REPORT

GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENCE
2366 Rainbow Court
Hayward, California

for
Joyce Steinfeld
19281 Mountain Way
Los Gatos, California 95030

Project No. 184920
August 2018

Project 201804682 SPR
2366 Rainbow Court

MILSTONE GEOTECHNICAL

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MILSTONE GEOTECHNICAL

August 13, 2018
Project No. 184920

Joyce Steinfeld
19281 Mountain Way
Los Gatos, CA 95030

SUBJECT: **Geotechnical Report Update**
Proposed Residence
2366 Rainbow Court
Hayward, California

Dear Ms. Steinfeld,

Milstone Geotechnical has completed a geotechnical investigation for the above referenced site in accordance with your authorization. The accompanying report presents the results of the investigation with conclusions and geotechnical design criteria for the proposed development.

Based on the results of this investigation we are pleased to report that, from a geotechnical perspective, the site is suitable for the residence if properly designed and constructed. It has been a pleasure providing professional services to you on this project and we are looking forward to assisting you and your design and construction team through project construction.

Please phone or e-mail if you have any questions regarding the contents of this report or require additional assistance.

Sincerely,
MILSTONE GEOTECHNICAL

Barry S. Milstone, G.E. 2111
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
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PROPOSED RESIDENCE
2366 Rainbow Court
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GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENCE
2366 Rainbow Court
Hayward, California

INTRODUCTION

This report presents the findings, conclusions, and updated recommendations resulting from our supplemental geotechnical investigation related to the construction of a proposed new residence at 2366 Rainbow Court in Hayward, California (Figure 1).

Project Description

Based on our discussions and my review of the provided schematic site improvement drawings, it is my understanding that the project will involve the construction of a new, multi-level, single-family residence at a currently undeveloped site.

Purpose and Scope of Investigation

The site was previously investigated by Geotechnical Engineering Inc. (GEI). The purposes of this investigation were to characterize the geotechnical conditions of the site and provide specific recommendations for the geotechnical aspects of the proposed improvements. Our investigation was supplemented by data provided by Geotechnical Engineering Inc. (GEI) in their 1995[1] and 2017[2] geotechnical investigations of the site.

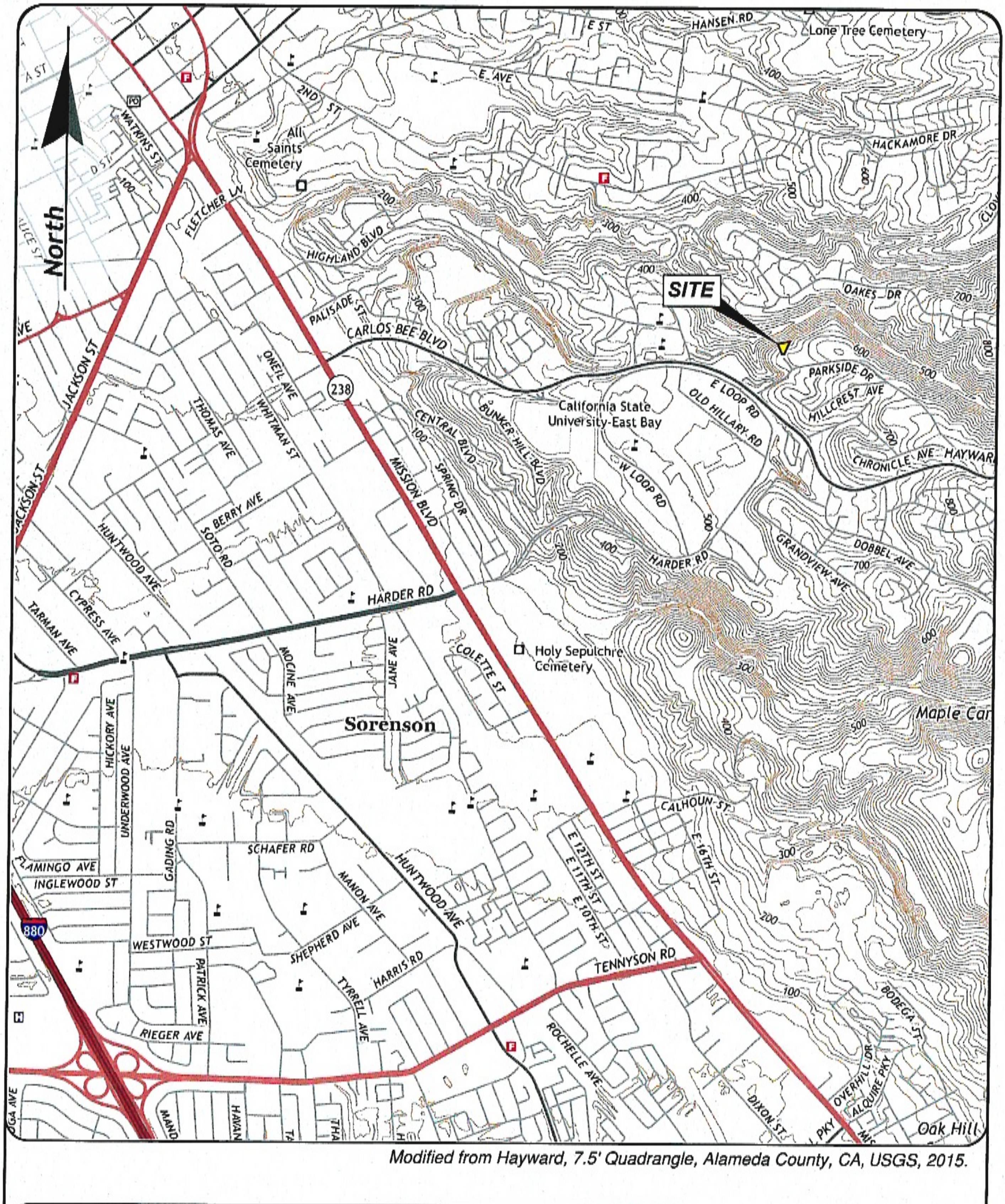
The scope of services performed for this investigation included the following tasks:

- Compilation and review of available engineering and geologic data relevant to site improvements including previous geotechnical investigation reports at the site prepared by GEI[1,2];
- Limited geotechnical mapping of the site using the provided site plan to identify pertinent surficial features;
- Hand-drilling, logging, in-situ testing, and sampling of one (1) small diameter exploratory borehole;

---

1 Geotechnical Engineering Inc., November 2, 1989, Geotechnical Investigation, Report, Supplementary Investigation and Geologic Reconnaissance, Proposed Residential Development, Parkside Drive & Rainbow Court, Tract 3992, Hayward California for Victoria Court Management, 1221 State Street, Suite 203, Santa Barbara, CA 93101.

2 Geotechnical Engineering Inc., September 6, 2017, Geotechnical Investigation, Report – Soil Investigation, Planned Single Family Residence, 2366 Rainbow Court, Hayward, California for Robert Jay and Joyce Steinfeld, 19281 Mountain Way, Los Gatos, CA.



Modified from Hayward, 7.5' Quadrangle, Alameda County, CA, USGS, 2015.

MILSTONE GEOTECHNICAL

SITE LOCATION MAP

PROPOSED STEINFELD RESIDENCE
2366 Rainbow Court
Hayward, California

FIGURE NO. **1**

Date:	Scale:	Drawn by:	Project No.
August 2018	1 inch = 2,000 feet	BSM	184920

- Laboratory testing of representative soil samples to verify field classifications, characterize the subsurface materials, and determine index properties and pertinent engineering characteristics for analysis and design;
- Engineering analysis of the resulting data and formulation of geotechnical design criteria;
- Preparation of this report and the accompanying illustrations.

GEOLOGIC SETTING

Regional Geologic Setting

The subject property is located near the base of the east flank of the northern Santa Cruz Mountains and within the Coast Ranges Geomorphic Province of Northern California. These mountains are composed primarily of tertiary, sedimentary, and small amounts of igneous rock. The crustal bedrock have been uplifted, folded and faulted into their present form, and marine terraces, colluvium and alluvium have subsequently been deposited on the range's flanks. Structurally, the region is dominated by northwest trending faults and folds. Due to on-going plate tectonic activity, structural deformation of the Santa Cruz Mountain area continues into the present. In more recent geologic time, the dominant sense of movement is right lateral motion concentrated along the active San Andreas Fault zone located about 2.9 miles southwest of the site.

Local Geologic Conditions

Graymer and others[3] indicate that the site is underlain by Jurassic-age Knoxville Formation materials (Figure 2) consisting of "mainly dark greenish-gray silt or clay shale with thin sandstone interbeds". Locally, the bedrock is mapped as favorably bedded with a northwest strike and dipping generally cross slope at an inclination of about of 46 degrees. A northwest striking contact fault adjoining Late-Cretaceous-age Joaquin Miller Formation consisting of "thinly bedded shale with minor sandstone" is mapped approximately 200 feet to the northeast.

