ATTACHMENT IX

LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

FAULT HAZARD EVALUATION

SUBMITTED TO
Mr. Gant Bowman
Eden Housing, Inc.
22645 Grand Street
Hayward, CA 94541

PREPARED BY
ENGEO Incorporated

April 23, 2020

PROJECT NO.
15577.000.000

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GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
15577.000.000

April 23, 2020

Mr. Gant Bowman
Eden Housing, Inc.
22645 Grand Street
Hayward, CA 94541

Subject: La Vista Residential Development
Hayward Parcel 3
Hayward, California

FAULT HAZARD EVALUATION

Dear Mr. Bowman:

With your authorization, we have completed this fault rupture hazard assessment for the proposed development of Parcel 3, located in Hayward, California. The accompanying report presents the findings of our paper studies, field exploration and reconnaissance mapping, and recommendations regarding geologic hazards at the site.

We are pleased to be of service to you on this project. If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

ROBERT BOECHE
No. 2318

Curtis E. Hall, PG

Robert H. Boeche, CEG

ceh/rhb/cjn

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this study has been to evaluate the potential for surface fault rupture within proposed development areas and conclude feasibility of development for Eden Housing, Inc. This study included the following scope of services:

- Review available literature and published geologic maps pertinent to the site.
- Perform a geologic site reconnaissance to observe site conditions and evidence (if any) of active fault creep.
- Review of available aerial photographs to identify geomorphic features that may be related to faulting, landsliding, and other geologic conditions.
- Excavation and detailed logging of two exploratory trenches.
- Preparation of this report summarizing our findings, conclusions, and recommendations to assist in site planning.

We prepared this report exclusively for you and your design team consultants. ENGEO Incorporated (ENGEO) should review any changes made in the character, design or layout of the development to modify conclusions and recommendations contained in this report, as necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO.

1.2 SITE DESCRIPTION AND PROPOSED DEVELOPMENT

Figure 1 displays a Site Vicinity Map. The site is located east of Mission Boulevard and north of Valle Vista Avenue (a renamed section of Tennyson Road east of Mission Boulevard), and consists of Alameda County Assessor's Parcel Numbers (APN) 78C-626-3-9, and 78C-626-1-7. In general, the site has remained generally undeveloped, and currently contains access roadways and minor structures, including horse stalls, storage sheds, and a riding ring.

Elevations across the site range from a low of 116 feet (msl), in the southwest corner, up to 275 feet along the northeast portion of the site. Slope gradients in this area generally range from 10:1 (horizontal:vertical) to 2:1 or steeper.

Based on the site plan provided by you, and developed by Architects Orange, we understand that the development will consist of multi-family residences, a charter school, and associated improvements, parking, and roadways.

2.0 BACKGROUND AND PREVIOUS STUDIES

Portions of the site are located within a State of California Earthquake Fault Zone for the active Hayward fault. The main trace of the Hayward fault is mapped by the California Geological Survey (CGS) just east of the site with two active splays mapped through the parcels. The site was part of a fault exploration performed by ENGEO in 2016 for William Lyon Homes, and several fault studies have been performed on the nearby La Vista Quarry and Ersted properties, as discussed below.

2.1 BERLOGAR GEOTECHNICAL CONSULTANTS (2000, 2001, AND 2005)

In 2000, Berlogar Geotechnical Consultants (BGC) performed a fault study, which included excavating several trenches throughout the La Vista Quarry property. Several faults were logged by BGC and shown projecting into the site (Figure 3). The 2001 BGC report states:

"…several subsidiary faults (Faults A, B, C, D and suspected Fault E) were mapped. These faults appear to be part of a system of relatively short fault segments included in the Hayward fault zone that trend northwest. These faults appear to constitute a zone of shearing and/or compression between the concentrated fault zone and a possible thrust fault (Crane 1988) located near the base of the hills west of the site."

In 2001, BGC performed additional fault trenching, which included trenches crossing the main Hayward fault trace. BGC encountered fault features in multiple trenches, including offset of relatively young soil and fault bound soil and bedrock contact. In conclusion, BGC mapped "Concentrated Fault Zone" that is generally in line with the mapped State trace.

Additional fault trenching was performed by BGC in 2005 associated with development of a community center site on the La Vista Quarry Property. As shown on Figure 2, Trenches T-18 to T-21 were excavated and logged as well as a bedrock exposure located within an old excavation (E-1). Greenstone and shale bedrock were exposed in E-1 and T-18 and a portion of T-19. Trench T-20 encountered 15 feet of unsheared soil that was described by BGC as a "Pleistocene weathering profile". Trench T-21 encountered unfaulted soil, described as Pleistocene age, over greenstone bedrock. In conclusion, BGC delineated an area acceptable for development based on the absence of faulting.

2.2 JUDD AND CRAWFORD & ASSOCIATES (1975 AND 2012)

As summarized in an ENGEO peer review in 2013, a secondary fault zone approximately 900 feet west of the main Hayward fault trace, was logged by Judd (1975) and Crawford & Associates (2012) east of the site. Due to previous grading activities that have resulted in loss of surficial soil, the age of faulting could not be determined and structural setback was recommended. This fault zone projects northwest towards the site.

2.3 ENGEO INCORPORATED (2005 AND 2007)

In 2005 and 2007, ENGEO performed fault explorations for the Ersted property located east of the site along Mission Boulevard. The exploration included excavating and logging over 1,500 linear feet of trench to identify the presence of active faulting. Two of the fault trenches crossed a fault trace previously mapped by Soil Engineering Construction, Inc. (SEC) in 1973. The fault trace mapped by SEC was shown striking northwest and dipping 40 to 60 degrees to the northeast. Four fault traces were encountered by ENGEO in Trench ET-1. The fault traces, which include the approximately 30- to 100-foot-wide zone, were observed to show signs of active faulting, including Franciscan bedrock juxtaposed to colluvial deposits. The eastern portion of ET-4 extended into sheared serpentinite in contact with soil units and was determined to be the western limit of the main trace of the Hayward fault zone.

2.4 ENGEO INCORPORATED (2016)

In 2016, ENGEO performed fault explorations within the current site as part of a larger fault exploration. The exploration included excavating and logging six trenches within the site, totaling approximately 1,455 linear feet of trench, to identify the presence of active faulting. Figure 2 shows the approximate locations of these trenches. As shown in Figure 3, Trenches T-2, T-3, T-5, and T-8 show a wide zone of faulting and shearing, with more distinct faulting observed in Trenches T-4 and T-6.

3.0 GEOLOGY

3.1 REGIONAL GEOLOGIC SETTING

The site is located in the Coast Ranges geomorphic province of California. The Coast Ranges are characterized by a series of northwest-trending valleys and mountain ranges. The bedrock in this region has been folded and faulted in a tectonic setting that is experiencing translational and compressional deformations of the earth’s crust.

As depicted on Figure 4, regional geologic mapping by Graymer (2000), the site is mapped as predominantly underlain by Cretaceous to late Jurassic Knoxville formation of the Great Valley Sequence, with a sliver of Jurassic Keratophyre mapped just east of the study area. Geologic mapping by BGC on the adjacent property (La Vista Quarry) depicts beds of Franciscan greenstone within the Knoxville formation west of the Hayward fault.

Regional landslide mapping by Nilsen (1975) depicts swales and low-lying areas adjacent to the foothills as colluvial or alluvial deposits, with no mapped landslides crossing the site. Mapping by Dibblee (2005) shows a large landslide west of the Hayward fault, with portions crossing areas of the site.

3.2 FAULTING AND SEISMICITY

As noted above, portions of the site are located within a State of California Earthquake Fault Zone for the active Hayward fault (Figures 3 and 6). The main creeping trace of the Hayward fault is mapped by the CGS approximately 50 feet east of the site, with two splays mapped west of the main trace directly through the site. A recently active fault trace is defined by the state of California as displaying signs of displacement within the last 11,000 years (Hart, 2007).

3.2.1 Hayward Fault

The Hayward fault is one of the main branches of the San Andreas fault displaying predominantly right-lateral displacement. The approximately 60-mile-long fault extends from San Jose along the East Bay Hills to Point Pinole and possibly right stepping and passing strain to the active and en echelon Rodgers Creek fault system (Lienkaemper, 2008). Although much of the area surrounding the fault system has experienced rapid urbanization over the last several decades, many geomorphic features indicative of strike-slip faulting can still be observed along the fault trace, such as, right-laterally offset drainages, shutter ridges, sag ponds and rift or hillside valleys. The Hayward fault has been extensively studied and the active creeping trace is well defined in the site vicinity.

The first regional maps of the Hayward fault were produced by Radbruch (1969), which included the possible location of the 27-mile long surface rupture associated with the 1868 earthquake. The Radbruch mapping included noted geomorphic features and compiled evidence of creep along the fault system. Radbruch’s mapping was the basis for the original 1974 Special Studies Zone map for the area.

The USGS Quaternary Fold and Fault Database (QFFD) is a nationwide GIS-based database that identifies fault locations and classifies faults based on estimated age. In California, the QFFD is jointly maintained by the USGS and the California Geological Survey (CGS). The QFFD shows the Hayward fault system in the area to be consistent with Earthquake Fault Zone map (Figure 6).

Previous studies by BGC have defined a “concentrated fault zone” for the Hayward fault as depicted east of the study area. This zone represents the main trace of the Hayward fault, is described by BGC as a band of sheared soil and rock, and is shown to be as wide as 300 feet and narrow to 100 feet to the north. The Hayward fault zone as shown by BGC is generally in-line with the 1868 fault rupture mapped by the USGS and CGS.

3.2.2 Seismicity

The San Francisco Bay Area has experienced numerous significant earthquakes in recorded history. In 1868, a major earthquake occurred along the Hayward fault causing a nearly 27-mile long surface rupture and a reported displacement of up to 3 feet. The surface rupture was not mapped until after the 1906 San Francisco earthquake by Lawson (1908) and the direction of displacement is not well documented.

The 2014 Working Group on California Earthquake Probabilities evaluated the regional seismicity of the Bay Area and published their results as The Uniform California Earthquake Rupture Forecast, Version 3 (UCERF 3). The Working Group periodically attempts to summarize seismic risk in California with time-dependent earthquake rupture forecasts, in which the probabilities of future events are conditioned upon the dates of previous earthquakes. According to UCERF 3, there is an aggregated 72 percent probability of 6.7 MW or greater earthquake on an active Bay Area fault over the next 30 years. The probability of a 6.7MW or greater earthquake on the Hayward fault, Calaveras and San Andreas faults are 14, 7, and 6 percent, respectively, over the next 30 years.

4.0 CURRENT INVESTIGATION

Based on the proposed development areas, discussions with you, aerial photograph review, and the extensive faulting observed during previous explorations within the site and adjacent parcels, we located our subsurface excavations to evaluate the potential for surface fault rupture and observe the geologic conditions at the site. Per your request, we focused on a portion of the site along East 16th Street (Figure 2). Our exploration began on May 1, 2019 and continued until May 13, 2019. Numerous faults were observed during our exploration and a majority determined to be active based on displacements within overlying modern soil.

4.1 AERIAL PHOTOGRAPH INTERPRETATION

Stereo-paired aerial photographs were reviewed as part of our study to observe the presence of geomorphic features indicative of faulting such as, linear discontinuities in rock or soil, offset drainages, linear scarps, topographic lows and/or vegetation patterns or breaks in slope.

We observed two photo lineaments in the 1957 and 1959 photographs that represent tonal variations associated with vegetation and topographic break in slope. These lineaments appear to be in-line with faulting observed by ENGEO (2005) and the State-mapped fault trace. Right laterally offset drainages north of the site were observed in-line with the main trace of the Hayward fault.

4.2 EXPLORATORY TRENCH EXCAVATIONS

Two exploratory trenches were excavated in an effort to identify shear features or other evidence of surface fracturing or displacement from seismic activity at the site at the locations shown in Figures 2 and 3. Trench logs completed in this study are presented in Figure 7. The trench locations shown in Figures 2 and 3 were determined by measuring from site landmarks. At the completion of the field exploration, the excavations were loosely backfilled using a track-mounted excavator and slightly mounded with soil to limit ponding of stormwater. Some settling of this non-engineered fill should be expected over time. If further development occurs in the area of the backfilled trenches, the non-engineered fill should be removed and replaced as compacted engineered fill.

The trenches range in depth from approximately 8 feet below existing ground surface (bgs) to approximately 12 feet bgs. We cleaned the southern walls of the trenches with hand tools prior to logging. An ENGEO Certified Engineering Geologist directed the field study with assistance from other office and field staff. We logged the exposures at a scale of 1 inch to 5 feet in the field. These field logs were used to create the report logs included at a scale of 1 inch to 10 feet as Figure 7.

4.3 GEOLOGIC MAPPING

Surface geologic mapping based on photo review, site reconnaissance, and exploratory trenching was performed as part of this study (Figure 3). Below are descriptions of the geologic units observed during mapping of the site.

4.3.1 Artificial Fill (Qaf)

Artificial manmade fill was mapped in the northern portion of the site, and are anticipated to consist of on-site materials. These area are outside of the current proposed development area.

4.3.2 Landslide Deposits (Qls)

Landslide deposits were mapped based on findings within our 2016 Trenches T-2 and T-4. Trench T-4 exposed features indicative of deep-seated bedrock landsliding within the notoriously landslide-rich Knoxville formation. The western end of Trench T-4 transitions into relatively thick, displaced colluvial deposits that are extensively sheared with a continuous slickensided shear plane observed at the base of the trench. Trench T-2 exposed landslide deposits within the Franciscan bedrock, with a distinct basal slip plane observed towards the west, near a natural break in slope topography. Landslide deposits mapped may be on the order of 20 to 40 feet thick, with features indicative of relatively recent movement, including very well developed striated shearing and distortion of Holocene active faulting.

4.3.3 Surface Soil and Colluvium (Qc)

Colluvial deposits were mapped in low-lying drainages with increased vegetation and are anticipated to consist of transported surficial soil derived from the site bedrock.

4.3.4 Bedrock Formations (KJkc, KJfm, and Jsv)

Current Trenches T-1 and T-2 exposed Cretaceous Knoxville formation (KJkc). The Knoxville formation observed on the site consisted mostly of yellowish brown silt and clay shale with sandstone interbeds.

Heavily faulted Franciscan Mélange bedrock (KJfm) was observed at depth in the southern portions of the current trenches, consisting of serpentinite, sandstone, and shale. Additionally, large boulders of Franciscan greenstone were observed within both trenches.

Jurassic Keratophyre (Jsv) was previously mapped in the eastern portion of the site, but was not observed within the current trenches. Graymer (2000) describes the keratophyre as consisting of highly altered intermediate and silicic volcanic and hypabyssal rocks, previously mapped as Leona and Northbrae rhyolite.

5.0 GROUNDWATER

Localized seeps were observed during our site reconnaissance and in aerial photographs. A well-defined seep is located within the northern portion of the site, evidenced by increased vegetation.

At the time of our investigation, groundwater was not observed in our trenches.

6.0 CONCLUSIONS AND RECOMMENDATIONS

As depicted in Figure 3, numerous active faults were observed during our exploration, offsetting soil believed to be late Holocene in age. The faulting observed is likely the western extension of the Hayward fault, currently observed as east-dipping thrust faults. Additionally, active landslides were observed within the site, including a deep-seated bedrock slide within the Knoxville formation and located between the trenches (Figure 3). Based on the observed shearing, it is anticipated and highly likely that the presence of active faulting exists below the landslide deposits, of which age and location are unknown.

Based on the current findings, the proposed development appears feasible. The current layout shows structures setback at least 50 feet from the western most fault trace as depicted on Figure 3.

Although it is outside the scope of this study, given the serpentinite encountered, the site may require a Bay Area Air Quality Management District (BAAQMD) approved monitoring plan prior to construction. A design-level geotechnical exploration should be performed as project planning progresses, which should include borings and/or test pits, and as-needed laboratory testing to provide data for preparation of specific recommendations regarding site grading, corrective grading measures, foundations, and drainage for the proposed residential development.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; no warranty is express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document’s applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO’s documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO’s scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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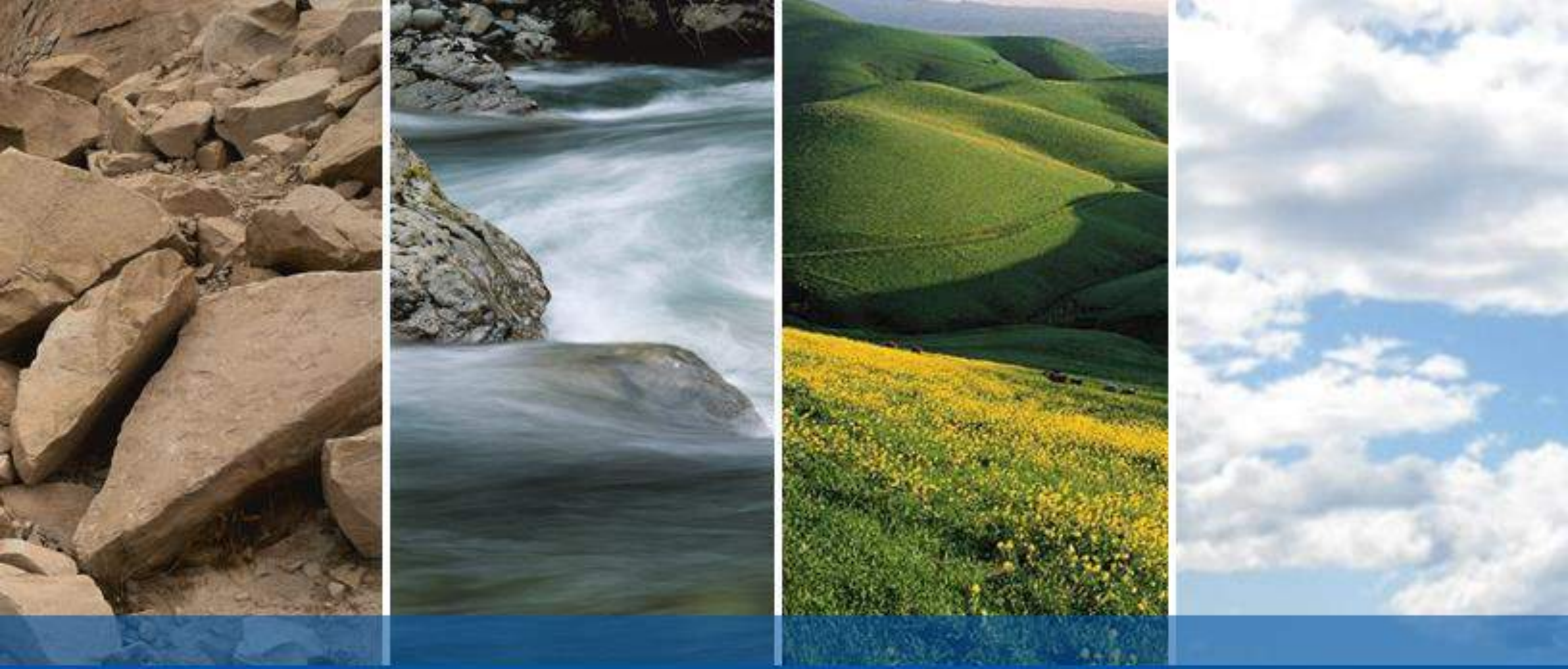

FIGURES

BASEMAP SOURCE: ESRI MAPPING SERVICE 2017

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VICINITY MAP

LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.000
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: RHB
FIGURE NO. 1

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LAYOUT: VICINITY USER: JVERGARA

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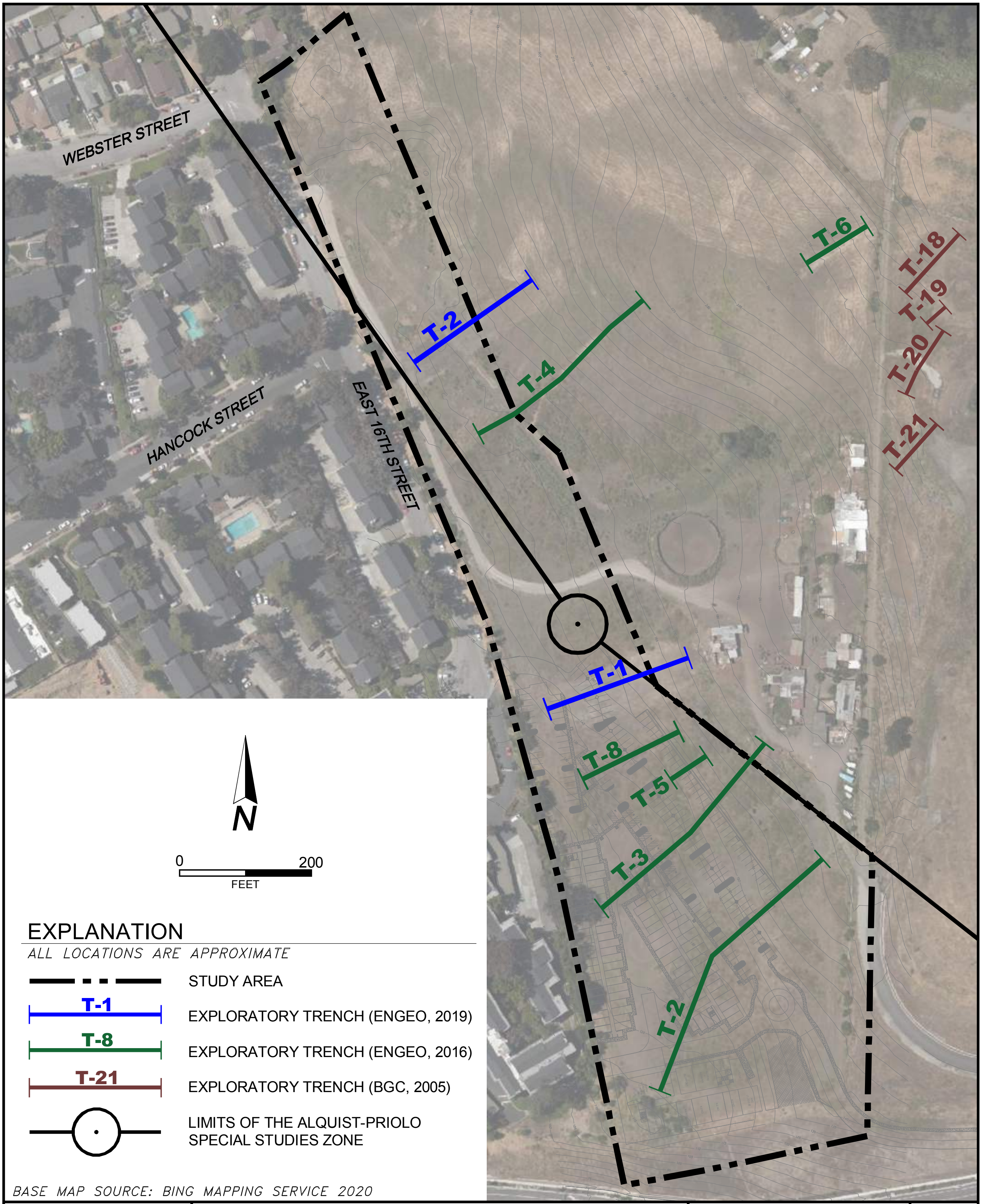

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SITE PLAN WITH TRENCH LOCATIONS

LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.000
SCALE: AS SHOWN
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FIGURE NO.
2

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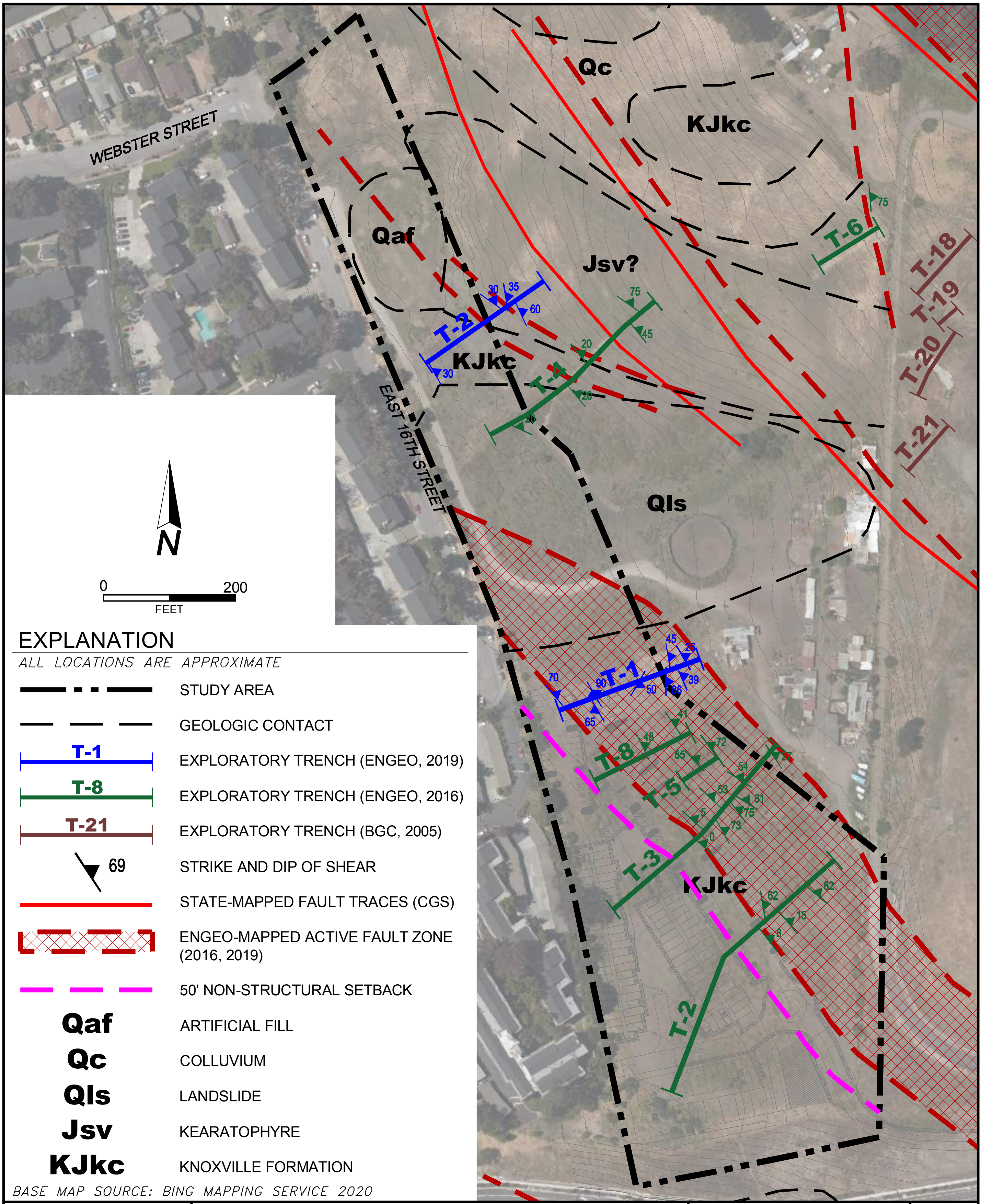

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

▬ · ▬	STUDY AREA
— — —	GEOLOGIC CONTACT
T-1	EXPLORATORY TRENCH (ENGEO, 2019)
T-8	EXPLORATORY TRENCH (ENGEO, 2016)
T-21	EXPLORATORY TRENCH (BGC, 2005)
69	STRIKE AND DIP OF SHEAR
▬	STATE-MAPPED FAULT TRACES (CGS)
▨	ENGEO-MAPPED ACTIVE FAULT ZONE (2016, 2019)
— —	50' NON-STRUCTURAL SETBACK
Qaf	ARTIFICIAL FILL
Qc	COLLUVIUM
Qls	LANDSLIDE
Jsv	KEARATOPHYRE
KJkc	KNOXVILLE FORMATION

BASE MAP SOURCE: BING MAPPING SERVICE 2020

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SITE GEOLOGY AND SEISMICITY
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

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FIGURE NO.
3

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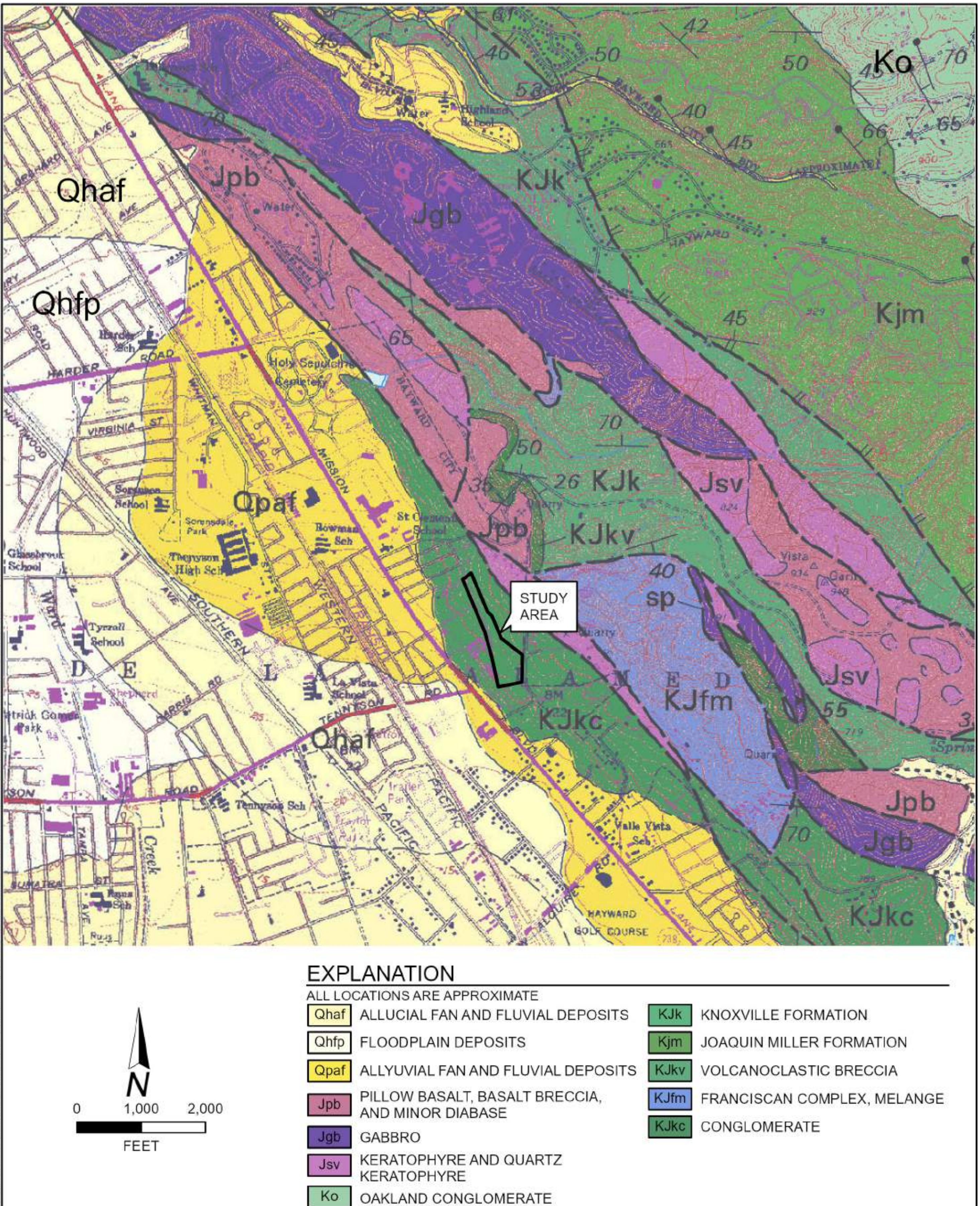

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qhaf	ALLUCIAL FAN AND FLUVIAL DEPOSITS
Qhfp	FLOODPLAIN DEPOSITS
Qpaf	ALLYUVIAL FAN AND FLUVIAL DEPOSITS
Jpb	PILLOW BASALT, BASALT BRECCIA, AND MINOR DIABASE
Jgb	GABBRO
Jsv	KERATOPHYRE AND QUARTZ KERATOPHYRE
Ko	OAKLAND CONGLOMERATE
KJk	KNOXVILLE FORMATION
Kjm	JOAQUIN MILLER FORMATION
KJkv	VOLCANOCLASTIC BRECCIA
KJfm	FRANCISCAN COMPLEX, MELANGE
KJkc	CONGLOMERATE

BASEMAP SOURCE: GRAYMER, 2000

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REGIONAL GEOLOGIC MAP
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

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SCALE: AS SHOWN
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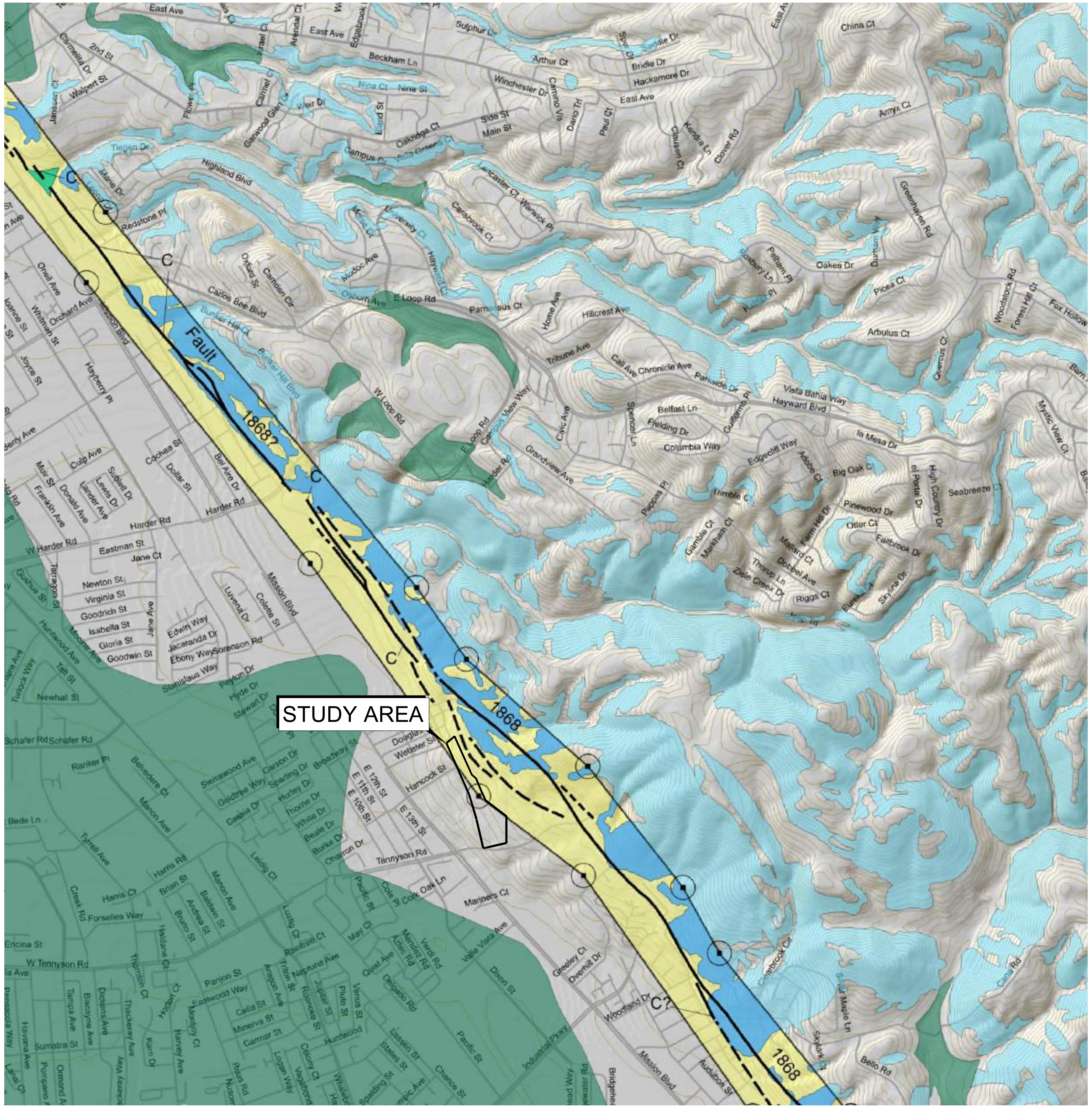
FIGURE NO. 4



EXPLANATION

EARTHQUAKE FAULT ZONES

ACTIVE FAULT TRACES:

FAULTS CONSIDERED TO HAVE BEEN ACTIVE DURING HOLOCENE TIME AND TO HAVE POTENTIAL FOR SURFACE RUPTURE; SOLID LINE WHERE ACCURATELY LOCATED, LONG DASH WHERE APPROXIMATELY LOCATED, SHORT DASH WHERE INFERRED, DOTTED WHERE CONCEALED; QUERY (?) INDICATES ADDITIONAL UNCERTAINTY. EVIDENCE OF HISTORIC OFFSET INDICATED BY YEAR OF EARTHQUAKE-ASSOCIATED EVENT OR C FOR DISPLACEMENT CAUSED BY FAULT CREEP.

