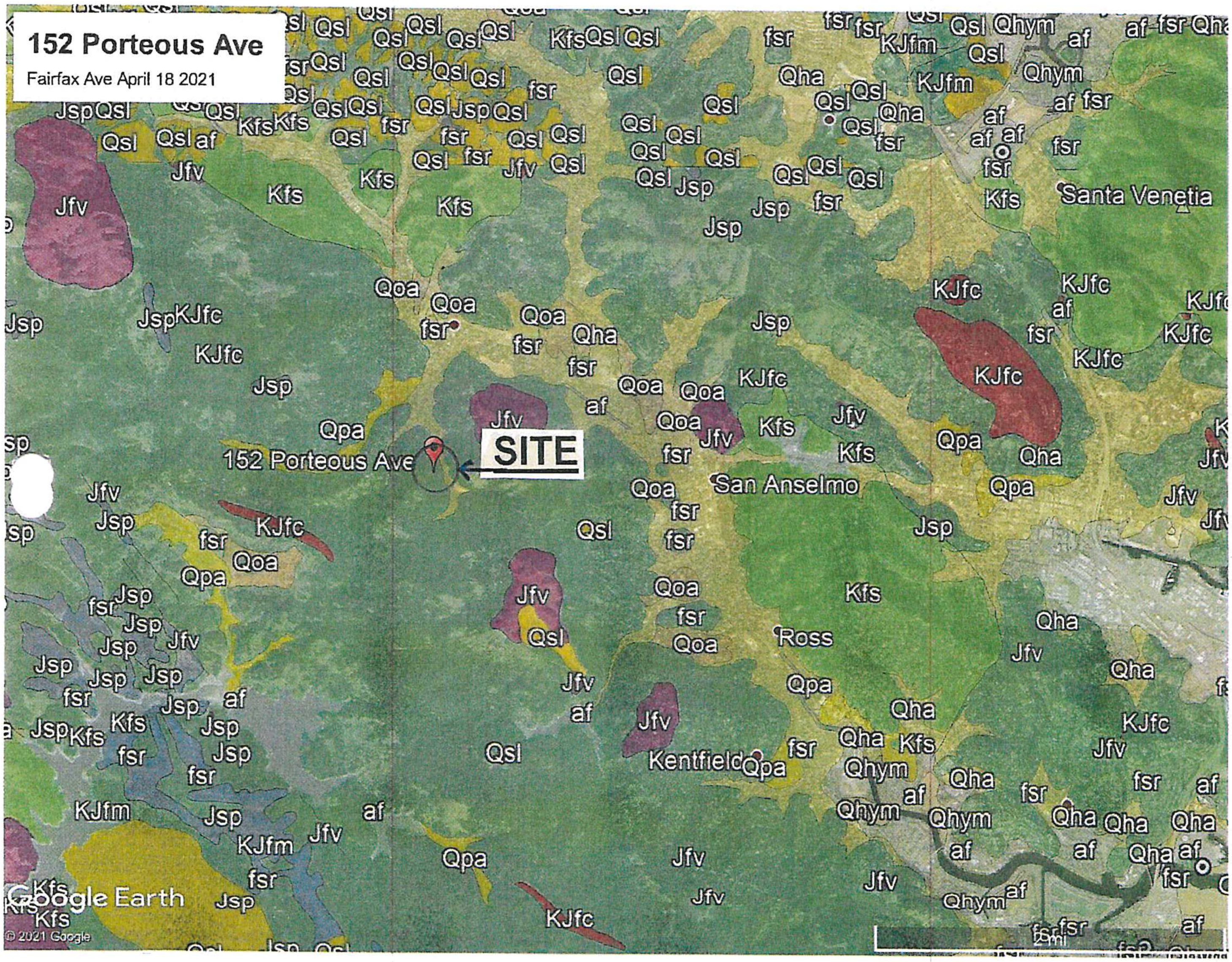
A Comparison with the Loma Prieta Earthquake