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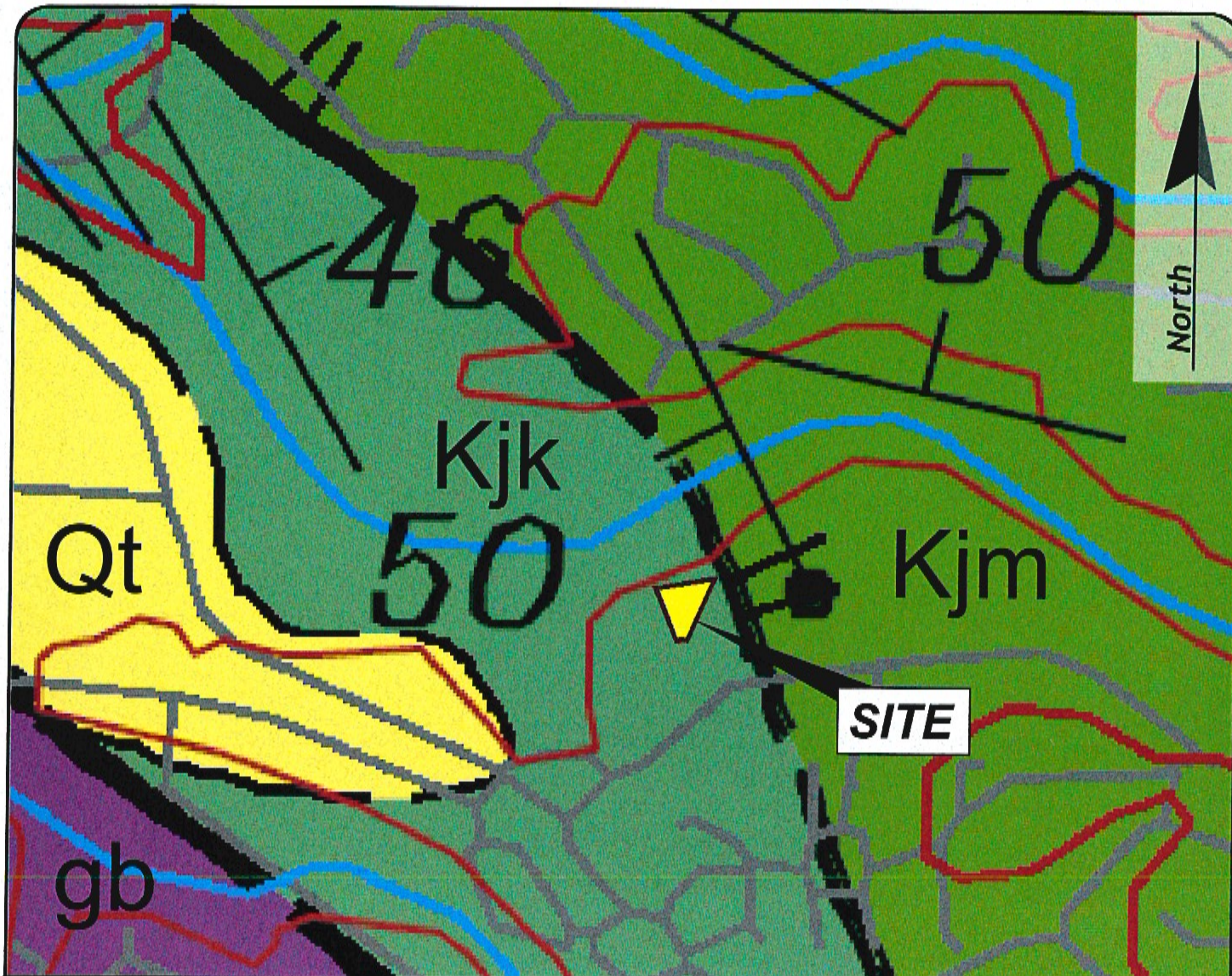
[3] Graymer, R.W., Jones, D.L., and Brabb, E.E., 1996, Preliminary geologic map emphasizing bedrock formations in Alameda County, California: A digital database, U.S. Geological Survey Open-File Report 96-252.



Modified from: Graymer, R.W., Jones, D.L., and Brabb, E.E., 1996, Preliminary geologic map emphasizing bedrock formations in Alameda County, California: A digital database, U.S. Geological Survey Open-File Report 96-252.

EARTH MATERIALS

- **Kjm** - Joiquin Miller Formation (Late Cretaceous, Cenomanian)
- **Kjk** - Knoxville Formation (Late Jurassic and Early Cretaceous)
- **gb** - Gabbro (Jurassic)
- **Qt** - Terrace Deposits (Holocene and Pleistocene)

MAP SYMBOLS

- Geologic contact, dashed where approximately located
- Fault; dashed were approximately located
- Oblique fault with normal component, dashed where approximately located
- 30 — Strike and dip of bedding
- 50 — Strike and dip bedding, top direction known

Modified from Hayward, 7.5' Quadrangle, Alameda County, CA, USGS, 2015.

MILSTONE GEOTECHNICAL	REGIONAL GEOLOGIC MAP PROPOSED STEINFELD RESIDENCE 2366 Rainbow Court Hayward, California			FIGURE NO. 2
	Date: August 2018	Scale: 1 inch = 500 feet	Drawn by: BSM	Project No. 184920

The site is located within a State-designated Earthquake-Induced Landslide Hazard Zone[4] (Figure 3). The Alameda County landslide map prepared by Roberts and others[5] does not depict any landslides in the general vicinity nor anywhere along the neighboring Ward Creek creekbanks that are located in the Seismic Hazard Zone. Visual reconnaissance of the site did not reveal surface geomorphology or disturbance to the mature tree cover that would be indicative of recent or historic slope stability.

Faulting and Seismicity

Although no faults are known to traverse the property, the site is located within the influence of several active and potential active faults with potential to generate significant ground shaking. Structurally, the region is dominated by northwest trending faults and folds with structural deformation continuing into the present. The regional seismic setting is dominated by stress associated with the oblique collision of the Pacific tectonic plate with the North American tectonic plate. Throughout coastal California, the surface expression of this interface is the San Andreas fault, including its principal northwest-aligned branches. In the San Francisco Bay Region, the San Andreas fault system includes several major branches in addition to maintaining a relatively continuous main trace. The study area is part of a structural slice within the Diablo Range between two such branches: the Hayward fault, which is located 0.9 miles to the southwest, and the Calaveras fault, located seven (7) miles to the northeast. These faults are well known active features exhibiting abundant geologic evidence of recurring movement and are the source of both nearly continuous micro-seismicity as well as several large historic earthquakes. Although these are considered to be the closest significant faults, other Bay Area faults such as the San Andreas located 19.4 miles to the southwest and the San Gregorio, located 26.6 miles to the southwest, are considered capable of significantly impacting the proposed development.

The site is not located within an Alquist-Priolo Earthquake Fault Zone[6] and published geologic maps do not indicate the presence of any faulting in the immediate vicinity. Although fault rupture is unlikely within the proposed site

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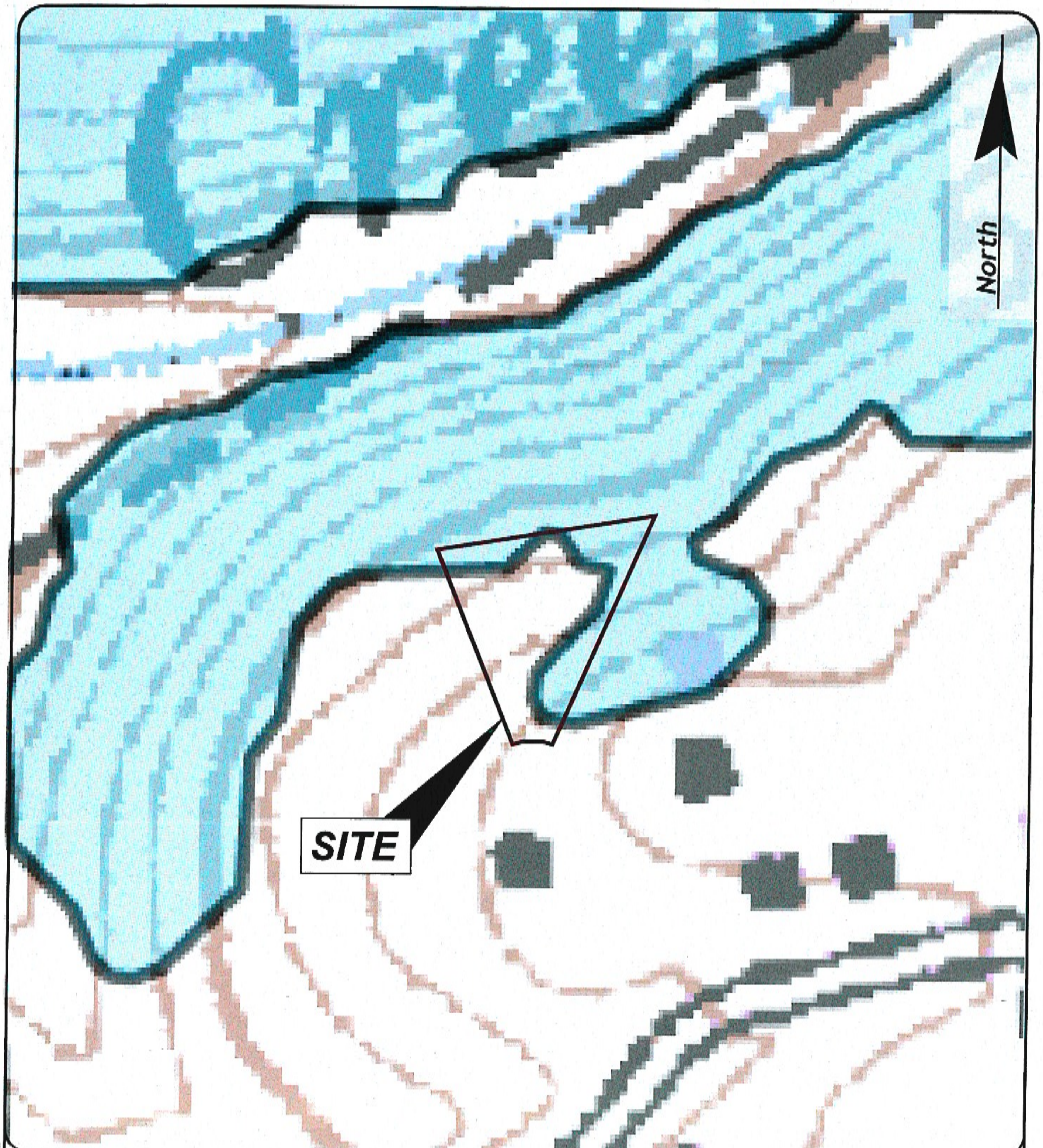
4 California Geologic Survey, 6/2/2003, Earthquake zones of required investigation, Hayward Quadrangle.