ZONES OF REQUIRED INVESTIGATION

EARTHQUAKE FAULT ZONES:

ZONES ARE AREAS DELINEATED AS STRAIGHT-LINE SEGMENTS THAT CONNECT ENCIRCLED TURNING POINTS ENCOMPASSING ACTIVE FAULTS THAT CONSTITUTE A POTENTIAL HAZARD TO STRUCTURES FROM SURFACE FAULTING OR FAULT CREEP SUCH THAT AVOIDANCE AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2621.5(A) WOULD BE REQUIRED

SEISMIC HAZARD ZONES

LIQUEFACTION:

AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED

EARTHQUAKE-INDUCED LANDSLIDES:

AREAS WHERE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED

OVERLAPPING ZONES:

OVERLAP OF EARTHQUAKE FAULT ZONE AND LIQUEFACTION ZONE AREAS THAT ARE COVERED BY BOTH EARTHQUAKE FAULT ZONE AND LIQUEFACTION ZONE. NOTE: MITIGATION METHODS DIFFER FOR EACH ZONE – AP ACT ONLY ALLOWS AVOIDANCE; SEISMIC HAZARD MAPPING ACT ALLOWS MITIGATION BY ENGINEERING/GEOTECHNICAL DESIGN AS WELL AS AVOIDANCE.

OVERLAP OF EARTHQUAKE FAULT ZONE AND EARTHQUAKE-INDUCED LANDSLIDE ZONE AREAS THAT ARE COVERED BY BOTH EARTHQUAKE FAULT ZONE AND EARTHQUAKE-INDUCED LANDSLIDE ZONE. NOTE: MITIGATION METHODS DIFFER FOR EACH ZONE – AP ACT ONLY ALLOWS AVOIDANCE; SEISMIC HAZARD MAPPING ACT ALLOWS MITIGATION BY ENGINEERING/GEOTECHNICAL DESIGN AS WELL AS AVOIDANCE.

N

0 2000
FEET

BASE MAP SOURCE: CGS, 2012

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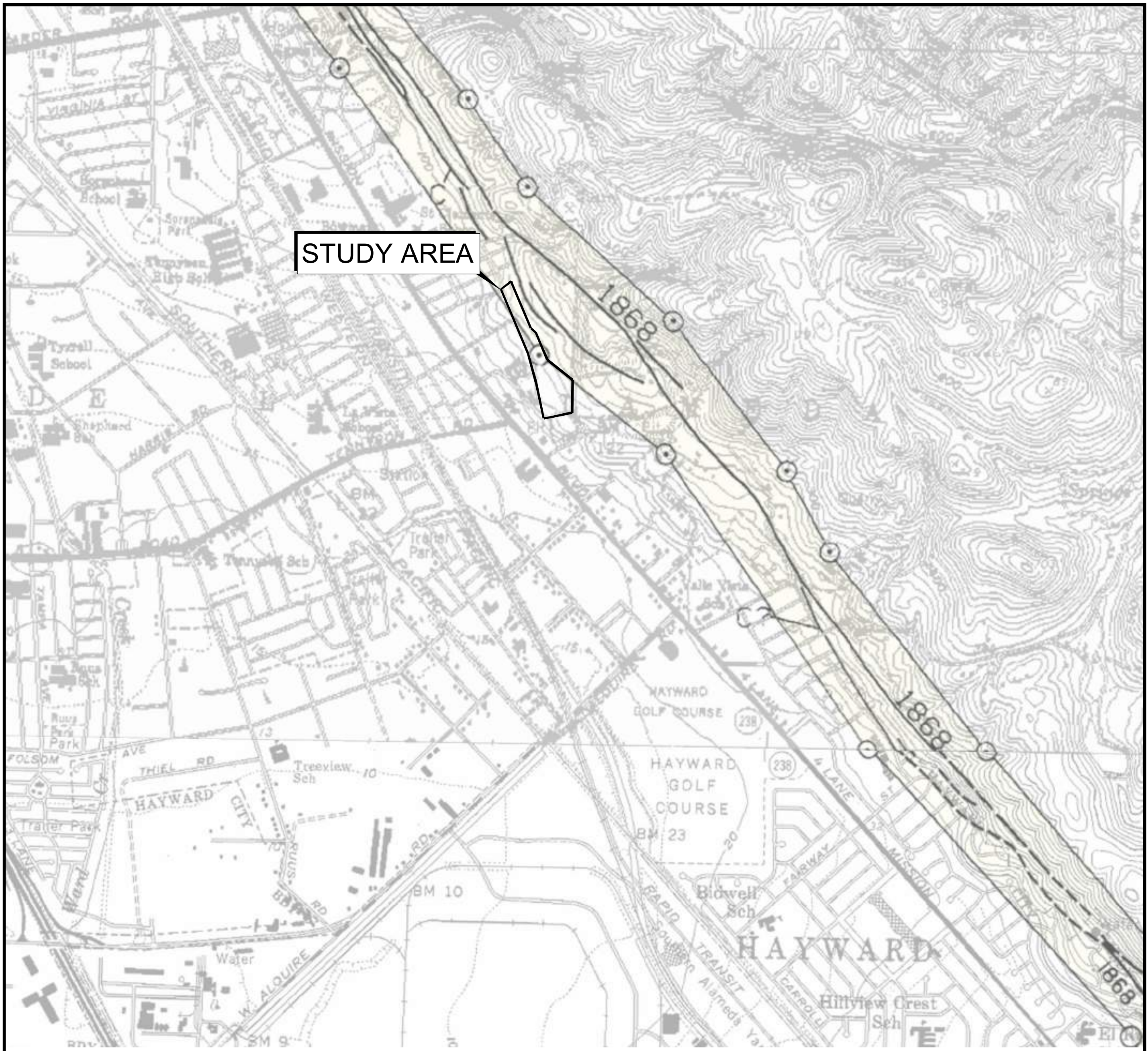
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

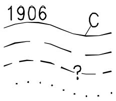
PROJECT NO.: 15577.000.000
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: RHB

FIGURE NO.
5

ORIGINAL FIGURE PRINTED IN COLOR

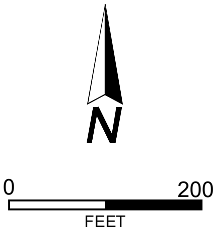


EXPLANATION



FAULTS CONSIDERED TO HAVE BEEN ACTIVE DURING HOLOCENE TIME AND TO HAVE A RELATIVELY HIGH POTENTIAL FOR SURFACE RUPTURE; SOLID LINE WHERE ACCURATELY LOCATED, LONG DASH WHERE APPROXIMATELY LOCATED, SHORT DASH WHERE INFERRED, DOTTED WHERE CONCEALED; QUERY (?) INDICATES ADDITIONAL UNCERTAINTY. EVIDENCE OF HISTORIC OFFSET INDICATED BY YEAR OF EARTHQUAKE-ASSOCIATED EVENT OR C FOR DISPLACEMENT CAUSED BY CREEP OR POSSIBLE CREEP

EARTHQUAKE FAULT ZONE BOUNDARIES; DELINEATED AS STRAIGHT-LINE SEGMENTS THAT CONNECT ENCIRCLED TURNING POINTS SO AS TO DEFINE EARTHQUAKE FAULT ZONE SEGMENTS



BASE MAP SOURCE: CDMG, 1993

EARTHQUAKE FAULT ZONE MAP
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.000	FIGURE NO. 6
SCALE: AS SHOWN	
DRAWN BY: JV	CHECKED BY: RHB

ORIGINAL FIGURE PRINTED IN COLOR

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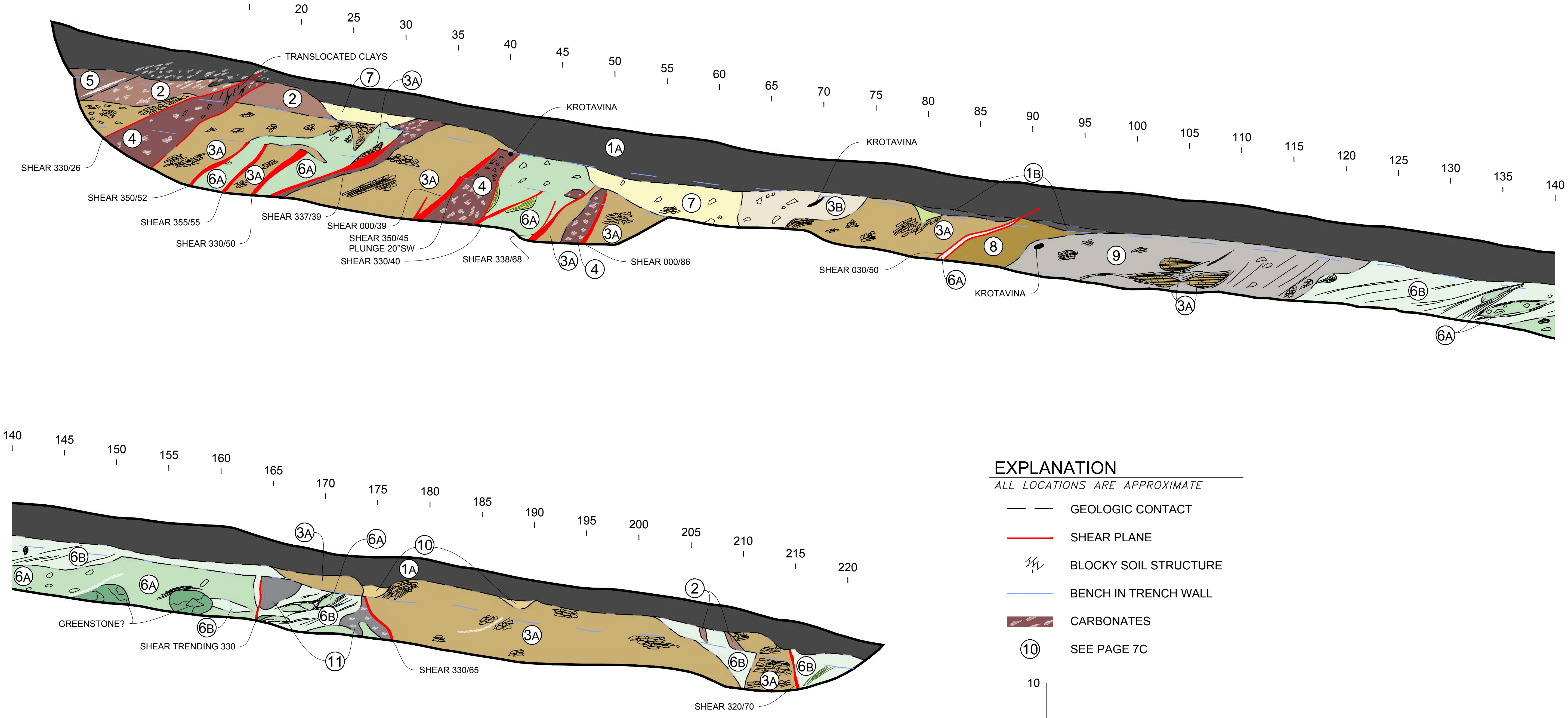
T-1

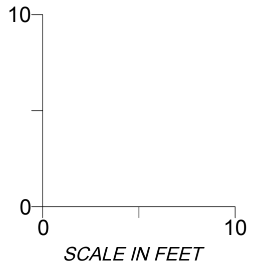
TREND N70°E



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

— —	GEOLOGIC CONTACT
🟥	SHEAR PLANE
⫽	BLOCKY SOIL STRUCTURE
—	BENCH IN TRENCH WALL
▨	CARBONATES
⑩	SEE PAGE 7C



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TRENCH LOGS
TRENCH T-1
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.000
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: RHB

FIGURE NO.
7A

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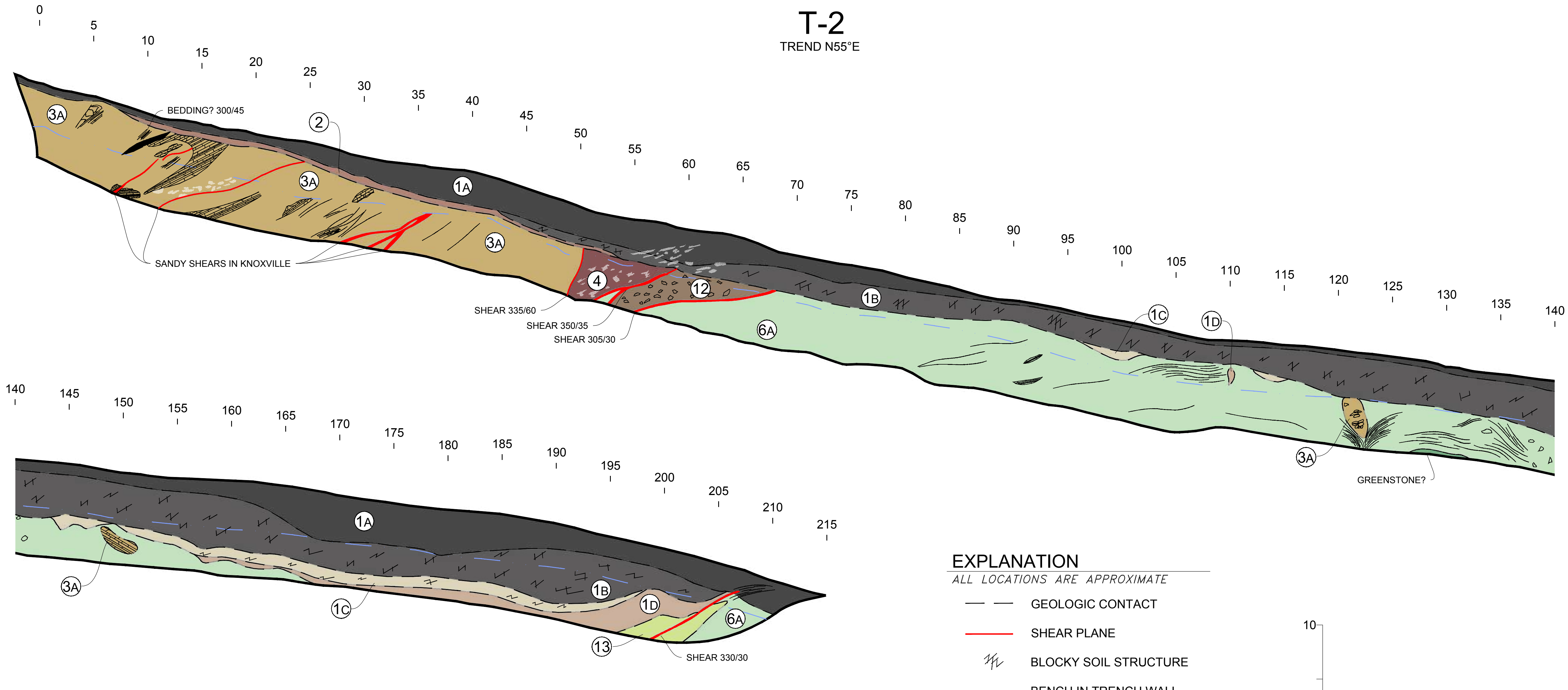
ORIGINAL FIGURE PRINTED IN COLOR

T-2

TREND N55°E



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

— — GEOLOGIC CONTACT

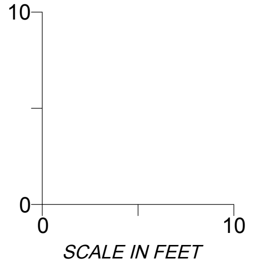
— SHEAR PLANE

⫽ BLOCKY SOIL STRUCTURE

— BENCH IN TRENCH WALL

▨ CARBONATES

Ⓐ SEE PAGE 7C



SCALE IN FEET

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TRENCH LOGS
TRENCH T-2
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.000
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: RHB

FIGURE NO.
7B

FILE PATH: G:\Drafting\PROJECTS_14000 to 15999\15577\15577000000\Geo\FEX\15577000000-2_3_7A_C-FEX-0420.dwg SAVE DATE: 4/23/2020 1:24:37 PM SAVED BY: jvergara

ORIGINAL FIGURE PRINTED IN COLOR

1A	SANDY CLAY (CH), very dark grayish brown (10YR 3/2), hard, moist, fine-grain sand, trace medium gravel, carbonates, rootlets [A-HORIZON]
1B	Development of coarse, blocky pedologic (ped) structure, fine clay films
1C	LEAN CLAY (CL), black (2.5YR 2.5/1), hard, moist, trace roots, little to no ped structure
1D	SANDY CLAY (CL), dark yellowish brown (10YR 4/4), hard, moist, abundant rounded black coarse-grain sand or fine-grain gravel, carbonates
2	FAT CLAY (CH), strong brown (7.5YR 5/8) with very dark grayish brown, very stiff, moist, trace carbonates, trace lithic fracgments, translocated clays, thick clay films on ped surfaces
3A	SANDSTONE AND SHALE, dark yellowish brown (10YR 3/4), medium strong, massive, closely fractured, moderately to highly weathered [KNOXVILLE]
3B	Highly altered and bleached Unit 3A
4	LEAN CLAY (CL), reddish brown (5YR 5/4), very stiff to hard, moist, trace carbonates, lithic fragments (sandstone and shale), moderately developed angular peds with clay films [PLEISTOCENE ALLUVIUM]
5	FAT CLAY (CH), brown (7.5 YR 5/4), very stiff, moist, trace carbonates and gravel
6A	CLAYEY SAND (SC), light gray (2.5Y 7/2) to pale green, very dense, dry, serpentine rich, highly sheared, fragments of serpentinite and greenstone(?)
6B	Very pale green, highly sheared, no clasts
7	CLAYEY SAND (SC), yellow to pale yellow (2.5Y 8/6 to 8/4), moist, large clasts of sandstone and volcanics [GRABEN FILL]
8	FAT CLAY (CH), strong brown (7.5YR 4/6), moist, very stiff, fine gravel, serpentinite fragments, well rounded gravel
9	CLAYEY SAND (SC) with gravel, light gray with brown (10YR 6/6), very dense predominantly sandstone and shale derived with trace serpentinite fragments, serpentinite rich gouge (pale yellow to pale green sand, soft, dry, altered serpentinite fragments)
10	SANDY CLAY (CH), yellowish brown (10YR 5/4) with translocated clays, very stiff to hard, fine gravel
11	LEAN CLAY (CL), dark grayish brown (2.5Y 4/2), moist, hard, trace serpentinite inclusions, bounded by serpentinite-rich shear/gouge
12	SANDY GRAVEL (SG), brownish yellow (10YR 6/6), very dense, moist, Knoxville-derived, sandstone and shale clasts, angular gravel up to 4" in diameter
13	FAT CLAY (CH), light gray (2.5Y 7/1) to pale olive, hard, moist, massive, serpentinite gravel/sand, carbonates

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TRENCH LOGS
UNIT DESCRIPTIONS
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.000
SCALE: NO SCALE
DRAWN BY: JV
CHECKED BY: RHB
FIGURE NO. 7C

ORIGINAL FIGURE PRINTED IN COLOR

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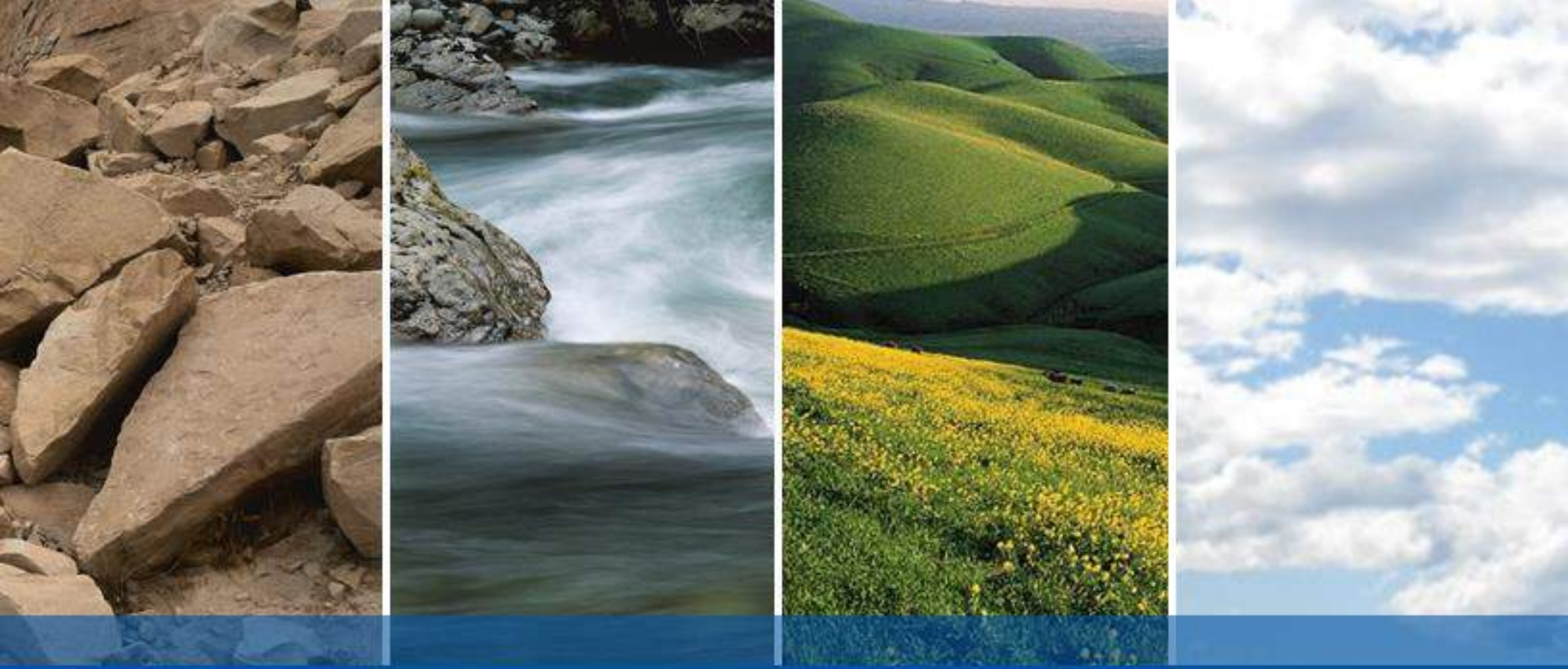

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PARCEL GROUP 3
LA VISTA RESIDENTIAL COMMUNITY AND CHARTER SCHOOL
HAYWARD, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Kate Blessing-Kawamura
Eden Housing, Inc.
22645 Grand Street
Hayward, CA 94541
and
Mr. Chris Grant
Pacific West Communities, Inc.
430 East State Street, Suite East 100
Eagle, ID 83616

PREPARED BY
ENGEO Incorporated

June 1, 2021

PROJECT NO.
15577.000.001

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GEOTECHNICAL
ENVIRONMENTAL
COASTAL/MARITIME
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
15577.000.001

June 1, 2021

Ms. Kate Blessing-Kawamura
Eden Housing, Inc.
22645 Grand Street
Hayward, CA 94541

Mr. Chris Grant
Pacific West Communities, Inc.
430 East State Street, Suite East 100
Eagle, ID 83616

Subject: Parcel Group 3
La Vista
Hayward, California

GEOTECHNICAL EXPLORATION

Dear Ms. Blessing-Kawamura and Mr. Grant:

We prepared this geotechnical report for Eden Housing, Inc. and Pacific West Communities, Inc. as outlined in our agreement dated March 29, 2021. We characterized the subsurface conditions at the site and reviewed relevant existing geotechnical information at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Alexander Patrick Light
No. 92370
CIVIL

Alex Light, PE

James Song Yang
No. 3153
GEOTECHNICAL

James Yang, GE

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No. 2671

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www.engeo.com

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TABLE OF CONTENTS (Continued)

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical report for design of the Parcel Group 3 project in Hayward, California. We prepared this report as outlined in our agreement dated March 29, 2021. Eden Housing, Inc. and Pacific West Communities, Inc. authorized us to conduct the following scope of services.

- Review of readily available maps and reports
- Subsurface field exploration
- Data analysis and conclusions
- Report preparation

For our use, we received the following.

1. A grading and utility plan, prepared by AO Architects, dated September 14, 2020.

2. A geotechnical report for the proposed charter school within the project, prepared by Krazan & Associates, Inc., dated February 21, 2020.

3. A geotechnical report for the proposed residential portion of the project, prepared by Krazan & Associates, Inc., dated February 18, 2020.

4. An engineering response to planning application, prepared by the City of Hayward, dated March 5, 2021.

5. An existing topography survey, prepared by Radman Aerial Surveys, dated July 19, 2019.

We previously performed the following subsurface explorations at the site.

- Fault Hazard Evaluation, Valle Vista (Various Parcels), dated August 15, 2016.
- Fault Hazard Evaluation, La Vista Residential Development, dated April 23, 2020.

We refer to these deliverables in more detail in the following sections.

This report was prepared for the exclusive use of our clients and their consultants for design of this project. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION AND PROPOSED DEVELOPMENT

Figure 1 displays a Site Vicinity Map. The site is located east of Mission Boulevard and north of Valle Vista Avenue (a renamed section of Tennyson Road east of Mission Boulevard), and consists of Alameda County Assessor’s Parcel Numbers (APNs) 78C-626-3-9, and 78C-626-1-7.

Based on our review of available project documents, we understand the development will consist of two 5-story wood-framed over concrete podium structures for the residential portion, and a 1- to 2-story structure for the charter school, as well as associated improvements such as site retaining walls, underground utilities, roadways, flatwork, bioretention basins, and landscaping.

2.0 FINDINGS

2.1 PREVIOUS STUDIES

Portions of the site are located within a State of California Earthquake Fault Zone for the active Hayward fault. The main trace of the Hayward fault is mapped by the California Geological Survey (CGS) east of the site with two active splays mapped through the parcels. Numerous fault studies have been conducted in the vicinity and within the current development site. These studies have been summarized in our most recent fault exploration performed in 2020. Below we include summaries of studies performed within the current development site.

Please refer to our 2020 report for more detailed fault characterization affecting the site and our published trench logs.

2.1.1 ENGEO Incorporated (August 2016)

In 2016, we performed a feasibility study and fault exploration that included the current site as part of a larger project. The study included site-specific geologic mapping, review of stereo-paired aerial photographs, and compiled published fault explorations and fault traces from numerous reports adjacent to the current development site. The exploration included excavating and logging six trenches within the site, totaling approximately 1,455 linear feet of trench, to identify the presence of active faulting. Figure 2 shows the approximate locations of these trenches.

2.1.2 ENGEO Incorporated (November 2016)

In late 2016, we performed a feasibility study and exploration that included the current site as part of a larger project for the City of Hayward. The study included site-specific geologic mapping, review of stereo-paired aerial photographs, and excavating and logging nine test pits and drilling and logging nine borings within the site. Findings from this report have been incorporated into the current study as necessary. Figure 2 shows the approximate locations of the borings and test pits.

2.1.3 ENGEO Incorporated (2020)

In 2020, we performed a fault exploration within the current site. The study included site-specific geologic mapping and review of stereo-paired aerial photographs to determine additional exploration locations to further evaluate the extents of active faulting at the site. The exploration included excavating and logging two exploratory trenches within the site, totaling approximately 440 linear feet of trench, to identify shear features or other evidence of surface fracturing or displacement from seismic activity. Figures 2 and 3 shows the approximate locations of these trenches.

Based on the findings of our report and in accordance with the Alquist-Priolo Earthquake Fault Zone Act and CGS Special Publication 42, we determined the location of a 50-foot non-structural setback from active faulting. The structural setback is located along the eastern extent of the current development area, and buildings are currently shown outside of the setback.

2.1.4 Krazan & Associates, Inc. (2020) – Proposed Charter School

In 2020, Krazan and Associates performed a geotechnical engineering investigation for the proposed charter school which included drilling three borings to approximately 10 to 50 feet below existing grade, one percolation test, and laboratory testing on select soil samples.

2.1.5 Krazan & Associates, Inc. (2020) – Proposed Residential Community

In 2020, Krazan and Associates performed a geotechnical engineering investigation for the proposed residential buildings which included drilling eight borings to approximately 10 to 50 feet below existing grade, one percolation test, and laboratory testing on select soil samples.

Borings and laboratory testing performed as part of the Krazan deliverables are included in Appendix C.

2.2 GEOLOGY AND SEISMICITY

2.2.1 Regional Geologic Setting

The site is located in the Coast Ranges geomorphic province of California. The Coast Ranges are characterized by a series of northwest-trending valleys and mountain ranges. The bedrock in this region has been folded and faulted in a tectonic setting that is experiencing translational and compressional deformations of the earth’s crust.

As depicted on Figure 4, regional geologic mapping by Graymer (2000), the site is mapped as predominantly underlain by Cretaceous to late Jurassic Knoxville formation of the Great Valley Sequence, with a sliver of Jurassic Keratophyre mapped just east of the study area.

Regional landslide mapping by Nilsen (1975) depicts swales and low-lying areas adjacent to the foothills as colluvial or alluvial deposits, with no mapped landslides crossing the site. Mapping by Dibblee (2005) shows a large landslide west of the Hayward fault, with a portion crossing the southeast corner of the site.

2.2.2 Geologic Mapping

We compiled surface geologic mapping based on aerial photo review, site reconnaissance, and exploratory trenching from our 2016 and 2020 explorations. Below are descriptions of the geologic units observed during mapping of the site and included on Figure 3.

2.2.2.1 Artificial Fill (Qaf)

Artificial fill was encountered within the upper 3½ feet of our Test Pits 2-TP3, 2-TP11, and TP-12, as well as Krazan Borings B1 and B7 performed within the proposed residential portion of the site. Encountered material consisted of on-site derived lean and fat clays in our test pits, and clayey sand within the Krazan borings. Figure 3 depicts a larger area of mapped fill that is likely 5 to 10 feet thick and is anticipated to consist of on-site derived materials.

2.2.2.2 Landslide Deposits (Qls)

Landslide deposits were mapped based on findings during our 2016 and 2020 explorations. Landslide debris was encountered during our current exploration in Test Pits 2-TP7 and 2-TP9.

Landslide deposits mapped may be on the order of 20 to 40 feet thick, with features indicative of relatively recent movement, including very well developed striated shearing and distortion of Holocene active faulting.

2.2.2.3 Colluvium (Qc)

Colluvial deposits were mapped in low-lying drainages with increased vegetation and are anticipated to consist of transported surficial soil derived from the site bedrock. Colluvial deposits were encountered to depths of 16 feet during our current exploration, as described in Test Pit 2-TP2.

2.2.2.4 Shear Zone (Qfs)

Based on compilation of numerous explorations logs and observations during site reconnaissance mapping, we have identified a larger “shear zone” that consists of faulted slivers of alluvium, colluvium, Knoxville shale and sandstone, landslide debris, and serpentinitic gouge. The shear zone is depicted on Figure 3 between more prominent fault features “Fault 1” and “Fault 3.” Numerous secondary active faults were identified within this shear zone and predominantly dip east, back in the direction of the Hayward Fault.

2.2.2.5 Bedrock Formations (JKk, JKks, and Jsv)

Trenches T-1 and ET-4 completed during our 2016 and 2020 investigations encountered Cretaceous Knoxville formation (JKk). The Knoxville formation observed on the site consisted mostly of yellowish brown silt and clay shale with sandstone interbeds.

Based on previous explorations and Test Pits 2-TP1 and 2-TP10, the Knoxville formation bedrock underlying the proposed development site (JKks) consists of interbedded sandstone, shale, and conglomerate that is generally weaker and more sheared from tectonic activity affecting the site. Jurassic Keratophyre (Jsv) was previously encountered in the eastern portion of the site, but was not observed within the current test pits. Graymer (2000) describes the keratophyre as consisting of highly altered intermediate and silicic volcanic and hypabyssal rocks, previously mapped as Leona and Northbrae rhyolite.

2.2.3 Faulting and Seismicity

As noted above, portions of the site are located within a State of California Earthquake Fault Zone for the active Hayward fault (Figures 5 and 6). The main creeping trace of the Hayward fault is mapped by the CGS approximately 900 feet east of the site, with two splays mapped west of the main trace directly through the adjacent open space parcel. A recently active fault trace is defined by the state of California as displaying signs of displacement within the last 11,000 years (Hart, 2007).

2.2.3.1 Hayward Fault

The Hayward fault is one of the main branches of the San Andreas fault displaying predominantly right-lateral displacement. The approximately 60-mile-long fault extends from San Jose along the East Bay Hills to Point Pinole and possibly right stepping and passing strain to the active and en echelon Rodgers Creek fault system (Lienkaemper, 2008). Although much of the area surrounding the fault system has experienced rapid urbanization over the last several decades, many geomorphic features indicative of strike-slip faulting can still be observed along the fault trace, such as, right-laterally offset drainages, shutter ridges, sag ponds, and rift or hillside valleys. The Hayward fault has been extensively studied and the active creeping trace is well defined in the site vicinity.

The first regional maps of the Hayward fault were produced by Radbruch (1969), which included the possible location of the 27-mile-long surface rupture associated with the 1868 earthquake. The Radbruch mapping included noted geomorphic features and compiled evidence of creep along the fault system. Radbruch's mapping was the basis for the original 1974 Special Studies Zone map for the area.

The USGS Quaternary Fold and Fault Database (QFFD) is a nationwide GIS-based database that identifies fault locations and classifies faults based on estimated age. In California, the QFFD is jointly maintained by the USGS and the California Geological Survey (CGS). The QFFD shows the Hayward fault system in the area to be consistent with Earthquake Fault Zone map (Figure 6).

Within the fault zone there are blocks of more competent and stable rocks such as Jurassic-age volcanic and intrusive rocks (basalt, keratopyre, and gabbro), and relatively intact blocks of Jurassic sedimentary rock (Knoxville Formation) separated by wide bands of highly sheared and deformed rocks and soil that include fault-bounded slivers of bedrock interspersed with highly weathered and altered serpentinite, old alluvium and landslide debris.

In the site vicinity, the most recent faulting has been concentrated at the west edge of the zone, extending from approximately the base of the hills at Mission Boulevard to the main trace east of the site. The western limit of the most complexly sheared materials appears to be a low-angle, east-dipping zone of sheared serpentinite, designated as "Fault 3" on Figure 3. This zone of shearing was also associated with extensive groundwater seepage in the trenches.

2.2.3.2 Seismicity

Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future. Other nearby active faults include other subsections of the Hayward fault, the Mission fault, and the Calaveras fault, as summarized in Table 2.2.3.2-1. To determine nearby active faults that are capable of generating strong seismic ground shaking at the site, we utilized the USGS Unified Hazard Tool and disaggregated the hazard at the peak ground acceleration (PGA), and for spectral periods of 0.2, 0.5, and 1.0 seconds for a 2,475-year return period, with the resulting faults listed below in Table 2.2.3.2-1.

TABLE 2.2.3.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site (Latitude: 37.6380° Longitude: -122.0526°)

SOURCE	R_{RUP}		MOMENT MAGNITUDE M_W
	(KM)	(MILES)	
Hayward (So) [5]	0.3	0.2	7.07
Hayward (So) [4]	3.7	2.3	6.79
Mission (connected) [0]	1.1	0.7	6.85
Calaveras (No) [3]	12.7	7.9	7.28
Hayward (So) [3]	8.5	5.3	6.80

*USGS Unified Hazard Tool - Edition: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

The Uniform California Earthquake Rupture Forecast (UCERF 3) (Field et al., 2015) estimates the 30-year probability (as of 2014) for a magnitude 6.7 or greater earthquake in the San Francisco region at approximately 72 percent, considering the known active seismic sources in the region.

2.3 FIELD EXPLORATION

Our field exploration included excavating 12 test pits, retrieving a near-surface soil sample for laboratory testing, and performing geophysical testing. We also performed geologic field mapping concurrently to update our previous geologic mapping. The location and elevations of our explorations are approximate and were estimated by using commercially available GPS services; they should be considered accurate only to the degree implied by the method used.

2.3.1 Test Pits

We observed excavation of 12 test pits at the locations shown on the Site Plan, Figures 2 and 3. Our representative observed the test pit excavation and logged the subsurface conditions at each location. We retained a Caterpillar 318 excavator to excavate the test pits and logged the type, location, and uniformity of the underlying soil/rock. The maximum depth penetrated by the test pits was 16 feet.

We obtained bulk soil samples from the test pits using hand-sampling techniques. The test pit logs present descriptions of the subsurface conditions encountered.

We used the field logs to develop the report logs in Appendix A. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.3.2 Geophysical Testing

We retained a geophysical subcontractor to perform the active-source, multi-channel analysis of surface waves (MASW) technique to prepare a shear wave velocity model of the upper 30 meters (approximately 100 feet) of the site. The surface wave testing occurred on April 11, 2021, and the results of the testing are included as Appendix E.

2.3.3 Geologic Field Mapping

During our field explorations, our geologist observed the surface conditions and visible geologic features at the site. We confirmed mapping and geologic features presented in our 2016 and 2020 studies and summarize our findings on Figure 3.

2.4 SURFACE CONDITIONS

Elevations across the site range from approximately 70 feet (NAVD88) in the southwest corner up to approximately 160 feet (NAVD88) along the eastern portion of the site. Slope gradients generally range from 10:1 (horizontal:vertical) to 2:1, with some local areas with steeper slopes.

In general, the site has remained generally undeveloped, and currently contains access roadways and minor structures, including horse stalls, storage sheds, and a riding ring.

We observed the following site features during our previous and current site visits.