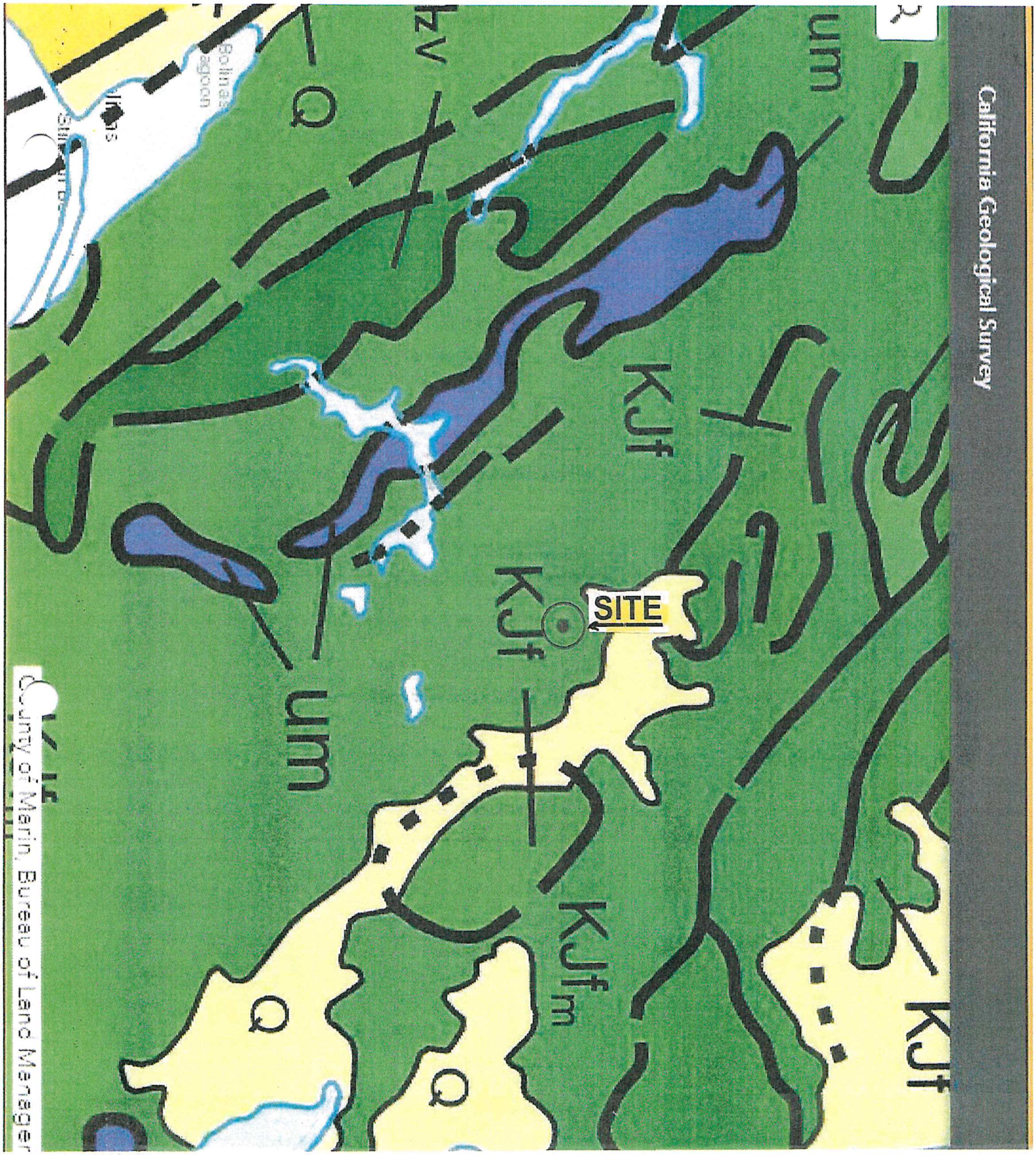
For the cities indicated, each colored bar shows how many times stronger a magnitude 7 earthquake on the same-colored fault segment is likely to feel in relation to the Loma Prieta earthquake. For example, a magnitude 7 earthquake on the northern Hayward fault segment is likely to cause shaking in Oakland that is about twelve times greater than shaking felt during the Loma Prieta earthquake.

- San Andreas fault (peninsula segment)
- Southern Hayward fault
- Northern Hayward fault