5 Roberts, S, Roberts, M.A., and Brennan, E.M., 1999, Landslides in Alameda County, California, A digital database extracted from preliminary photointerpretation maps of surficial deposits by T.H. Nilsen in USGS Open-File Report 75-277, U.S. Geological Survey, Open-File Report 99-504.

6 California Geologic Survey, 9/21/12, Earthquake zones of required investigation, Hayward Quadrangle.



California Geologic Survey, 6/2/2003, Earthquake zones of required investigation, Hayward Quadrangle.

MILSTONE GEOTECHNICAL	SEISMIC HAZARD ZONES PROPOSED STEINFELD RESIDENCE 2366 Rainbow Court Hayward, California			FIGURE NO. 3
	Date: August 2018	Scale: 1 inch =100 feet	Drawn by: BSM	Project No. 184920

development area, strong to violent ground shaking due to local fault activity will probably occur sometime during the economic lifetime of the development. Historic data suggests the most severe ground shaking induced by fault rupture will most likely be generated by a major event along the nearby active Hayward fault system. When calculating seismic hazards, the US Geologic Survey[7] assumes a maximum moment magnitude of 7.3 for the combined branches of the Hayward-Rodgers Creek fault, 7.0 for the combined branches of the Calaveras fault, and 7.7 for the combined northern and peninsular segments of the San Andreas.

Based on work performed by the National Earthquake Hazard Reduction Program, the USGS[8] has classified the subject area as within a Site Class B shaking hazard zone.

Anticipated Ground Surface Acceleration

Based on the most recent earthquake forecasts published by the Working Group on California Earthquake Probabilities[9], there is estimated to be there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2014 and 2044. The property is expected to experience violent ground shaking during large earthquakes on the nearby faults. Based on the site location (lat. 37.6608, long. -122.0505), the peak ground acceleration with a 10% probability of being exceeded in 50 years is estimated to be 0.65g using the probabilistic seismic evaluation tools provided by the U.S. Geologic Survey[10].

As a minimum, the proposed structure should be designed in accordance with the current California Building Code (CBC) standards for static and seismic design. More specific seismic design criteria are presented in the Geotechnical Design Criteria section. It should be noted that there is a paucity of data

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7 Petersen, M.D. and others, 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps, United States Geological Survey, Open File Report 2008-1128.

8 United States Geological Survey, undated, Soil type and shaking hazard in the San Francisco Bay Area, https://earthquake.usgs.gov/hazards/urban/sfbay/soiltype/.

9 Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y., 2013, Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model: U.S. Geological Survey Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, http://pubs.usgs.gov/of/2013/1165/.

10 US Geologic Survey, 2/10/11, Earthquake Ground Motion Parameters V.5.1.0.

available for near field sites, such as the subject site, and that it is possible that actual ground surface accelerations will exceed the current estimates.

GENERAL SITE CONDITIONS

Site Setting

The site is located on a plateau in the southwest foothills of the Diablo Range. The truncated-triangular-shaped, one-third-acre property is situated at the crest of the south flank of the west-flowing Ward Creek drainage channel. Locally, the property is situated on a northwest-trending spur ridge created by Ward Creek and an unnamed drainage to the west. The property is accessed from the north side of the Rainbow Court cul-de-sac, approximately 230 feet northwest of its intersection with Parkside Drive. The property is bordered by the Rainbow Court cul-de-sac to the south, by a wooded slope flanking Ward Creek to the north, by undeveloped land to the west, and by a residentially developed property to the east

Existing Improvements

The site is currently undeveloped although fence posts, an abandoned concrete foundation, and surface debris suggest that the area had been used historically for agricultural, and possibly other, purposes. Aerial photographs suggest that the structure was demolished sometime between 1993 and

Topography

The project is located approximately 570 feet above sea level. Ground surfaces in the vicinity of the proposed residence at the southern portion of the property slope moderately toward the north with inclinations ranging from about four to one (4 to 1) horizontal to vertical to five to one (5 to 1). A localized fill slope traverses the building pad at an inclination of about three to one (3 to 1) horizontal to vertical. Beyond a slope break located approximately 20 feet north of the north of the proposed residence, the slopes incline steeply toward Ward Creek at an inclination of about 1.8 to one (1.8 to 1) horizontal to vertical.

Site Drainage

Surface drainage of storm water occurs by sheet flow to the north toward the south flank of the west-flowing Ward Creek. We observed no indications of concentrated surface runoff such as rills or channels.

Vegetation

The proposed building pad is vegetated with wild grasses. Tightly spaced, predominantly oak and bay, trees of varying age and size, ranging up to at least 40-inch diameter, blanket the northern slope.

SUBSURFACE CONDITIONS

Subsurface Investigation

The subsurface conditions at the site were investigated by drilling, logging, and sampling one (1) small-diameter exploratory borehole to practical refusal at a depth of 4.5 feet and by reviewing data published by GEI from four (4) boreholes that they previously advanced on the site from 3.0 to 8.5 feet deep. Our subsurface investigation is described in more detail in Appendix A. The exploratory borehole locations are depicted on Figure 4 (Site Plan and Exploration Map). Graphical logs of the boreholes are presented in Appendix A of this report.

Subsurface Materials

In general, the proposed development area is underlain by up to two (2) feet of artificial fill blanketing weathered Knoxville Formation siltstone with lesser amounts of sandstone. The upper two to three (2 to 3) feet of the bedrock exhibits advanced weathering to a residual soil. Our findings are similar to those encountered by GEI[1],during previous geotechnical investigations. The encountered earth materials are described below in order of decreasing age. Pertinent field and laboratory test results are summarized at the end of this section.

Weathered Siltstone

At a depth of about three (3) feet beneath the ground surface, our borehole encountered silty gravel that appears to have weathered out of the underlying siltstone. The encountered materials are very dense and consist of about 40 percent hard, angular, siltstone fragments in a matrix of about 40 percent fine to coarse grained sand and 20 percent non-plastic fines. GEI[1] reported a standard penetration blowcount of 85 blows in 11 inches at depth of about five (5) feet near the proposed residence. Pocket penetrometer resistance in these materials exceeded 4.5 tons per square foot (tsf).

Two (2) representative samples of these materials demonstrated an average dry density of 102.7 pounds per cubic foot (pcf) and an average moisture content of 7.5 percent. Unconfined compression test results of 3,540 and 1,750 pounds per square foot (psf) exhibited generally brittle failure and are believed to be a lower-bound representation of the in-situ compressive strengths. The California Geological Survey[11] has published a compilation of laboratory tests on Knoxville Formation materials and, based on 11 tests, report a friction angle of 32 degrees consistent with apparent cohesion of 621 psf.

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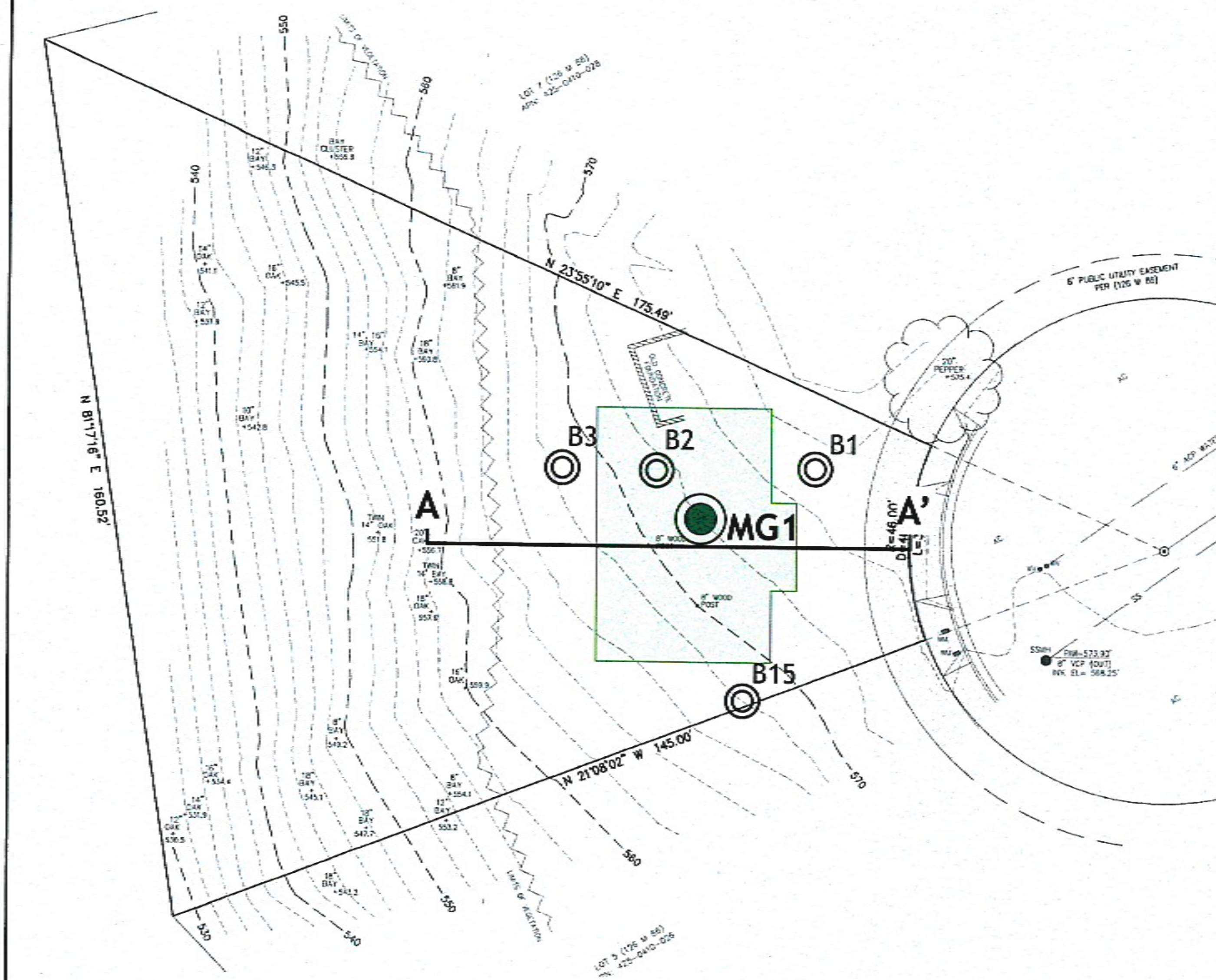
[11] California Geological Survey, 2003, State of California Seismic Hazard Zones, Hayward 7.5-Minute Quadrangle, California: Seismic Hazard Zone Report 091.