- The western portion of the property is covered by a light to dense growth of trees. The site is covered by a moderate growth of grasses and weeds.
- Several modular structures and a horse-riding ring are located within the parcel extents to the east of the project boundary. We understand that these will be demolished as part of the future La Vista Park construction.
- A gravel road enters the site between the extensions of Hancock Street and Webster Street and runs along the western boundary parallel to East 16th Street and outlets to Tennyson Road to the southeast of the site.
- A visible scarp was observed in the northeastern portion of the site east of the gravel road.

Please refer to the Site Plan, Figure 2, for more information on site features.

2.5 SUBSURFACE CONDITIONS

Test Pits 2-TP1, 2-TP2, 2-TP3, 2-TP11 and 2-TP12 were excavated within mapped colluvial deposits (Qc) overlying Knoxville Formation bedrock (JKks), Figure 3. Colluvial deposits consisting of hard fat clay to dense poorly graded sand with gravel were encountered to a depth of 16 feet as observed in 2-TP2. Very weak and highly weathered sandstone and shale was encountered at depths of 6 and 10 feet, underlying colluvial deposits in 2-TP1 and 2-TP3, respectively. Artificial fill deposits ranging from 1½ to 2 feet thick overlying colluvial deposits were encountered in Test Pits 2-TP3, 2-TP11, and 2-TP12. The fill deposits generally consist of on-site derived soils.

Test Pits 2-TP4, 2-TP5, TP6, and 2-TP8 were excavated within mapped “shear zone” deposits (Qfs), Figure 3. Hard colluvial deposits consisting of sandy lean clays to fat clays were encountered to depths ranging from 1½ to 7½ feet below ground surface (bgs). Underlying the surficial colluvial deposits, slivers of weak sandstone, shale, strong conglomerate, and hard serpentinite rich clays were encountered to 15 feet bgs.

Test Pits 2-TP7 and 2-TP9 were excavated within mapped landslide deposits (Qls) as depicted on Figure 3. Stiff to hard fat and lean clays with dense poorly graded sand with clay was observed to a depth of 16 feet bgs, the depth of exploration. Shear planes were observed in samples recovered from 2-TP9, indicative of landsliding.

Test Pit 2-TP10 was excavated within mapped Knoxville Formation bedrock (JKks), Figure 3. Strong slightly weathered conglomerate was observed to a depth of 11 feet in 2-TP10, the depth of exploration.

Consult the Site Plan and exploration logs for specific subsurface conditions at each location. We include our exploration logs from our current study in Appendix A. The logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration.

2.5.1 Geophysical Testing

Based on the results of the MASW testing, shear wave velocities ranged from 154 to 226 m/s in the upper 4 meters, before increasing to 332 to 424 m/s in the underlying 4 to 30 meters. The results of the geophysical testing are included in Appendix E.

2.6 GROUNDWATER CONDITIONS

Previous explorations at the site encountered groundwater in the explorations. We summarize the groundwater measurements in the table below:

TABLE 2.6-1: Groundwater Observations

EXPLORATION LOCATION	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)
B1 (Krazan 2020, Residential Community)	36	14(1)
B2 (Krazan 2020, Charter School)	33	17(1)

(1) Elevations based on values provided in Krazan 2020 Charter School and Residential Community reports, datum not specified

Further, localized seeps were observed during previous site visits and in review of aerial photographs. A well-defined seep is located within the northern portion of the site, evidenced by increased vegetation.

Based on the Seismic Hazard Report for the Hayward 7.5-minute Quadrangle, the depth to historic high groundwater in the vicinity of the site is approximately 20 feet.

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

2.7 LABORATORY TESTING

As part of the current scope of work, we performed laboratory tests on select near-surface samples. These include remolded triaxial strength testing, plasticity index, grain size, and hydrometer testing. We also reviewed laboratory testing performed by Krazan & Associates, Inc.; this included consolidation, direct shear, grain size, expansion index, plasticity index, and R-value testing.

Further, sulfate test results were included within the Krazan report, although the actual test results are not included in the Krazan report appendices. Krazan reports sulfate values of less than 0.02 percent.

Previous laboratory data is included in Appendix A; moisture contents and dry densities are also recorded on the boring logs in Appendix A. Current laboratory testing is included in Appendix C.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications.

The primary geotechnical concerns that could affect development on the site are slope stability of the eastern slope, potential creep resulting from encroachment of traces of the Hayward fault into the project limits, existing landslide, expansive soil, rock excavatability, and existing fill.

3.1 SLOPE STABILITY ANALYSES

3.1.1 Method of Analysis

We performed two-dimensional limit-equilibrium slope stability analysis of the existing slope east of the proposed development with the computer slope stability software Slide Version 9.0 using Spencer's method (Spencer, 1967). We performed slope stability analysis on four generalized cross sections of the slope (Sections 1-1', 2-2', 3-3', and 4-4', shown on Figure 3) under static and seismic loading. We considered sections in existing conditions as well as proposed conditions following corrective grading as discussed in Section 7.

Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (CGS, 2008), is currently used in best practice to evaluate seismic stability of slopes in California. Note 48, which is used for Public Schools, Hospitals, and Essential Service Buildings, advises the procedure recommended in SP117A in addition to using a design-level ground motion based on geometric mean and without risk coefficient (i.e. $PGA_M/1.5$). We estimate PGA_M to be 0.935g based on our site-specific seismic hazard analysis, as discussed in Section 4. To develop the design-level PGA, we multiplied the PGA_M by 2/3 to yield a value of 0.623g. SP117A states that slopes that have a pseudostatic factor of safety greater than 1.0 using a seismic coefficient derived from the screening analysis procedure of Stewart and others (2003) can be considered stable. We used a pseudostatic coefficient of 0.24g based on a 15-centimeter (6-inch) threshold of displacement as recommended by Steward and others (2003). For sections that showed a pseudostatic factor of safety less than 1.0, we estimated displacement with methods described by Bray and Travasarou (2007).

Bray and Travasarou (2007) provides a simplified semi-empirical predictive relationship for estimating permanent displacements due to earthquake-induced deviatoric deformations. The seismic displacement model is developed based primarily on the influence of the system's yield coefficient (ky), its initial fundamental period (T_S), and the ground motions spectral acceleration at a degraded period equal to $1.5T_S$. The ground motion used in the relationship is that of the base of the slope; therefore, the response spectra developed for the site-specific hazard analysis was used to estimate earthquake-induced deformations in Section 2-2'.

3.1.2 Acceptable Factors of Safety

The minimum allowable factor of safety with respect to slope stability commonly ranges from 1.5 to 3 for static cases and 1.0 to 1.5 for seismic cases.

Based on local geotechnical practice, in our opinion, we recommend that a static factor of safety of 1.5 and a pseudostatic factor of safety of 1.0 be considered adequate for the site slope conditions. We considered the various levels of conservatism involved in determining the engineering properties of the soil (density, shear strength, unit weight, permeability, etc…), the assumptions made in the method of analysis, and potential variations in field conditions.

3.1.3 Geometry and Idealized Soil Profiles

We received topographic survey data (Radman 2019) for use in creating the two-dimensional cross-sections used in our analysis.

We performed unconsolidated undrained triaxial and unconfined compression testing of bulk samples of near-surface site soils to estimate remolded strength of material which we propose to be placed as compacted engineered fill. We reviewed the raw lab strength data and compared it with plasticity index (PI), soil type, and observations made during test pit and previous trench excavations. Shear strengths of landslide material and bedrock were modeled based on previous lab testing results. The bedrock material was modeled using Generalized Hoek-Brown shear-normal function. The idealized soil profiles used on our analysis are shown in Appendix F.

3.1.4 Results of Analyses

We performed slope stability analyses for both static and pseudostatic conditions. Results of analysis performed on the existing and proposed conditions are shown in Tables 3.1.4-1 and 3.1.4-2 below.

TABLE 3.1.4-1: Summary of Slope Stability Analyses with Existing Conditions

	FACTOR OF SAFETY		SEISMIC DEFORMATION
LOCATION	**STATIC**	**PSEUDOSTATIC (0.24g)**	**Calculated Earthquake-Induced Deviatoric Deformation (inches)**
Section 1-1’	1.8	1.0	N/A
Section 2-2’	1.2	0.5	132-192
Section 3-3’	1.7	1.0	N/A
Section 4-4’	2.1	0.9	6-18

As shown above, our analysis indicates that the existing slope conditions in Sections 2-2’ and 4-4’ are anticipated to undergo seismic-induced deformation during the design event.

TABLE 3.1.4-2: Summary of Slope Stability Analyses with Proposed Improvements

	FACTOR OF SAFETY		SEISMIC DEFORMATION
LOCATION	**STATIC**	**PSEUDOSTATIC (0.24g)**	**Calculated Earthquake-Induced Deviatoric Deformation (inches)**
Section 1-1’	1.8	1.0	N/A
Section 2-2’	1.6	0.6	12-36
Section 3-3’	1.6	1.2	N/A
Section 4-4’	1.9	1.0	N/A

Appendix F presents the results of our static and pseudostatic stability analyses. As shown above, the static factor of safety under existing conditions is less than 1.5 only at Section 2-2’. For pseudostatic analysis, a factor of safety greater than 1.0 will experience approximately 6 inches of lateral displacement or less during a design seismic event. Based on our analysis, Section 2-2’ is anticipated to undergo approximately 12 to 36 inches of earthquake-induced deformation as calculated using Bray and Travasarou (2007).

3.2 EXISTING FILL

Our Test Pits 2-TP3, 2-TP11, and 2-TP12 indicate that portions of the site are underlain by non-engineered fill. Fill material was encountered within the upper approximately 1½ to 3½ feet of these test pits and generally consisted of on-site clayey materials. Additionally, an area of existing fill on the northern portion of the site is anticipated to be 5 to 10 feet thick (Figure 3).

Non-engineered fills can undergo excessive settlement, especially under new fill or building loads. Without proper documentation of existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. We present fill removal recommendations in Section 7.

3.3 EXPANSIVE SOIL

We observed potentially expansive clay and sandy clay near the surface of the site in the explorations. Our laboratory testing indicates that this soil exhibits high shrink/swell potential with variations in moisture content with Plasticity Index laboratory results for soil in the upper 5 feet ranging from 28 to 46.

Expansive soil can change in volume with changes in moisture. It can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Building damage due to volume changes associated with expansive soil can be reduced by: (1) deepening conventional shallow footings to below the zone of significant seasonal moisture fluctuation, (2) using a rigid mat foundation that is designed to resist the settlement and heave of expansive soil, or (3) blanketing the footprint of the building pad with non-expansive soil. We provide foundation recommendations in Section 8 of this report.

Successful performance of structures on expansive soil requires special attention during construction. It is imperative that exposed soil be kept moist prior to placement of concrete for foundation construction. It can be difficult to remoisturize clayey soil without excavation, moisture conditioning, and recompaction.

We have also provided specific grading recommendations for compaction of clay soil at the site. The purpose of these recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the amount of compaction. Expansive soil mitigation recommendations are presented in Sections 7, 8, and 9 of this report.

3.4 EXCAVATABILITY

Based on field observations during excavation of trenches and test pits at the subject site, blow counts recorded in the exploratory borings and laboratory test results, it is our opinion that, in general, bedrock should be rippable with conventional heavy construction equipment (such as a Caterpillar D-8). Localized well-cemented beds and occasional well-cemented concretions may be encountered that will require more ripping effort. Trenching for utilities should be possible with conventional equipment. As noted above, localized well-cemented beds may be encountered that may necessitate use of heavy equipment such as track-mounted excavators.

In general, all soil and bedrock materials observed on the site appear suitable for use as engineered fill, if properly processed. If rocks greater than 6 inches in diameter are encountered during grading, these should be placed in accordance with recommendations provided in the Section 7.3.

3.5 LANDSLIDE REMOVALS

We anticipate the landslides mapped on the site (shown on Figure 2) will require removal. The approximate limits of these landslides are outside of the project boundary in some areas. Therefore, complete removal of the landslide will involve coordination with the adjacent property owner. Alternatively, a structural solution such as a pier buttress along the project boundary can be designed to arrest movement from portions of the landslide outside of the project boundary.

3.6 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking and ground lurching. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, soil liquefaction, lateral spreading, tsunamis, flooding or seiches is considered low to negligible at the site.

3.6.1 Ground Rupture

Since active fault traces have been identified crossing the property in previous investigations and the site is located within an Earthquake Fault Special Study Zone, it is our opinion that ground rupture is possible at the site within the mapped active fault zone. Active fault traces encountered at the site are generally considered secondary traces branching off of the main trace of the Hayward fault and extend through a wide shear zone. Proposed structures are planned to be a minimum of 50 feet from any active fault trace identified at the site.

3.6.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the current California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.6.3 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded, fine-grained sand.

Review of the borings performed at the site by Krazan showed high blow counts (minimum of 36 blows per foot with a standard penetration test) at the site below the historic high water level. Further, the site is mapped as a shallow bedrock site, which we also observed in the test pits we performed at the site. The site is not mapped within a liquefaction hazard zone by CGS.

For these reasons and based upon engineering judgment, it is our opinion that the potential for liquefaction at the site is low during seismic shaking.

3.6.4 Seismically Induced Landsliding

Seismically induced landslides are triggered by earthquake ground shaking. The risk of this hazard is greatest in the late winter when groundwater levels are highest and hillside colluvium is saturated. As with all slopes in the region, this risk is also present at the site to varying degrees depending on the slope conditions and time of year. The hazard of seismically induced landslides to the proposed structures can be best mitigated by properly engineered stabilization of landslides and removal of landslide deposits as recommended in this report.

As discussed in Section 3.1, our analysis indicates approximately 12 to 36 inches of earthquake-induced deviatoric deformation at Section 2-2’ during the design seismic event; this deformation is anticipated at the northern portion of the site where parking is planned. We provide discussion specific to a Geologic Hazard Abatement District (GHAD) to provide a long-term fund for maintenance and repair related to geologic hazards such as seismically induced landsliding in Section 6.

3.7 NATURALLY OCCURRING ASBESTOS

During our explorations, we encountered bedrock containing serpentine. Serpentine can contain the fibrous mineral chrysotile, which is considered an asbestiform mineral. In other locations in San Francisco, chrysotile and other asbestiform minerals have been identified in the Franciscan formation.

It is our opinion that the project will be required to follow the rules and regulations outlined in the Asbestos Airborne Toxic Control Measure (ATCM) for Construction, Grading, Quarrying, and Surface Mining Operations established by the Bay Area Air Quality Management District (District) under California Code of Regulations, Title 17, Section 93015. The purpose of this regulation is to reduce public exposure to naturally occurring asbestos (NOA) from construction and mining activities that emit dust, which may contain NOA. The ATCM requires regulated operations engaged in road construction and maintenance activities, construction and grading operations, and quarrying and surface mining operations in areas where NOA is likely to be found, to employ the best available dust mitigation measures in order to reduce and control dust emissions.

As part of compliance with the ATCM, an Asbestos Dust Mitigation Plan (ADMP) should be prepared by a qualified representative for approval by the BAAQMD and for inclusion in the contract documents. Our experience indicates that dust monitoring during ground disturbing activities may be required.

3.8 SOIL CORROSION POTENTIAL

As part of this study, we obtained a representative soil sample and submitted to a qualified analytical lab for determination of pH, resistivity, sulfate, and chloride. The results are included in Appendix B and summarized in the table below.

TABLE 3.8-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH (FT)	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
2-TP2	1	6.07	1,420	4.2	1.9

* ASTM D4327

The 2019 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Section 19.3.1 for concrete durability requirements. ACI Table 19.3.1.1 provides the following exposure categories and classes, and Table 19.3.2.1 provides requirements for concrete in contact with soil based upon the exposure class.

In accordance with the criteria presented in the above table, the representative soil is categorized as within the S0 sulfate exposure class. Considering a ‘Not Applicable’ sulfate exposure, there is no requirement for cement type or water-cement ratio; however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

The value of resistivity is generally considered highly corrosive to buried metal piping in direct contact with the soil. The range of pH measured is not generally considered corrosive.

If desired to investigate this further, we recommend a corrosion consultant be retained to evaluate if specific corrosion recommendations are advised for the project. Note that ASTM Test Method D4327 was used in lieu of the ACI designated sulfate test methods, but our experience indicates this difference should not impact the findings.

3.9 STATIC AND PERCHED GROUNDWATER

It does not appear that the static groundwater level beneath the site is likely to affect the proposed development. However, perched water can:

1. Impede grading activities.
2. Cause moisture damage to sensitive floor coverings.
3. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.
4. Cause premature pavement failure if hydrostatic pressures build up beneath the section.

We provide recommendations to reduce the effects of perched water in sections addressing subsurface drainage facilities, site drainage, and lime treatment.

3.10 SOIL CREEP

Soil creep is a natural process that involves slow downhill movement of soil mantle on a slope. Soil creep consists of lateral extension and vertical settlement. Soil creep results when surficial expansive soil is subjected to wetting and drying cycles caused by seasonal moisture changes, precipitation, and/or long-term landscape irrigation; by the growth of roots; and by burrowing animals. The surface manifestations of soil creep include tipping/tilting of fence posts, separations of exterior concrete slabs or other landscape elements from residential buildings, and cracks with vertical and/or horizontal offsets in surface and near-surface improvements. The amounts of vertical and horizontal movement are a function of the soil physical characteristics, such as plasticity, height, and gradient of the downhill slope, and the depth of wetting and drying cycles. Improvements constructed on or near downhill slopes will be impacted by soil creep. Any improvements such as site retaining walls planned on or near the tops of downhill slopes should consider soil creep. Further, a long-term vehicle for maintenance and repair related to soil creep can be mitigated by the establishment of a GHAD.

4.0 SITE-SPECIFIC SEISMIC HAZARD ANALYSIS

This section describes the site-specific seismic-hazard analysis that we performed for the proposed development at the site. We performed our analysis in accordance with the 2019 California Building Code (2019 CBC). The 2019 CBC utilizes the seismic design criteria described in the 2016 ASCE/SEI 7 Standard (ASCE 7-16)[1].

We classified the site as Site Class D per ASCE 7-16 based on multi-channel array surface wave (MASW) measurements that we collected at the project site (Array 1). According to the 2019 CBC and ASCE 7-16, a site-specific seismic hazard analysis is required for Site Class D sites where S_1 is greater than or equal to 0.2. Since the project site meets this criterion, we performed a site-specific seismic-hazard analysis to evaluate the seismic design parameters. We completed the following tasks to develop Risk-Targeted, Maximum-Rotated Maximum Considered Earthquake (MCE_R) and Design Earthquake (DE) response spectra for this site:

- Perform probabilistic seismic-hazard analysis (PSHA) to develop a risk-targeted, maximum-rotated response spectrum corresponding to a 2-percent probability of exceedance in 50 years (2,475-year return period).
- Perform deterministic seismic-hazard analysis (DSHA) to develop an 84th-percentile maximum-rotated response spectrum.
- Compare the DSHA response spectrum with the Deterministic Lower Limit in accordance with Section 21.2.2 of ASCE 7-16 and Supplement No. 1.
- Compare the risk-targeted and maximum-rotated probabilistic and the maximum-rotated deterministic response spectra to obtain the site-specific MCE_R response spectrum for the site.
- Multiply the site-specific MCE_R response spectrum by two-thirds to obtain the site-specific DE response spectrum for the site.
- Compare the MCE_R and DE response spectra developed in the previous step with their corresponding 80-percent mapped response spectra to develop the recommended site-specific MCE_R and DE response spectra.
- Develop seismic design parameters per Sections 21.4 and 21.5 of ASCE 7-16.

4.1 GROUND MOTION MODELS AND SITE PARAMETERS

We used four semi-empirical ground motion models (GMMs) from Next Generation Attenuation West 2 (NGA West 2) project in the seismic-hazard analysis for this project. These include Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and

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[1] Minimum Design Loads for Buildings and Other Structures

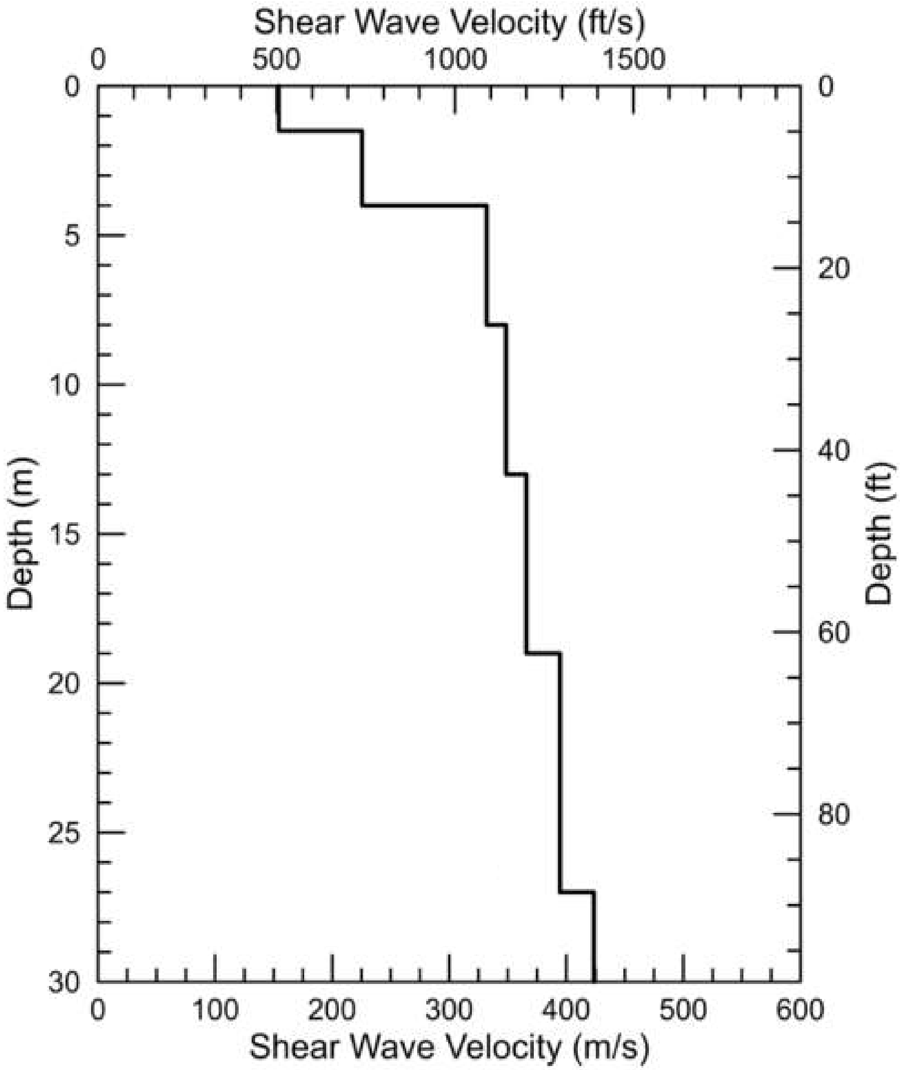
Youngs (2014). We performed our analysis using all four GMMs for a spectral damping of 5 percent of critical damping. We used the logic-tree approach and assigned equal weight (0.25) to the four GMMs in our analysis.

The ground-motion models incorporate "site parameters" to model how subsurface soil will amplify or attenuate ground motions as they propagate from underlying bedrock. These site parameters include:

- Time-averaged shear-wave velocity over the top 100 feet or 30 meters (V_{S30})
- Depth at which the shear-wave velocity (V_S) reaches 3,280 feet/sec or 1.0 kilometer/sec ($z_{1.0}$)
- Depth at which V_S reaches 8,200 feet/sec or 2.5 kilometers/sec ($z_{2.5}$)

A profile of shear-wave velocity (V_S) is needed to compute V_{S30}. We estimated a V_{S30} value of 1,082 feet/sec (330 meters/sec) based the V_S profile measured at the site, as shown in Exhibit 4.1-1.

EXHIBIT 4.1-1: Shear-Wave Velocity Profiles, V_S, at Array 1



We initially obtained $z_{1.0}$ and $z_{2.5}$ estimates from the USGS Bay Area Velocity Model version 8.3.0 Basin Depth model as implemented in the USGS Site Data Application Software (OpenSHA); however, these estimates were inconsistent with our measured V_S data. Therefore, we utilized the V_{S30}-based relationships described in the NGA West 2 GMM references (Abrahamson et al., 2014; Cambell and Bozorgnia, 2014; Chiou and Youngs, 2014) to estimate $z_{1.0}$ and $z_{2.5}$. We used $z_{1.0}$ values of 1,417 and 1,421 feet (432 and 433 meters) for the Abrahamson et al. (2014) and Chiou and Youngs (2014) GMMs, respectively, based on the suggested values in the associated references. Note that these values are nearly identical. Boore et al. (2014) recommends the same value as recommended by Chiou and Youngs (2014) in their GMM. Therefore, we used the same $z_{1.0}$ value for the BSSA GMM. We used a $z_{2.5}$ value of 5,171 feet (1,576 meters) for the Campbell and Bozorgnia (2014) GMM.

4.2 PROBABILISTIC SEISMIC HAZARD ANALYSIS

4.2.1 Fault Database and Probabilistic Model

We performed a probabilistic seismic-hazard analysis (PSHA) for the project site for a return period of 2,475 years. We utilized the Third California Earthquake Rupture Forecast model (UCERF3). This is the most up-to-date rupture forecast model for the state of California and is required by ASCE 7-16. We calculated the seismic hazard using the standard methodology for hazard analysis (McGuire, 2004). The seismic-hazard calculations can be represented by the following equation, which is an application of the total-probability theorem.

$$H(a) = \sum_{i} v_i \iint P[A > a|m, r]\, f_{Mi}(\mathrm{m})\, f_{Ri|Mi}(\mathrm{r}, \mathrm{m}) drdm$$

In this equation, the hazard **H*(a)*** is the annual frequency of earthquakes that produce a ground motion amplitude ***A*** higher than ***a***. Amplitude ***A*** may represent peak ground acceleration, velocity, or it may represent spectral pseudo-spectral acceleration (PSa) at a given frequency. The summation in the equation shown extends over all sources (i.e. over all faults and areas). In the above equation, $\boldsymbol{v_i}$ is the annual rate of earthquakes (with magnitude higher than some threshold $\mathbf{M_i}$) in source ***i***, and $\boldsymbol{f_{Mi}}$ ***(m)*** and $\boldsymbol{f_{Ri|Mi}}$ ***(r,m)*** are the probability density functions on magnitude and distance, respectively. ***P[A > a|m, r]*** is the probability that an earthquake of magnitude ***m*** at distance ***r*** produces a ground-motion amplitude ***A*** at the site that is greater than ***a***. Seismic sources may be either faults or area sources; the specification of source geometries and the calculation of $\boldsymbol{f_{Ri}|_{Mi}}$, are performed differently for these two types of sources.

4.2.2 Disaggregation of the Seismic Hazard

We disaggregated the seismic-hazard associated with the 2,475-year return period at the peak ground acceleration, and at periods of 0.2, 0.5, and 1.0 seconds. These disaggregation results are presented in Appendix D. We summarize the dominant scenarios and their relative contributions to the hazard at each period in Table 4.2.2-1. These results represent sources contributing at least one percent to the seismic hazard at the site for the spectral periods considered and for the given return period. Gridded or areal sources are not presented. Bracketed numbers represent the UCERF3 subsection for a given fault.

TABLE 4.2.2-1: Summary of Disaggregation Results for a 2,475-Year Return Period*

SOURCE	R_{RUP}		M_W	PERCENT CONTRIBUTION			
	(KM)	(MILES)		PGA	0.5 SEC	1.0 SEC	3.0 SEC
Hayward (So) [5]	0.3	0.2	7.07	65.9	63.6	66.9	68.6
Hayward (So) [4]	3.7	2.3	6.79	15.5	15.5	14.5	13.9
Mission (connected) [0]	1.1	0.7	6.85	9.3	8.7	8.9	8.7
Calaveras (No) [3]	12.7	7.9	7.28	3.2	4.1	3.5	3.4
Hayward (So) [3]	8.5	5.3	6.80	2.2	2.9	2.3	1.1

*Based on USGS Unified Hazard Tool: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

4.3 DETERMINISTIC SEISMIC HAZARD ANALYSIS

The deterministic seismic hazard analysis (DSHA) involves developing the 84th percentile (i.e., lognormal mean plus one standard deviation) maximum-rotated response spectrum for a spectral damping of 5 percent of critical damping considering characteristic magnitudes of significant faults, without background seismicity, and the aforementioned ground-motion models. However, it is important to note that the definition of the characteristic magnitude is ambiguous when using the UCERF3 model due to its complexity. Based on our communications with developers of ASCE 7-16 and the 2020 NEHRP Provisions, in deterministic analyses, “scenario” earthquakes with significant contribution to hazard should be used in lieu of “characteristic” earthquakes when using UCERF3. We identified the scenario earthquakes by considering the results of the disaggregation of the PSHA results. Accordingly, we considered the scenarios in Table 4.2.2-1, as described below.

We considered the magnitudes in Table 4.2.2-1 and associated distances (R_{RUP}, R_{JB}, R_X) to calculate the deterministic response spectrum. We estimated additional ground motion model parameters (e.g., rupture width, depth to top of rupture, etc.) for each fault/scenario based on fault-specific information published on the United States Geologic Survey (USGS) website. Our analyses indicate a controlling event on the Hayward (So) fault with a moment magnitude (M_W) of 7.07 within 0.2 mile (0.3 kilometer) of the site. We found this scenario controlled for all spectral periods presented.

4.4 RESULTING SURFACE RESPONSE SPECTRA

Following the steps described above, we developed probabilistic and deterministic median-component (RotD50) response spectra. To convert the RotD50 response spectra to maximum-rotated response spectra, we applied the maximum rotation factors discussed in Shahi and Baker (2014). We also applied the mapped risk factors defined in Section 21.2.1.1 of ASCE 7-16 to the probabilistic response spectrum in order to develop a risk-targeted spectrum. We then compared the maximum-rotated deterministic response spectrum with the lower limit deterministic response spectrum defined in Section 21.2.2 of ASCE 7-16 and Supplement No. 1 to finalize the deterministic spectrum.

According to Section 21.2.3 of ASCE 7-16, the MCE_R is controlled by the lesser of the maximum-rotated and risk-targeted probabilistic and the 84th percentile maximum-rotated deterministic response spectra. At this site, the spectral accelerations associated with the deterministic response spectrum are less than the probabilistic response spectrum. Additionally, the MCE_R and DE are not permitted to be lower than 80 percent of the mapped MCE_R and DE response spectra (i.e., the code minimum), respectively. Exhibit 4.4-1 presents the development of the max-rotated 84th percentile deterministic and risk-targeted and max-rotated probabilistic response spectra. Table 4.4-1 and Exhibit 4.4-2 depict the recommended site-specific MCE_R and DE spectra for the project site. Finally, Table 4.4-2 presents site-specific seismic design parameters based on ASCE 7-16 Sections 21.4 and 21.5.

EXHIBIT 4.4-1: (a) Deterministic and (b) Probabilistic Seismic Hazard Analysis Results

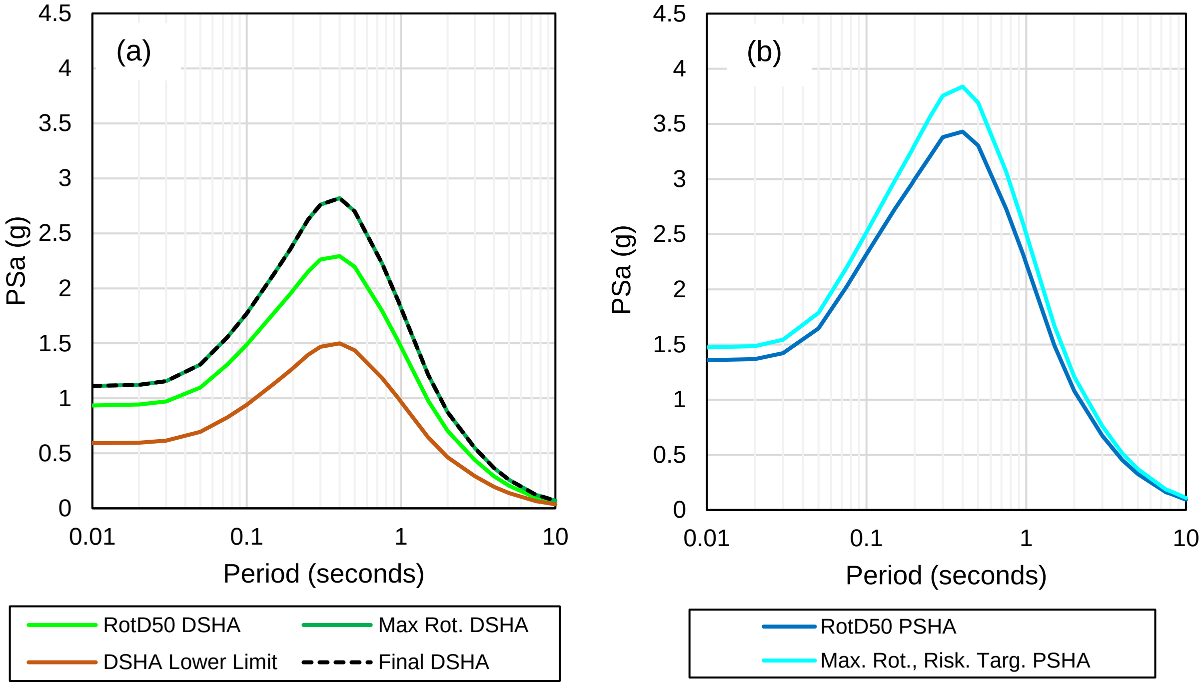


TABLE 4.4-1: Recommended Site-Specific Spectra

PERIOD (SECONDS)	RECOMMENDED SPECTRAL ACCELERATION (g)	
	RISK TARGETED – MAXIMUM-ROTATED MCE_R	MAXIMUM-ROTATED DE
0.01	1.112	0.741
0.02	1.123	0.748
0.03	1.156	0.771
0.05	1.307	0.871
0.075	1.557	1.038
0.10	1.770	1.180
0.15	2.130	1.420
0.193	2.361	1.574
0.20	2.402	1.601
0.25	2.626	1.750
0.30	2.761	1.840
0.40	2.820	1.880
0.50	2.701	1.800
0.75	2.231	1.487
0.963	1.880	1.253
1.0	1.819	1.212

PERIOD (SECONDS)	RECOMMENDED SPECTRAL ACCELERATION (g)	
	RISK TARGETED – MAXIMUM-ROTATED MCE$_R$	MAXIMUM-ROTATED DE
1.5	1.212	0.808
2.0	0.874	0.582
3.0	0.561	0.374
4.0	0.421	0.281
5.0	0.337	0.225
7.5	0.225	0.150
8.0	0.211	0.140
10.0	0.135	0.090

EXHIBIT 4.4-2: Recommended Site-specific MCE$_R$ and DE Response Spectra

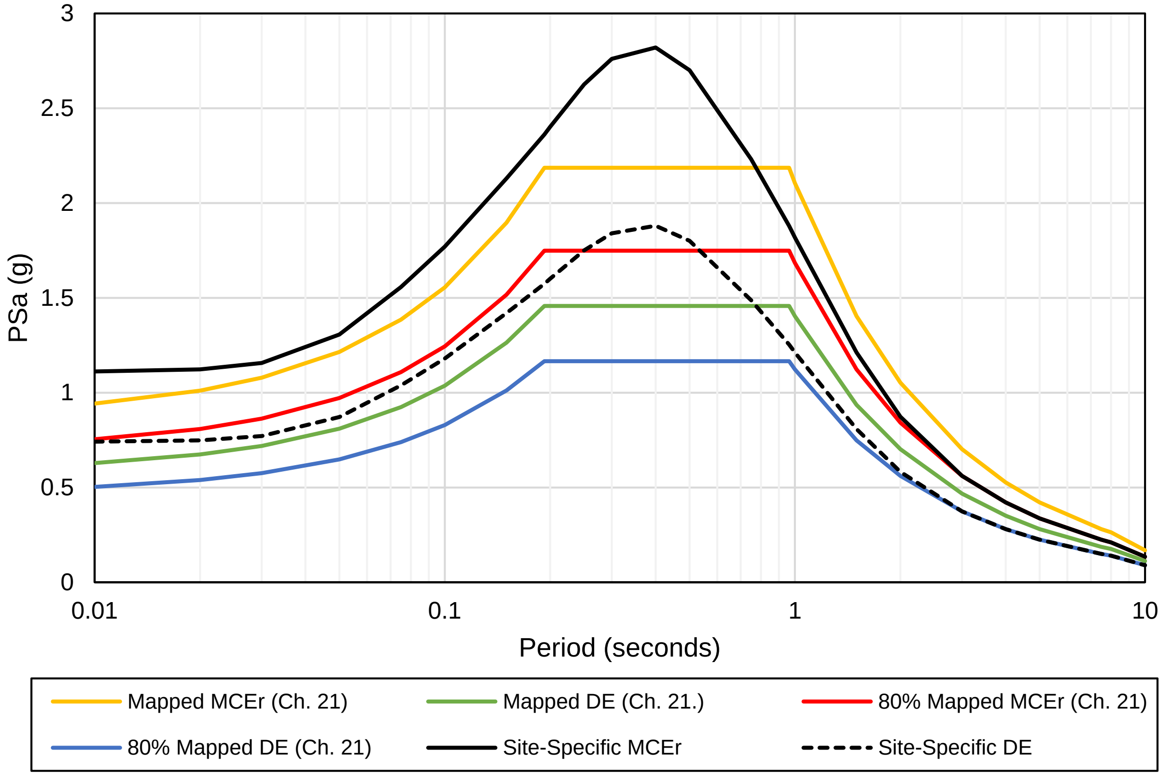


TABLE 4.4-2: Design Acceleration Parameters based on ASCE 7-16 Sections 21.4 and 21.5 (Latitude: 37.6380° Longitude: -122.0526°)

PARAMETER	VALUE
Site Class	D
Mapped MCE$_R$ Spectral Response Acceleration at Short Periods, S_S (g)	2.186
Mapped MCE$_R$ Spectral Response Acceleration at 1-second Period, S_1 (g)	0.842
MCE$_R$ Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.538
MCE$_R$ Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.819
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.692
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	1.212
MCE$_G$ peak ground acceleration adjusted for site class effects, PGA_M (g)	0.935

5.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design, or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party’s interpretation of our report (and subsequent addenda, letters, and verbal discussions).