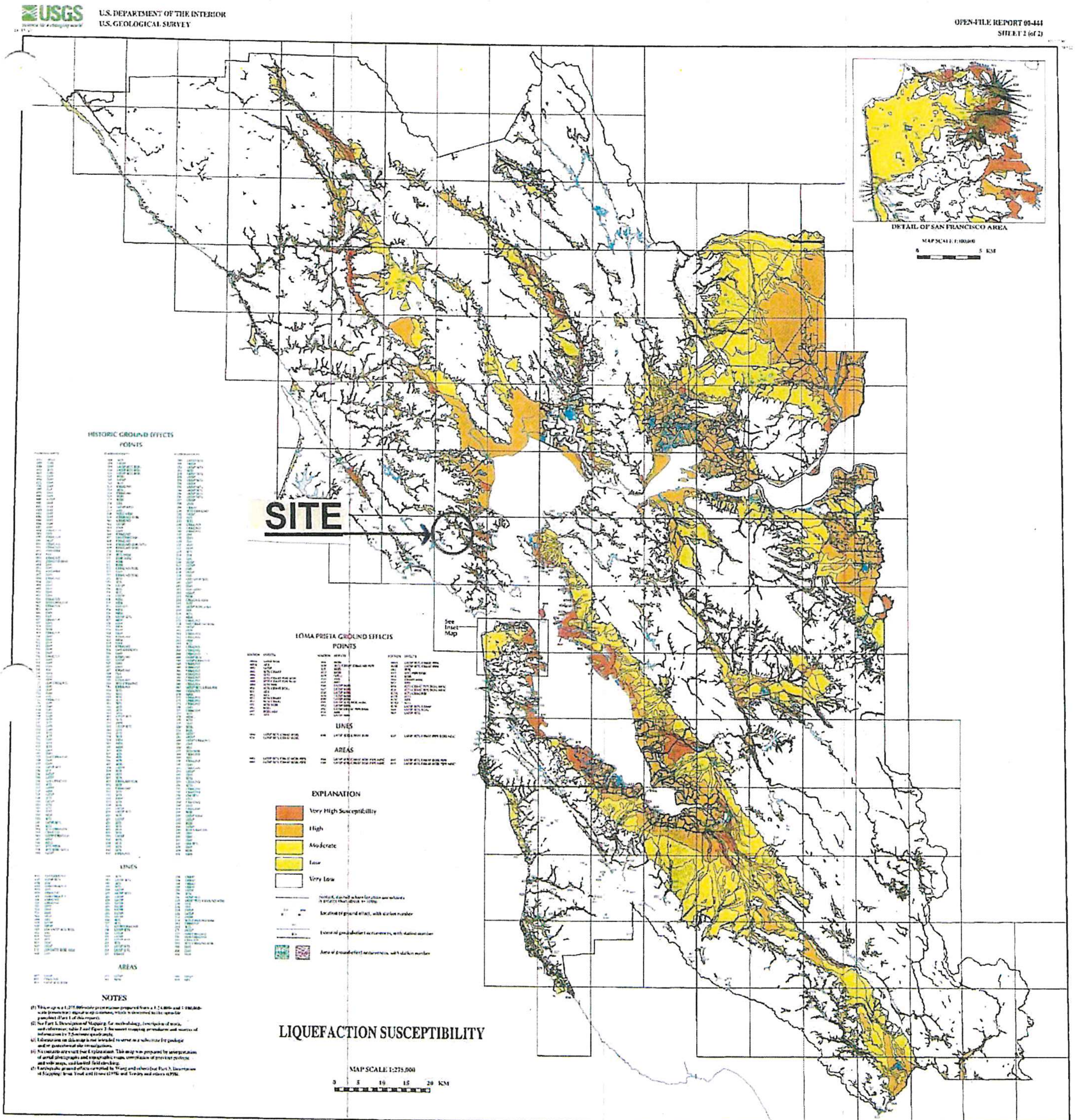
CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
SEISMICITY MAP VS. LOMA PRIETA	DATE:	4/18/2021
	FIGURE:	4



CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
GEOLOGY MAP 1	DATE:	4/18/2021
	FIGURE:	5