EXPLANATION

MG1 — Milstone Geotechnical exploratory borehole

B1 — Geoforensics (1997, 2017) exploratory borehole

A — A' Approximate location of Geotechnical Cross Section.

Limits of proposed residence.

NOTES: Surface topography and Milstone Geotechnical borehole location determined by tape and compass methods. B1 through B3 borehole locations derived from 9/6/17 GEI Soil Investigation Report. B15 borehole location derived from 11/2/89 GEI Investigation and Geologic Reconnaissance Report. Limits of proposed residence derived from undated preliminary layout sketch provided by client. This figure is not intended to be used for construction purposes.

MILSTONE GEOTECHNICAL

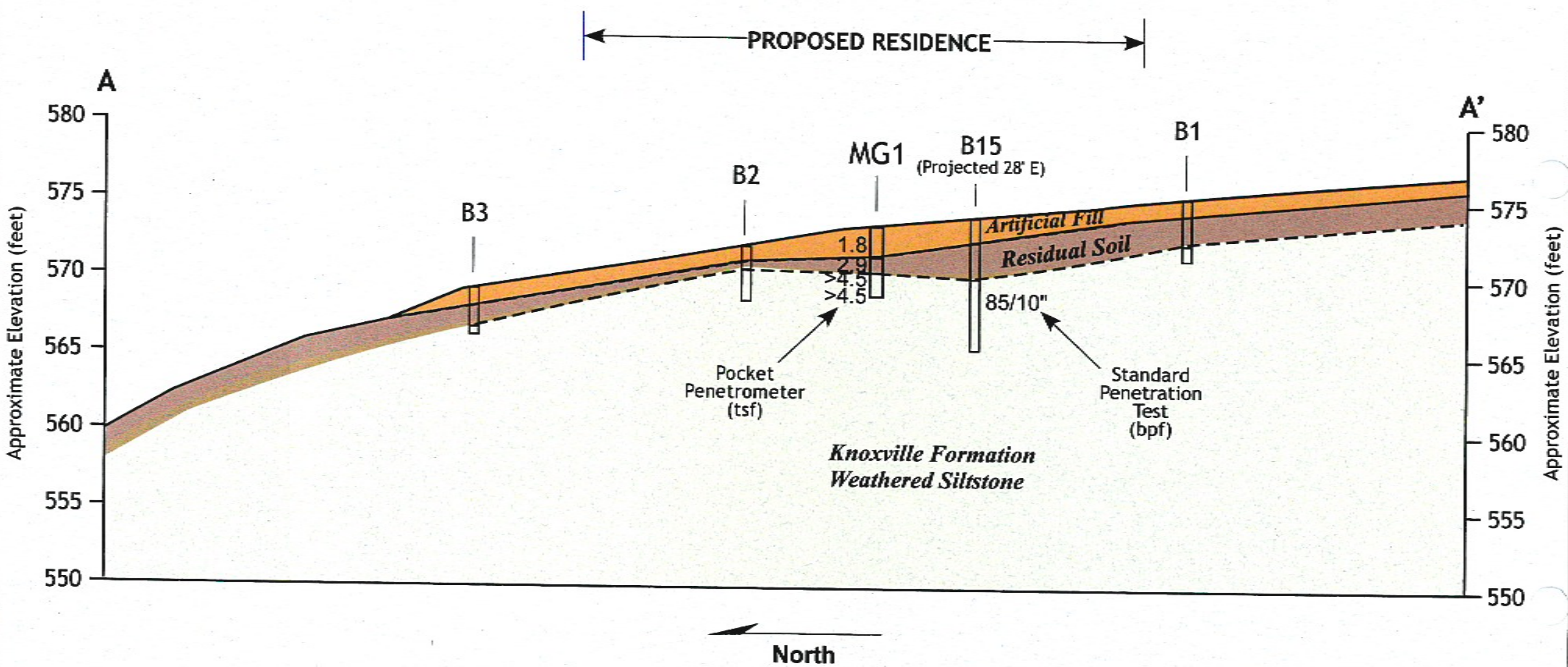
SITE PLAN AND EXPLORATION MAP

PROPOSED STEINFELD RESIDENCE
2366 Rainbow Court
Hayward, California

Date	Scale	Engineer	Project No.
August 2018	1 inch = 30 feet	BSM	174840

FIGURE NO. 4

GEOTECHNICAL CROSS SECTION A-A'



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MILSTONE GEOTECHNICAL	GEOTECHNICAL CROSS SECTION A-A'			FIGURE NO.
	PROPOSED STEINFELD RESIDENCE 2366 Rainbow Court Hayward, California			5
	Date August 2018	Scale 1 inch = 10 feet	Engineer BSM	Project No. 174840

Residual Soil The upper two to three (2 to 3) feet of the Knoxville Formation has decomposed through weathering to residual soil consisting of medium stiff to stiff, tough, low to medium plasticity clay and medium dense, clayey, fine-grained sand. Pocket penetrometer testing in these materials indicate and average unconfined compressive strength of about 2.6 (tsf). One (1) representative samples of the residual soil exhibited a dry density of 101.9 psf with a corresponding moisture content of 14.2 percent.

Artificial Fill The site is blanketed by up to two (2) feet of artificial fill placed during original grading operations to establish the cul-de-sac. The encountered fill is characterized as loose to medium dense, moist, silty sand with about 20 percent low plasticity fines, rootlets near the surface, a minor amount of construction debris, and isolated zones of silty gravel.

Two (2) representative samples of the fill demonstrated dry densities of 96.0 and 107.2 pcf with corresponding moisture contents of 13.2 and 10.7 percent. One representative sample of the silty sand exhibited an unconfined compressive strength of 1,700 psf.

Summary of Laboratory Tests

Borehole/ Sample No.	Depth (ft)	Earth Material	Moisture Content (%)	Dry Density (pcf)	Unconfined Compression (psf)
MG1/T1	0.75	Fill	13.2	96.0	1,700
MG1/T2	1.5	Fill	10.7	107.2	-
MG1/T3	2.5	Residual Soil	14.4	101.9	-
MG1/T4	3.0	W. Siltstone	22.0	101.8	3,540
MG1/T4	4.0	W. Siltstone	15.2	103.6	1,750

GROUND WATER

Ground water was not encountered in the borehole advanced for this investigation and the previous investigator did not encounter ground water in their four boreholes that extended to a maximum depth of 8.5 feet. It should be noted that ground water conditions at other locations and times, or during different weather conditions might differ from those encountered in our test boreholes. Nevertheless, based on the results of our subsurface investigation, it is anticipated that construction of the proposed improvements will not be adversely affected by ground water if constructed during the dry season.

SEISMIC SCREENING ANALYSIS

Methodology

Because a portion of the site is located within a seismic hazard zone[4] the property has been evaluated evaluated following the guidelines presented by CGS in Special Publication 117[12] (SP-117). Subsequent to the publication of SP-117, the Southern California Earthquake Center (SCEC) published recommended guidelines[13] for the implementation of SP-117. Based on personal communication with Tim McCrink of the CGS, it is my understanding that CGS recognizes the SCEC procedures to be acceptable, and in many ways preferred to the original SP-117 seismic analysis techniques. Although there is some disagreement within the Bay Area geotechnical community regarding the appropriate use of the SCEC document in Northern California, the seismic stability of the property was evaluated using the procedures described therein. As described by SCEC, the site was subjected to a seismic deformation screening analysis that has been modified from the Seed procedure[14] described in SP-117.

Slope stability was evaluated using SLIDE[15], a limit equilibrium computer program developed by Rocscience, Inc. An idealized slope model was developed for property using site geometry, subsurface stratigraphy, ground water conditions, and engineering properties of the site soils as summarized below. Thousands of potential circular and non-circular failure surfaces were evaluated with the SLIDE software using Bishop's and Spencer's methods with continued model refinement to result in the lowest achievable factors of safety for static and seismic conditions. The analyses considered potential landslides that extend below the surficial soils. The factor of safety is defined as the ratio of forces resisting failure to those that tend to induce failure. Seismic slope analyses were performed by applying a "pseudostatic" horizontal force component to simulate earthquake loading on the subject slope. This was done both by applying a psuedostatic horizontal component to the critical static

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[12] California Department of Conservation, Division of Mines and Geology, 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CDMG Special Publication 117.