6.0 POST-CONSTRUCTION MONITORING

As discussed in Section 3.1, potential slope deformation of up to 12 to 36 inches is possible at Section 2-2’ during a seismic event. Furthermore, while a slope setback has been recommended in the 2020 fault exploration report, we understand that retaining walls are planned within the area identified as having active faulting and will therefore potentially be exposed to creep movement. Finally, we anticipate that corrective grading will be required to accommodate the proposed slopes along the eastern portion of the site. For these reasons, we recommend a funding vehicle, such as a Geologic Hazard Abatement District, be established to address future long-term maintenance and repair costs related to geologic hazards.

7.0 EARTHWORK RECOMMENDATIONS

7.1 CORRECTIVE GRADING PLANS

Due to the complex geology and hillside topography, we recommend that we be retained to prepare corrective grading plans for this project as the plans progress. This is important to clarify our geotechnical recommendations related to keyways, benches, cut/fill transition subexcavations, and subdrains. In preparing these plans, we intend to overlay the grading plans with graphic representations of our grading and subsurface drainage recommendations presented in this report. This allows the unique hillside geotechnical recommendations to be clearly displayed on the grading plans and can assist in obtaining more accurate earthwork bids as well as clarifying the geotechnical recommendations as they apply to the final grading plan.

7.2 GRADING

The following grading recommendations are provided for the project based upon the current site grading depicted on the project plans dated May 12, 2021, prepared by AO Architects. As discussed in the section above, final corrective grading plans should be provided prior to the start of site grading.

The Geotechnical Engineer should be notified at least 3 days prior to grading in order to coordinate its schedule with the grading contractor. Grading operations should be observed and tested by the Geotechnical Engineer.

7.3 SELECTION OF MATERIALS

With the exception of the organically contaminated near-surface material, the site soil and rocks containing less than 2 percent organics are suitable for use as engineered fill.

Imported fill material should meet the above requirements and have a plasticity index similar to on-site soil material. We should be given the opportunity to sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

Placement of oversized rock in the fill should be done according to the following specifications.

- The rock should be placed in the deep fills. No rock fragments larger than 6 inches in diameter shall be placed in the upper 10 feet of finished grade.
- The rock size to be placed in the engineered fill should not exceed 18 inches in any dimension. Larger rock sizes should be broken mechanically either by the heavy bulldozers rolling on them or by a pneumatic hammer mounted on a backhoe.
- The rock should be spread and mixed with fines and should not be allowed to nest. Engineered fill consisting of rock fragments only are not allowed. The rock should be mixed with fines at a ratio of 1 to 10, or one load of rock fragments to 10 loads of fines, as approved by us during grading.

7.4 DEMOLITION AND STRIPPING

Grading should begin with the removal of existing structures, if any, including their foundations. Underground structures, such as buried pipes, septic tanks, and leach fields, if any, which will be abandoned or are expected to deteriorate, should be removed from the project site entirely.

All existing artificial fill, vegetation, and soft or compressible soil should be removed as necessary for project requirements. The depth of removal of these materials should be determined by the Geotechnical Engineer’s qualified representative in the field at the time of grading. Evaluation of unsuitable deposits should be performed during grading by sampling and laboratory analyses.

Areas to receive fill or structures and those areas that serve as borrow for fill should be stripped of existing vegetation. Topsoil is estimated to be from 3 to 6 inches in thickness depending on location. Tree roots should be removed to a depth of at least 3 feet below finished grade in cut areas and 3 feet below original grade in fill areas. Subject to approval by the Landscape Architect, strippings and organically contaminated soils that are not suitable for use as engineered fill may be used in landscape areas. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with the mass grading.

Within the development areas, excavations resulting from demolition and stripping which extend below final grades should be cleaned to firm undisturbed soil as determined by the Geotechnical Engineer's representative. Following clearing and grubbing, all depressions in areas to be filled should be scarified, moisture conditioned and backfilled with compacted engineered fill. The requirements for backfill materials and placement procedures are the same as those for engineered fill as described in the "Monitoring and Testing" section.

7.5 KEYWAYS

After stripping, mass grading should begin with construction of toe keys and subdrains. All fills should be adequately keyed into firm natural materials or previously placed engineered fill unaffected by shrinkage cracks. Typical minimum key sizes and typical keyway subdrains are shown in Figures 8 and 9, respectively. Slope keys or benches should be constructed above toe keys as filling progresses. Unless otherwise recommended by the Geotechnical Engineer, such keys should be placed at vertical height intervals of not less than 5 feet and should be excavated into firm competent soil or bedrock. Corrective grading plans should be provided separate from this report based on the final grading plans. Anticipated keyway configurations are depicted on Figure 7, Cross Sections. The actual size of the keyways will be determined by the Geotechnical Engineer in the field during grading.

All keyway excavations should be examined and approved by the Engineering Geologist during grading for extents of adverse bedding, seepage, or bedrock conditions that may affect slope stability. In the event that adverse geologic conditions are detected during grading, additional recommendations may be necessary.

7.6 LANDSLIDE REMOVAL

Landslide deposits at the site are anticipated to be up to 20 to 40 feet thick and will require partial removal based on the current proposed development plan.

The corrective grading plans should include removal of landslide deposits as they affect the proposed development. All removals should be examined and approved by the Engineering Geologist during grading. In the event that adverse geologic conditions are detected during grading, additional recommendations may be necessary.

7.7 GRADED SLOPES AND SLOPE STABILIZATION

The following guidelines may be used when considering slope gradients, heights, and retaining walls.

- In general, graded 2:1 (horizontal: vertical) slopes may be constructed up to 15 feet in vertical height. If higher vertical 2:1 slopes are planned, 2:1 slopes up to a maximum vertical height of 30 feet high may be constructed with reinforcement such as geogrid.
- Graded 2½:1 slopes (unreinforced with geogrid) may be constructed up to 30 feet in vertical height. Slopes exceeding 30 feet in height should include benches and/or concrete ditches, as designed by the Civil Engineer.
- Graded 3:1 slopes or flatter may exceed 30 feet and do not require benches and/or concrete ditches. Major slopes exceeding 50 feet in vertical height should include a minimum 20-foot-wide debris bench along the base of slope.
- Graded cut and fill slopes exceeding 30 feet in height should include benches and/or concrete ditches, as designed by Civil Engineer.

The above guidelines may be considered in combination with vertical retaining wall systems, such as MSE retaining walls; however, such walls upon shallow foundations with downslope conditions will require greater embedment than walls with level foreground at slope bottoms. As an alternate to the slope gradient recommendations described above, if steeper slope gradients exceeding the above-noted maximum vertical heights are desired, such slopes could be specially designed using appropriate geogrid reinforcement. Geogrid-reinforced slopes should be designed and may include primary and secondary geogrid reinforcement layers. As a general design consideration for geogrid reinforced slopes, if selected, the primary horizontal reinforcement would extend on the order of 1.5 times the total vertical slope height of the slope, comprising the outer geogrid reinforced buttress zone of the slope; the typical vertical spacing of geogrid layers would be anticipated ranging from 1.5- to 2.5-foot spacing depending on the total vertical slope height.

7.8 SUBSURFACE DRAINAGE FACILITIES

Subsurface drainage systems are planned for keyways, and at the base of removal areas, as a minimum. Secondary bench subdrains may also be required, depending upon the height of the fill slope and the slope of the underlying native terrain. In addition, observed seepage areas or suspected spring areas should be controlled in development areas through the use of subdrains. Positive fall of at least 1 percent towards an approved outlet should also be provided for all subdrains.

As shown on Figure 9, subdrain systems should consist of a minimum 6-inch-diameter perforated SDR35 pipe encased in Caltrans Class 2 Permeable Material, or crushed rock wrapped in filter fabric. As an alternative, prefabricated geocomposite drainage material (such as SKAPS TNS 220-6) could be considered in lieu of the granular medium above the subdrain zone.

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by us. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

TABLE 7.8-1: Perforated Pipe Requirements

PIPE TYPE	STANDARD	TYPICAL SIZE (INCHES)	PIPE STIFFNESS (PSI)
BELOW 50 FEET OF FINISHED GRADE			
PVC Schedule 80	ASTM D1785	6	530
BETWEEN 15 AND 50 FEET OF FINISHED GRADE			
PVC SDR 23.5	ASTM D3034	6	153
PVC Schedule 40	ASTM D1785	6	135
BETWEEN 0 TO 15 FEET OF FINISHED GRADE			
PVC SDR 35	ASTM D3034	6	46

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

Discharge from the subdrains will generally be low but in some instances may be continuous. Subdrains should outlet into the storm drain system or other approved outlets, and their locations should be surveyed and documented by the project Civil Engineer for future maintenance and for documentation as they relate to the proposed buildings.

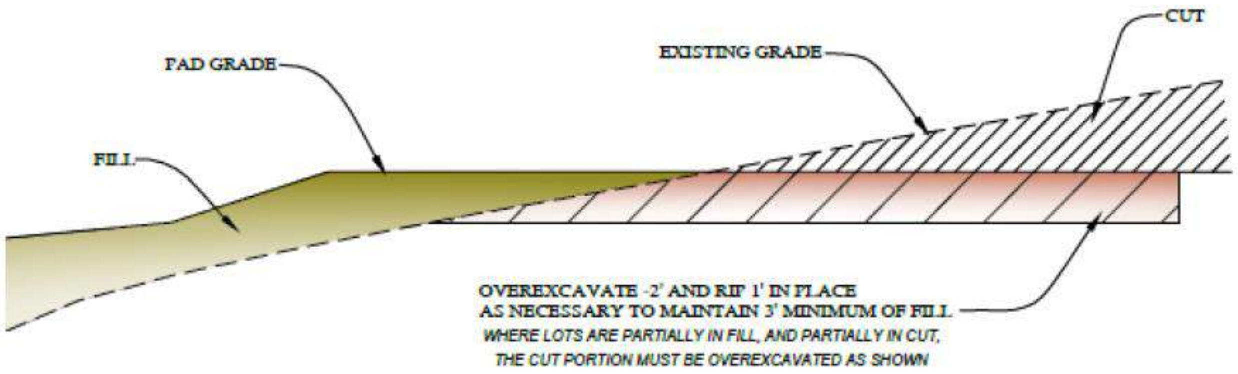
Not all sources of seepage are evident during the time of field work because of the intermittent nature of some of these conditions and their dependence on long-term climatic conditions. Furthermore, new sources of seepage may be created by a combination of changed topography, manmade irrigation patterns, and potential utility leakage. Since uncontrolled water movements are one of the major causes of detrimental soil movements, it is of utmost importance that a Geotechnical Engineer be advised of any seepage conditions so that remedial action may be initiated, if necessary.

Proposed subdrain locations should be included on corrective grading plans prior to the start of site grading. Actual subdrain locations will be determined in the field during grading activities.

7.9 CUT LOTS AND CUT-FILL TRANSITION LOTS

Some buildings in this project will likely be entirely in cut or traversed by a cut-fill transition. We anticipate that significant variations in material properties may occur in areas of cut or cut-and-fill daylighting if not mitigated during site grading. Atterberg Limits and swell test data indicate that there is a potential for a significant differential in swell characteristics across cut areas and cut/fill transitions. Such situations can be detrimental to building performance. As such, it is recommended to overexcavate the cut portions of transition pads and replace the excavated materials with properly compacted, engineered fill. This can be accomplished by subexcavating the natural soil cover and the native rock and replacing the subexcavated material with engineered fill as shown in the exhibit below. The subexcavation depth should be 3 feet for cut-fill transition lots. In addition, cut lot building areas should be overexcavated and reworked to at least 3 feet below rough pad grade.

EXHIBIT 7.9-1: Cut-Fill Transition Detail



WHERE LOTS ARE PARTIALLY IN FILL, AND PARTIALLY IN CUT,
THE CUT PORTION MUST BE OVEREXCAVATED AS SHOWN

7.10 DIFFERENTIAL FILL THICKNESS

We recommend that building sites have a differential fill thickness of less than 10 feet across the building footprint. Local subexcavation of soil/bedrock material and replacement by engineered fill may be necessary to achieve this requirement.

In some cases, keyway excavation adjacent to proposed buildings may create a differential fill thickness in excess of 10 feet and will require additional recommendations in the field by us. Corrective grading recommendations for specific buildings should be provided once final grading plans are reviewed and prior to the start of grading.

7.11 PLACEMENT OF FILL

After removal of soft soil and loose fill, the exposed non-yielding surface of all areas to receive minor fills, secondary slabs-on-grade, or pavements should be scarified to a depth of 12 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fill should be placed in thin lifts. The lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less. Test procedures should be determined in accordance with ASTM D-1557. Additional samples will be collected during site grading and transported to our laboratory for compaction curve testing.

TABLE 7.11-1: Fill Placement Specifications

LOCATION	MINIMUM RELATIVE COMPACTION	MINIMUM MOISTURE CONTENT (PERCENT ABOVE OPTIMUM)
General Fill (< 50 feet deep)	90	4
General Fill (≥ 50 feet deep)	95	2
Keyway	95	2

The contractor should compact finish subgrade and aggregate base to a relative compaction of 95 percent. Aggregate base should meet the requirements for ¾-inch maximum Class 2 AB per Section 26 of the latest Caltrans Standard Specifications.

7.12 MONITORING AND TESTING

It is important that all site preparations for site grading be performed under the observation of the Geotechnical Engineer’s field representative. The Geotechnical Engineer’s field representative should observe all graded area preparation, including demolition and stripping. The final grading plans should be submitted to the Geotechnical Engineer for review.

As used in this report, relative compaction refers to the in-place dry unit weight of soil expressed as a percentage of the maximum dry unit weight of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure, latest edition. Compacted soil is not acceptable if it is unstable; it should exhibit only minimal flexing or pumping, as observed by our representative. The term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define “structural areas” as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

7.13 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed.

As a minimum, we recommend the following.

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.

2. Consider the use of rear lot surface drainage collection systems to reduce overland surface drainage from back to front of lot.

3. Do not allow water to pond near foundations, pavements, or exterior flatwork.

We provide recommendations for subsurface drainage facilities associated with remedial grading in Section 7.8.

7.14 STORMWATER INFILTRATION AND SELECT PROJECT RISK LEVEL FACTORS

Percolation testing performed during the Krazan 2020 geotechnical investigations suggests that the near-surface soil is not conducive to infiltration. Unless subdrains are installed, the near-surface site soil is expected to have a low permeability value for stormwater infiltration in grassy swales or permeable pavers. Therefore, Best Management Practices should assume that limited stormwater infiltration will occur at the site.

7.15 STORMWATER BIORETENTION AREAS

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, one of the following options should be followed.

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

Site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements subject to lateral loads (such as from impact or traffic patterns), additional design considerations may be recommended. If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend we be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

7.16 LANDSCAPING CONSIDERATION

As the near-surface soil is moderately to highly expansive, we recommend greatly restricting the amount of surface water infiltration near structures, pavements, flatwork, and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements.
- Using low precipitation sprinkler heads.
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system.
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements.
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements.
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that these items be incorporated into the landscaping plans.

8.0 FOUNDATION RECOMMENDATIONS

The proposed buildings will need to be able to address the shrink-swell of the surface soil.

Based on our experience and the anticipated building types, the proposed structures can be founded on either spread footings with slab-on-grade or a structural mat foundation. If a spread footing option is selected additional earthwork will be required to prepare the building pad and mitigate expansive soil as described below.

Following are our recommendations for design of the anticipated foundation types.

8.1 SETTLEMENT

Structural loads have not been provided to us; however, based on our understanding of the project and proposed buildings, we anticipate total load-induced settlement of up to 1 inch and differential load-induced settlement of ½ inch over 50 feet may occur following construction.

8.2 CONVENTIONAL FOOTINGS WITH SLAB-ON-GRADE

The proposed charter school and residential structures may be supported by a conventional spread footing system that consists of a perimeter strip footing with interior column spread footings. Provided the upper 18 inches of building pad subgrade consists of low to non-expansive import fill or 18 inches of calcium quicklime amended soil, the following soil design criteria may be considered.

8.2.1 Footing Dimensions and Allowable Bearing Capacity

Footings should be designed with the minimum dimensions as follows in the table below.

TABLE 8.2.1-1: Minimum Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (INCHES)	MINIMUM WIDTH (INCHES)
Continuous	30	12
Isolated	30	18

* below lowest adjacent pad grade

The minimum footing depths shown above are taken from lowest adjacent pad grade.

Foundations meeting the minimum dimensions recommended above can be designed for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for dead-plus-live loads. This bearing capacity can be increased by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

8.2.2 Waterstop

If a two-pour system is used for footings and slab, the cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent finish exterior grade. If this is not done, then we recommend the addition of a waterstop between the two pours to reduce moisture penetration through the cold joint and migration under the slab. Use of a monolithic pour would eliminate the need for the waterstop.

8.2.3 Reinforcement

The structural engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Continuous footings should be reinforced with at least top and bottom steel to provide structural continuity and to permit spanning of local irregularities. At a minimum, continuous footings should be designed to structurally span a clear distance of 5 feet.

To help resist expansive soil movement, continuous footings should be reinforced with at least four No. 4 steel reinforcement bars, two top and two bottom.

8.3 CONVENTIONALLY REINFORCED MAT FOUNDATIONS

The proposed residential structures may also be supported on a rigid mat foundation bearing on compacted fill engineered without regard for the presence of expansive soil. The mat may be designed for an average allowable bearing pressure of 1,500 psf for dead-plus-live loads. This allowable bearing pressure may be increased to 2,000 psf in areas of loading concentration. The allowable bearing pressure can be increased by one-third for short-term loading that includes wind or seismic load combinations.

If a spring constant is needed for design, a modulus of subgrade reaction (k_s) of 100 pounds per square inch per inch of deflection can be used; this modulus is for a 1-foot-by-1-foot area. If the modulus is applied to larger areas, the modulus should be modified by multiplying the k_s value by 1/B where B is the side dimension of the larger area.

To address expansive soil in the building pad, the mats should be designed for an edge cantilever condition of 6 feet and an internal span of 20 feet. If low expansive engineered fill or lime treatment is used for the foundation subgrade, the reinforced mat foundation may be designed without this consideration.

8.4 LIME TREATMENT

If calcium quicklime treatment is used to mitigate expansive soil conditions in the building pad, we recommend uniformly mixing the subgrade soil with at least 5 percent high calcium lime by dry weight; this percentage should be verified by lab testing of building pad soil prior to mixing. The soil should be moisture conditioned to at least 4 percentage points above the optimum moisture content before mixing. The mixing should be performed in accordance with the current version of Caltrans Standard Specifications with the following exceptions.

1. Following mixing, the treated soil should be allowed to fully hydrate prior to compaction.

2. Following hydration, the treated soil should be compacted according to ASTM D-1557 to not less than 90 percent relative compaction at a moisture content at least 2 percentage points above the optimum to a non-yielding surface.

8.5 SLAB MOISTURE VAPOR REDUCTION

When buildings are constructed with concrete slab-on-grade floors, including structural mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. A moisture retarder system should be constructed directly beneath the slab on-grade that consists of the following:

    a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs". The vapor retarder should be **underlain by**

    b. 4 inches of clean crushed rock to act as a capillary break. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve. If a structural mat is used, this capillary break may be omitted.

2. Concrete should have a concrete water-cement ratio of no more than 0.50.

3. Inspection and testing should be performed during concrete placement to check that the proper concrete and water cement ratio are used.

4. The slab should be moist cured for a minimum of 3 days or use other equivalent curing specified by the structural engineer should be implemented.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing. If a sand or pea gravel is used above the vapor retarder membrane along with a structural mat, the edge of the mat above should be thickened to cut off water getting in between the slab and the membrane. The thickened edge should be as thick as the sand or pea gravel layer and at least 12 inches wide.

8.6 FOUNDATION LATERAL RESISTANCE

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design.

- Passive Lateral Pressure: 300 pcf
- Coefficient of Friction: 0.30

The above allowable values include a factor of safety of 1.5. The above values may be increased by one-third for the short-term effects of wind or seismic loading.

Passive lateral pressure should not be used for footings on or above slopes.

9.0 SLABS-ON-GRADE

9.1 INTERIOR CONCRETE FLOOR SLAB IN CONJUNCTION WITH FOOTINGS

Due to the high expansion potential of the near-surface soil, we recommend that interior floor slab be supported on non-expansive fill to reduce the likelihood of slab damage from heave or shrinkage. We recommend the slab-on-grade be at least 5 inches in thickness and reinforced with at least No. 4 rebar spaced at 18-inch centers each way; reinforcement steel should be in the middle one-third of the slab.

The structural engineer should provide final design thickness and additional reinforcement, if necessary, for the intended structural loads.

9.2 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum section of 6 inches of concrete over 4 inches of aggregate base. Compact the aggregate base to at least 90 percent relative compaction (ASTM D1557). Thicken flatwork edges to at least 10 inches to help control moisture variations in the subgrade and place wire mesh or rebar within the middle third of the slab to help control the width and offset of cracks. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

10.0 RETAINING WALLS

Based on our 2016 and 2020 fault exploration reports, the site retaining walls are proposed within the mapped active fault zone. Retaining walls within the active fault zone should be designed to either 1) incorporate joints to accommodate potential fault creep over time, or 2) consider long-term maintenance. We understand that the project is contemplating annexing into a Geologic Hazard Abatement District (GHAD) to accommodate maintenance of the proposed retaining walls and open space within the site as needed.

10.1 LATERAL SOIL PRESSURES

The proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Recommendations for wall drainage are presented in a later section. Proposed walls should be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

Basement walls or retaining walls supporting building loads should be designed for at-rest lateral loading conditions. The recommended lateral equivalent fluid pressures (static case) are presented below.

TABLE 10.1-1: Lateral Earth Pressures

BACKFILL SLOPE CONDITION	EQUIVALENT FLUID PRESSURES (PCF)			
	CANTILEVERED (ACTIVE)		RESTRAINED (AT-REST)	
	WITHOUT HYDROSTATIC PRESSURES (PCF)	WITH HYDROSTATIC PRESSURES (PCF)	WITHOUT HYDROSTATIC PRESSURES (PCF)	WITH HYDROSTATIC PRESSURES (PCF)
Level	55	95	80	120
4:1	60	100	80	120
3:1	60	100	85	125
2:1	65	105	90	130

Surcharge loads from buildings, vehicles, hardscape, or paving should be included in the wall design if the surcharge loading is situated above a 1:1 line of projection from the base of the retaining wall footing or bottom block. We understand that the majority of site retaining walls will be constructed using a tiered system. Wall tiers constructed within a 1:1 (horizontal:vertical) influence line of overlying tiers should be designed to account for the surcharge load of walls above. For sloping foreground conditions of the upper wall tiers, footings should be deepened such that a minimum of 10 feet horizontal distance to daylight is achieved between the footing bottom and the slope. We provide recommendations for drilled piers and helical pile foundation alternatives.

Passive pressures acting on footings may be assumed as 300 pounds per cubic foot (pcf) provided that the area in front of the retaining wall is level for a distance of at least 10 feet. The upper 1 foot of soil should be excluded from passive pressure computations unless it is confined by pavement or a concrete slab. The friction factor for sliding resistance may be assumed as 0.30.

The above lateral earth pressures assume sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Under seismic conditions, the active incremental seismic force along the face of a retaining wall should be added to the static active pressures, and can be calculated as following, in Table 10.1-2.

TABLE 10.1-2: Seismic Increment

BACKFILL SLOPE CONDITION	SEISMIC INCREMENT (ΔP)
Level	$15 \times H^2$
4:1	$20 \times H^2$
3:1	$30 \times H^2$
2:1	$40 \times H^2$

H is the design height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at 0.3 x H from the base of the wall. Since seismic loading requires soil movement, evaluation of the seismic case should comprise adding the seismic increment to the active soil pressure for all wall types.

Global stability analysis should be performed as part of retaining wall designs. Construct a drainage system, as recommended below, to reduce hydrostatic forces behind the retaining wall.

10.2 RETAINING WALL DRAINAGE

Either graded rock drains or geosynthetic drainage composites should be constructed behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives.

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. The rock drain should be placed directly behind the walls of the structure.

2. The rock drains should extend from the wall base to within 12 inches of the top of the wall.

3. A minimum of 4-inch-diameter perforated pipe (glued joints and end caps) should be placed at the base of the wall, inside the rock drain and fabric, with perforations placed down.

4. The pipe should be placed at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

We should review and approve geosynthetic composite drainage systems prior to use.

10.3 BACKFILL

Backfill behind the retaining walls should be placed and compacted in accordance with Section 7.11. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

10.4 FOUNDATIONS

10.4.1 Continuous Footings

Retaining walls may be supported on continuous footings designed in accordance with recommendations presented in Section 8.2, except the minimum embedment depth should be increased to 36 inches below lowest adjacent soil grade. For sloping foreground conditions footings should be deepened such that a minimum of 10 feet horizontal distance to daylight is achieved between the footing bottom and the slope. Footings should be deepened such that the footing bottom extends below a 1:1 (horizontal:vertical) influence line from building pads.

10.4.2 Drilled Piers

The proposed retaining walls may be founded on cast-in-place, drilled piers. The minimum diameter for the piers should be 12 inches, and these should be spaced no closer than 3 pier diameters apart. An allowable skin friction of 500 psf may be used in design; skin friction should be disregarded in the upper 2 feet of embedment. The allowable skin friction may be increased by one-third when considering seismic or wind loads.

Lateral loads may be resisted by passive pressures generated by the soil below a depth of 2 feet of the pier. Resistance to lateral loads can be obtained from passive resistance against the drilled pier face. For passive resistance, an equivalent fluid weight of 250 pcf acting on 2 times the pier diameter may be used for the portions below a depth of 2 feet. For piers on or near slopes, the portion of the pier within 10 feet from the face of the slope should be neglected when computing passive resistance. The passive resistance may be increased by one-third to include short-term wind or seismic effects.

“Mushrooming” at the top of the piers should be avoided to prevent unnecessary uplift forces from being applied to the piers, and forming the upper portion of piers or other alternatives to removing excess concrete at the top of the piers may be necessary. Additionally, to further reduce panel movement, we recommend the panels be underlain with a degradable material such as “surevoid,” or equivalent material, at least 2 inches thick between the bottom of the panels and the supporting soil. The use of a void forming material will reduce potential vertical panel movement.

Pier-drilling operations and concrete placement should be coordinated such that pier holes are left open a minimum amount of time. Pier holes should not be allowed to desiccate visibly before placing concrete. Depressions at the tops of the piers resulting from drilling operations or from any other cause should be backfilled to prevent ponding. In order to minimize potential future pier settlements, loose soil “slough” should be removed from the bottom of pier holes prior to placing concrete. If water collects in the pier shaft, it should be pumped out prior to the placement of concrete. Concrete should be placed by means of a tremie pipe or similar device to avoid concrete contamination by soil dislodging from the pier shaft.

We recommend that the excavation of piers be performed under our direct observation to establish that the piers are founded in suitable materials. Due to the potential for caving, each shaft may need to be cased. If groundwater is encountered, remove it from excavations prior to concrete placement. If groundwater cannot be removed from excavations prior to concrete placement, then we recommend that concrete be placed by tremie pipe. The concrete should be tremied to the bottom of the hole keeping the tremie pipe below the surface of the concrete to avoid entrapment of water in the concrete. As concrete is poured, water is displaced out of the hole. Drilling into bedrock may be difficult and require drill rigs capable of drilling the bedrock materials described in previous sections. The use of rock barrels/buckets may be needed to maintain plumbness and integrity of pier holes.

10.4.3 Helical Piles

If helical piles are contemplated, we should be retained to review the proposed design-build package. We also recommend designing a load-test program to confirm the vertical and lateral capacity and the torque force. Finally, we should be retained to provide the following.

- Observation of helical pile installation
- Observation of reinforcing steel placement
- Concrete – placement observations, sampling and field testing, and laboratory testing
- DSM Wall Grout– placement observations, sampling and field testing, and laboratory testing

11.0 PAVEMENT DESIGN

11.1 FLEXIBLE PAVEMENTS

Based on our exploration, we anticipate an R-value of 5 will be appropriate for preliminary design of flexible pavements. Using estimated traffic indexes for various pavement loading requirements, we developed the following recommended pavement sections using Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

TABLE 11.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (inches)	CLASS 2 AGGREGATE BASE (inches)
5	3	10
6	3½	13
7	4	16
8	5	18

The civil engineer should determine the appropriate traffic indexes based on the estimated traffic loads and frequencies.

11.2 RIGID PAVEMENTS

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements.

- Use a minimum section of 6 inches of Portland Cement concrete over 6 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Minimum control joint spacing should be in accordance with Portland Cement Association guidelines.

11.3 SUBGRADE AND AGGREGATE BASE COMPACTION

The finished subgrade and aggregate base should be compacted in accordance with Section 7.11. Aggregate base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with the latest Caltrans Standard Specifications.

11.4 CUT-OFF CURBS

Overly wet pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

12.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.2 for the Parcel Group 3 project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document’s applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO’s documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO’s scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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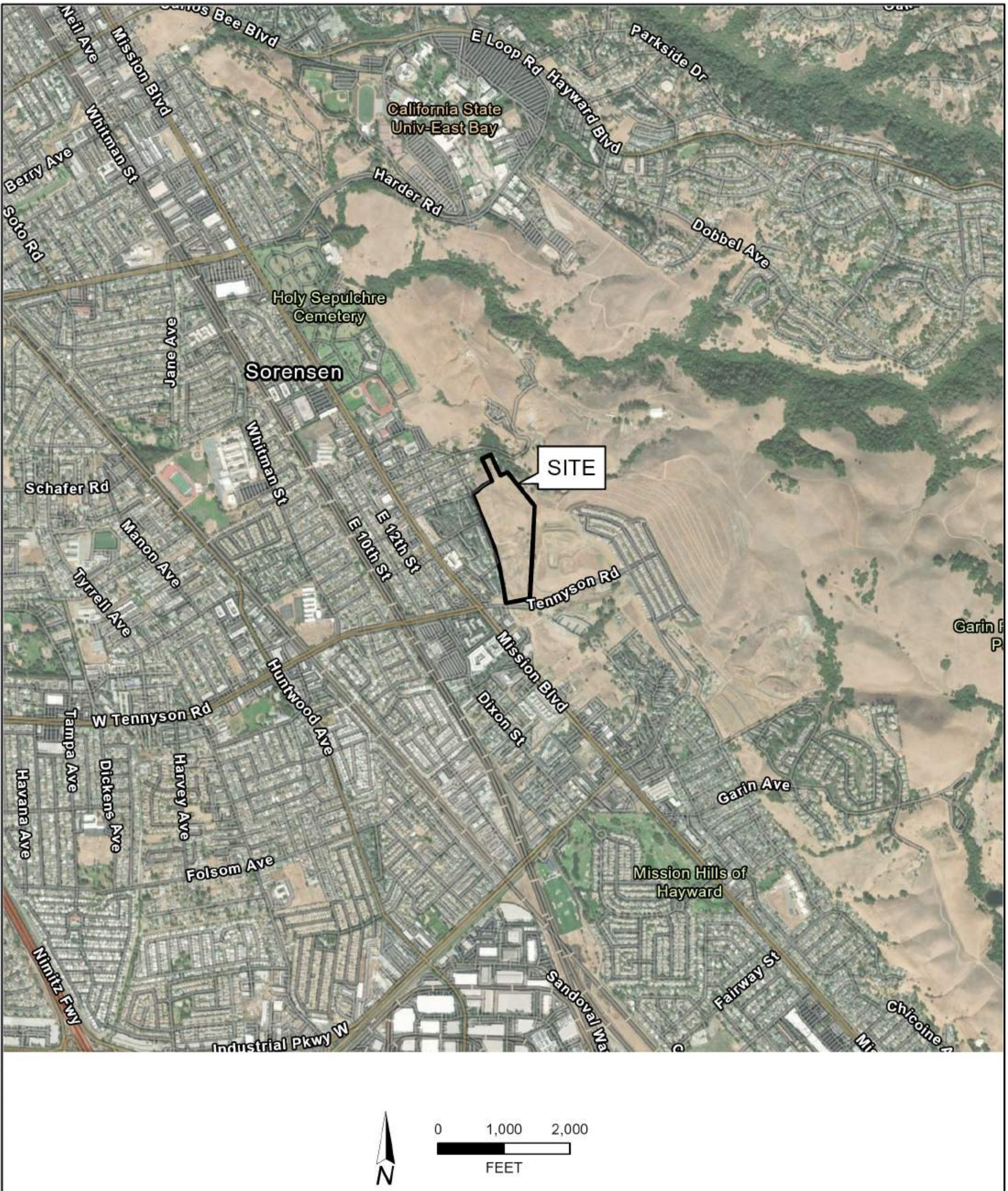
FIGURES



BASEMAP SOURCE: ESRI MAPPING SERVICE 11/4/2019

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VICINITY MAP
PARCEL GROUP 3
HAYWARD, CALIFORNIA

PROJECT NO. : 15577.000.001
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: RHB
FIGURE NO. 1

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LAYOUT: VICINITY USER: JVERGARA

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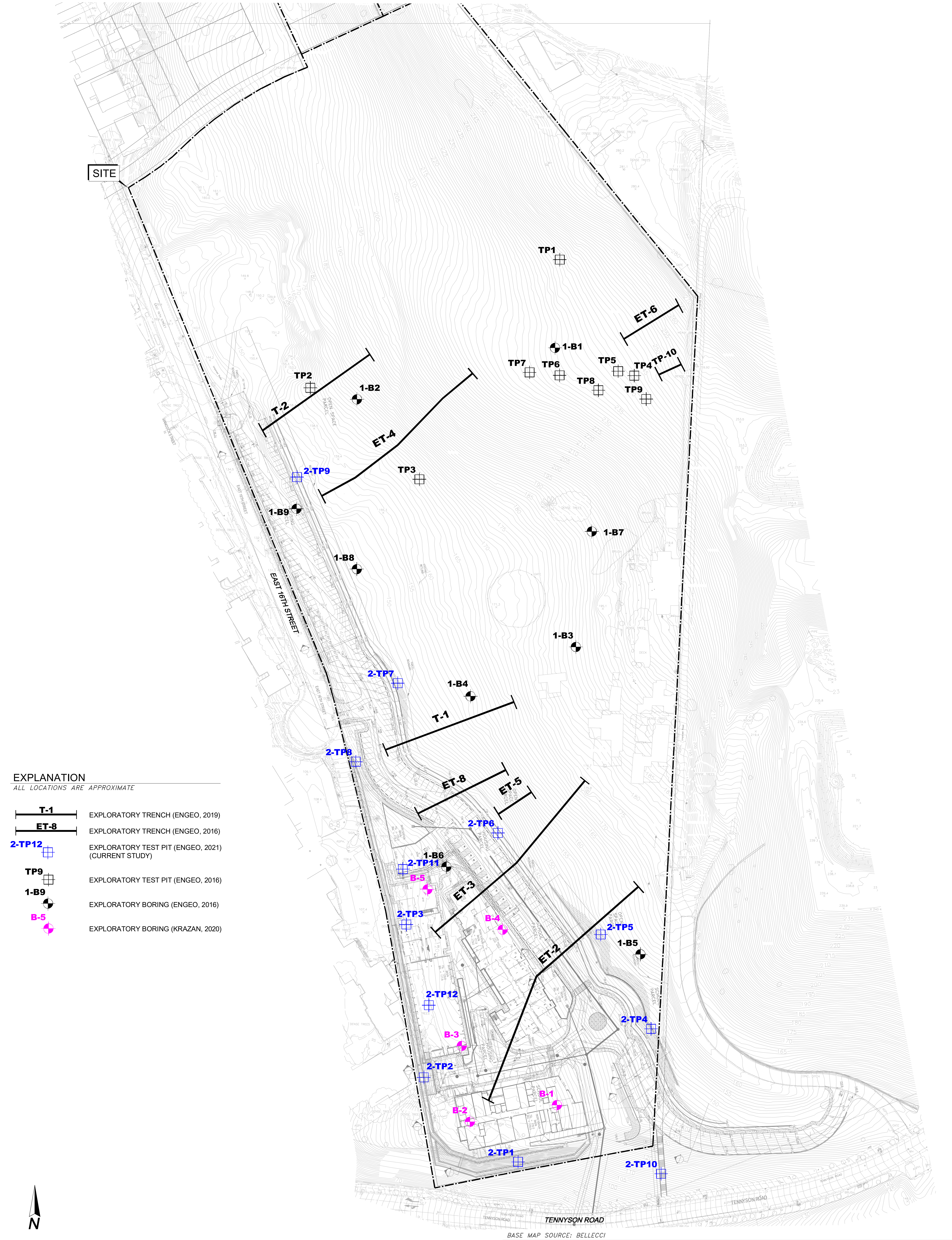