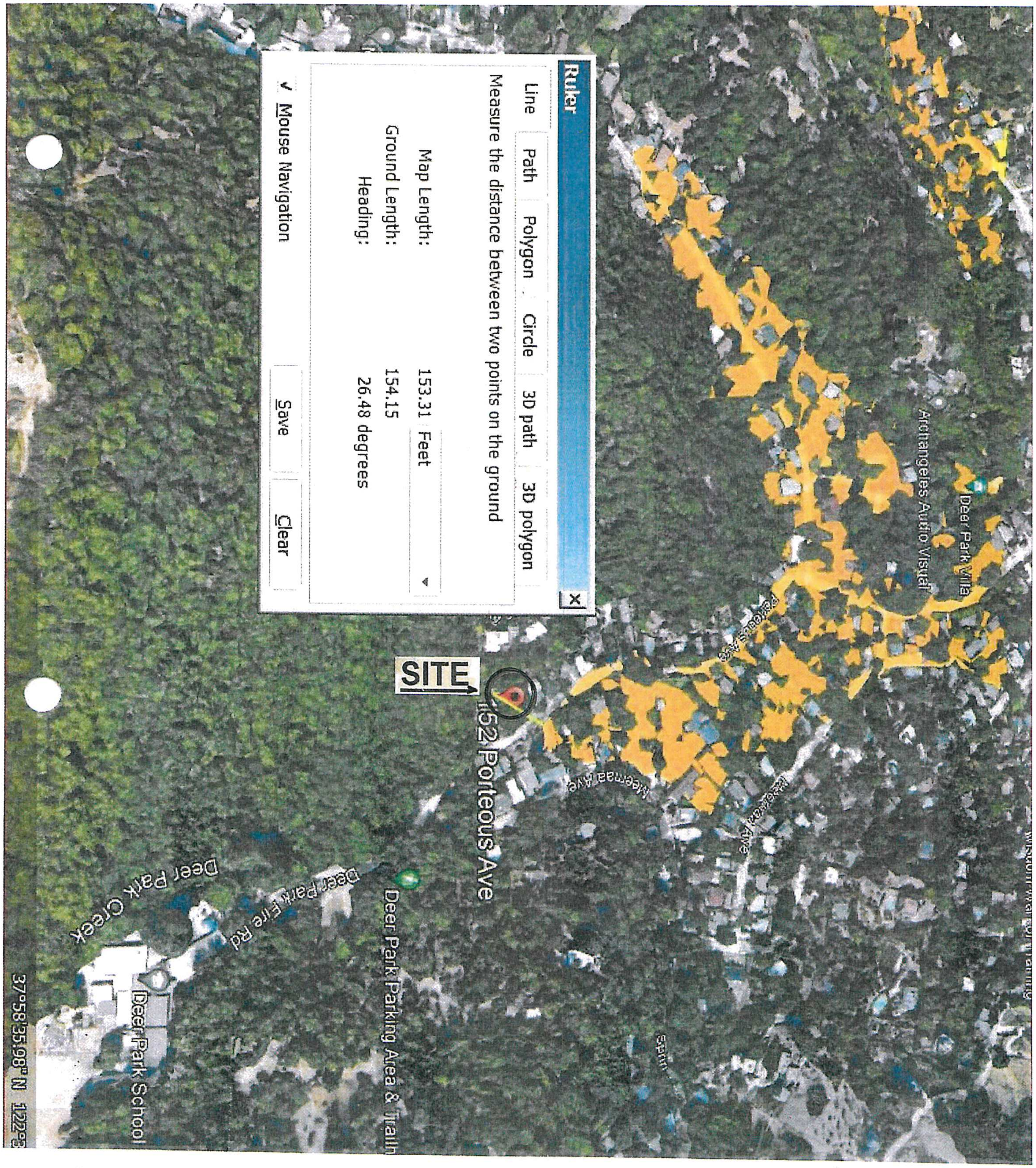


CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
GEOLOGY MAP 2	DATE:	4/18/2021
	FIGURE:	5-1