[13] Blake, T.F. and others, ed., 2002, Recommended procedures for implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California, Southern California Earthquake Center.

[14] Seed, H.B., 1979, "Considerations in the earthquake-resistant design of earth and rockfill dams," Geotechnique, 29(3), 215-263.

[15] Rocscience, Inc., SLIDE version 5.044

surface as recommended by Stark[16] and by conducting a search for the critical surface under seismic loading conditions.

Geometry Although most of the site below the proposed residence is not included in the State Seismic Hazard zone, for conservatism the analyzed cross section trends through the proposed building site along Section A-A' and extends downslope approximately 125 feet. The surface topography was determined by tape, hand-level, electronic distance meter.

Soil Properties Due to similar engineering characteristics, the fill and residual soils are combined for purposes of stability analysis. Three representative samples exhibited an average moist density of 115 pcf. Two representative samples of the encountered bedrock exhibited an average moist density of 110 pcf. As described previously in this report, direct shear testing of a representative weathered bedrock sample by a previous investigator yielded a friction angle of 33 degrees and apparent cohesion of 157 psf within the range of confining pressures under consideration. This compares with a friction angle of 32 degrees and cohesion of 621 psf published by CGS[11] for similar materials.

Ground Water Five (5) boreholes advanced on the property by the current and previous investigators did not encounter ground water to the maximum depth explored of 8.5 feet. The local ground water conditions are likely to be significantly influenced by the deep drainages located to the immediate north and west. Consequently, the ground water level is assumed to be lower than the analyzed section.

Seismic Loading For this residential project, we applied a horizontal ground acceleration with a 10 percent probability of exceedance during a 50-year period as calculated by the CGS[17] to be 0.65g. Using the 15-centimeter displacement criteria as suggested by Mr. McCrink for an assumed magnitude 7.8 event, a pseudostatic reduction factor of 0.49 was applied to the probabilistically determined seismic coefficient, yielding a pseudostatic seismic coefficient of 0.32g.

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[16] Stark, T.D., 2003, Analysis of Landslides: Shear Strengths, Testing, and Stability Methods, Short Course.

[17] California Geologic Survey, 2005, Probabilistic seismic hazards mapping ground motion page, http://www.consrv.ca.gov/CGS/rghm/pshamap/pshamap.asp.

Findings

The factor of safety is the ratio of available forces to resist failure, such as friction and cohesion, to the forces that would tend to induce failure, such as gravity and seismic loading. Limited Based on the described site properties and anticipated seismic loading conditions, described seismic screening analysis yielded a factor of safety against earthquake induced landsliding of the subject slope of 1.15 (Figure 6). These values exceed the minimum screening analysis factor of safety criteria of 1.0.

Based on the results of the analyses discussed herein, the risk of seismically induced landsliding adversely impacting the proposed development is judged to be low.

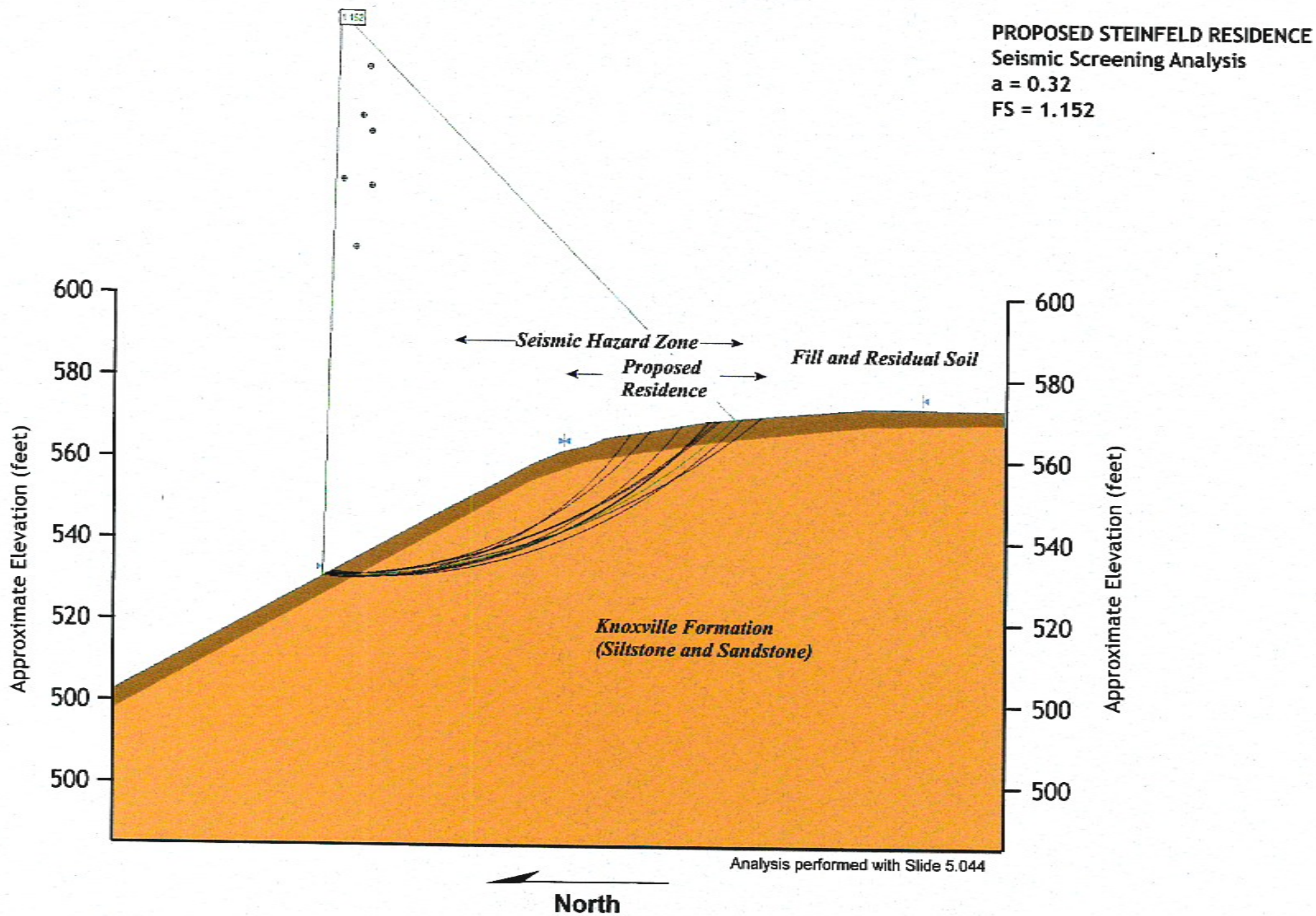
CONCLUSIONS

Based on the findings of this investigation, it is our opinion that the geotechnical conditions of the site are suitable for the proposed landscape improvements provided that the geotechnical criteria presented in this report are incorporated into the design and construction. We conclude that the primary geotechnical factors affecting the design and construction of the proposed improvements are the presence of relatively weaker and potentially expansive-prone near-surface soils and the potential for significant ground shaking caused by an earthquake on the nearby active San Andreas fault and Hayward fault systems.

GEOTECHNICAL DESIGN CRITERIA

The following recommendations are presented as guidelines for subsequent stages of development. These recommendations shall be incorporated into the siting and design of the proposed site improvements. Final detailing of concrete elements and reinforcing steel is to be designed by a qualified structural engineer in accordance with the provided geotechnical criteria.

To assure that the intent of these recommendations is included in the project plans and specifications, we request an opportunity to review the plans prior to initiation of construction. It has been our experience that the permit process is often expedited when we review the plans prior to submittal. References to ASTM test designations are intended to indicate the most recent version at the time of construction.



MILSTONE GEOTECHNICAL

SEISMIC SCREENING ANALYSIS			FIGURE NO.
PROPOSED STEINFELD RESIDENCE 2366 Rainbow Court Hayward, California			6
Date August 2018	Scale 1 inch = 10 feet	Engineer BSM	Project No. 174840

Grading

It is currently anticipated that grading will be limited to, minor cuts and fills to achieve design subgrade elevations and limited amounts of retaining wall backfill. Based on the experience of exploratory borehole drilling, it is expected that proposed site excavations can be performed with conventional earthmoving equipment. It is anticipated that much of the excavated weathered bedrock will be suitable for use as engineered fill.

Clearing and Site Preparation

All areas to be graded should be cleared of organic laden soil and obstructions such as buried utility lines. Stripped materials should be removed from the property for proper disposal. Holes created by the removal of root balls or other debris extending below the proposed finished subgrade should be backfilled with engineered fill as described below. Disturbed soil subgrades to receive fill, should be scarified to a depth of six (6) inches, moisture conditioned to within two (2) percent of optimum, and recompacted to a minimum of 90 percent of the maximum dry density as determined by the ASTM D1557 test method.

Material for Fill

Any fill to be placed at the site should not contain rocks or lumps greater than four (4) inches in greatest dimension and should not contain greater than 15 percent (by dry weight) larger than two-and-one-half (2.5) inches. Fill material in areas to receive structures or within five (5) feet of the ground surface should have a maximum plasticity index of 12. Minimum 50-pound samples of materials to be used as engineered fill should be submitted to the project geotechnical engineer for review and approval prior to placement. Granular soil from the proposed excavations, with the exception of surficial soils and oversized rock fragments, is expected to be suitable for use as engineered fill.

Fill Placement and Compaction

Fill should be moisture conditioned to within two (2) percent of optimum, spread in horizontal lifts not exceeding eight (8) inches in loose thickness, and compacted with an approved mechanical compactor to a minimum of 95 percent of the maximum dry density as determined by the ASTM D1557 test method. Fill placed in landscape areas that will not support structures or vehicular traffic may be compacted to a minimum of 90 percent. The upper 12 inches of fill in landscape areas may be compacted to a minimum of 85 percent to promote growth of vegetation.