BASE MAP SOURCE: BELLECCI

ENGEO Expect Excellence	SITE PLAN PARCEL GROUP 3 HAYWARD, CALIFORNIA	PROJECT NO.: 15577.000.001 SCALE: AS SHOWN DRAWN BY: JSW CHECKED BY: RHB	FIGURE NO. 2

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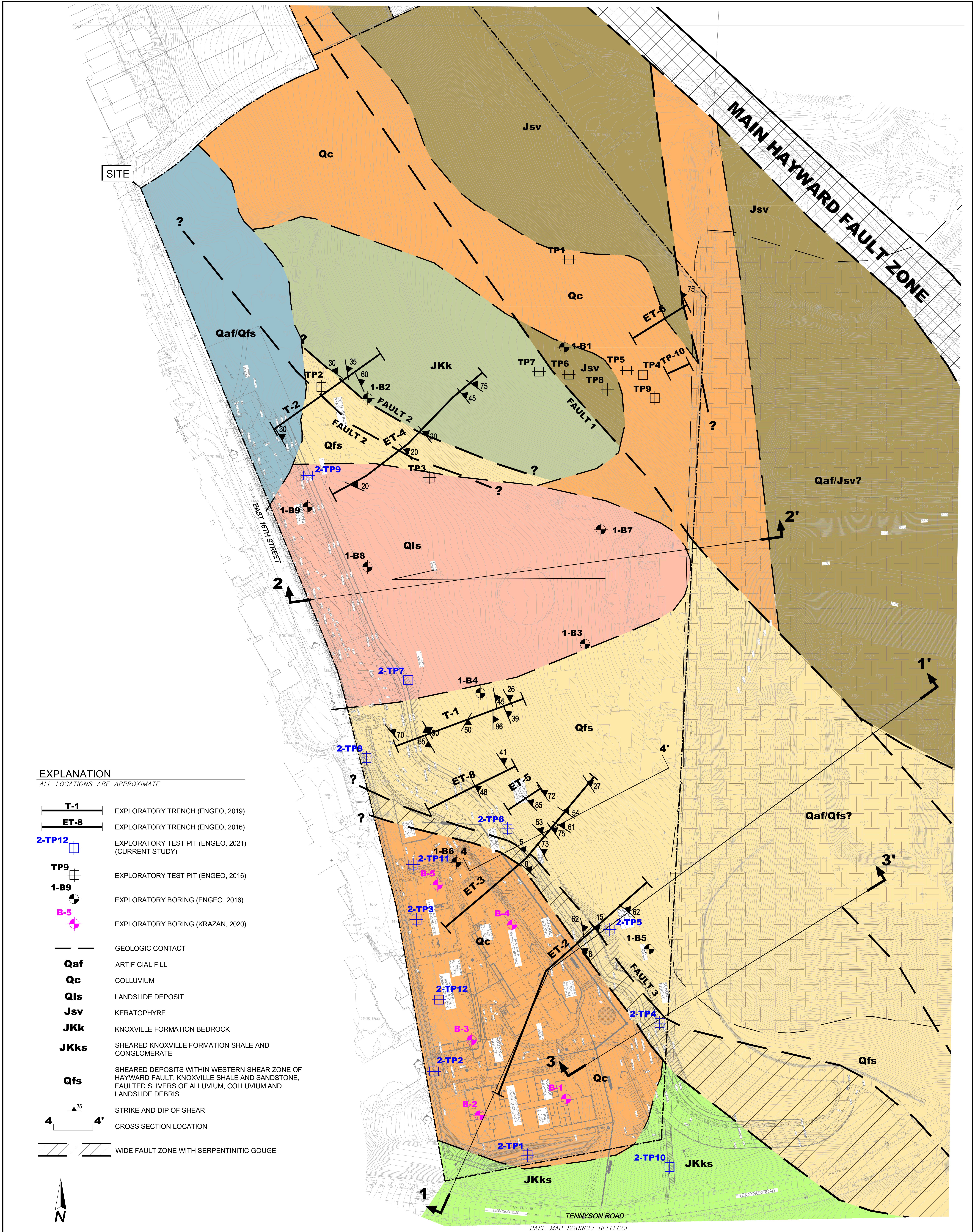
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

T-1	EXPLORATORY TRENCH (ENGEO, 2019)
ET-8	EXPLORATORY TRENCH (ENGEO, 2016)
2-TP12 ⊞	EXPLORATORY TEST PIT (ENGEO, 2021) (CURRENT STUDY)
TP9 ⊞	EXPLORATORY TEST PIT (ENGEO, 2016)
1-B9 ◐	EXPLORATORY BORING (ENGEO, 2016)
B-5 ◐	EXPLORATORY BORING (KRAZAN, 2020)
— —	GEOLOGIC CONTACT
Qaf	ARTIFICIAL FILL
Qc	COLLUVIUM
Qls	LANDSLIDE DEPOSIT
Jsv	KERATOPHYRE
JKk	KNOXVILLE FORMATION BEDROCK
JKks	SHEARED KNOXVILLE FORMATION SHALE AND CONGLOMERATE
Qfs	SHEARED DEPOSITS WITHIN WESTERN SHEAR ZONE OF HAYWARD FAULT, KNOXVILLE SHALE AND SANDSTONE, FAULTED SLIVERS OF ALLUVIUM AND LANDSLIDE DEBRIS
75	STRIKE AND DIP OF SHEAR
4 4'	CROSS SECTION LOCATION
▨	WIDE FAULT ZONE WITH SERPENTINITIC GOUGE

BASE MAP SOURCE: BELLECCI

ENGEO
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SITE GEOLOGY AND FAULTING
PARCEL GROUP 3
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.001
SCALE: AS SHOWN
DRAWN BY: JSW
CHECKED BY: RHB
FIGURE NO.
3

ORIGINAL FIGURE PRINTED IN COLOR

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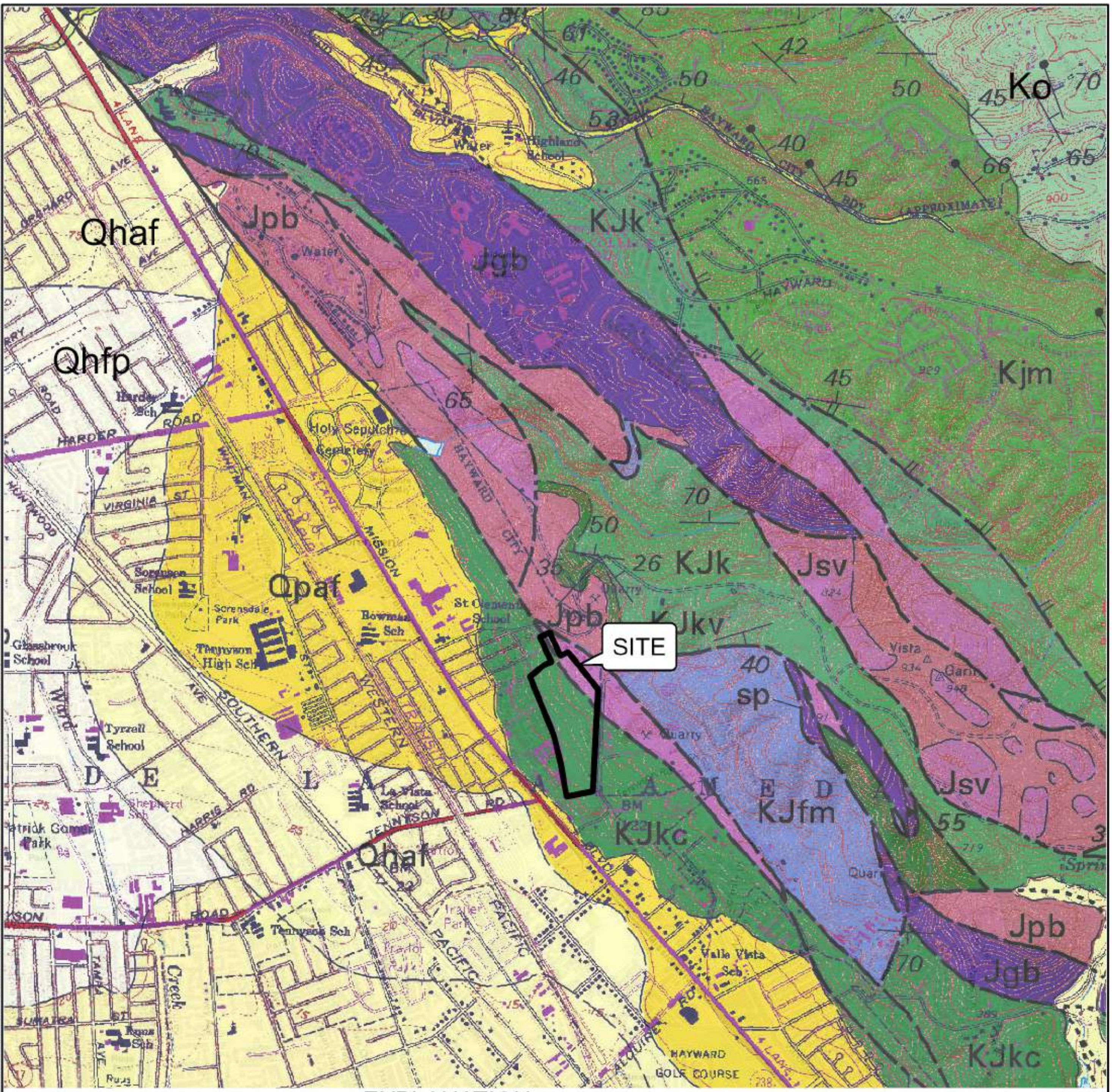

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qhaf	ALLUCIAL FAN AND FLUVIAL DEPOSITS
Qhfp	FLOODPLAIN DEPOSITS
Qpaf	ALLYUVIAL FAN AND FLUVIAL DEPOSITS
Jpb	PILLOW BASALT, BASALT BRECCIA, AND MINOR DIABASE
Jgb	GABBRO
Jsv	KERATOPHYRE AND QUARTZ KERATOPHYRE
Ko	OAKLAND CONGLOMERATE
KJk	KNOXVILLE FORMATION
Kjm	JOAQUIN MILLER FORMATION
KJkv	VOLCANOCLASTIC BRECCIA
KJfm	FRANCISCAN COMPLEX, MELANGE
KJkc	CONGLOMERATE

N

0 1,000 2,000

FEET

BASEMAP SOURCE: GRAYMER, 2000

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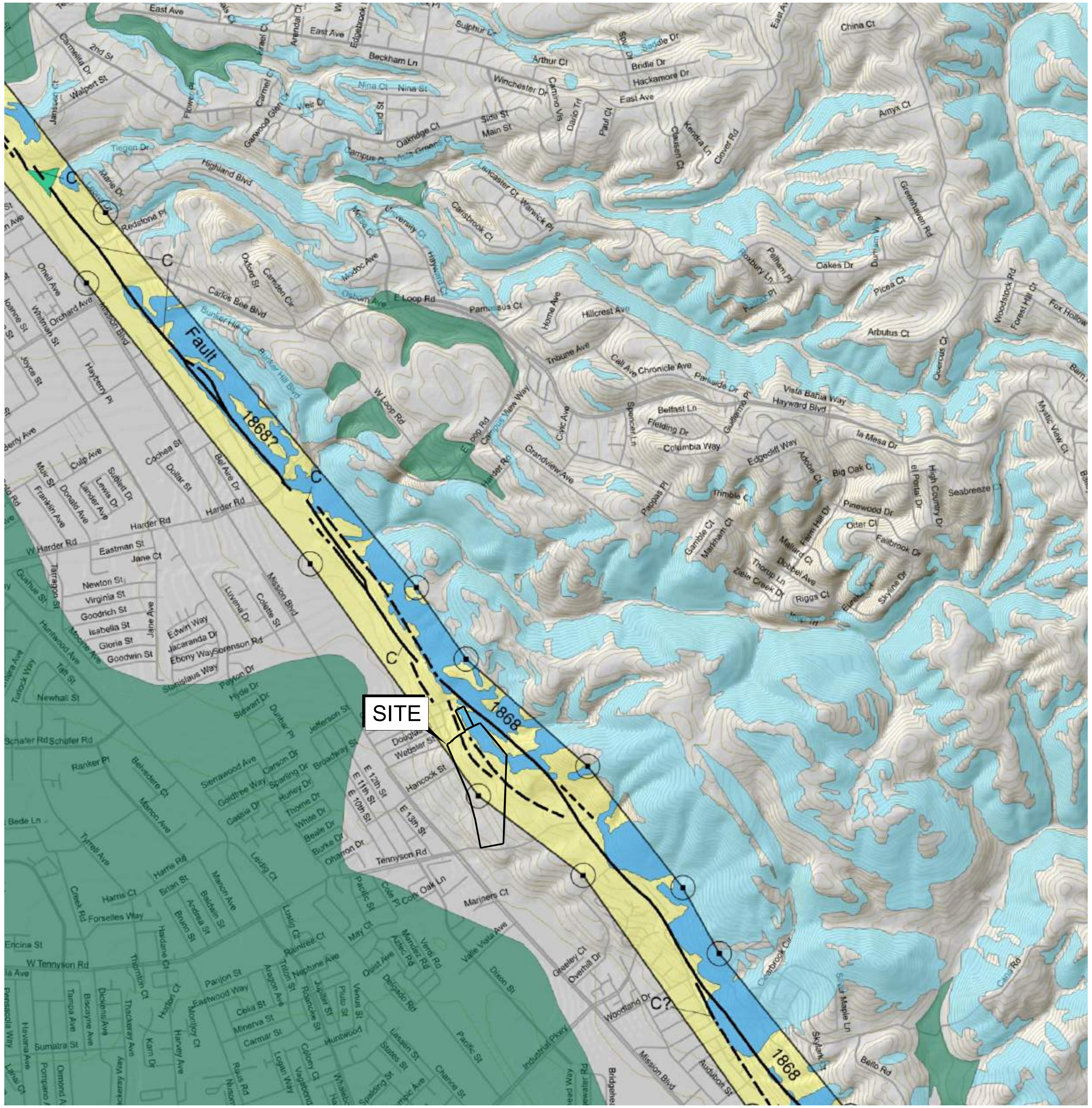
REGIONAL GEOLOGIC MAP
PARCEL GROUP 3
HAYWARD, CALIFORNIA

PROJECT NO. : 15577.000.001
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: RHB
FIGURE NO.
4



EXPLANATION

EARTHQUAKE FAULT ZONES

1906 C ? ?

ACTIVE FAULT TRACES:

FAULTS CONSIDERED TO HAVE BEEN ACTIVE DURING HOLOCENE TIME AND TO HAVE POTENTIAL FOR SURFACE RUPTURE; SOLID LINE WHERE ACCURATELY LOCATED, LONG DASH WHERE APPROXIMATELY LOCATED, SHORT DASH WHERE INFERRED, DOTTED WHERE CONCEALED; QUERY (?) INDICATES ADDITIONAL UNCERTAINTY. EVIDENCE OF HISTORIC OFFSET INDICATED BY YEAR OF EARTHQUAKE-ASSOCIATED EVENT OR C FOR DISPLACEMENT CAUSED BY FAULT CREEP.

ZONES OF REQUIRED INVESTIGATION

🟨 EARTHQUAKE FAULT ZONES:

ZONES ARE AREAS DELINEATED AS STRAIGHT-LINE SEGMENTS THAT CONNECT ENCIRCLED TURNING POINTS ENCOMPASSING ACTIVE FAULTS THAT CONSTITUTE A POTENTIAL HAZARD TO STRUCTURES FROM SURFACE FAULTING OR FAULT CREEP SUCH THAT AVOIDANCE AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2621.5(A) WOULD BE REQUIRED

SEISMIC HAZARD ZONES

🟩 LIQUEFACTION:

AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED

🟦 EARTHQUAKE-INDUCED LANDSLIDES:

AREAS WHERE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(C) WOULD BE REQUIRED

OVERLAPPING ZONES:

🟨🟩 OVERLAP OF EARTHQUAKE FAULT ZONE AND LIQUEFACTION ZONE AREAS THAT ARE COVERED BY BOTH EARTHQUAKE FAULT ZONE AND LIQUEFACTION ZONE. NOTE: MITIGATION METHODS DIFFER FOR EACH ZONE – AP ACT ONLY ALLOWS AVOIDANCE; SEISMIC HAZARD MAPPING ACT ALLOWS MITIGATION BY ENGINEERING/GEOTECHNICAL DESIGN AS WELL AS AVOIDANCE.

🟦🟨 OVERLAP OF EARTHQUAKE FAULT ZONE AND EARTHQUAKE-INDUCED LANDSLIDE ZONE AREAS THAT ARE COVERED BY BOTH EARTHQUAKE FAULT ZONE AND EARTHQUAKE-INDUCED LANDSLIDE ZONE. NOTE: MITIGATION METHODS DIFFER FOR EACH ZONE – AP ACT ONLY ALLOWS AVOIDANCE; SEISMIC HAZARD MAPPING ACT ALLOWS MITIGATION BY ENGINEERING/GEOTECHNICAL DESIGN AS WELL AS AVOIDANCE.

BASE MAP SOURCE: CGS, 2012

ENGEO
Expect Excellence

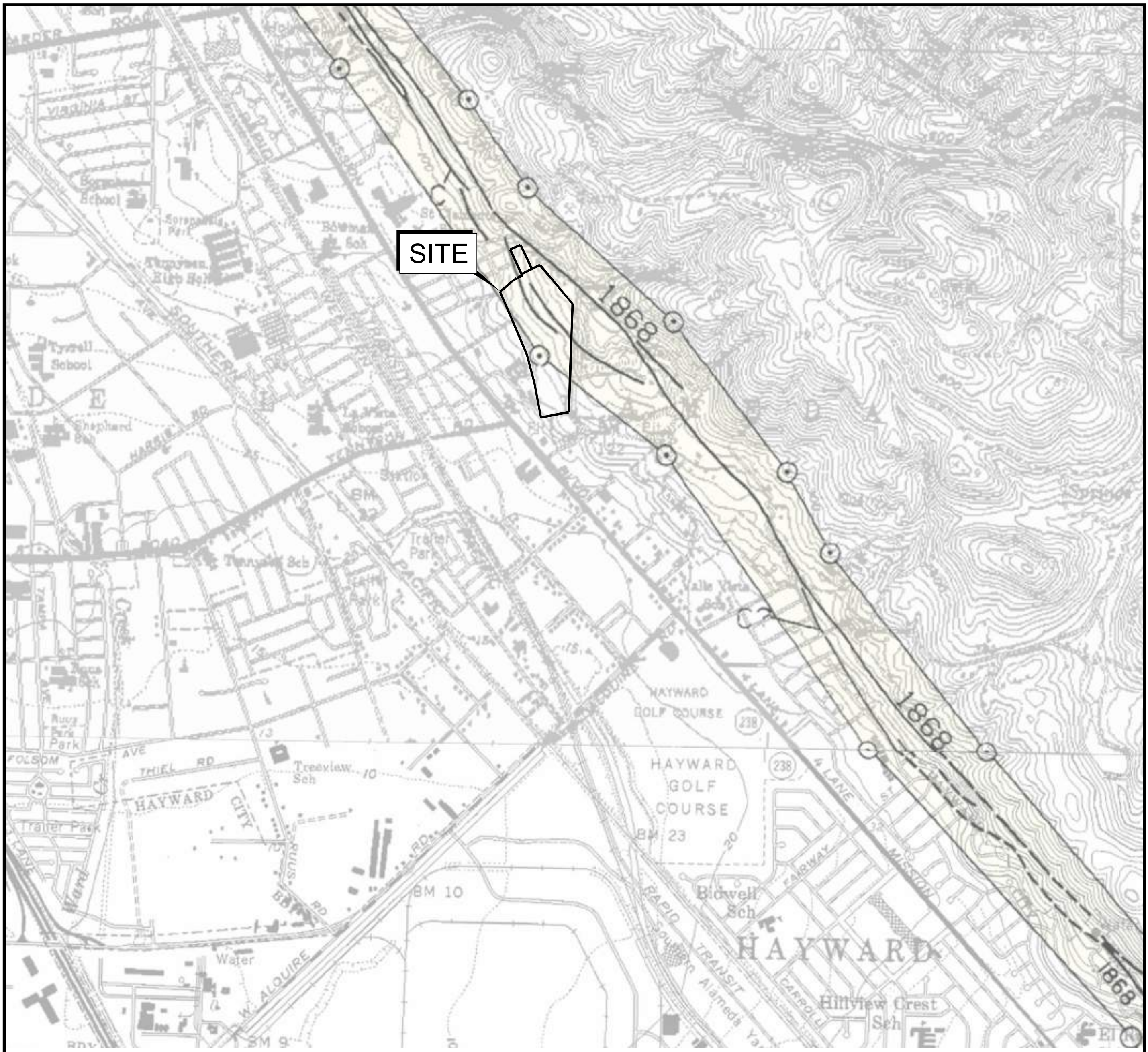
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION
PARCEL GROUP 3
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.001
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: RHB
FIGURE NO.
5

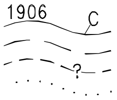
FILE PATH: G:\Drafting\PROJECTS_14000 to 15999\15577\15577000000\Geo\GEX\15577000001-GEX-5-EarthquakeZones-0521.dwg SAVE DATE: 6/1/2021 10:51:06 AM SAVED BY: jvergara

ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\PROJECTS_14000 to 15999\15577\15577000000\Geo\GEX\15577000001-GEX-6-EarthquakeFault-0521.dwg SAVE DATE: 5/28/2021 4:21:58 PM SAVED BY: jvergara

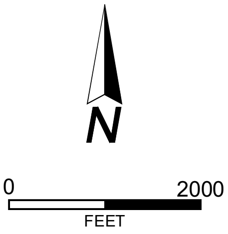


EXPLANATION



FAULTS CONSIDERED TO HAVE BEEN ACTIVE DURING HOLOCENE TIME AND TO HAVE A RELATIVELY HIGH POTENTIAL FOR SURFACE RUPTURE; SOLID LINE WHERE ACCURATELY LOCATED, LONG DASH WHERE APPROXIMATELY LOCATED, SHORT DASH WHERE INFERRED, DOTTED WHERE CONCEALED; QUERY (?) INDICATES ADDITIONAL UNCERTAINTY. EVIDENCE OF HISTORIC OFFSET INDICATED BY YEAR OF EARTHQUAKE-ASSOCIATED EVENT OR C FOR DISPLACEMENT CAUSED BY CREEP OR POSSIBLE CREEP

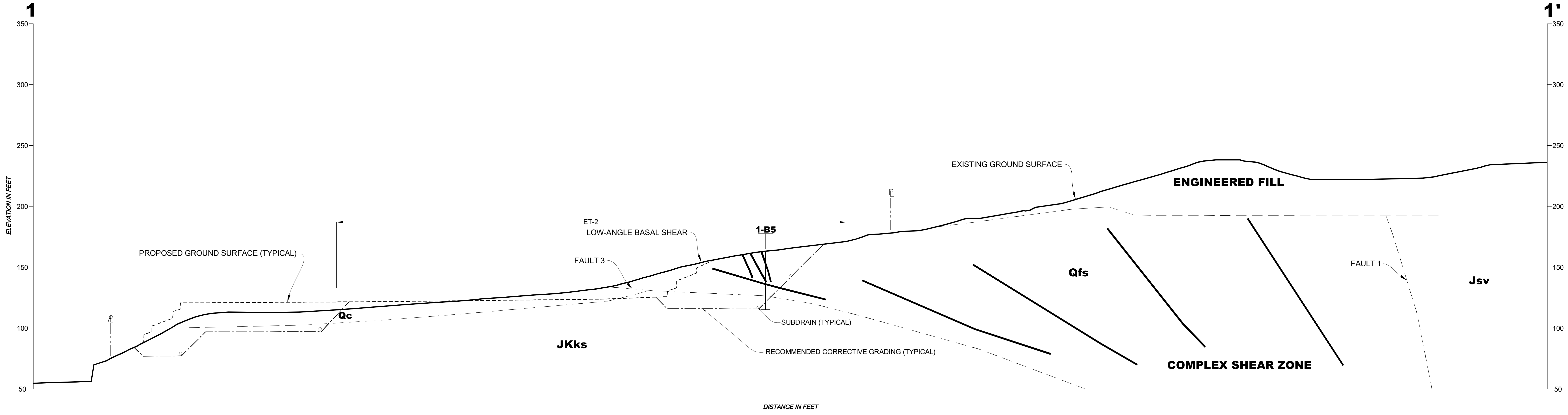
⊙———⊙ EARTHQUAKE FAULT ZONE BOUNDARIES; DELINEATED AS STRAIGHT-LINE SEGMENTS THAT CONNECT ENCIRCLED TURNING POINTS SO AS TO DEFINE EARTHQUAKE FAULT ZONE SEGMENTS



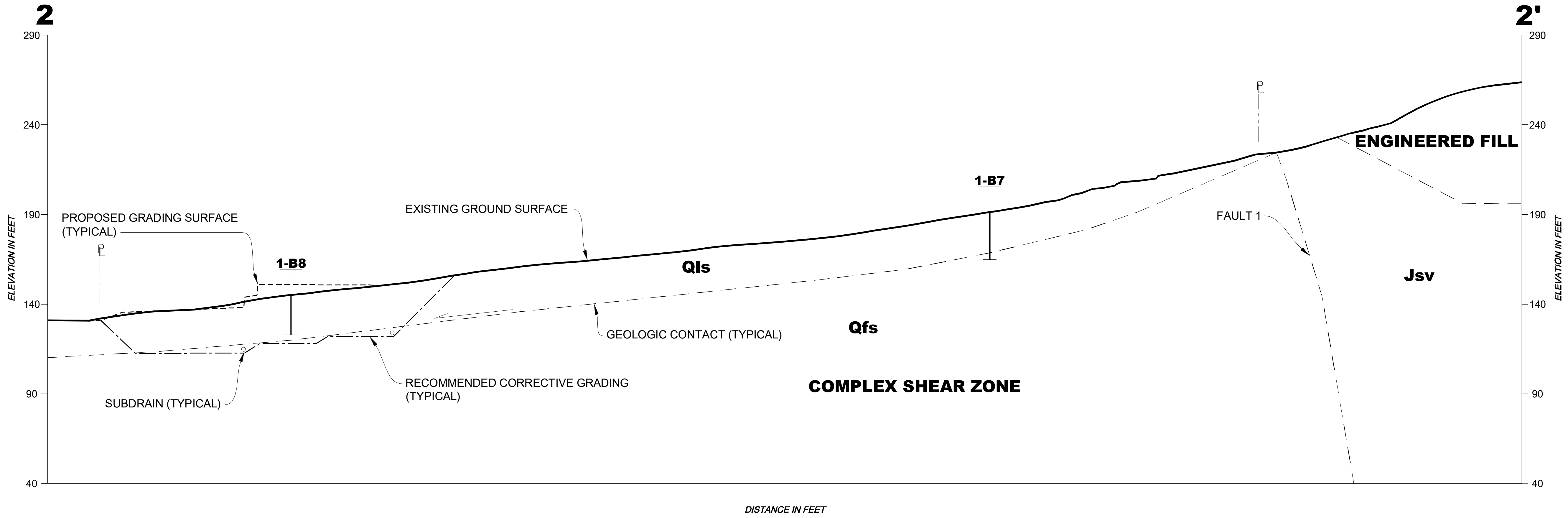
BASE MAP SOURCE: CDMG, 1993

ENGEO Expect Excellence	EARTHQUAKE FAULT ZONE MAP PARCEL GROUP 3 HAYWARD, CALIFORNIA	PROJECT NO.: 15577.000.001 SCALE: AS SHOWN DRAWN BY: JV CHECKED BY: RHB	FIGURE NO. 6

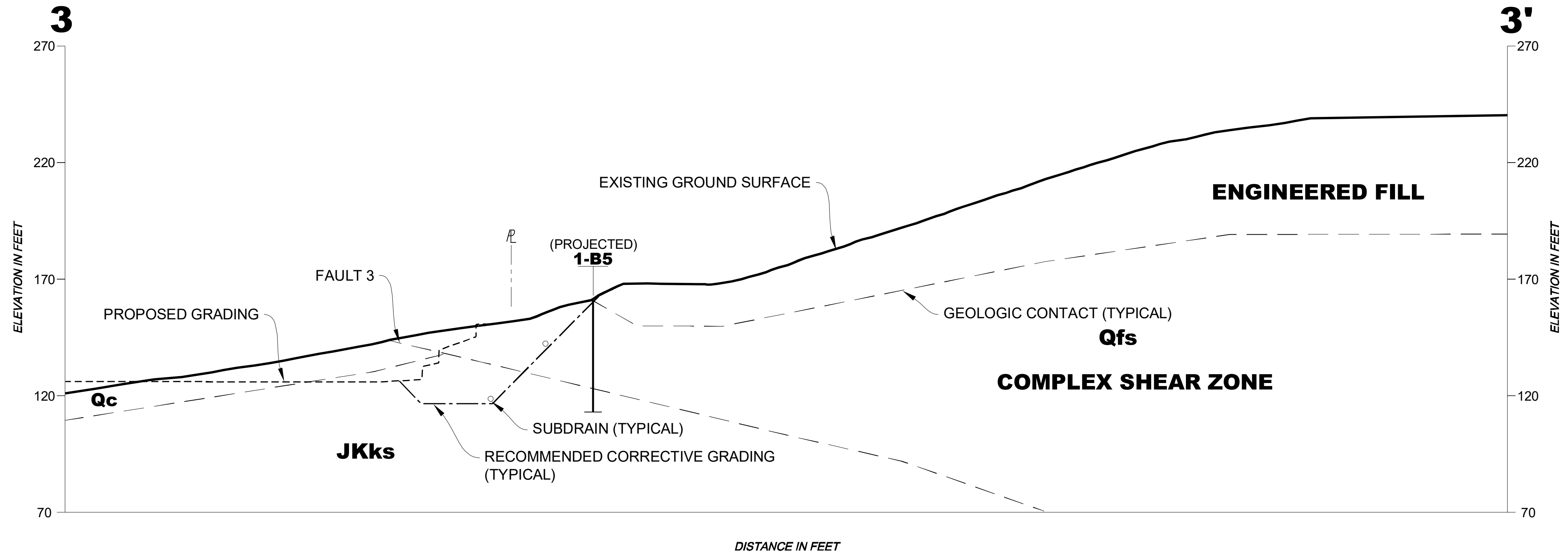
ORIGINAL FIGURE PRINTED IN COLOR



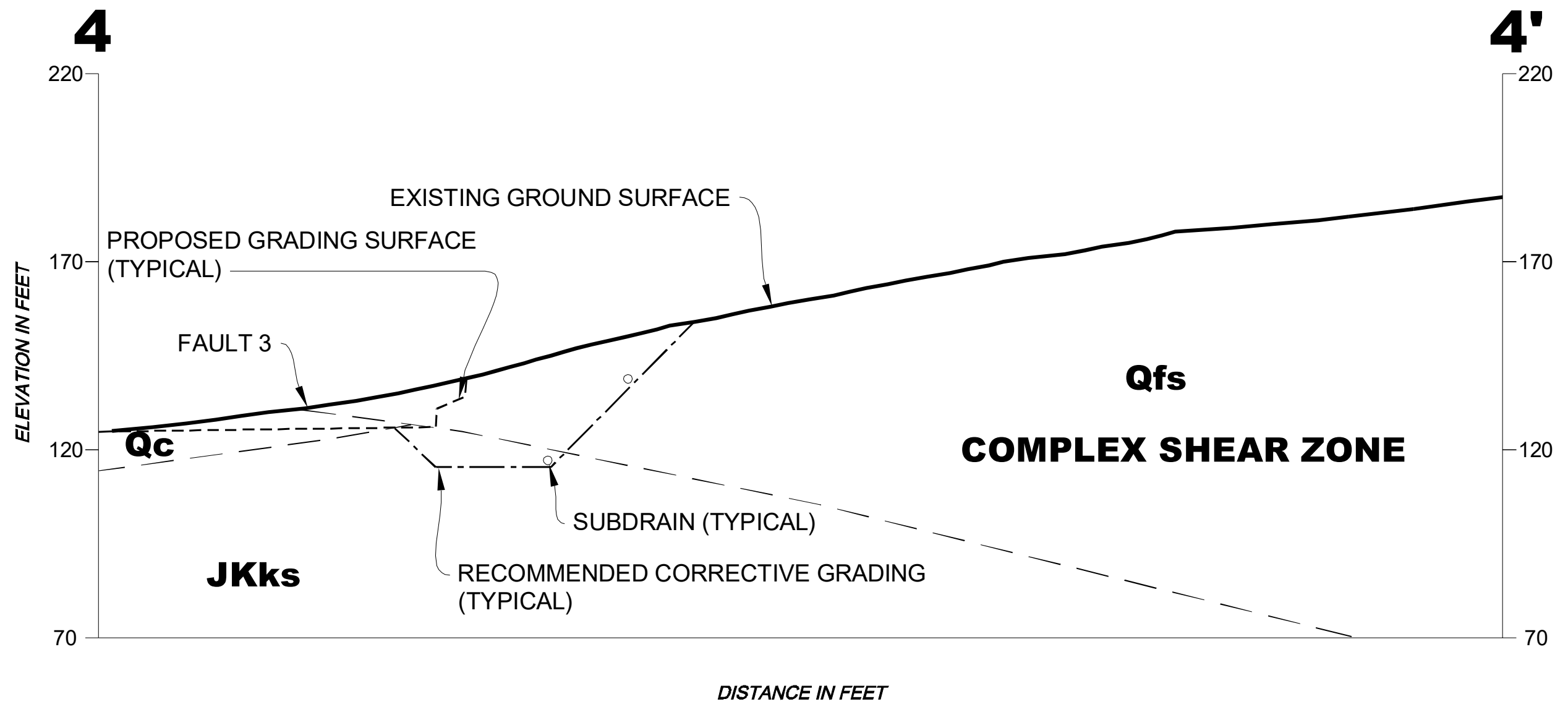
SECTION 1-1'
1"=50'



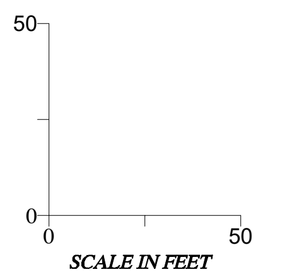
SECTION 2-2'
1"=50'



SECTION 3-3'
1"=50'



SECTION 4-4'
1"=50'



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qc	COLLUVIUM
Qls	LANDSLIDE DEPOSIT
Jsv	KERATOPHYRE
JKk	SHEARED KNOXVILLE FORMATION SHALE AND CONGLOMERATE
JKks	SHEARED DEPOSITS WITHIN WESTERN SHEAR ZONE OF HAYWARD FAULT, KNOXVILLE SHALE AND SANDSTONE, FAULTED SLIVERS OF ALLUVIUM, COLLUVIUM AND LANDSLIDE DEBRIS

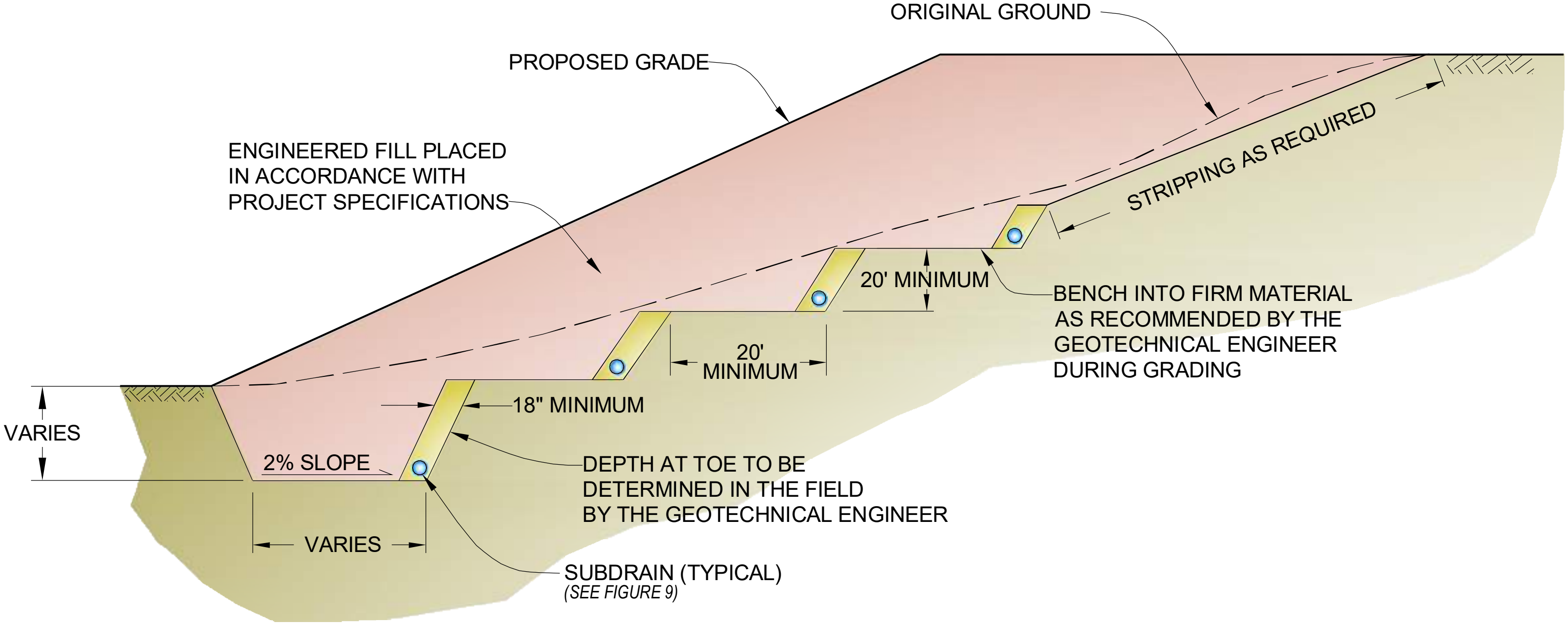
ENGEO
Expect Excellence

CROSS SECTIONS
PARCEL GROUP 3
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.001
SCALE: AS SHOWN
DRAWN BY: JSW
CHECKED BY: RHB
FIGURE NO.
7

ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\PROJECTS_14000 to 15999\15577\15577000000\Geo\GEX\15577000001-GEX-8-TypKeyway-0521.dwg Plot Date:6-01-21 jvergara



ENGEO
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TYPICAL KEYWAY AND BENCHING DETAIL

PARCEL GROUP 3

HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.001

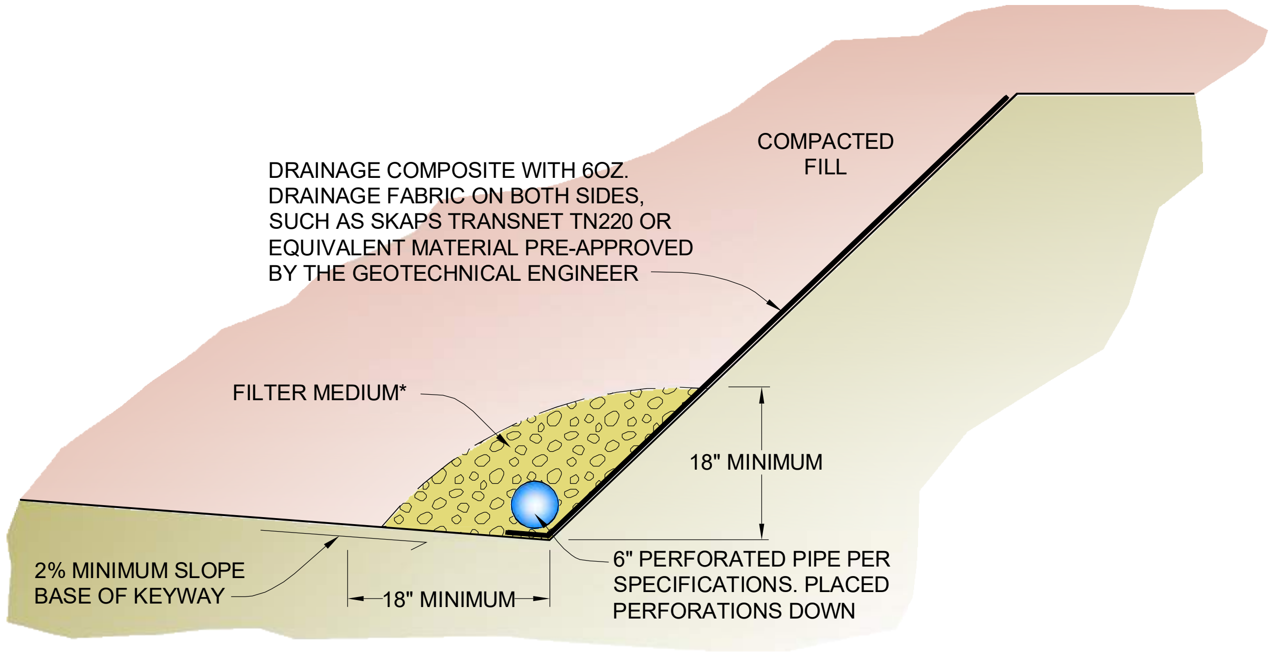
SCALE: NO SCALE

DRAWN BY: JSW

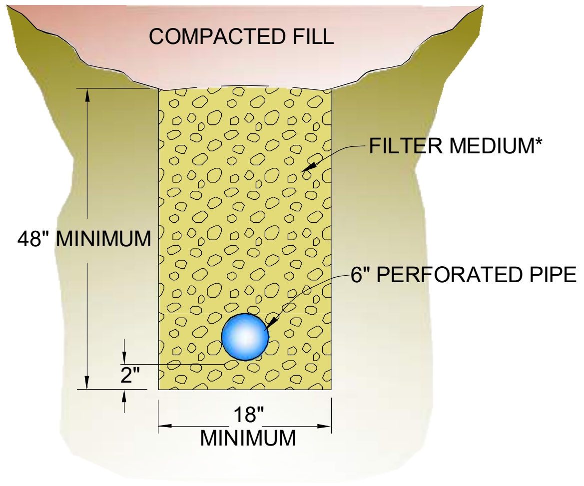
CHECKED BY: RHB

FIGURE NO.
8

ORIGINAL FIGURE PRINTED IN COLOR



KEYWAY SUBDRAIN - OPTION 1



SWALE SUBDRAIN

NOTES:

1. ALL PIPE JOINTS SHALL BE GLUED
2. ALL PERFORATED PIPE PLACED PERFORATIONS DOWN
3. 1% FALL (MINIMUM) ON ALL TRENCHES AND DRAIN LINES

*FILTER MEDIUM

ALTERNATIVE A

CLASS 2 PERMEABLE MATERIAL

MATERIAL SHALL CONSIST OF CLEAN, COARSE SAND AND GRAVEL OR CRUSHED STONE, CONFORMING TO THE FOLLOWING GRADING REQUIREMENTS:

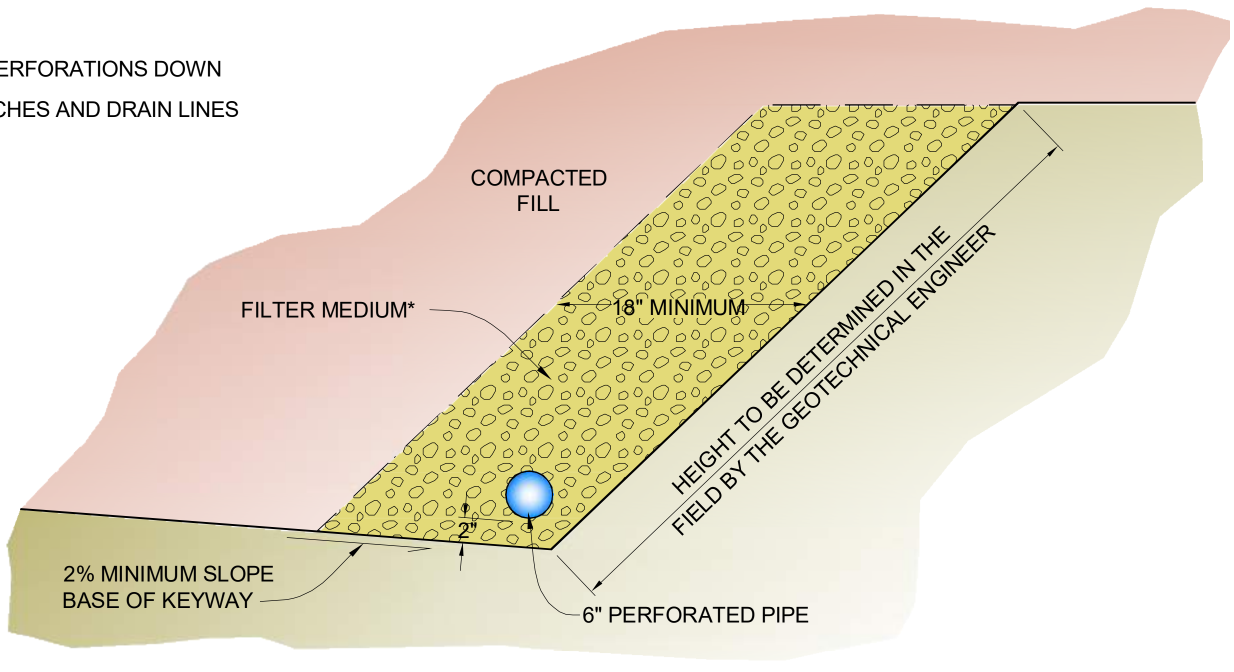
SIEVE SIZE	% PASSING SIEVE
1"	100
3/4"	90-100
3/8"	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

ALTERNATIVE B

CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

ALL FILTER FABRIC SHALL MEET THE FOLLOWING MINIMUM AVERAGE ROLL VALUES UNLESS OTHERWISE SPECIFIED BY ENGEO:

GRAB STRENGTH (ASTM D-4632) MASS	180 lbs
PER UNIT AREA (ASTM D-4751)	6 oz/yd^2
APPARENT OPENING SIZE (ASTM D-4751)	70-100 U.S. STD. SIEVE
FLOW RATE (ASTM D-4491) PUNCTURE	80 gal/min/ft
STRENGTH (ASTM D-4833)	80 lbs



KEYWAY SUBDRAIN - OPTION 2

ENGEO
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TYPICAL SUBDRAIN DETAILS
PARCEL GROUP 3
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.001
SCALE: NO SCALE
DRAWN BY: JSW
CHECKED BY: RHB
FIGURE NO. 9

ORIGINAL FIGURE PRINTED IN COLOR

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APPENDIX A

CURRENT EXPLORATION LOGS

ENGEO
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TEST PIT LOG

La Vista Residential Comm.
Hayward, CA
15577.000.001

Logged By: CMJ
Logged Date: 3/31/2021
Equipment: CAT 318

Test Pit Number	Depth (Feet)	Description
2-TP1	0 – 6	Clayey SAND with gravel (SP), yellow red, moist, dense, fine-to-coarse sand, fine-to-coarse gravel, iron-oxide, carbonates. (Qc) PP > 4.5
	6 – 10	Sandstone and Shale, pale olive and pale yellow, very weak, highly weathered, massive, blocky, iron-oxide, carbonates. (JKks)
		Bottom of test pit at 10 feet. Groundwater not encountered.
2-TP2	0 – 1	Lean CLAY (CL), dark brown, moist, very stiff, fine-to-medium sand. (Qc) PP: 2.5
	1 – 7	Sandy fat CLAY (CH), yellow red, moist, hard, fine-to-coarse sand, fine-to-coarse gravel, iron-oxide. (Qc) PP > 4.5
	7 – 13	Poorly graded sand with clay (SP-SC), yellow brown, moist, fine-to-coarse sand, fine-to-coarse gravel. (Qc)
	13 - 16	Clayey SAND (SC), yellow red, moist, fine-to-coarse sand, fine-to-coarse gravel, iron-oxide. (Qc)
		Bottom of test pit at 16 feet. Groundwater not encountered.

ENGEO
Expect Excellence

TEST PIT LOG

La Vista Residential Comm.
Hayward, CA
15577.000.001

Logged By: CMJ
Logged Date: 3/31/2021
Equipment: CAT 318

Test Pit Number	Depth (Feet)	Description
2-TP3	0 – ½	Lean CLAY (CL), dark brown, moist, hard, fine-to-medium sand, rootlets. (Fill) PP > 4.5
	½ - 1½	Lean CLAY (CL), yellow red, moist, hard, fine-to-medium sand, wood fragments. (Fill) PP > 4.5
	1½ – 3½	Fat CLAY (CH), black, moist, hard, rock fragments, cobbles-to-boulders. (Fill) PP > 4.5
	3½ - 10	Poorly graded SAND with clay (SP-SC), dark yellow red, moist, fine-to-coarse sand, fine-to-coarse gravel, iron-oxide. (Qc)
	10 – 14	Sandstone and Shale, pale olive and pale yellow, very weak, highly weathered, massive, blocky, iron-oxide, carbonates. (JKks)
		Bottom of test pit at 14 feet. Groundwater not encountered.

ENGEO
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TEST PIT LOG

Hayward Parcel 3
Hayward, CA
15577.000.001

Logged By: CMJ
Logged Date: 3/31/2021
Equipment: CAT 318

Test Pit Number	Depth (Feet)	Description
2-TP4	0 – ½	Lean CLAY (CL), dark brown, moist, hard, fine-to-medium sand, rootlets. (Qc) PP > 4.5
	½ - 2½	Fat CLAY (CH), black, moist, hard. (Qc) PP > 4.5
	2½ – 4	Poorly graded clayey SAND with gravel (SC), yellow red and pale yellow, moist, fine-to-coarse sand, fine-to-coarse gravel, cobbles, iron-oxide, and carbonates. (Qc)
	4 - 11	Conglomerate, yellow red, strong, moderately weathered, massive, fine-to-coarse sand, fine-to-coarse gravel, cobbles, rounded clasts. (Shear Zone?)
		Bottom of test pit at 11 feet. Groundwater not encountered.
2-TP5	0 – 1½	Sandy lean CLAY (CL), black, moist, hard, rootlets. (Qc) PP > 4.5
	1½ – 12	Lean CLAY with sand (CL), pale olive, moist, hard, fine-to-coarse sand, fine-to-coarse gravel, carbonate lenses, serpentine. (Shear Zone) PP > 4.5
	12 - 13	Lean CLAY with sand (CL), Yellow red mottle with pale olive, moist, fine-to-coarse sand, fine-to-coarse gravel, abundant carbonates. (Shear Zone)
		Bottom of test pit at 13 feet. Groundwater not encountered.

ENGEO
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TEST PIT LOG

Hayward Parcel 3
Hayward, CA
15577.000.001

Logged By: CMJ
Logged Date: 3/31/2021
Equipment: CAT 318

Test Pit Number	Depth (Feet)	Description
2-TP6	0 – 7	Fat CLAY (CH), black, moist, hard, rootlets. (Qc) PP > 4.5
	7 -13	Lean CLAY with sand (CL), dark yellow brown, moist, hard, fine-to-coarse sand, fine-to-coarse gravel. (Shear Zone) PP > 4.5
		Bottom of test pit at 13 feet. Groundwater not encountered.
2-TP7	0 – 3½	Fat CLAY (CH), black, moist, hard, rootlets. (Landslide Debris)
	3½ - 6	Lean CLAY (CL), light olive brown to dark yellowish brown, moist, very stiff, medium to coarse grained sand. (Landslide Debris)
	6 – 16	Poorly graded SAND with clay (SP-SC), light yellowish brown, moist, medium to coarse grained sand, serpentinite. (Landslide Debris?)
		Bottom of test pit at 16 feet. Groundwater not encountered.

ENGEO
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TEST PIT LOG

Hayward Parcel 3
Hayward, CA
15577.000.001

Logged By: CMJ
Logged Date: 3/31/2021
Equipment: CAT 318

Test Pit Number	Depth (Feet)	Description
2-TP8	0 – 1½	Fat CLAY (CH), black, moist, hard, rootlets. (Qc) PP > 4.5
	1½ - 5	Sandy fat CLAY (CH), very dark yellow brown, moist, hard, fine-to-coarse sand, fine-to-coarse gravel, iron-oxide. (Qc) PP > 4.5
	5 – 7½	Sandy lean CLAY (CL), dark yellow brown, moist, hard, fine-to-coarse sand, fine-to-coarse gravel, iron-oxide. (Qc) PP > 4.5
	7½ - 15	Sandstone and Shale, yellow red, weak, highly weathered, massive, iron-oxide. (Shear Zone?)
		Bottom of test pit at 15 feet. Groundwater not encountered.
2-TP9	0 – 4	Lean CLAY (CL), dark olive brown, moist, hard, rootlets, fine-to-medium sand. (Landslide Debris) PP > 4.5
	4 – 9	Fat CLAY (CH), black, moist, hard. (Landslide Debris) PP > 4.5
	9 - 16	Lean CLAY with sand (CL), red yellow mottle with pale olive, moist, stiff-to-very stiff, fine-to-medium sand, contains coarse sand, shear planes in samples. (Landslide Debris) PP: 2.0
		Bottom of test pit at 16 feet. Groundwater not encountered.

ENGEO
Expect Excellence

TEST PIT LOG

Hayward Parcel 3
Hayward, CA
15577.000.001

Logged By: CMJ
Logged Date: 3/31/2021
Equipment: CAT 318

Test Pit Number	Depth (Feet)	Description
2-TP10	0 – 11	Conglomerate, gray, strong, slightly weathered, massive, closely jointing, iron-oxide. (JKks)
		Bottom of test pit at 11 feet. Groundwater not encountered.
2-TP11	0 – 2	Fat CLAY CH), black, moist, very stiff. (Fill) PP: 3.5
	0 - 4	Fat CLAY (CH), black, moist, hard. (Qc) PP > 4.5
		Bottom of test pit at 4 feet. Groundwater not encountered.

ENGEO
Expect Excellence

TEST PIT LOG

Hayward Parcel 3
Hayward, CA
15577.000.001

Logged By: CMJ
Logged Date: 3/31/2021
Equipment: CAT 318

Test Pit Number	Depth (Feet)	Description
2-TP12	0 – 1½	Lean CLAY (CL), dark brown, moist, hard, contains fine-to coarse gravel. (Fill) PP > 4.5
	1½ - 5	Poorly graded SAND with clay (SP-SC), dark yellow red, moist, fine-to-coarse sand, fine-to-coarse gravel, iron-oxide. (Qc)
		Bottom of test pit at 5 feet. Groundwater not encountered.

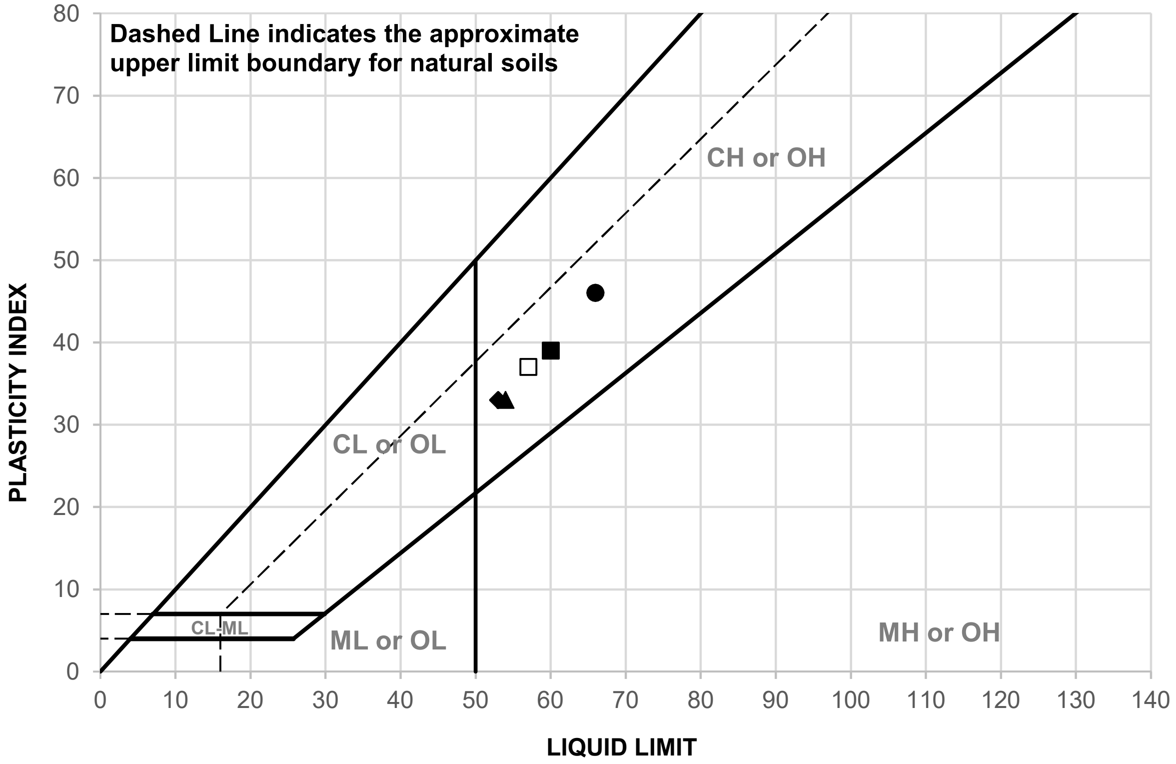
APPENDIX B

CURRENT LABORATORY TEST DATA

Liquid and Plastic Limits Test Report
Compaction Curve Report
Particle Size Distribution Report
Unconfined Compression Test
Isotropic Unconsolidated Undrained Triaxial Test Report
Isotropic Consolidated Undrained Triaxial Test Report

LIQUID AND PLASTIC LIMITS TEST REPORT

ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	2-TP1@3	3 feet	See exploration logs	54	21	33
◆	2-TP2@3	3 feet	See exploration logs	53	20	33
☐	2-TP3@2	2 feet	See exploration logs	57	20	37
●	2-TP7@2	2 feet	See exploration logs	66	20	46
■	2-TP8@3	3 feet	See exploration logs	60	21	39

	SAMPLE ID	TEST METHOD	REMARKS
▲	2-TP1@3	PI: ASTM D4318, Wet Method	
◆	2-TP2@3	PI: ASTM D4318, Wet Method	
☐	2-TP3@2	PI: ASTM D4318, Wet Method	
●	2-TP7@2	PI: ASTM D4318, Wet Method	
■	2-TP8@3	PI: ASTM D4318, Wet Method	

ENGEO
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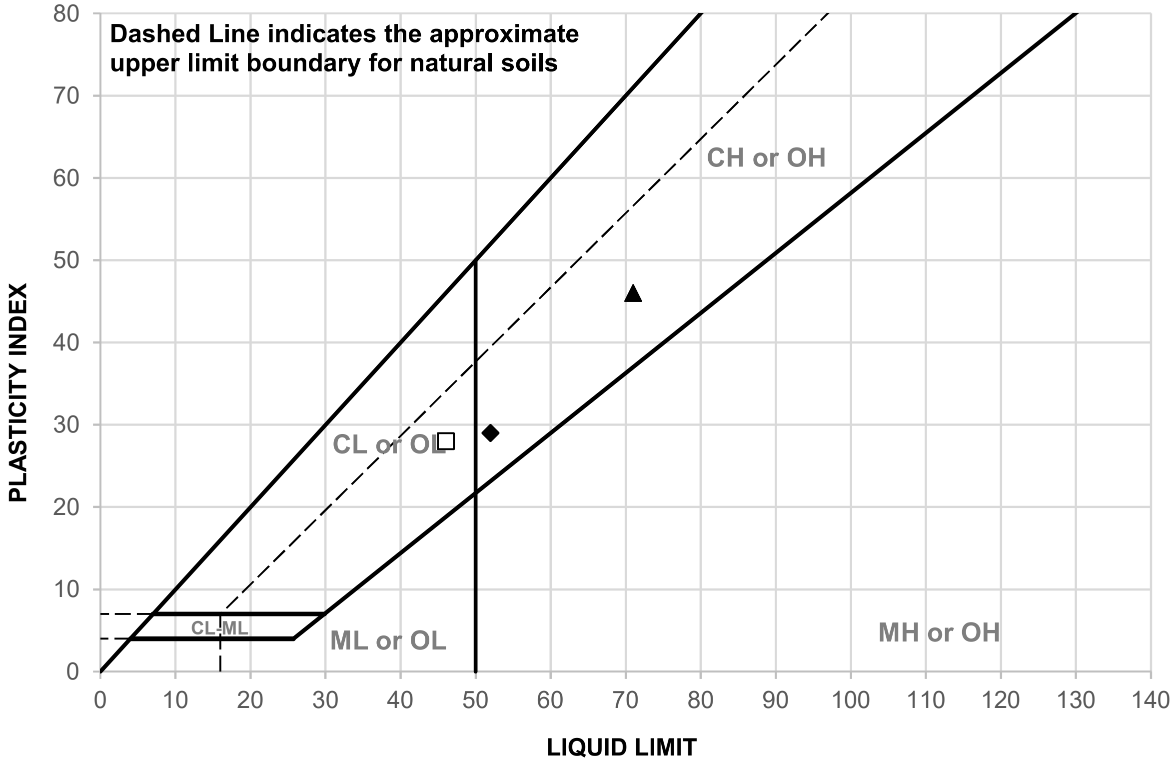
CLIENT: Eden Housing, Inc.
PROJECT NAME: Hayward Parcel 3 - La Vista
PROJECT NO: 15577.000.001
PROJECT LOCATION: Hayward, CA
REPORT DATE: 4/27/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

LIQUID AND PLASTIC LIMITS TEST REPORT

ASTM D4318



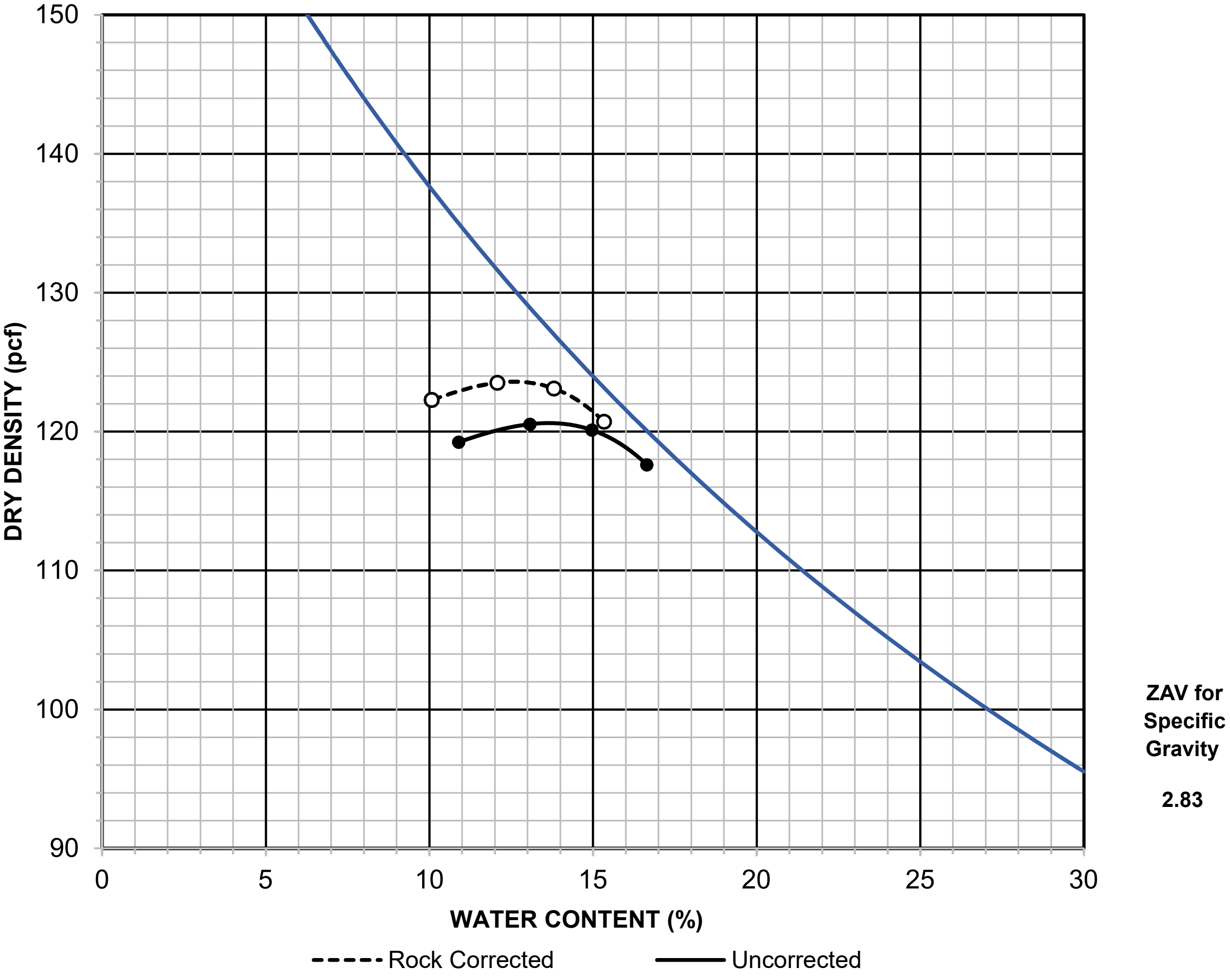
	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	2-TP9@5	5 feet	See exploration logs	71	25	46
◆	2-TP13@0.5	0.5 feet	See exploration logs	52	23	29
□	2-TP5@1	1.0 foot	See exploration logs	46	18	28

	SAMPLE ID	TEST METHOD	REMARKS
▲	2-TP9@5	PI: ASTM D4318, Wet Method	
◆	2-TP13@0.5	PI: ASTM D4318, Wet Method	
□	2-TP5@1	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Eden Housing, Inc.
PROJECT NAME: Hayward Parcel 3 - La Vista
PROJECT NO: 15577.000.001 PH001
PROJECT LOCATION: Hayward, CA
REPORT DATE: 4/27/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

COMPACTION CURVE REPORT
ASTM D1557



Curve Number: 1

Test Specification: Method A

Sample Location: 2-TP5@1

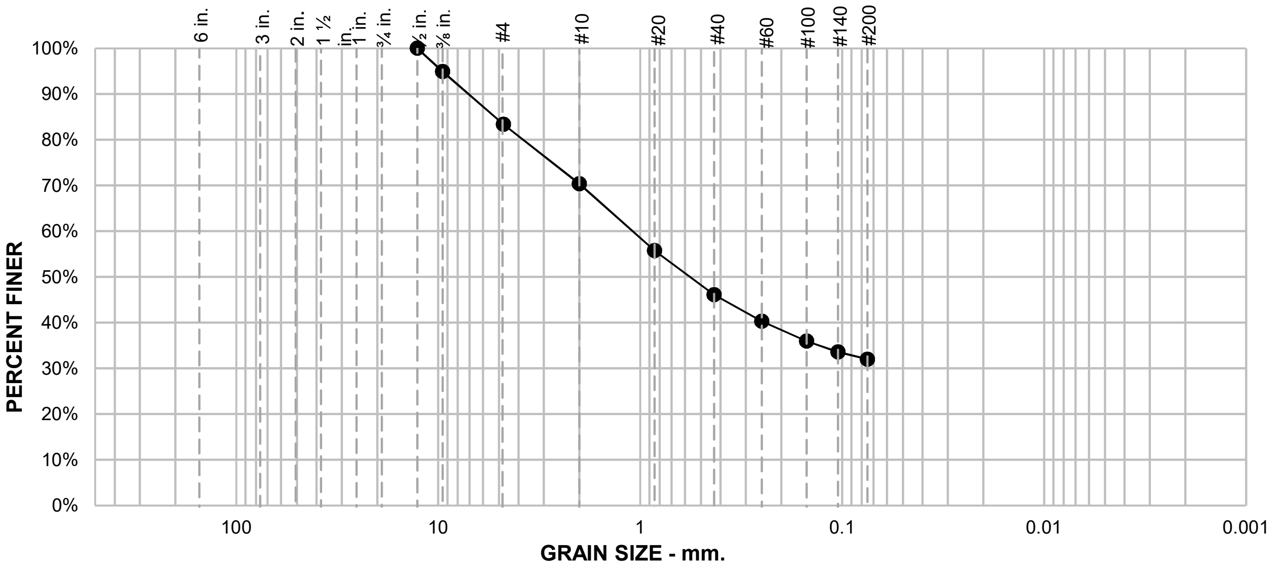
	ROCK CORRECTED	UNCORRECTED	MATERIAL DESCRIPTION - D2487
Maximum Dry Density, pcf	123.6	120.6	See exploration logs
Optimum Moisture Content, %	12.6	13.6	
Remarks			

ENGEO
Expect Excellence

CLIENT: Eden Housing, Inc.
PROJECT NAME: Hayward Parcel 3 - La Vista
PROJECT NO: 1557.000.001 PH001
PROJECT LOCATION: Hayward, California
REPORT DATE: 4/30/2021
TESTED BY: D. Bryant
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 2-TP1@3

DEPTH (ft): 3

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		16.7	12.9	24.3	14.1	32.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
½ in.	100.0		
⅜ in.	94.9		
#4	83.3		
#10	70.4		
#20	55.8		
#40	46.1		
#60	40.3		
#100	36.0		
#140	33.6		
#200	32.0		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL =	LL =	PI =

COEFFICIENTS

D_{90} = 7.0994 mm	D_{85} = 5.2599 mm	D_{60} = 1.0872 mm
D_{50} = 0.5655 mm	D_{30} =	D_{15} =
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS =

REMARKS

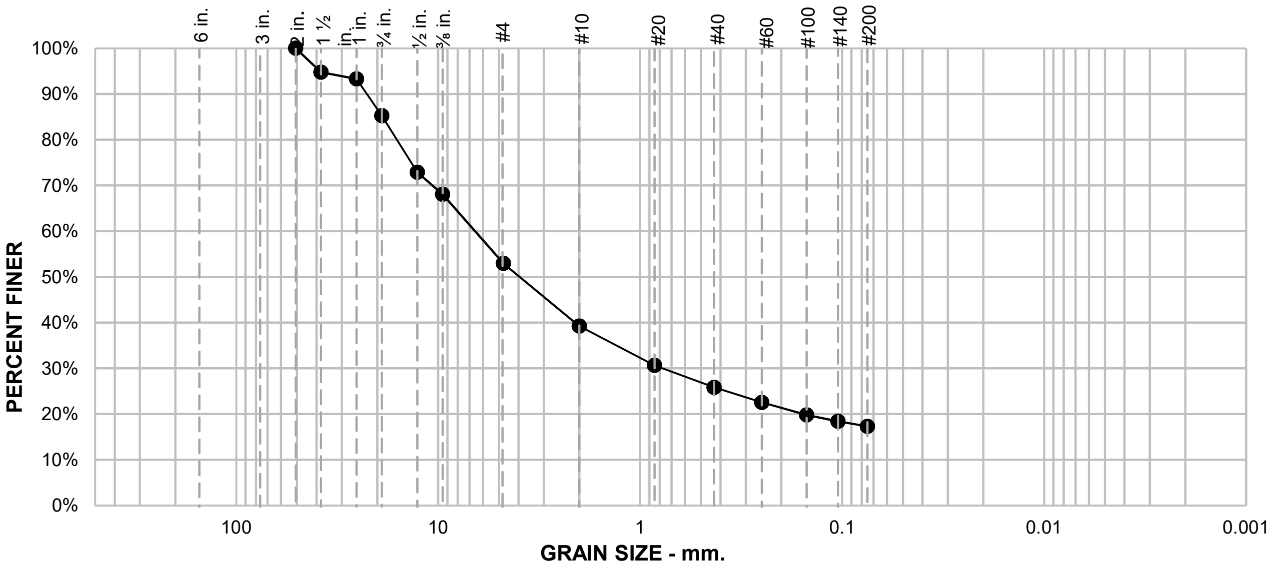
* (no specification provided)

ENGEO
Expect Excellence

CLIENT: Eden Housing, Inc.
PROJECT NAME: Hayward Parcel 3 - La Vista
PROJECT NO: 15577.000.001
PROJECT LOCATION: Hayward, CA
REPORT DATE: 4/27/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 2-TP4@3

DEPTH (ft): 3

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	14.7	32.3	13.7	13.5	8.5	17.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2 in.	100.0		
1-½ in.	94.8		
1 in.	93.3		
¾ in.	85.3		
½ in.	72.9		
⅜ in.	68.1		
#4	53.0		
#10	39.3		
#20	30.7		
#40	25.8		
#60	22.5		
#100	19.8		
#140	18.4		
#200	17.3		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} = 22.5578 mm D_{85} = 18.8640 mm D_{60} = 6.5580 mm

D_{50} = 3.9304 mm D_{30} = 0.7712 mm D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS =

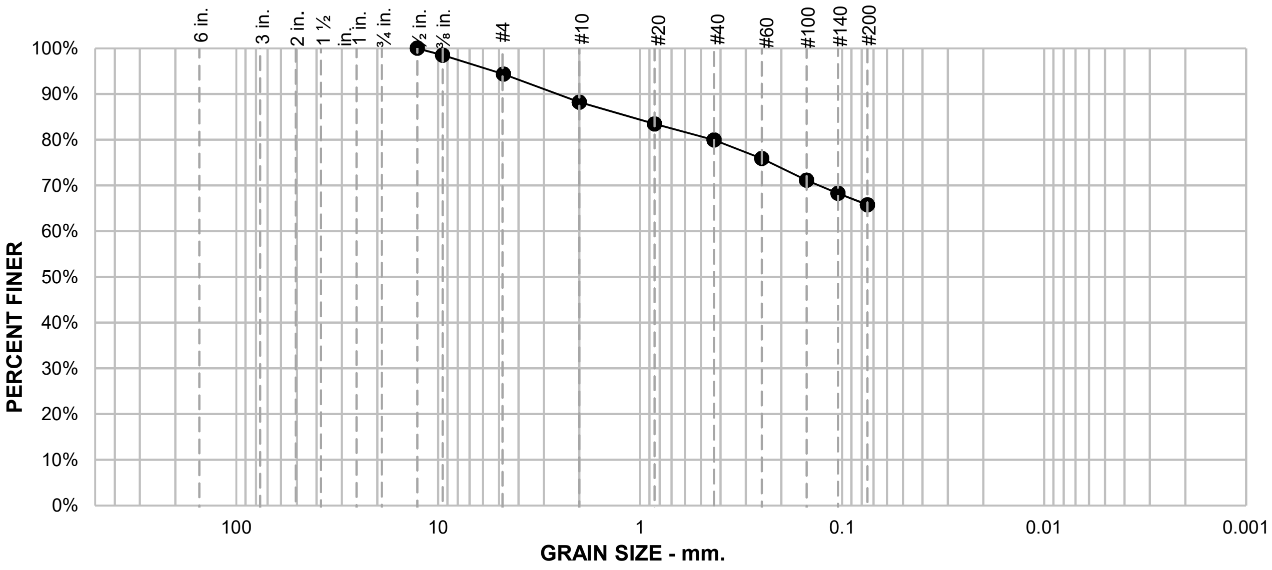
REMARKS

* (no specification provided)

CLIENT: Eden Housing, Inc.
PROJECT NAME: Hayward Parcel 3 - La Vista
PROJECT NO: 15577.000.001
PROJECT LOCATION: Hayward, CA
REPORT DATE: 4/27/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 2-TP5@12

DEPTH (ft): 12

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		5.6	6.2	8.3	14.1	65.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
½ in.	100.0		
⅜ in.	98.5		
#4	94.4		
#10	88.2		
#20	83.5		
#40	79.9		
#60	75.9		
#100	71.1		
#140	68.3		
#200	65.8		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL =
LL =
PI =

COEFFICIENTS

D_{90} = 2.5709 mm	D_{85} = 1.1169 mm	D_{60} =
D_{50} =	D_{30} =	D_{15} =
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS =

REMARKS

* (no specification provided)

ENGEO
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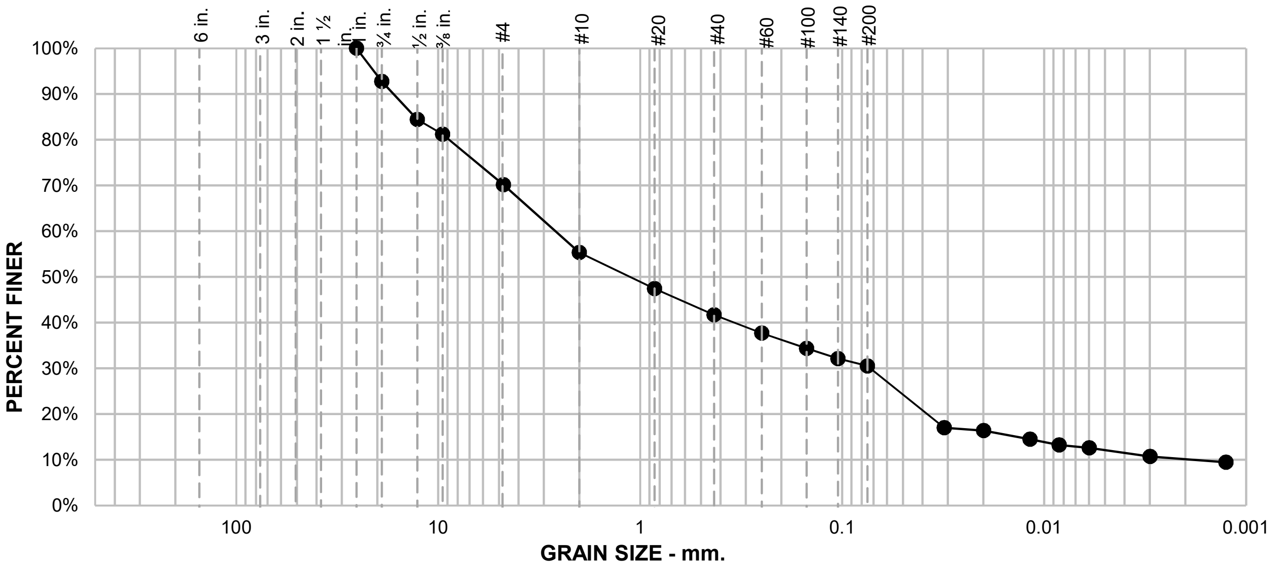
CLIENT: Eden Housing, Inc.
PROJECT NAME: Hayward Parcel 3 - La Vista
PROJECT NO: 15577.000.001
PROJECT LOCATION: Hayward, CA
REPORT DATE: 4/27/2021
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D422



SAMPLE ID: 2-TP2@3

DEPTH (ft): 3

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	7.2	22.6	14.9	13.6	11.2	10.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
¾ in.	92.8		
½ in.	84.4		
⅜ in.	81.2		
#4	70.2		
#10	55.3		
#20	47.4		
#40	41.7		
#60	37.7		
#100	34.3		
#140	32.1		
#200	30.5		
0.0313 mm.	17.0		
0.0199 mm.	16.4		
0.0117 mm.	14.5		
0.0084 mm.	13.2		
0.0060 mm.	12.6		
0.0030 mm.	10.7		
0.0013 mm.	9.5		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL =
LL =
PI =

COEFFICIENTS

D_{90} = 16.6417 mm
D_{85} = 13.0732 mm
D_{60} = 2.6274 mm
D_{50} = 1.1265 mm
D_{30} = 0.0726 mm
D_{15} = 0.0135 mm
D_{10} = 0.0018 mm
C_u = 1457.67
C_c = 1.11

CLASSIFICATION

USCS =

REMARKS

Silt/clay division of 0.002mm used

ENGEO
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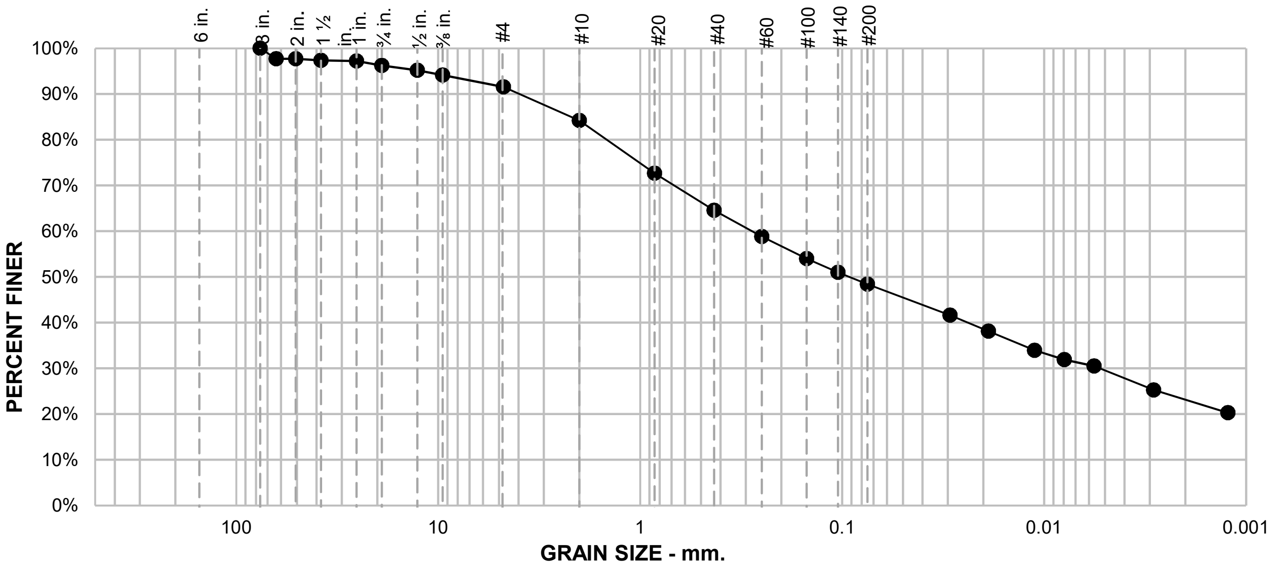
CLIENT: Eden Housing, Inc.
PROJECT NAME: Hayward Parcel 3 - La Vista
PROJECT NO: 15577.000.001 PH001
PROJECT LOCATION: Hayward, California
REPORT DATE: 5/3/2021
TESTED BY: M. Quasem
REVIEWED BY: G. Criste

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D422



SAMPLE ID: 2-TP5@1

DEPTH (ft): 1

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	3.7	4.8	7.3	19.6	16.2	25.3	23.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3 in.	100.0		
2.5 in.	97.7		
2 in.	97.7		
1-½ in.	97.4		
1 in.	97.2		
¾ in.	96.3		
½ in.	95.2		
⅜ in.	94.1		
#4	91.5		
#10	84.2		
#20	72.7		
#40	64.6		
#60	58.8		
#100	54.0		
#140	51.0		
#200	48.4		
0.0293 mm.	41.6		
0.0189 mm.	38.1		
0.0111 mm.	33.9		
0.0080 mm.	31.9		
0.0057 mm.	30.5		
0.0029 mm.	25.3		

* (no specification provided)

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 18 LL = 46 PI = 28

COEFFICIENTS

D_{90} = 3.9765 mm D_{85} = 2.1989 mm D_{60} = 0.2797 mm

D_{50} = 0.0923 mm D_{30} = 0.0053 mm D_{15} =

D_{10} = C_u = C_c =

CLASSIFICATION

USCS = SC

REMARKS

Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

ENGEO
Expect Excellence

CLIENT: Eden Housing, Inc.

PROJECT NAME: Hayward Parcel 3 - La Vista

PROJECT NO: 15577.000.001 PH001

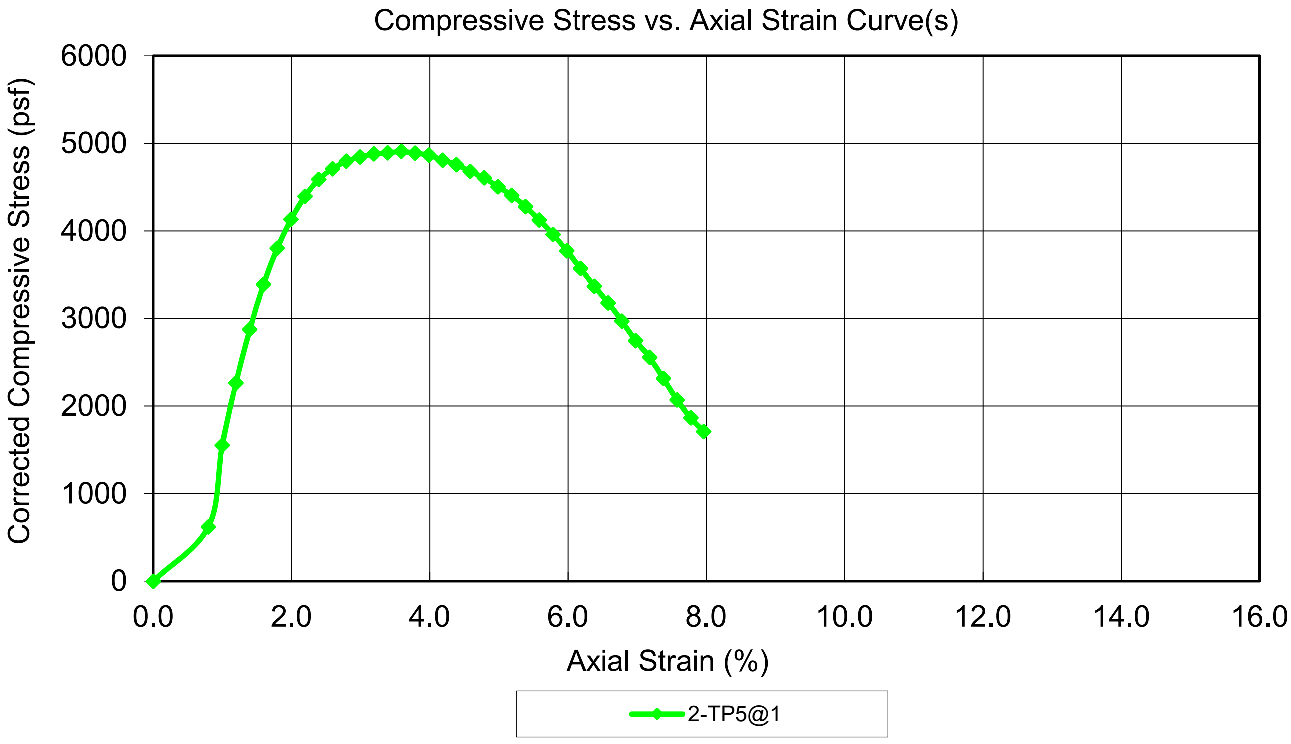
PROJECT LOCATION: Hayward, California

REPORT DATE: 5/3/2021

TESTED BY: G. Criste

REVIEWED BY: K. Lecce

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



	SPECIMEN
BEFORE TEST	**2-TP5@1**
Test Moisture Content (%)	16.94
Dry Density (pcf)	110.9
Saturation (%)	81.0
Void Ratio	0.59
Diameter (in)	2.412
Height (in)	5.016
Height-To-Diameter Ratio	2.08
TEST DATA	
Unconfined Compressive Strength (psf)	4908
Undrained Shear Strength (psf)	2454.14
Strain Rate (in/min)	0.050
Specific Gravity (MEASURED)	2.829
Strain at Failure(%)	3.59
Test Remarks	Remolding specs: 90% relative compaction of 123.6 pcf at 16.6%MC (+4% over opt)

SPECIMEN	DESCRIPTION
2-TP5@1	See exploration logs

PROJECT NAME: Hayward Parcel 3 - La Vista
PROJECT NO: 15577.000.001 PH001
CLIENT: Eden Housing, Inc.
LOCATION: Hayward, California

Test Date: 5/5/2021
Tested By: G. Criste
Reviewed By: K. Lecce

ENGEO
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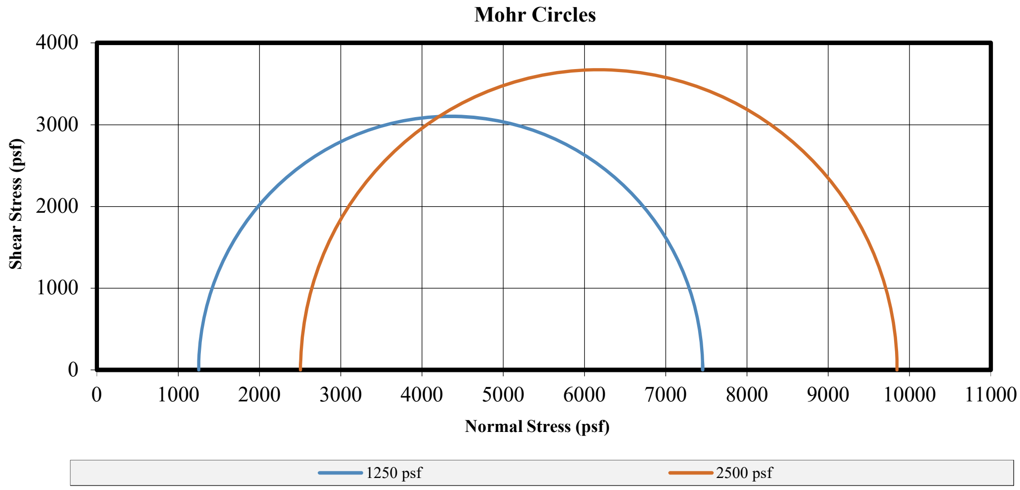
Isotropic Unconsolidated Undrained Triaxial Test
ASTM D2850

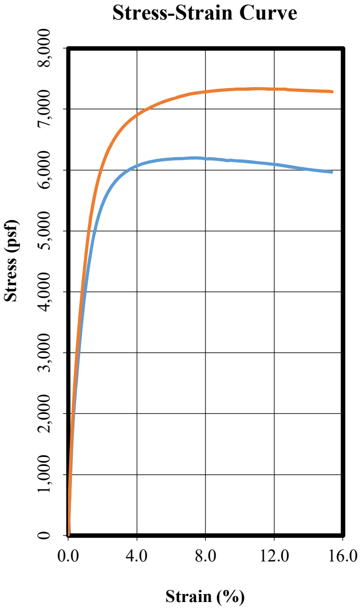
Date: 05/06/21

Checked By: K. Lecce

Date: 5/5/2021

Tested By: G. Criste





	Specimen			
Before Test	**1250 psf**	**2500 psf**		
Water Content (%)	16.94	16.94		
Dry Density (pcf)	111.70	111.40		
Saturation (%)	82.39	81.86		
Void Ratio	0.58	0.59		
Diameter (in)	2.412	2.412		
Height (in)	4.984	4.993		
Height-to-Diameter Ratio	2.066	2.070		
ASTM D4318 - Wet Method				
Liquid Limit	46	46		
Plastic Limit	18	18		
ASTM D854 - Assumed				
Specific Gravity	2.829	2.829		
After Test	**1250 psf**	**2500 psf**		
Water Content (%)	16.94	16.94		
Saturation (%)	82.39	81.86		
Strain Rate (%/min)	0.05	0.05		
Peak Deviator Stress (psf)	6201.5	7340.4		
Axial Strain @ Failure (%)	7.424	11.216		
	Cell Pressure			
Cell (psf)	1252.8	2505.6		
Back (psf)	n/a	n/a		
	Principle Stresses at Failure			
σ1 (psf)	7454.3	9846.0		
σ3 (psf)	1252.8	2505.6		
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)	
Cohesion, c (psf)	n/a
Friction Angle Ø	n/a

	Cohesion at Failure with a Zero Friction Angle (Ø=0)			
	3100.8	3670.2		
	n/a	n/a		

Project Information	
Project Name:	Hayward Parcel 3 - La Vista
Project Number:	15577.000.001 PH001
Project Location:	Hayward, California
Client:	Eden Housing, Inc.
Description:	See exploration logs
Test Remarks:	Remolding specs: 90% relative compaction of 123.6 pcf at 16.6%MC (+4% over opt)

ENGEO
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Isotropic Unconsolidated Undrained Triaxial Test
ASTM D2850

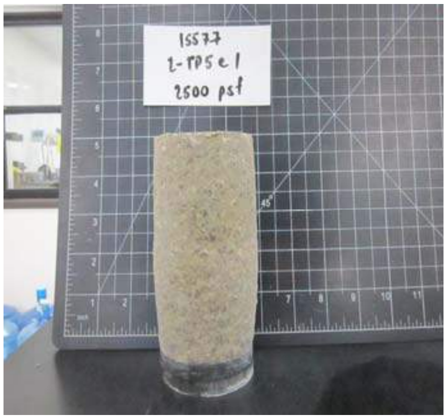
Tested By: G. Criste

Date: 5/5/2021

Checked By: K. Lecce

Date: 05/06/21

SPECIMEN PHOTOS

SAMPLE NUMBER: 1250 psf	SAMPLE NUMBER: 2500 psf
	

Project Information	
Project Name:	Hayward Parcel 3 - La Vista
Project Number:	15577.000.001 PH001
Project Location:	Hayward, California
Client:	Eden Housing, Inc.
Description:	See exploration logs
Test Remarks:	Remolding specs: 90% relative compaction of 123.6 pcf at 16.6%MC (+4% over opt)

Isotropic Consolidated Undrained Triaxial Test

ASTM D4767

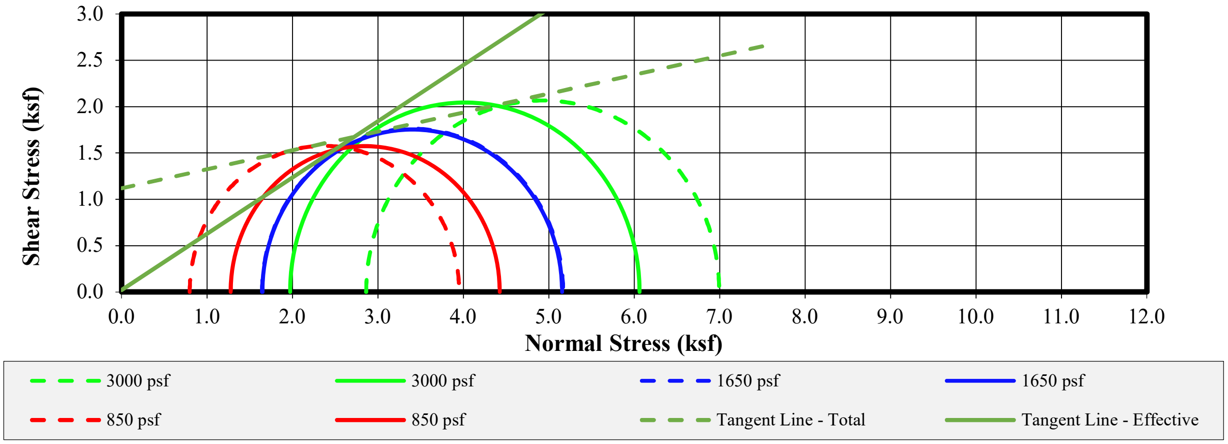
Date: 5/28/21

Checked By: D. Seibold

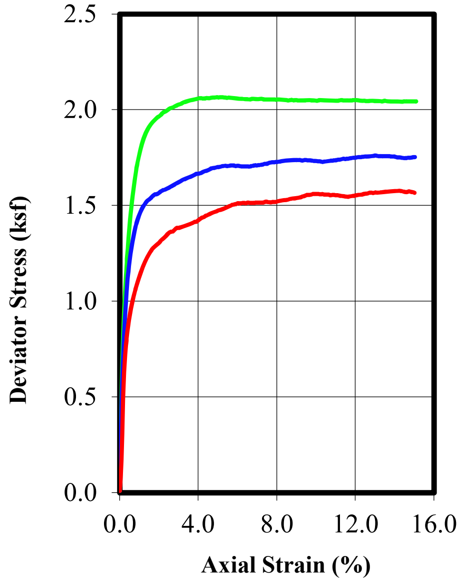
Date: 5/28/21

Tested By: G. Criste

Effective Stress at Maximum Deviator Stress Criterion



Deviator Stress Vs. Axial Strain



Initial	3000 psf	1650 psf	850 psf	
Height-to-Diameter Ratio	2.071	2.076	2.067	
Water Content (%)	16.9	16.9	16.9	
Dry Density (pcf)	111.2	111.0	111.4	
Saturation (%)	81.50	81.03	81.88	
Void Ratio	0.588	0.592	0.585	
Diameter (in)	2.413	2.413	2.414	
Height (in)	4.998	5.010	4.990	
ASTM D854 - Measured				
Specific Gravity	2.829	2.829	2.829	
ASTM D4318 - Wet Method				
Liquid Limit	46	46	46	
Plastic Limit	18	18	18	
After Consolidation	**3000 psf**	**1650 psf**	**850 psf**	
Void Ratio	0.646	0.655	0.680	
Water Content (%)	22.82	23.15	24.05	
Dry Density (pcf)	107.30	106.70	105.10	
Eff. Consol. Press. (ksf)	20.6	11.4	5.5	
Back Pressure (ksf)	43.0	43.0	43.2	
Saturation (%)	100.0	100.0	100.0	
B-Value	0.98	0.95	0.97	
Strain Rate (%/hr)	0.00083	0.00083	0.00083	
Height-to-Diameter Ratio	2.060	2.074	2.067	
After Shear	**3000 psf**	**1650 psf**	**850 psf**	
σ'1 at Failure (ksf)	6.0	5.1	4.4	
σ'3 at Failure (ksf)	1.9	1.6	1.3	
t_{50} (min)	0.694	0.694	0.694	
t_{90} (min)	2.249	2.249	2.249	
Corrected Peak Deviator Stress (ksf):	4.1	3.5	3.2	

Maximium Deviator Stress Criterion	
C (psf)	1.1
C' (psf)	0.0
Ø (deg)	11.5
Ø' (deg)	31.2

Project:	Hayward Parcel 3 - La Vista
Location:	Hayward, California
Project Number:	15577.000.001 PH001
Boring Number:	2-TP5
Sample Number:	2-TP5@1
Depth:	1.0 foot
Sample Type:	Remolded
Description:	See exploration logs
Test Type	Consolidated Undrained
Remarks	Consolidation data inconclusive. Default shear rate used. Remolding specs: 90% relative compaction of 123.6 pcf at 16.6% moisture (+4% over opt)

ENGEO

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Isotropic Consolidated Undrained Triaxial Test

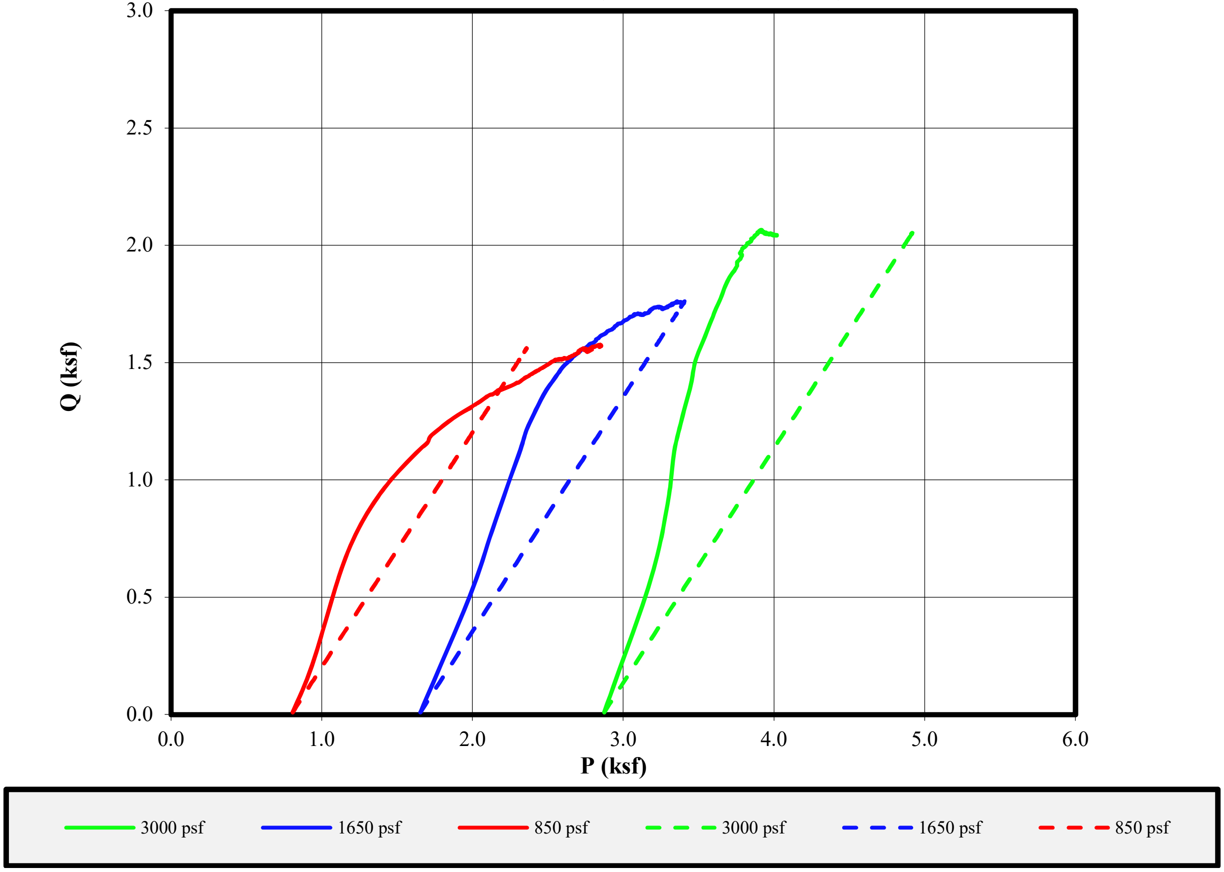
ASTM D4767

Date: 5/28/21

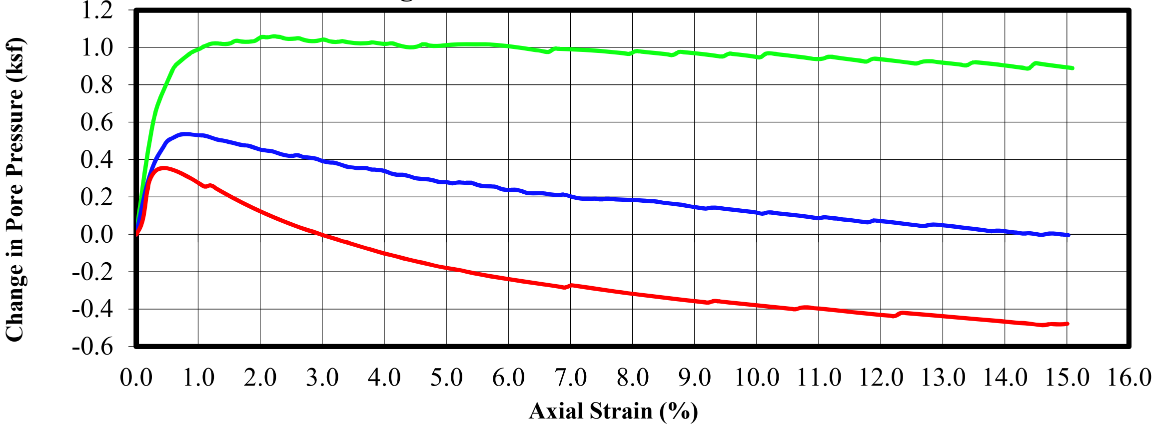
Checked By: D. Seibold

Date: 5/28/21

Tested By: G. Criste



Stress Paths (Total & Effective)



Change in Pore Pressure vs. Axial Strain

Project:	Hayward Parcel 3 - La Vista
Location:	Hayward, California
Project Number:	15577.000.001 PH001
Boring Number:	2-TP5
Sample Number:	2-TP5@1
Depth:	1.0 foot
Sample Type:	Remolded
Description:	See exploration logs
Test Type:	Consolidated Undrained
Remarks:	Consolidation data inconclusive. Default shear rate used. Remolding specs: 90% relative compaction of 123.6 pcf at 16.6% moisture (+4% over opt)

ENGEO

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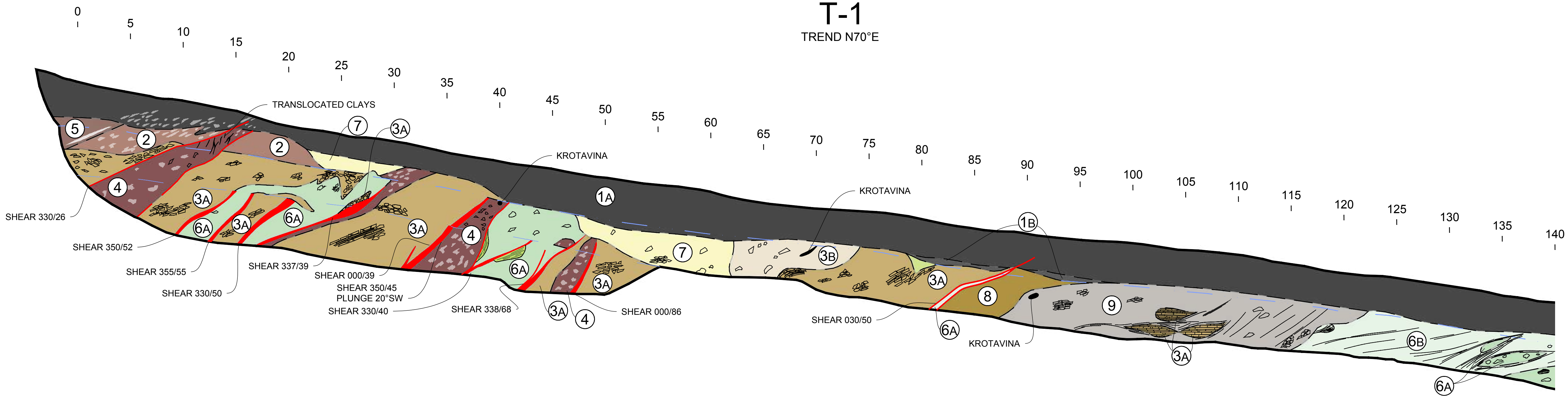
APPENDIX C

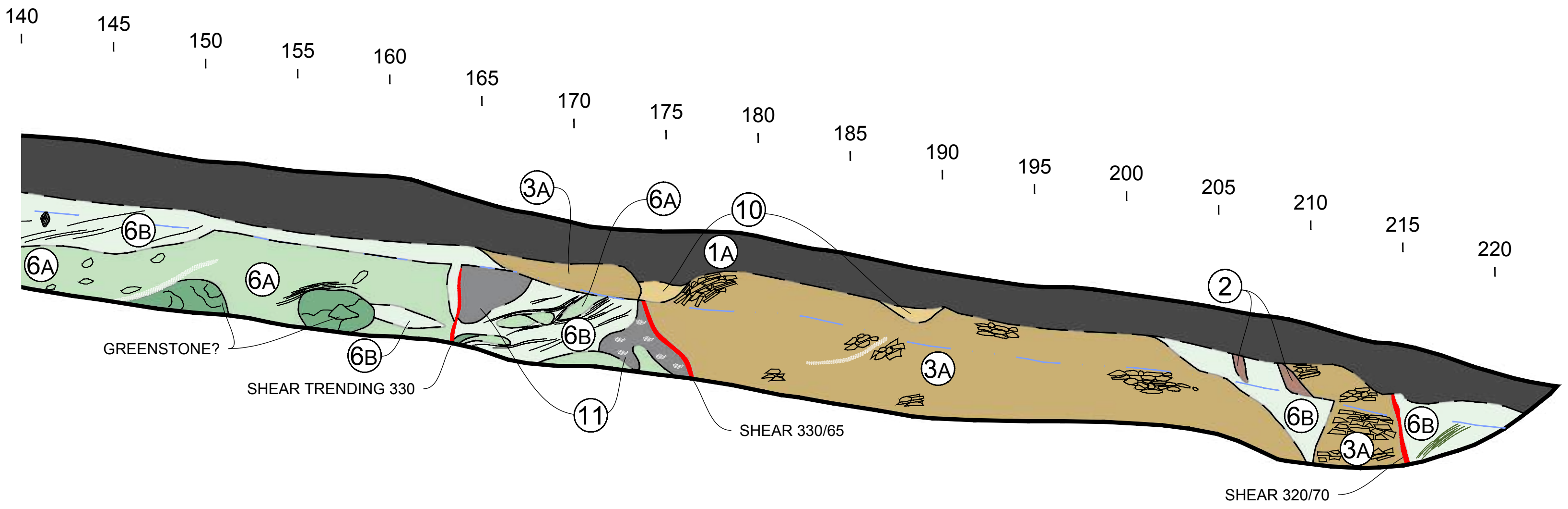
PREVIOUS EXPLORATION LOGS
AND LABORATORY TEST DATA

Trench Logs (ENGEO, 2020)
Boring Logs and Laboratory Tests (Krazan, 2020)
Boring Logs and Laboratory Tests (Krazan, 2020)