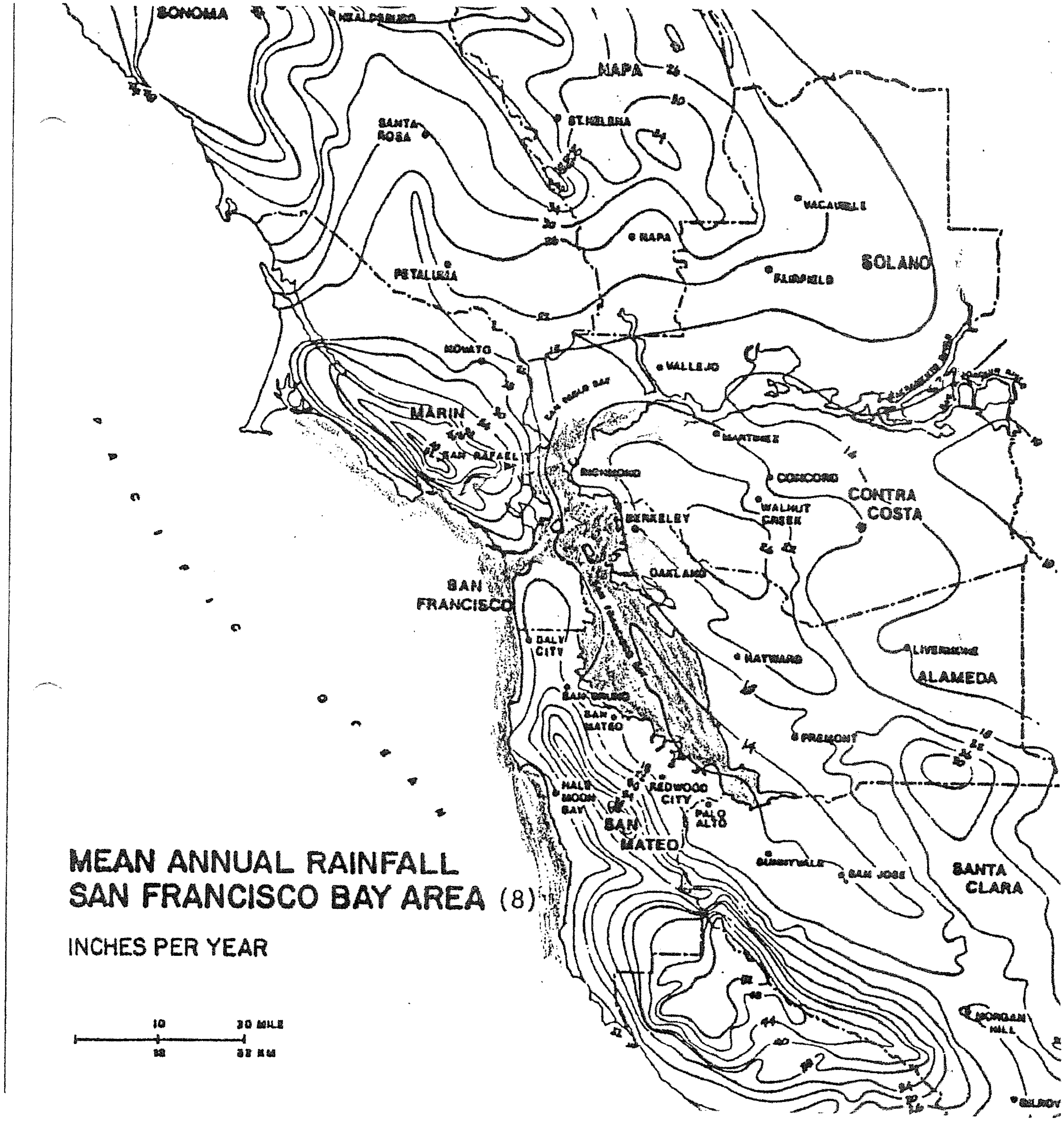


PRELIMINARY MAPS OF QUATERNARY DEPOSITS AND LIQUEFACTION SUSCEPTIBILITY, NINE-COUNTY SAN FRANCISCO BAY REGION, CALIFORNIA

CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
LIQUEFACTION POTENTIAL	DATE:	4/18/2021
	FIGURE:	5A



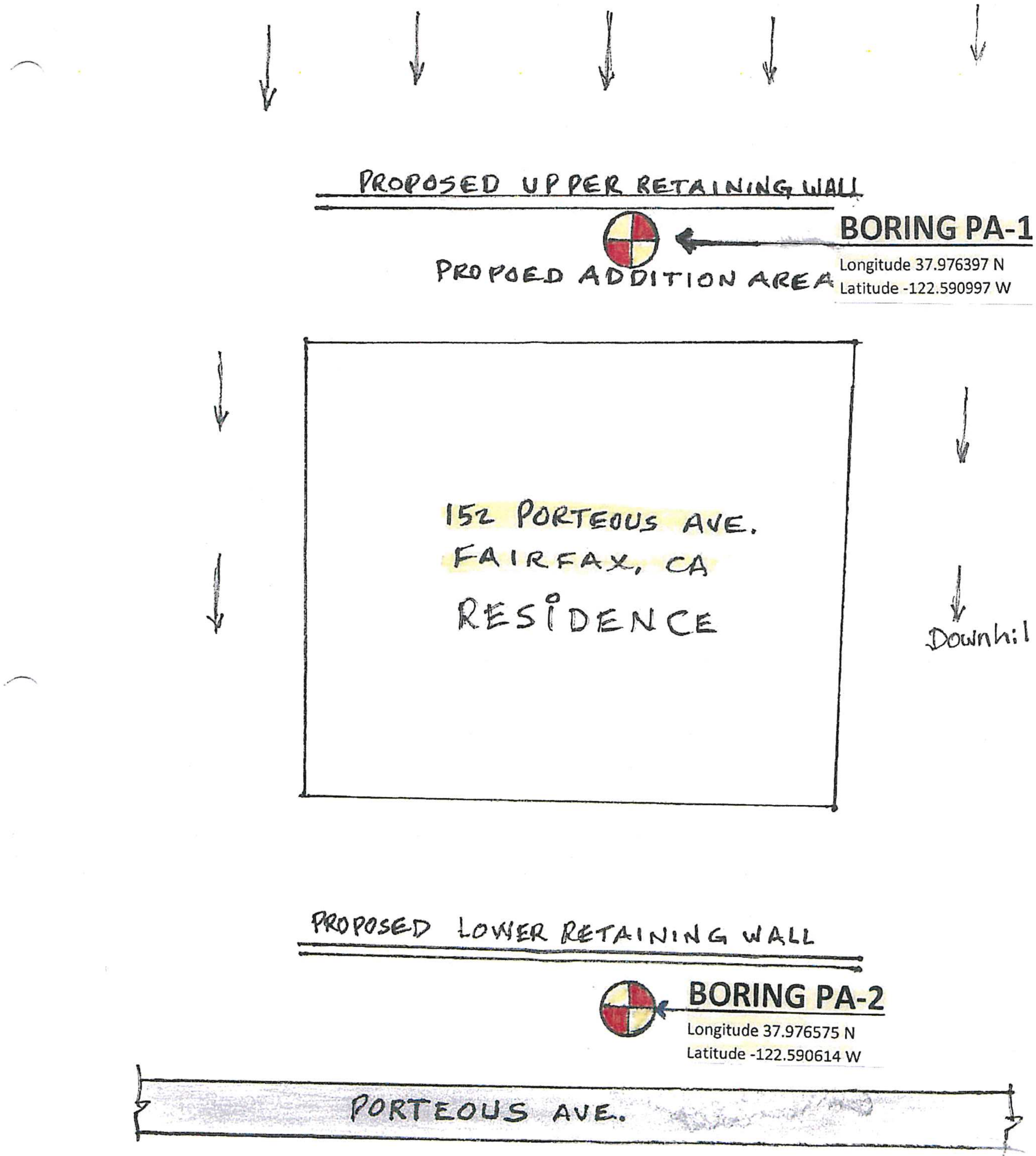
CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO: CEICO 3162021
	LOCATION: 152 Porteos Ave. Fairfax, CA 91930
LIQUEFACTION POTENTIAL	DATE: 4/18/2021
	FIGURE: 5B