Cut and Fill Slope Design: New permanent cut slopes and backfill slopes should not exceed inclinations of two to one (2 to 1) horizontal to vertical.

Building Foundations

Due to the presence of relatively weak near-surface soils, hillside setting, and anticipated seismic shaking, the residence foundation should be supported on a drilled, cast-in-place, concrete piers and grade beams that extend through the surficial and residual soils to derive bearing through friction in the underlying weathered siltstone and sandstone. All foundation piers should be interconnected by grade beams or tie beams. Maximum total and differential settlement of the drilled pier and grade beam supported foundations is estimated to be one-half (1/2) inch or less. Final design of foundation configuration, connections, and reinforcement to be determined by a qualified structural engineer based on the following geotechnical design criteria:

Minimum Pier Diameter	16 inches.
Minimum Pier Depth	Ten (10) feet into competent weathered siltstone that is estimated to be encountered ad depths ranging from about two to four (2 to 4) feet below the existing ground surface.
Minimum Pier Spacing	Three (3) pier diameters, center to center.
Maximum Pier Spacing	10 feet.
Allowable Shaft Friction	In competent weathered sandstone: 600 psf in compression; 500 psf in uplift resistance. Neglect shaft friction in overlying colluvial and fill soils. Increase by 33% for transient loads such as wind or seismic.
Lateral Resistance	350 pounds per cubic foot per foot (pcf/f) equivalent fluid pressure in weathered sandstone. Apply resistance over two (2) pier diameters.

Neglect passive resistance in overlying surficial soils.
Increase by 33% for seismic or wind loads.

Minimum Pier Reinforcement Four (4) - vertical No. 4 bars with No. 3 spirals or ties at maximum 12-inch spacing. Reinforcement to be provided with a minimum of three (3) inches concrete cover. Reinforcing cages to be constructed to allow introduction of tremie pipe to bottom of pier.

Grade Beams Perimeter grade beams should be embedded a minimum of 18 inches below adjacent exterior grade and 12 inches below the lowest adjacent interior grade. As a minimum, all grade beams should be reinforced with two (2) No. 4 bars, top and bottom with No. 3 stirrups at maximum 12-inch spacings.

Construction Contractors should be made aware that exploratory boreholes met considerable resistance at relatively shallow depths. Consequently, they should mobilize appropriately-sized drilling equipment to achieve the required depths.

Pier holes should be free of standing water and cleared of all loose debris prior to placement of concrete. Although not currently anticipated, if standing water collects in the pier excavations, the water should be pumped out or the concrete should be placed by the tremie method with the concrete displacing the water from the bottom up. If casing is required to maintain excavation stability, the casings shall be removed during placement of the concrete so that the concrete will cure in contact with native soil. Uncased holes that encounter groundwater should be poured within 24 hours of drilling.

All pier excavations should be inspected and approved by the project geotechnical engineer prior to the placement of reinforcing steel. Concrete over-pour ("mushrooming") of piers and grade beams should be prevented with the use of "sono-tubes" where required.

Retaining Walls

Foundation retaining walls required to achieve grade along the upgradient sides of the structure should be constructed integrally with the building foundation. Site retaining walls may be supported on drilled pier and grade beam foundations as described for the building foundation or on continuous cantilever footings bearing on approved weathered siltstone that is encountered below the surficial soils. Total and differential settlements of retaining walls supported on shallow footings are estimated to be less than one-half (½) inch. Retaining walls are to be designed in accordance with the following geotechnical criteria:

Lateral Loading — Unrestrained: 45 pcf/f equivalent fluid pressure
Restrained: 60 pcf/f equivalent fluid pressure.

Seismic Surcharge — As described by Lew and others[18], the evaluation of seismic earth pressures for unrestrained walls less than 12 feet tall is not necessary provided the walls are designed for a factor of safety of at least 1.5.

Wall Drainage — Positive drainage to daylight must be provided behind all retaining walls exceeding 18 inches in height. The drain should consist of a minimum 12-inch wide vertical blanket of Caltrans Class 2 permeable material or clean, one-half to three-quarter (1/2 to 3/4)-inch drainrock that is completely enveloped by filter fabric such as Mirafi 140N. Drainage materials should be left 12 inches below the ground surface and the top 12 inches of wall backfill should consist of compacted, low permeability material separated from the drainrock by a double layer of non-woven filter fabric. Due to the low likelihood of collected water, the walls may be drained by screened, minimum two (2)-inch diameter weep holes located at maximum four (4) foot spacings that are integrated with the back drain.

If weep holes are not desired, an approved, minimum four (4)-inch diameter, perforated, rigid, smooth-wall, drain-pipe (or approved functional equivalent) should be placed with perforations pointed downward on a minimum one (1)-inch thick drainrock layer over the retaining wall heel. The pipe should be sloped to drain at a minimum

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18 Lew, L., Sitar, N., Al Atik, L., Pourzanjani, M., and Hudson, M.B., 2010. "Seismic Earth Pressures on Deep Building Basements", SEAOC 2010 Convention Proceedings.

inclination of one (1) percent. The use of 90-degree angled connections should be strictly avoided in favor of long sweep-90 connections or combinations of maximum 45-degree angled connections. Drain lines should be provided with appropriate and sufficient cleanouts. Collected waters should be directed to an appropriate approved discharge location.

Wall Backfill — Retaining wall drainrock and backfill placement and compaction should conform to the requirements for engineered fill and be compacted with appropriate equipment and in a manner to prevent excessive loading to adjacent walls or damage to waterproofing or drainage systems. Waterproofing membranes should be inspected for integrity during backfill placement and compaction.

Deepened Footings Alternative — The proposed retaining walls may be founded on deepened continuous footings that bear in approved, competent, weathered siltstone materials that are anticipated to be encountered approximately three to four (3 to 4) feet below the existing ground surface.

Footing Embedment — Deeper of 18 inches or 12 inches into approved, competent, weathered siltstone that is encountered below the existing artificial fill and surficial soils.

Footing Width — Minimum 24 inches.

Bearing Capacity — 3,000 psf for dead and live loads;
4,000 psf for dead, live, and transient loads such as wind and seismic.

Passive Resistance — 300 pounds per cubic foot per foot (pcf/f) equivalent fluid pressure against the face of footings embedded in weathered bedrock. Neglect passive resistance within artificial fill and surficial soils. Alternatively, lateral resistance may be derived by friction along the base of the footing calculated using a friction factor or 0.3.

Drilled Pier Alternative — Proposed retaining walls may be founded on drilled, cast-in-place concrete piers and grade beams with piers deriving support through skin friction in the weathered siltstone that is encountered below the existing artificial fill and surficial soils. It is estimated that competent bearing materials will be

	encountered approximately three to four (3 to 4) feet below the existing ground surface.
Pier Embedment	Minimum eight (8) feet into approved, competent, weathered bedrock.
Pier Diameter	Minimum 16 inches.
Pier Spacing	Minimum three (3) pier diameters, edge to edge; Maximum eight (8) feet.
Shaft Friction	500 pounds per square foot (psf) in approved weathered bedrock; Neglect shaft resistance in artificial fill and surficial and residual soils.
Passive Resistance	300 pounds per cubic foot per foot in weathered bedrock applied across two (2) pier diameters. Neglect passive resistance within artificial fill and surficial soils. Alternatively, lateral resistance may be derived by friction along the base of the footing calculated using a friction factor or 0.3.
Minimum Reinforcement	Four (4) - vertical No. 4 bars with No. 3 spirals or ties at maximum 12-inch spacing. Reinforcement to be provided with a minimum of three (3) inches concrete cover. Reinforcing cages longer than 10 feet to be constructed to allow introduction of tremie pipe to bottom of pier.
Pier Construction	Pier holes should be free of standing water and cleared of all loose debris prior to pouring of concrete. Although not currently anticipated, if standing water collects in the pier excavations, the water should be pumped out or the concrete should be placed by the tremie method with the concrete displacing the water from the bottom up. Concrete in piers exceeding 10 feet should be placed using the tremie method. If casing is required to maintain excavation stability, the casings shall be removed during placement of the concrete so that the concrete will cure in contact with native soil. Uncased holes that encounter groundwater should be poured within 24 hours of drilling. All pier excavations should be inspected and approved by the project geotechnical engineer prior to the placement of reinforcing steel. Concrete

over-pour ("mushrooming") of piers and grade beams should be prevented with the use of "sono-tubes" where required.

Surface Drainage

Positive surface drainage, with a minimum slope five (5) percent, should be provided away from the structures for a minimum distance of 10 feet as mandated by the current California Building Code. Where this is not possible due to topographic considerations, alternate approaches such as lined surface swales or low permeability surface treatments should be considered to limit the introduction of surface runoff to the building foundation.

All roof sections should be provided with gutters connected via downspouts to a minimum four (4)-inch diameter, non-perforated, rigid, smooth-wall drain-pipes that have a minimum slope of one (1) percent to discharge at an appropriate discharge facility. The use of 90-degree angled connections should be strictly avoided in favor of long sweep-90 connections or combinations of maximum 45-degree angled connections. Drain lines should be provided with appropriate and sufficient cleanouts and isolated from subsurface drainage facilities.

Final siting of on-site storm drain discharge facilities, such as infiltration trenches or energy dissipaters, should avoid areas immediately downslope of proposed improvements and should be determined in the field by the project architect, civil engineer, and geotechnical engineer. The use of drought tolerant landscaping is encouraged to limit irrigation requirements.

Concrete Slabs-on-Grade

Exterior concrete slabs may be constructed on grade in accordance with the following recommendations. Slabs should bear on approved, competent, inorganic, native, silty sand or engineered fill that bears on approved subgrade soils, up to a maximum of 18 inches. Engineered fill beneath concrete slabs in living areas should be of uniform thickness.

The slabs-on-grade should be underlain by a minimum of six (6) inches of compacted Caltrans Class 2 permeable material and reinforced with a minimum of No. 4 bars on 18-inch spacings in both directions. Slabs should be provided with minimum eight (8)-inch by eight (8)-inch thickened edges. Final design of slab thickness, steel reinforcement, load-transfer devices, and crack control features should be determined by the structural engineer.

Interior slabs in living areas should be structurally tied to, or constructed integrally with, the footings. Exterior slabs should be structurally isolated from adjacent structures although a sleeved dowel connection may be used at entrances to limit differential vertical displacement.

Interior slabs should be provided with a comprehensive moisture/vapor barrier as described in a subsequent Moisture Control section of this report. Exterior slab moisture and potential efflorescence can be limited with a moisture barrier consisting of a minimum 10-mil thick waterproof membrane that is protected from construction-related damage.

Seismic Design Criteria

The site is expected to experience strong ground shaking from earthquakes along active faults located within the region during the design life of the project. Peak probable horizontal ground accelerations of 0.65g have been predicted by probabilistic methods. As a minimum, the structure should be designed to resist lateral loads resulting from ground shaking as provided in the current California Building Code (CBC) or other accepted design methods. Based on the observed site conditions, we conclude the following design parameters to be appropriate for design using the 2016 California Building Code design method:

Seismic Design Parameters

PARAMETER	VALUE
Site Class	B
S_S (0.2s Spectral Response Acc.) Default Site Class B	2.397
S_1 (1.0s Spectral Response Acc.) Default Site Class B	0.997
S_{MS} (0.2s Spectral Response Acc.)	2.397
S_{M1} (1.0s Spectral Response Acc.)	0.997
S_{DS} (0.2s Spectral Response Acc.)	1.598
S_{D1} (1.0s Spectral Response Acc.)	0.665
F_a (Site Class B)	1.0
F_v (Site Class B)	1.0

For additional guidance on reducing the risks associated with living in seismically active areas, owners may wish to consult "Putting Down Roots in Earthquake Country" [19] (available on-line at the US Geological Survey), which references additional useful documents.

Moisture Control

To minimize efflorescence at the face of exposed exterior walls, the blind sides of the walls may be sealed with a continuous, minimum 15-mil water/vapor barrier that is functionally equivalent to Tremco's *Paraseal LG* or Grace's *Bituthene 3000*.

Installation, lapping, and sealing of waterproofing membranes should be performed in accordance with the manufacturers' recommendations. It is recommended that return corners, such as at wall/footing joints, be provided with a cant strip or sloping infill to reduce the potential for damage to the overlying waterproofing membranes. Waterproofing membranes should be protected from drainrock and backfill with a rigid panel or prefabricated drainage panel. It is critical that waterproofing systems be installed correctly by qualified professionals.

Underground Utilities

Underground utility pipes and conduits should be bedded with approved free-draining sand or quarry-fines. Trenches should be backfilled with compacted on-site or import fill material that does not contain rocks or lumps greater than three (3) inches in size. The backfill should be moisture conditioned to within two (2) percent of optimum, placed in maximum six (6)-inch horizontal layers and compacted by mechanical means to 90 percent of the maximum dry density as determined by the ASTM D1557 test method. The upper 24 inches of fill below exterior surface improvements (such as paved areas) should be backfilled with non-expansive soil and compacted to 95 percent of the maximum dry density. Compaction of trench backfill by flooding, jetting, or other non-mechanical means shall not be permitted.

Sloping trenches should be provided with minimum 12-inch thick, low permeability cutoff walls (such as clay or controlled density pumpable fill (CDF)) at maximum lateral intervals of 25 feet to limit the migration of bedding soils.

[19] United States Geological Survey, 2005, Putting down roots in earthquake country, General Information Product 15, http://pubs.usgs.gov/gip/2005/15/.

Erosion Protection

Project contractors should be responsible to install and maintain adequate erosion protection facilities to protect offsite areas from construction activities throughout the project. At a minimum, erosion protection should consist of properly installed fiber rolls or erosion fencing below the downslope limits of grading. Disturbed slopes should be protected with appropriate erosion resistant matting or hydromulch.

TECHNICAL REVIEW

This report should be reviewed by the project architect, engineers, contractors, and potential sub-contractors prior to the next stage of development. A copy of this report should also be provided to the general contractor for reference during construction. Any questions or discrepancies should be brought to the attention of a representative of Milstone Geotechnical prior to the start of construction.

We request an opportunity to review the final plans, design calculations, and specifications prior to construction to confirm that our recommendations have been incorporated and, if necessary, to provide supplemental recommendations. It has been our experience that the permit process may be expedited if we review the plans prior to submittal.

CONSTRUCTION OBSERVATION

Foundation site preparation, footing and slab subgrade preparation, pier drilling, installation of waterproofing and drainage systems, and placement of engineered fill and backfill should be observed by the project geotechnical engineer (prior to placement of steel and pouring of concrete) to verify that the encountered site conditions are the same as those anticipated by this investigation and to verify conformance with our recommendations. A minimum of three (3) working-days notification prior to construction activities requiring inspection services is required. The cost of these services will be charged on a time-and-expenses basis.

Geotechnical plan review and construction observation are conducted to reduce - not eliminate - the risk of problems arising during construction, and provision of the service does not create a warranty or guarantee of any type. In all cases, contractors shall retain responsibility for the quality and completeness of their work, for adhering to the plans, specifications, and recommendations on which their work is based, and for contacting the appropriate parties in a timely manner regarding construction activities that require inspection or observation services.

It is suggested that an on-site pre-construction meeting be conducted with the owner, designer, geotechnical engineer, general contractor, and appropriate subcontractors (such as excavation and grading) prior to the start of construction to establish project expectations and communication protocol.

LIMITATIONS

These services consist of professional opinions and recommendations made in accordance with generally accepted engineering geologic and geotechnical engineering principles and practices in the San Francisco Bay Area at the time this report was written. The investigation was performed, and this report prepared, for the exclusive use of the client, and for specific application to proposed site development as outlined in the body of the report. No third-party shall have the right to rely on the findings, opinions, or recommendations rendered in connection with this investigation without the written consent of Milstone Geotechnical. No warranty, express or implied, or merchantability of fitness, is made or intended in connection with this work, by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings.

This report is issued with the understanding that the owners choose the risk they wish to bear by the expenditures and savings involved with the chosen construction alternatives. The recommendations and design criteria presented in this report are contingent upon a representative of Milstone Geotechnical being retained to review the final plans and specifications and to provide testing and inspection services for all earthwork and construction operations.

Unanticipated soils and geologic conditions are commonly encountered during construction and cannot be fully determined from existing exposures. If conditions encountered in the field are different than those anticipated by this report, our firm should be contacted immediately to provide any necessary revisions to the recommendations.

APPENDIX A
FIELD AND LABORATORY INVESTIGATION

Description of Small-Diameter Borehole Investigation
Soil Classification Chart
Log of Exploratory Borehole MG1
Log of GEI Borehole 15
Direct Shear Test Results (GEI)

SUBSURFACE INVESTIGATION DESCRIPTION

The small-diameter exploratory borehole MG1 was drilled and logged on February 21, 2018 at the location shown on Figure 4. The borehole was advanced using a 3.5-inch diameter hand auger. The borehole was drilled and sampled to a depth of 4.5 feet.

Earth materials encountered in the borehole were continuously logged and described in the field by a registered geotechnical engineer and representative soil samples were obtained at various depths. Relatively undisturbed samples were obtained with a three (3)-inch-outside-diameter, two-and-one-half (2.5)-inch-inside-diameter, sampler with a six (6)-inch-long, thin walled brass liner. The sampler was advanced using an 18-inch, 10-pound slide hammer. In-situ testing was performed at five (5) locations using a down-hole vane shear device.

Upon the completion of logging, the borehole was backfilled with loosely compacted drill cuttings. All soil samples were transported to the laboratory to verify field descriptions and perform index and strength testing. The laboratory test results are summarized in the body of this report.

A graphical log of the borehole and a key to soil classification follow in this appendix. The following log and related information show our interpretation of the subsurface conditions at the dates and locations indicated. It is not implied that they are representative of subsurface conditions at other locations or at other times.

CRITERIA FOR ASSIGNING GROUP SYMBOLS AND GROUP NAMES			SOIL CLASSIFICATION		
			GRAPHIC SYMBOL	USCS GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS LARGER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES		GW	Well graded gravel
				GP	Poorly graded gravel
		GRAVELS WITH MORE THAN 12% FINES		GM	Silty gravel
				GC	Clayey gravel
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES		SW	Well graded sand
				SP	Poorly graded sand
		SANDS WITH MORE THAN 12% FIINES		SM	Silty sand
				SC	Clayey sand
FINE-GRAINED SOILS MORE THAN HALF IS SMALLER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%	INORGANIC		ML	Low plasticity silt
				CL	Low plasticity clay, Lean clay
		ORGANIC		OL	Low plasticity organic silt, Low plasticity oganic clay
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%	INORGANIC		MH	High plasticity silt, Elastic silt
				CH	High plasticity clay, Fat clay
		ORGANIC		OH	Medium to high plasticity organic silt or clay
	HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER		PT	Peat

Note: Blow-counts reported for samplers other than a Standard Penetration Split Spoon Sampler were obtained by empirically converting the number of blows required to drive the sampler through the last 12 inches of an 18-inch penetration to the equivalent number of blows using a Standard Penetration Split Spoon Sampler.