MEAN ANNUAL RAINFALL
SAN FRANCISCO BAY AREA (8)

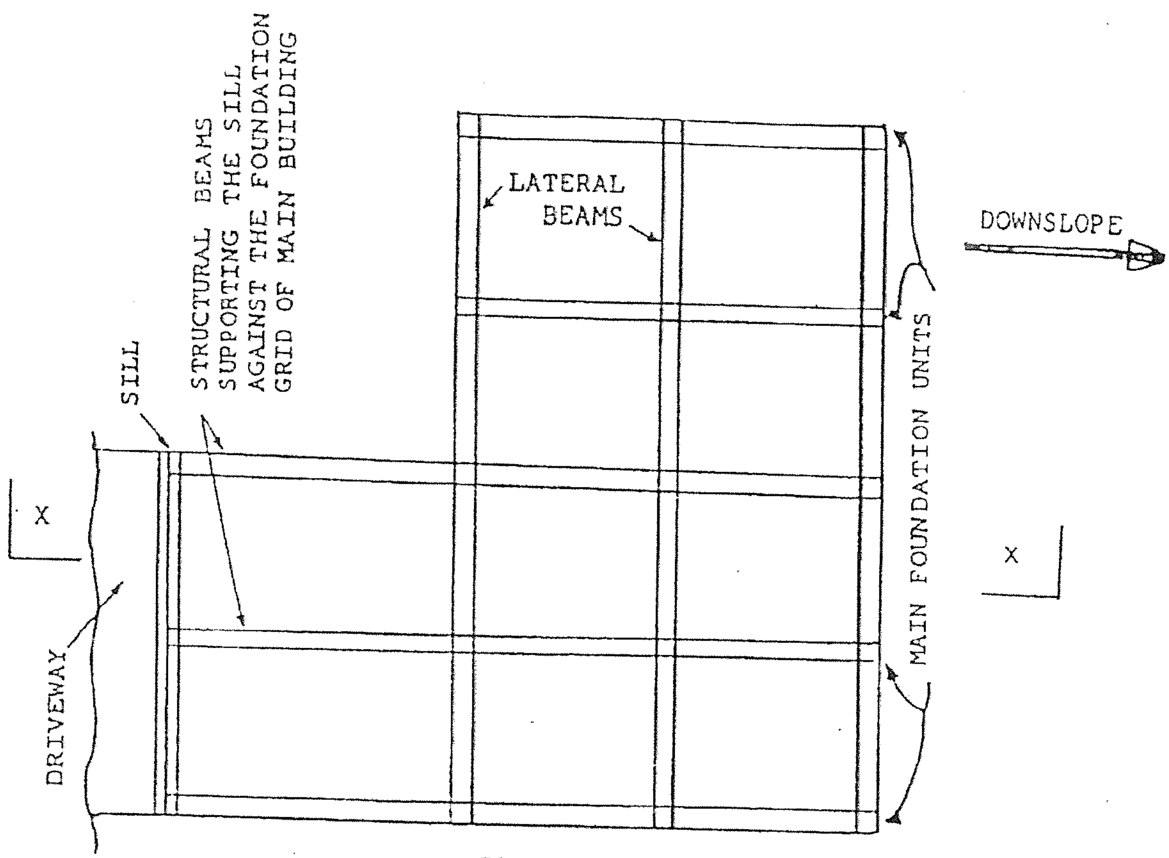
INCHES PER YEAR

CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
RAINFALL MAP	DATE:	4/18/2021
	FIGURE:	6



CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
BORING LOCATION	DATE:	4/18/2021
	FIGURE:	7

TYPICAL FOUNDATION GRID



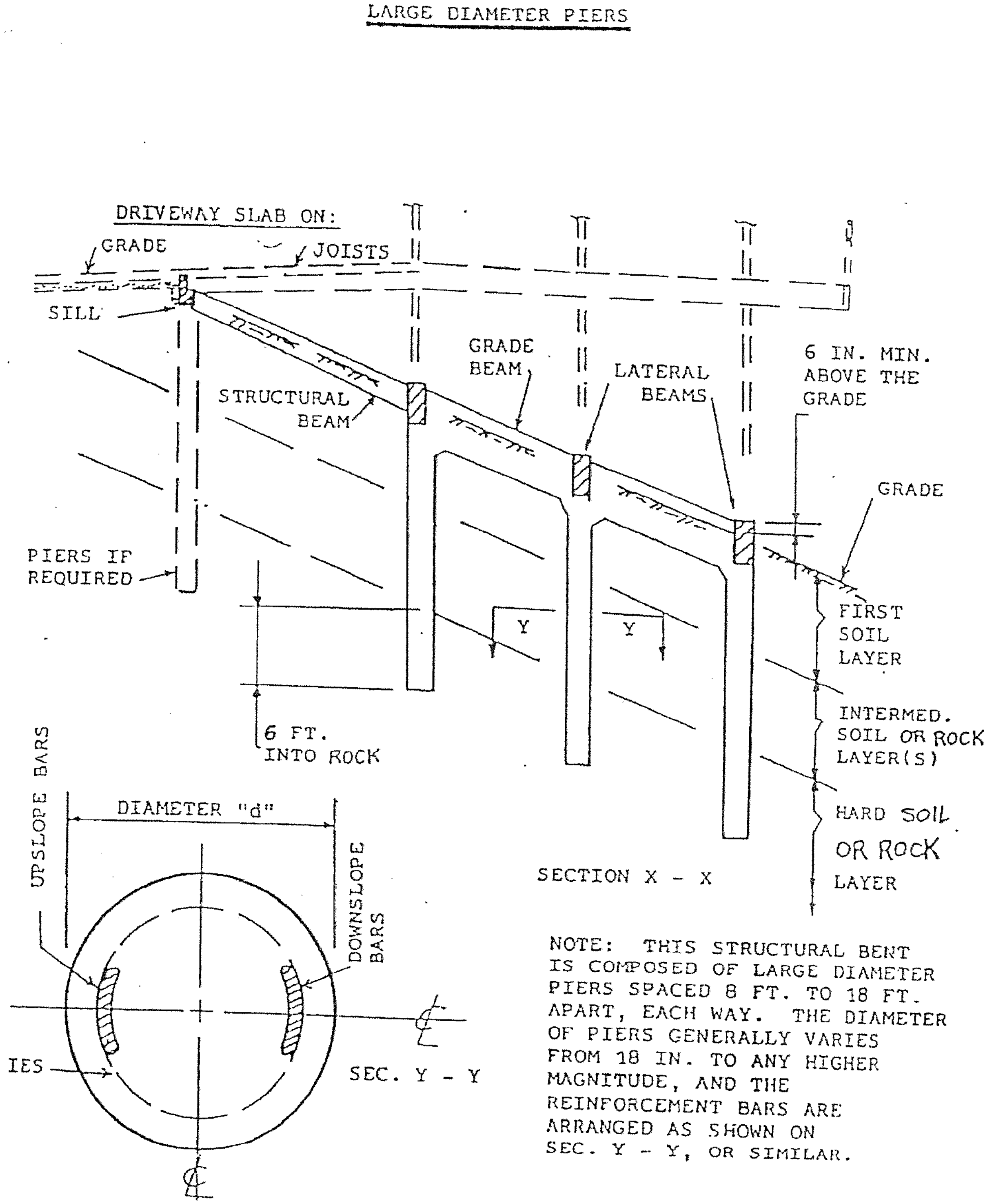
PLAN VIEW

NOTE: 1) ALL FOUNDATIONS MUST BE DESIGNED BY AN EXPERIENCED FOUNDATION ENGINEER AND DATA CONTAINED IN THE SOIL REPORT MUST BE USED SO THAT THE REQUIRED SAFETY FACTORS, AS BASED ON PROBABILITY CALCULUS, CAN BE OBTAINED.

2) THE FOUNDATION GRID MUST HAVE ALL UNITS, LIKE MAIN AND LATERAL BEAMS, MONOLITHICALLY CONNECTED WITH EACH OTHER SO THAT COMPLETE BENDING MOMENTS AND SHEARING FORCES TRANSFER WILL BE OBTAINED.