Note:The borehole logs depict our interpretation of the subsurface conditions at the dates and locations indicated. It is not warranted that they are representative of subsurface conditions at other times and locations. The lines separating strata on the boring logs represent approximate boundaries only. Actual transitions may be gradual.

ABBREVIATIONS

AD: Auger Drilling

HD: Modified California Sampler

T1: Tube Sample (undisturbed)

B1: Grab Sample (disturbed)

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SOIL CLASSIFICATION CHART
AND
KEY TO LOGS OF EXPLORATORY BOREHOLES

LOG OF EXPLORATORY BOREHOLE MG1

Project: Lands of Steinfeld
Location: 2366 Rainbow Court, Hayward, CA
Drilling Equipment: Hand Auger
Drilling Contractor: -

Project Elev.: ~570 feet
Hole Diameter: 3.5 inch
Surface: bare soil

Project Number: 174840

Logged By: BSM
Date: 11/29/17

GROUND WATER	POCKET TORVANE (tsf)	POCKET PENET. (tsf)	RECOVERY (%)	SPT (bpf) or PRESS. (psi)	SAMPLE OR DRILL MODE	SAMPLE DESIG-NATION
					AD	
		1.8	6/6		HD	T1
					AD	B1
		-	4/6		HD	T2
		2.9	3/6		HD	T3
					AD	B3
		>4.5	6/6		HD	T4
					AD	B4
		>4.5	5/6		HD	T5

DEPTH IN FEET: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

USCS DESIG.	GEOTECHNICAL DESCRIPTION
	ARTIFICIAL FILL
SM	Silty SAND: Dark yellowish brown (10YR3/4); ~80% fine to medium grained sand; ~20% low plasticity fines; loose; moist; rootlets in upper 3 inches; glass fragment.
GM	Silty GRAVEL: Dark yellowish brown (10YR3/4); ~60% hard, angular gravel; ~30% fine to medium grained sand; ~10% low plasticity fines; dense; moist.
	RESIDUAL SOIL
SC-SM	Clayey to silty SAND with Gravel: Dark yellowish brown (10YR4/4); ~15% fine, hard, angular, gravel; ~60% fine grained sand; ~25% medium plasticity, tough fines; medium dense; moist. Resistant drilling at 4 feet.
	WEATHERED SILTSTONE
GM-SM	Silty GRAVEL to Silty SAND: Dark yellowish brown(10YR4/4); ~45% medium hard, angular, siltstone fragments to 1/2-inch size; ~40% fine to coarse grained sand; ~15% low to medium plasticity fines; very dense; moist. Drilling refusal at 4.5 feet.

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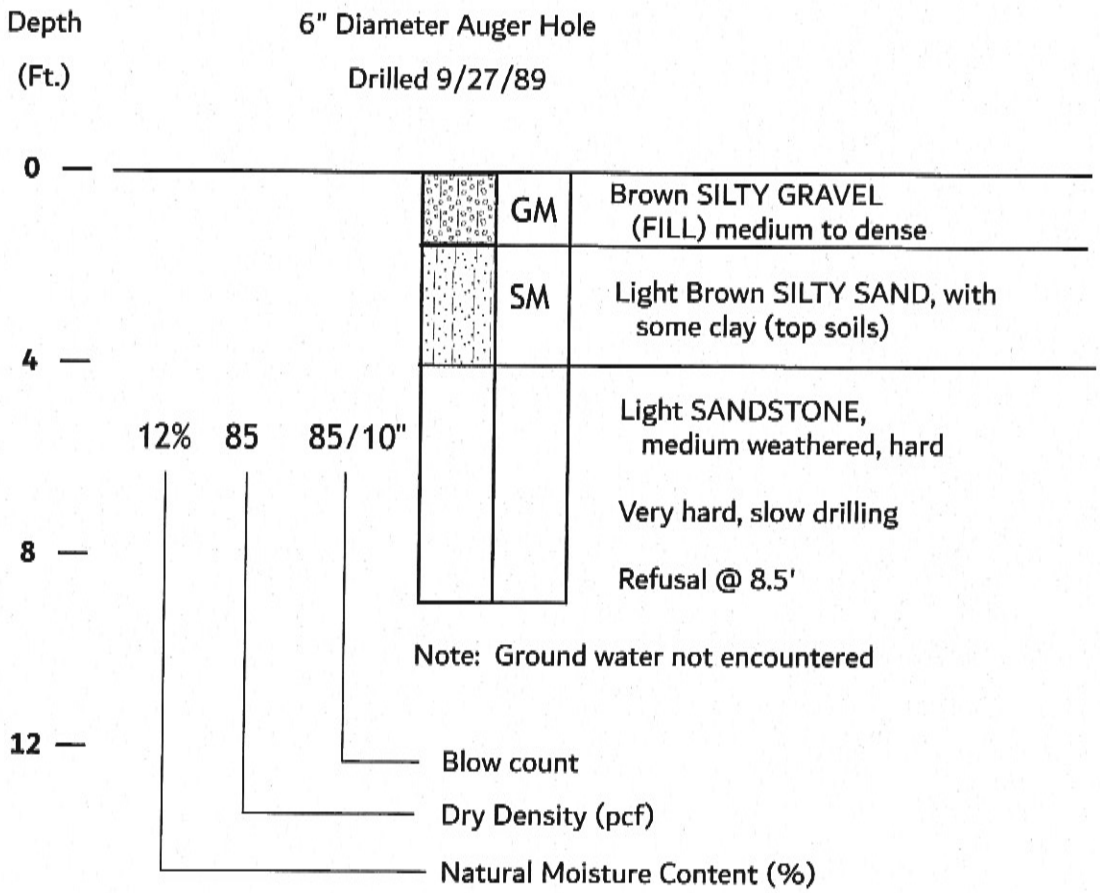
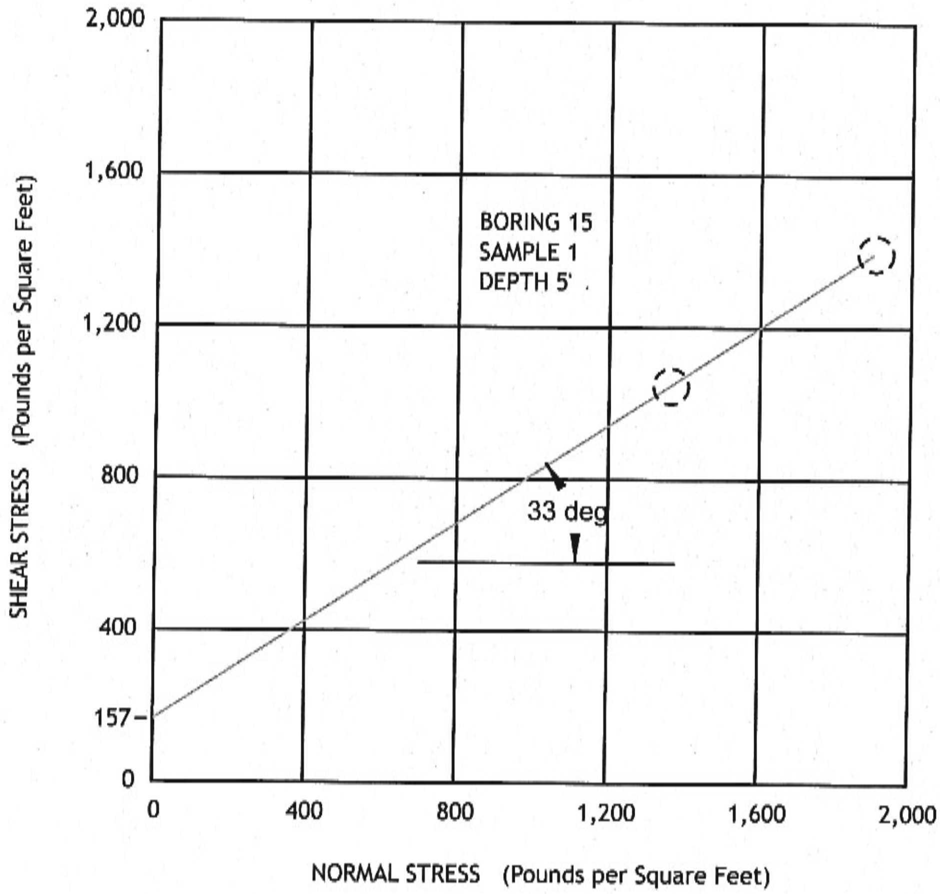
Remarks: Borehole terminated at 4.5 feet.
No ground water encountered.
Borehole backfilled with tamped cuttings.



Trace of borehole log presented in *Geotechnical Engineering Inc., November 2, 1989, Report - Supplementary Investigation and Geologic Reconnaissance, Proposed Residential Development, Parkside Drive & Rainbow Court, Tract 3992, Hayward, California* for Victoria Court Management.

MLSTONE GEOTECHNICAL	GEI BOREHOLE 15 STEINFELD RESIDENCE 2366 Rainbow Court Hayward, California			FIGURE NO.
	Date:	Scale:	Drawn by:	Project No. 184920

SATURATED DIRECT SHEAR TEST



Modified from direct shear test presented in *Geotechnical Engineering Inc., November 2, 1989, Report - Supplementary Investigation and Geologic Reconnaissance, Proposed Residential Development, Parkside Drive & Rainbow Court, Tract 3992, Hayward, California* for Victoria Court Management.

MLSTONE GEOTECHNICAL	DIRECT SHEAR TEST STEINFELD RESIDENCE 2366 Rainbow Court Hayward, California			FIGURE NO.
	Date: 11/2/89	Scale: as shown	Drawn by: BSM	Project No. 184920