3) IF PIERS ARE USED UNDER THE ABOVE FOUNDATION GRID, THEN THEY MUST ALSO BE MONOLITHICALLY CONNECTED WITH ANY OF THE BEAMS TO RESIST SEISMIC, WIND AND/OR SOIL FORCES.

CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
TYPICAL PIER FOUNDATION PLAN	DATE:	4/18/2021
	FIGURE:	8



CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
TYPICAL PIER FOUNDATION SECTIONS	DATE:	4/18/2021
	FIGURE:	9

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS. MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

DEFINITION OF TERMS

	U.S. STANDARD SERIES SIEVE 200	40	10	4	CLEAR SQUARE SIEVE OPENINGS 3/4"	3"	12"	
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

SANDS AND GRAVELS	BLOWS/FOOT †
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH ‡	BLOWS/FOOT †
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

† Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡ Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
KEY TO EXPLATORY BORING LOGS	DATE:	4/18/2021
	FIGURE:	10

FIELD EXPLORATION & LABORATORY TESTING DATA

BORING & SAMPLING

The site investigation-boring program was made using the services of Access Soil Drilling Inc. of San Mateo, CA.

Equipment: Minuteman Drilling and Sampling.
Samplers: Standard penetration Split Barrel Sampler 1 3/8" ID, 2" OD
Split Barrel Sampler (Modified California Sampler) 2" ID, 2 ½" OD
Sampling Method: 140 pound hammer, 30" drop, motorized Cathead Winch
Samples: 2.5" Sampler in 6" Stainless Steel Liners
Penetration: N, number of blows per foot based on SPT, 140 # hammer and 30 "drop

LABORATORY TESTING

Performed by Toscano Testing, Oakland, CA
The total number of samples: 8
The samples were extracted from the 6" long 2 ½" Diameter Stainless Steel Liners.
Water Content was analyzed and calculated for all the samples.
The boring logs show the water content as a percentage of the sample weight.

Note:
No water table / ground water was present in boreholes 1 or 2 at the time of investigation.

CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteos Ave. Fairfax, CA 91930
FIELD EXPLORATION & LABORATORY TESTING DATA	DATE:	4/18/2021
	FIGURE:	11

Drilled: 4/12/2021 BORING PA-1

DESCRIPTION	USCS Symbol	DEPTH FEET	SAMPLE NUMBER	N BLOWS PER FOOT	qu TSF	MOISTURE CONTENT %	DRY DENSITY PCF	NOTES
Tan brown silty sandy rocks, very dense	GM	1	1-1	51	4	7	115.6	This layer is 5-7'
Tan brown silty sandy rocks, very dense	GM	2	1-2	48	4	5.7	129	excavated cut
Tan brown silty sandy rocks, very dense	GM	3		69				
		4	1-3	61	4	7	119.1	Sieve Analysis
Tan brown silty sandy rocks, very dense		5		61				
	GM	6	1-4	62	4	6.3	112.6	
Practical refusal @ 5'-9"								

Sieve Analysis % Soil Passing PA-1-3@4'

Passing No. 30	37.89%
Passing No. 50	31.96%
Passing No. 100	25.25%
Passing No. 200	20.57%

Note:No water table encountered

Note: Drlling stated at -7' below ground elevation at bottom of cut

CAL ENGINEERING INT. CO. Geotechnical Engineers	PROJECT NO:	CEICO 3162021
	LOCATION:	152 Porteous Ave. Fairfax, CA
BORING LOG PA - 1	DATE:	4/18/2021
	FIGURE:	12

Drilled: 4/12/2021

BORING PA -2

DESCRIPTION	USCS Symbol	DEPTH FEET	SAMPLE NUMBER	N BLOWS PER FOOT	qu TSF	MOISTURE CONTENT %	DRY DENSITY PCF	NOTES
Tan brown silty sand, med dense	SM	1		14				
		2	2-1	15	2	15.5	98.3	Atterberg Limits
Tan brown silty sand, med dense	SM	3	2-2	17	2	12.5	92.2	
Tan brown silty sandy clay, very stiff	CL	4	2-3	32	4	12.2	98.6	
Tan brown silty sand, very dense	SM	5		37				
		6	2-4	71	4	10.1	96.4	
Practical refusal@6'								
Atterberg limits@ PA-2-1@2'								
Non-Plastic								
Note:No water table encountered								
Note: Drilling started at ground level								

CAL ENGINEERING INT. CO. Geotechnical Engineers	**PROJECT NO: CEICO 3162021** **LOCATION: 152 Porteous Ave. Fairfax, CA**
BORING LOG PA - 2	**DATE: 4/18/2021** **FIGURE: 13**