T-1

TREND N70°E





EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

— — GEOLOGIC CONTACT

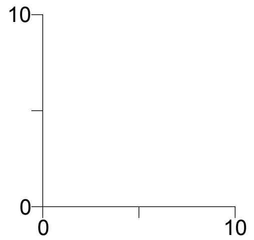
— SHEAR PLANE

⫽ BLOCKY SOIL STRUCTURE

— BENCH IN TRENCH WALL

▨ CARBONATES

⑩ SEE PAGE 7C



SCALE IN FEET

ENGEO
Expect Excellence

TRENCH LOGS
TRENCH T-1
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

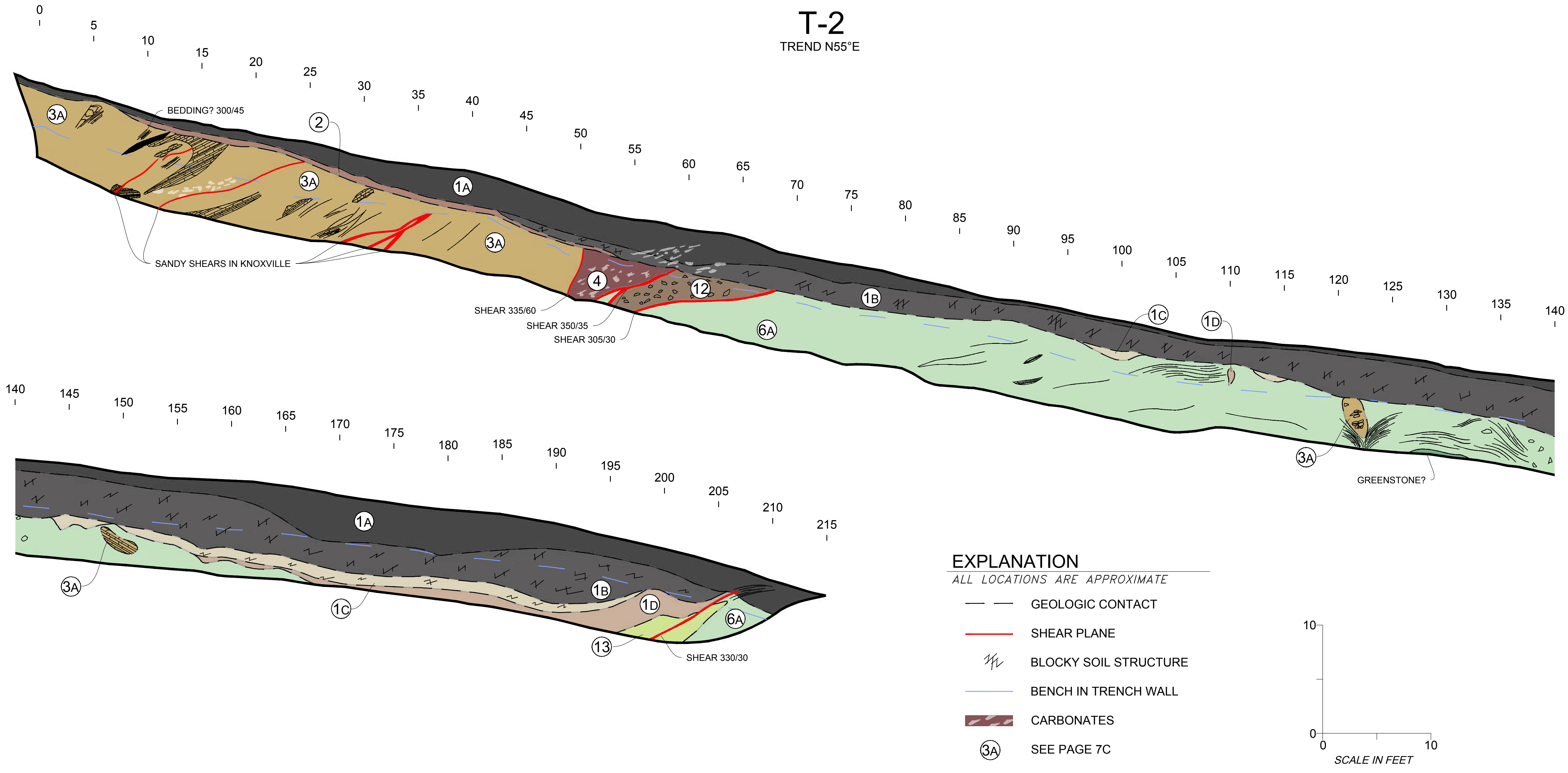
PROJECT NO.: 15577.000.000
SCALE: AS SHOWN
DRAWN BY: JV
CHECKED BY: RHB
FIGURE NO. 7A

ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\PROJECTS_14000 to 15999\15577\15577000000\Geo\FEX\15577000000-2_3_7A_C-FEX-0420.dwg SAVE DATE: 4/23/2020 1:24:37 PM SAVED BY: jvergara

T-2

TREND N55°E



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

— —	GEOLOGIC CONTACT
—	SHEAR PLANE
⫽	BLOCKY SOIL STRUCTURE
—	BENCH IN TRENCH WALL
▨	CARBONATES
③A	SEE PAGE 7C

ENGEO
Expect Excellence

TRENCH LOGS
TRENCH T-2
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.000	FIGURE NO.
SCALE: AS SHOWN	7B
DRAWN BY: JV / CHECKED BY: RHB	

ORIGINAL FIGURE PRINTED IN COLOR

(1A)	SANDY CLAY (CH), very dark grayish brown (10YR 3/2), hard, moist, fine-grain sand, trace medium gravel, carbonates, rootlets [A-HORIZON]
(1B)	Development of coarse, blocky pedologic (ped) structure, fine clay films
(1C)	LEAN CLAY (CL), black (2.5YR 2.5/1), hard, moist, trace roots, little to no ped structure
(1D)	SANDY CLAY (CL), dark yellowish brown (10YR 4/4), hard, moist, abundant rounded black coarse-grain sand or fine-grain gravel, carbonates
(2)	FAT CLAY (CH), strong brown (7.5YR 5/8) with very dark grayish brown, very stiff, moist, trace carbonates, trace lithic fracgments, translocated clays, thick clay films on ped surfaces
(3A)	SANDSTONE AND SHALE, dark yellowish brown (10YR 3/4), medium strong, massive, closely fractured, moderately to highly weathered [KNOXVILLE]
(3B)	Highly altered and bleached Unit 3A
(4)	LEAN CLAY (CL), reddish brown (5YR 5/4), very stiff to hard, moist, trace carbonates, lithic fragments (sandstone and shale), moderately developed angular peds with clay films [PLEISTOCENE ALLUVIUM]
(5)	FAT CLAY (CH), brown (7.5 YR 5/4), very stiff, moist, trace carbonates and gravel
(6A)	CLAYEY SAND (SC), light gray (2.5Y 7/2) to pale green, very dense, dry, serpentine rich, highly sheared, fragments of serpentinite and greenstone(?)
(6B)	Very pale green, highly sheared, no clasts
(7)	CLAYEY SAND (SC), yellow to pale yellow (2.5Y 8/6 to 8/4), moist, large clasts of sandstone and volcanics [GRABEN FILL]
(8)	FAT CLAY (CH), strong brown (7.5YR 4/6), moist, very stiff, fine gravel, serpentinite fragments, well rounded gravel
(9)	CLAYEY SAND (SC) with gravel, light gray with brown (10YR 6/6), very dense predominantly sandstone and shale derived with trace serpentinite fragments, serpentinite rich gouge (pale yellow to pale green sand, soft, dry, altered serpentinite fragments)
(10)	SANDY CLAY (CH), yellowish brown (10YR 5/4) with translocated clays, very stiff to hard, fine gravel
(11)	LEAN CLAY (CL), dark grayish brown (2.5Y 4/2), moist, hard, trace serpentinite inclusions, bounded by serpentinite-rich shear/gouge
(12)	SANDY GRAVEL (SG), brownish yellow (10YR 6/6), very dense, moist, Knoxville-derived, sandstone and shale clasts, angular gravel up to 4" in diameter
(13)	FAT CLAY (CH), light gray (2.5Y 7/1) to pale olive, hard, moist, massive, serpentinite gravel/sand, carbonates

ENGEO
Expect Excellence

TRENCH LOGS
UNIT DESCRIPTIONS
LA VISTA RESIDENTIAL DEVELOPMENT
HAYWARD, CALIFORNIA

PROJECT NO.: 15577.000.000
SCALE: NO SCALE
DRAWN BY: JV
CHECKED BY: RHB

FIGURE NO.
7C

ORIGINAL FIGURE PRINTED IN COLOR

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UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
	Clean Gravels (Less than 5% fines)	
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
	Clean Sands (Less than 5% fines)	
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)		
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of 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 50% or greater	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

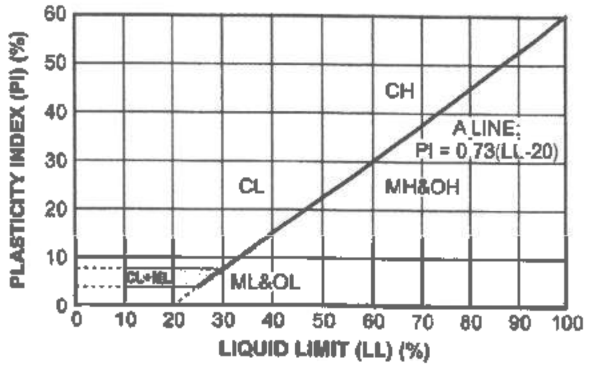
CONSISTENCY CLASSIFICATION

Description	Blows per Foot
Granular Soils	
Very Loose	< 5
Loose	5 – 15
Medium Dense	16 – 40
Dense	41 – 65
Very Dense	> 65
Cohesive Soils	
Very Soft	< 3
Soft	3 – 5
Firm	6 – 10
Stiff	11 – 20
Very Stiff	21 – 40
Hard	> 40

GRAIN SIZE CLASSIFICATION

Grain Type	*Standard Sieve Size*	*Grain Size in Millimeters*
Boulders	Above 12 inches	Above 305
Cobbles	12 to 13 inches	305 to 76.2
Gravel	3 inches to No. 4	76.2 to 4.76
Coarse-grained	3 to ¾ inches	76.2 to 19.1
Fine-grained	¾ inches to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
Coarse-grained	No. 4 to No. 10	4.76 to 2.00
Medium-grained	No. 10 to No. 40	2.00 to 0.042
Fine-grained	No. 40 to No. 200	0.042 to 0.074
Silt and Clay	Below No. 200	Below 0.074

PLASTICITY CHART



Log of Boring B1

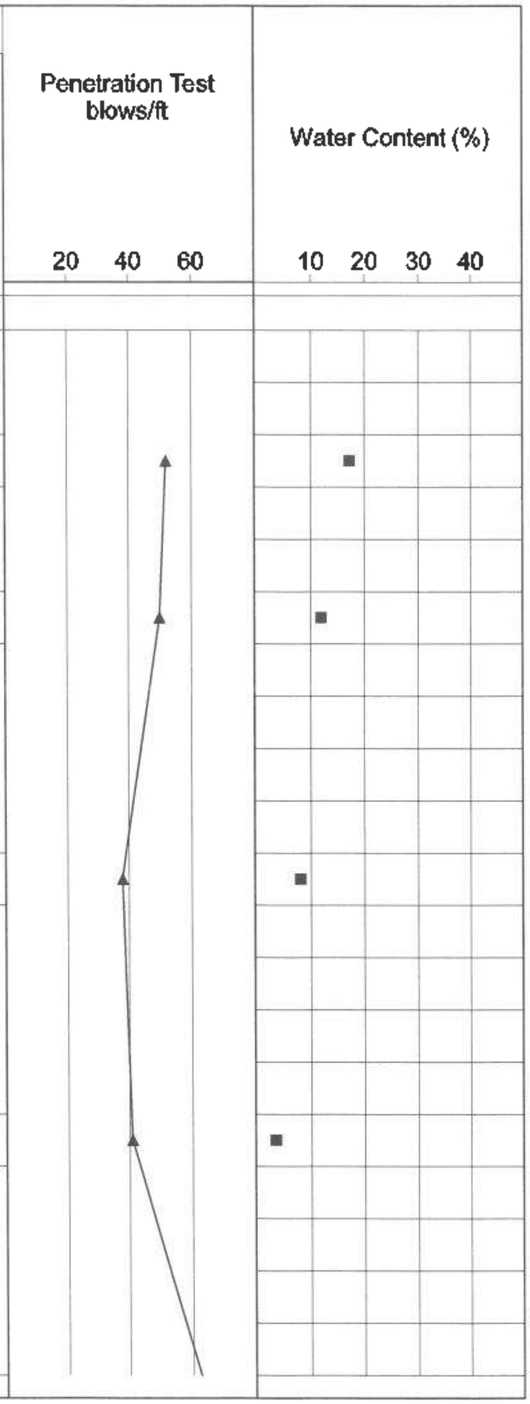
Project: La Vista Residential Community
Project No: 042-19042

Client: Eden Housing
Figure No.: A-1

Location: East 26th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water>
Initial: 38 Feet
At Completion: 36 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0		***CLAYEY SAND (SC)*** FILL, fine- to medium-grained; dark brown/black moist, drills easily				
2		***SANDY SILTY CLAY (CH)*** Hard, fine- to coarse-grained; dark brown/black, moist, drills firmly	103.3	17.3	■	52
4		***CLAYEY SAND (SC)*** Very dense, fine- to coarse-grained with trace GRAVEL; olive-brown, damp, drills hard	119.0	12.0	◢	50+
6						
8		***SILTY SAND (SM)*** Dense, fine- to coarse-grained with trace GRAVEL; olive-brown, damp, drills firmly				
10			122.0	8.1	◢	38
12						
14						
15			118.2	3.6	◢	41
16						
18						
20		Very dense, with increased GRAVEL and drills hard below 20 feet			◢	



Drill Method: Hollow Stem
Drill Date: 1-23-20

Drill Rig: CME 45C-1
Krazan and Associates
Hole Size: 6½ Inches

Driller: Chris Wyneken
Elevation: 50 Feet

Sheet: 1 of 3

Log of Boring B1

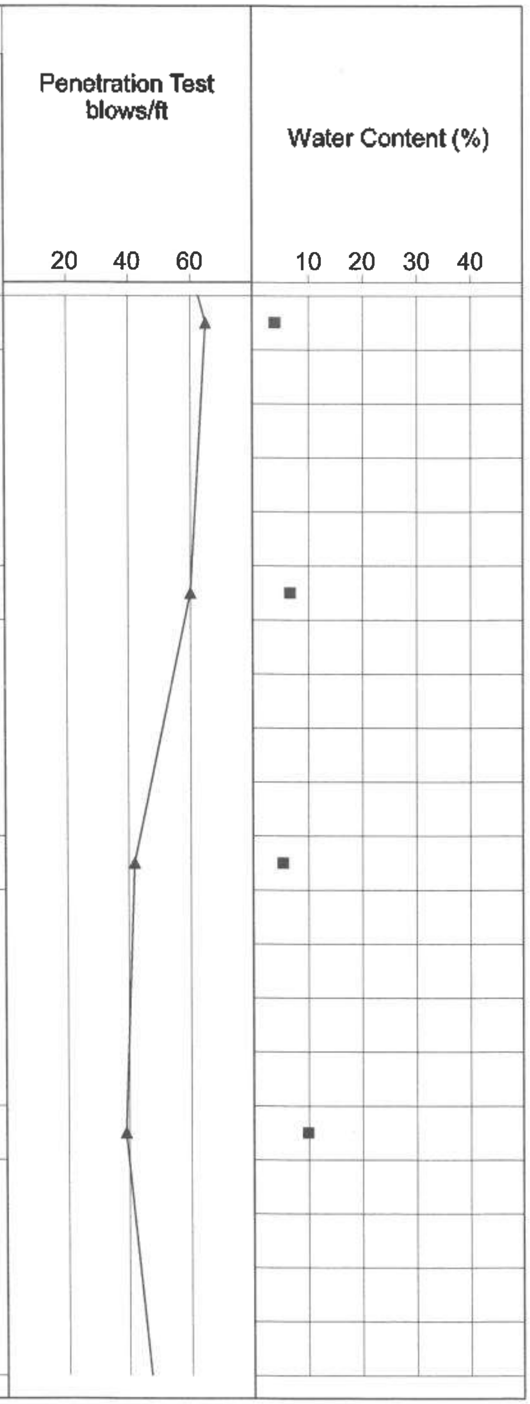
Project: La Vista Residential Community
Project No: 042-19042

Client: Eden Housing
Figure No.: A-1

Location: East 26th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water> **Initial:** 38 Feet **At Completion:** 36 Feet

SUBSURFACE PROFILE	SAMPLE			
Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
	119.5	4.0	◢	65
CLAYEY SAND (SC) Very dense, fine- to coarse-grained with trace GRAVEL; brown, damp, drills firmly				
	122.5	6.7	◢	60
SILTY SAND (SM) Dense, fine- to coarse-grained with trace GRAVEL; brown, damp, drills firmly				
	122.9	5.4	◢	42
CLAYEY SAND (SC) Dense, fine- to coarse-grained with trace GRAVEL; brown, moist, drills firmly				
	125.3	9.9	◢	39
▼ Saturated below 36 feet ▽				
Very dense below 40 feet			◢	



Drill Method: Hollow Stem
Drill Date: 1-23-20

Drill Rig: CME 45C-1
Hole Size: 6½ Inches

Krazan and Associates

Driller: Chris Wyneken
Elevation: 50 Feet

Sheet: 2 of 3

Log of Boring B1

Project: La Vista Residential Community

Project No: 042-19042

Client: Eden Housing

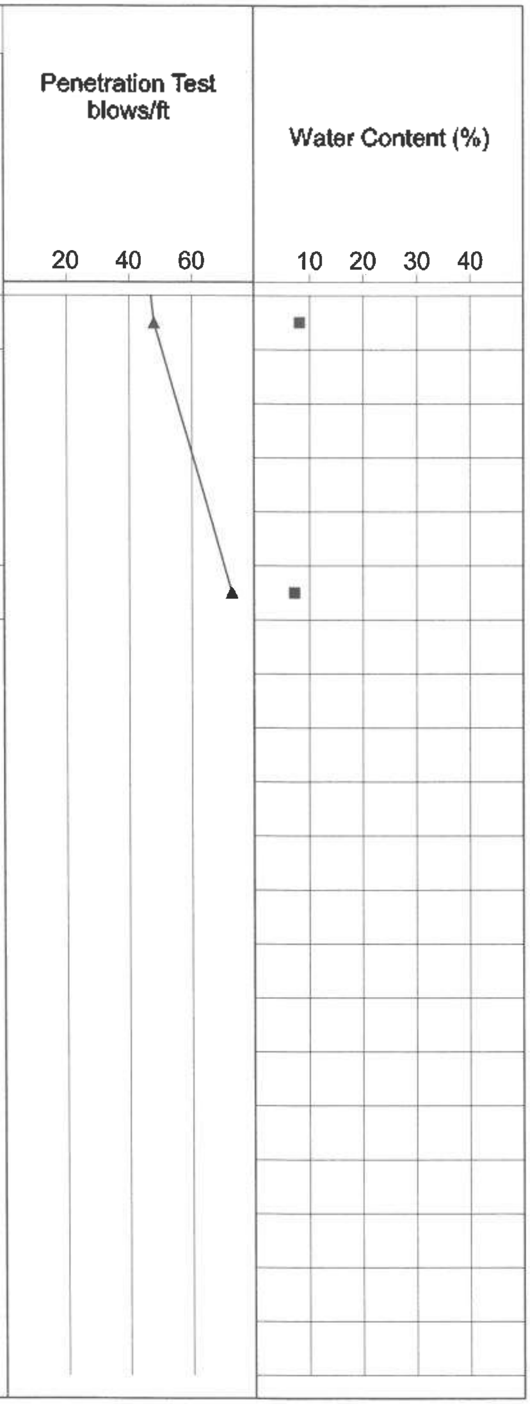
Figure No.: A-1

Location: East 26th Street and Tennyson Road, Hayward, California

Logged By: Wayne Andrade

Depth to Water> **Initial:** 38 Feet **At Completion:** 36 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
	▨		127.2	8.2	◢	48
42	▨					
44	▥	***SILTY SAND/SAND (SM/SP)*** Very dense, fine- to coarse-grained; brown, saturated, drills hard				
	▥		120.6	7.3	◢	73
46	▥					
48	▥					
50		End of Borehole				
52						
54						
56						
58						
60						



Drill Method: Hollow Stem

Drill Date: 1-23-20

Drill Rig: CME 45C-1

Krazan and Associates

Hole Size: 6½ Inches

Driller: Chris Wyneken

Elevation: 50 Feet

Sheet: 3 of 3

DRAFT

Log of Boring B2

Project: La Vista Residential Community

Project No: 042-19042

Client: Eden Housing

Figure No.: A-2

Location: East 26th Street and Tennyson Road, Hayward, California

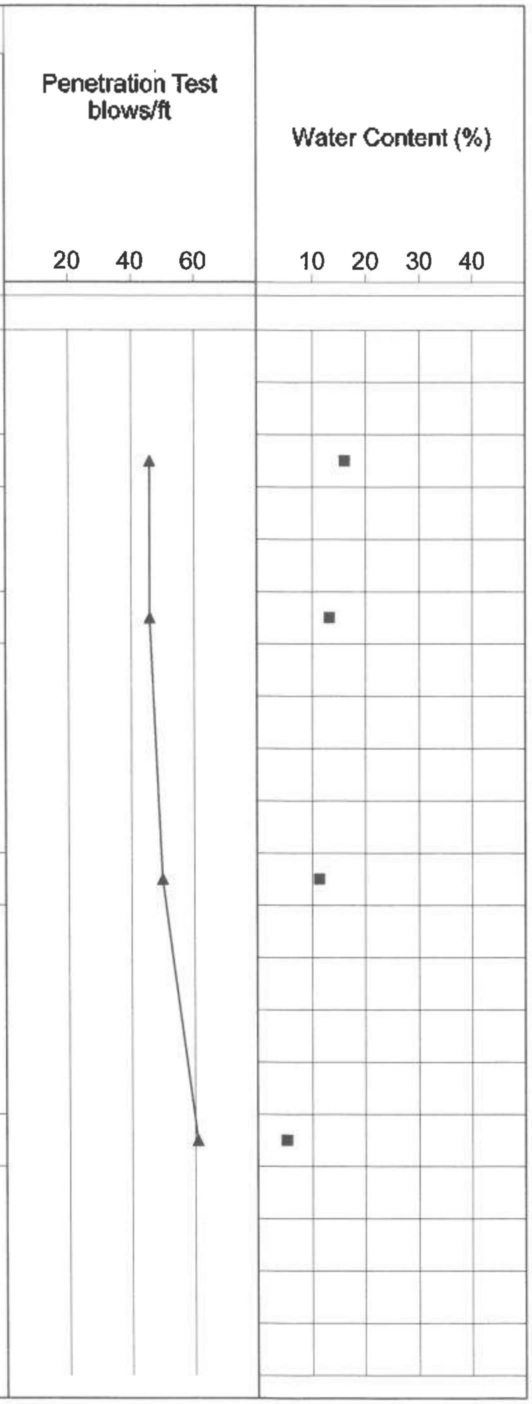
Logged By: Wayne Andrade

Depth to Water>

Initial: None

At Completion: None

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0	▨	***SILTY CLAY (CL/CH)*** Soft; dark brown moist, drills easily Stiff below 12 inches Hard and drills firmly below 2 feet				
2	▨		108.7	16.1	■	46
4	▨	With trace fine- to coarse-grained SAND below 4 feet				
6	▨		114.2	13.3	■	46
8	▨	***GRAVELLY CLAYEY SAND (SC)*** Very dense, fine- to coarse-grained; brown, damp, drills hard				
10	▨		127.5	11.4	■	50+
12	▨					
14	▨	Dense and drills firmly below 15 feet				
16	▨		130.0	5.4	■	61
18	▨					
20						



Drill Method: Solid Flight

Drill Date: 1-23-20

Drill Rig: CME 45C-1

Krazan and Associates

Hole Size: 4½ Inches

Driller: Chris Wyneken

Elevation: 20 Feet

Sheet: 1 of 1

Log of Boring B3

Project: La Vista Residential Community

Project No: 042-19042

Client: Eden Housing

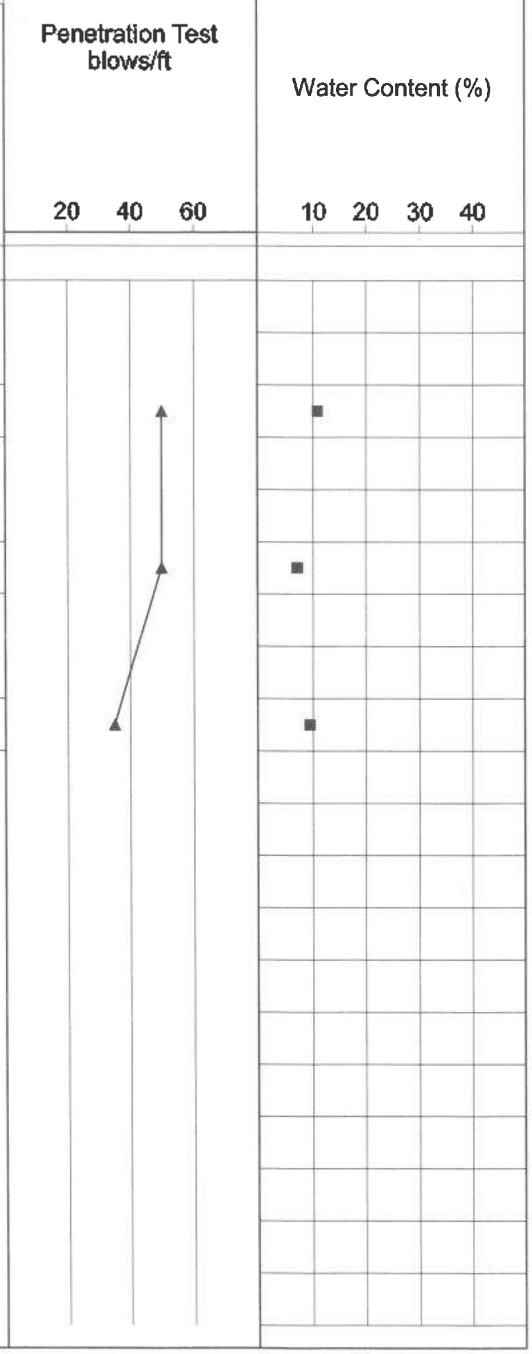
Figure No.: A-3

Location: East 26th Street and Tennyson Road, Hayward, California

Logged By: Wayne Andrade

Depth to Water> **Initial:** None **At Completion:** None

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0	▨	***SILTY CLAY (CL/CH)*** Soft; dark brown/black moist, drills easily Stiff below 12 inches				
2	▨	***GRAVELLY CLAYEY SAND (SC)*** Very dense, fine- to coarse-grained; brown, damp, drills hard	105.6	10.9	■	50+
4	▨					
	▨			7.2	■	50+
6	▨					
8	▨	Medium dense below 8 feet	110.6	9.5	■	35
10	▨					
12	▨					
14	▨					
		End of Borehole				
16						
18						
20						



Drill Method: Solid Flight

Drill Date: 1-23-20

Drill Rig: CME 45C-1

Krazan and Associates

Hole Size: 4½ Inches

Driller: Chris Wyneken

Elevation: 15 Feet

Sheet: 1 of 1

Log of Boring B4

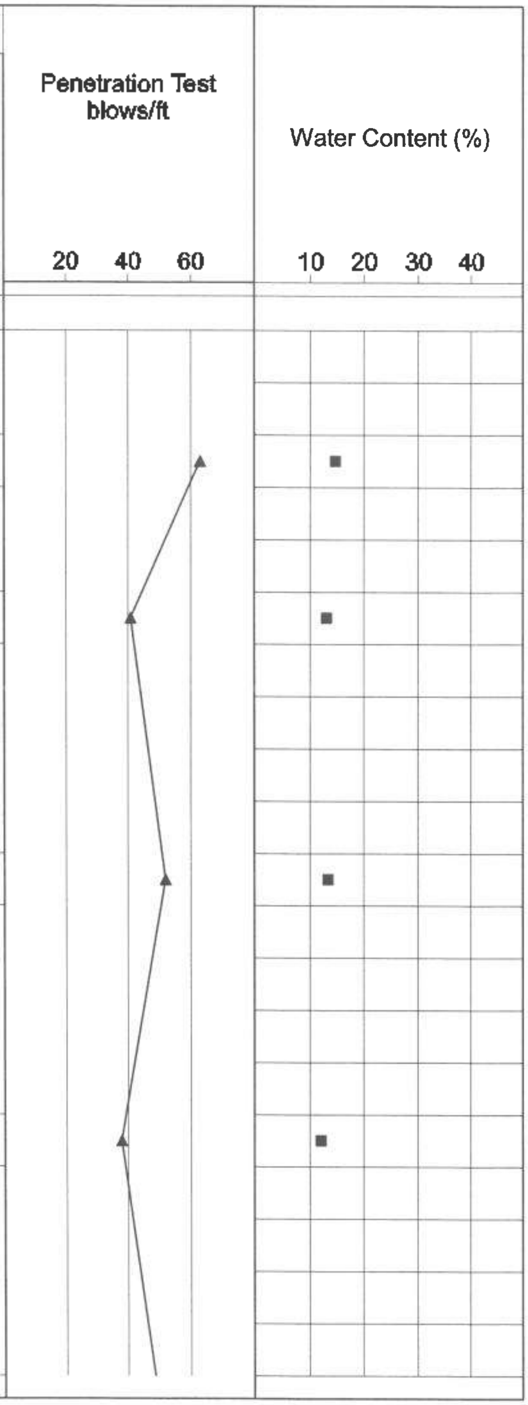
Project: La Vista Residential Community
Project No: 042-19042

Client: Eden Housing
Figure No.: A-4

Location: East 26th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water> **Initial:** None **At Completion:** None

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0	▨	***SILTY CLAY (CL/CH)*** Soft; dark brown/black moist, drills easily Stiff below 12 inches Hard, with trace fine- to coarse-grained SAND and drills firmly below 2 feet				
2	▨		110.1	14.8	■	63
4	▨		109.3	13.1	■	41
6	▨					
8	▨	***CLAYEY SAND (SC)*** Dense, fine- to coarse-grained with trace GRAVEL; brown, moist, drills firmly				
10	▨		111.5	13.4	■	52
12	▨					
14	▨	Medium dense below 15 feet	123.8	12.1	■	38
16	▨					
18	▨	***GRAVELLY CLAYEY SAND (SC)*** Very dense, fine- to coarse-grained; light brown, damp, drills hard				
20	▨				■	



Drill Method: Solid Flight
Drill Date: 1-23-20

Drill Rig: CME 45C-1
Hole Size: 4½ Inches

Krazan and Associates

Driller: Chris Wyneken
Elevation: 25 Feet

Sheet: 1 of 2

DRAFT

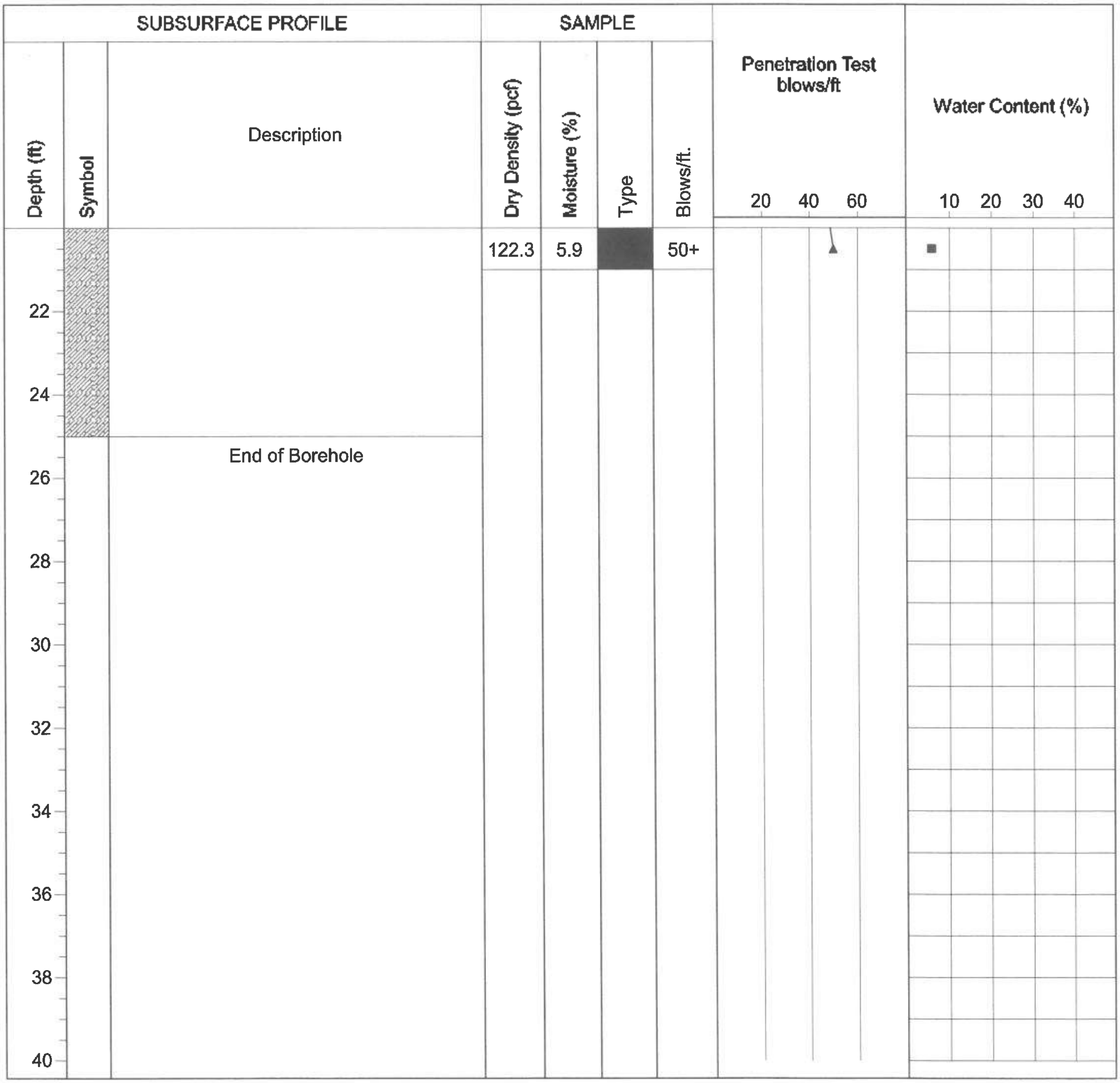
Log of Boring B4

Project: La Vista Residential Community
Project No: 042-19042

Client: Eden Housing
Figure No.: A-4

Location: East 26th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water> **Initial:** None **At Completion:** None



Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
	▨		122.3	5.9	■	50+
		End of Borehole				

Drill Method: Solid Flight
Drill Date: 1-23-20

Drill Rig: CME 45C-1
Krazan and Associates
Hole Size: 4½ Inches

Driller: Chris Wyneken
Elevation: 25 Feet

Sheet: 2 of 2

Log of Boring B5

Project: La Vista Residential Community

Project No: 042-19042

Client: Eden Housing

Figure No.: A-5

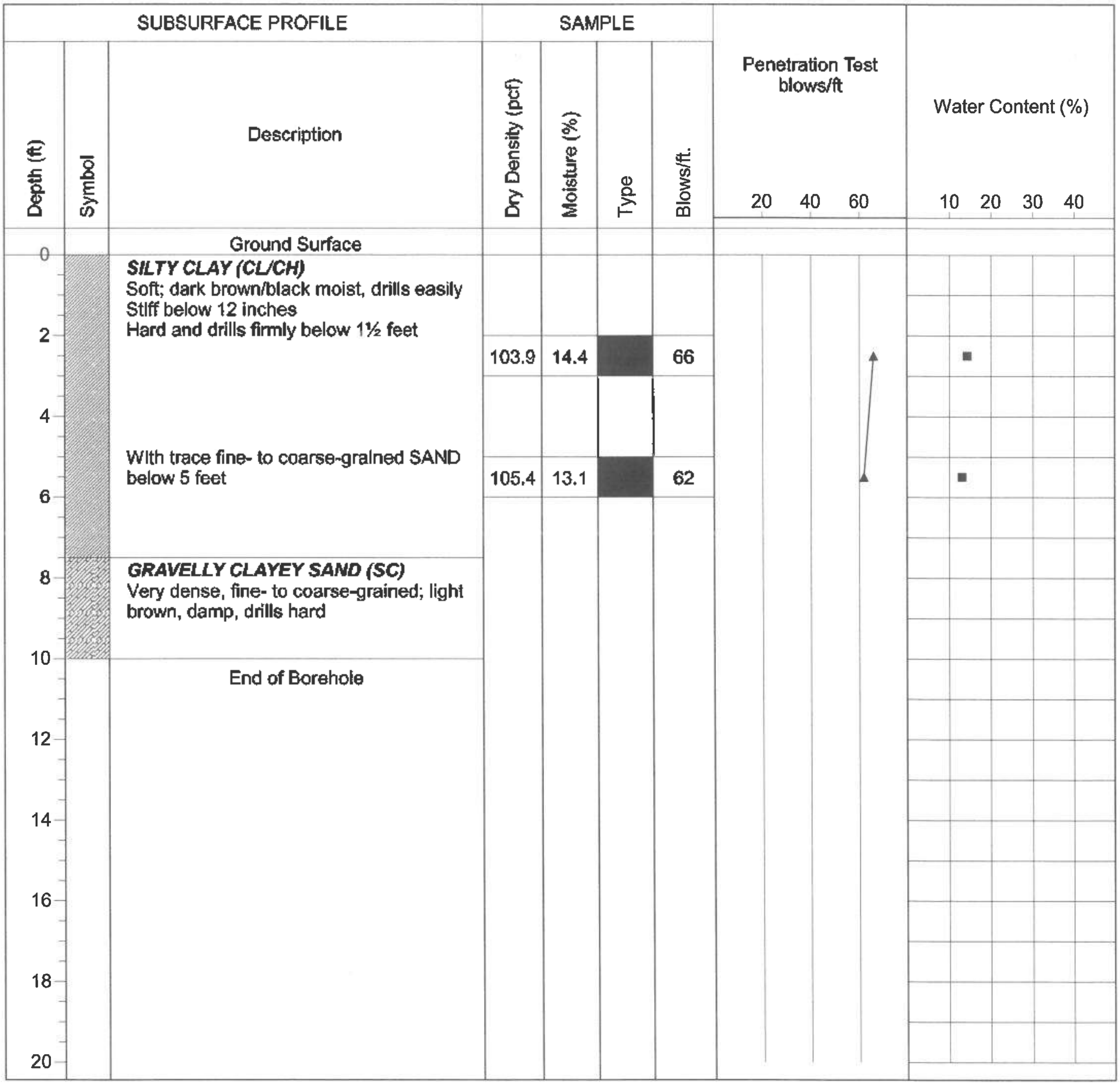
Location: East 26th Street and Tennyson Road, Hayward, California

Logged By: Wayne Andrade

Depth to Water>

Initial: None

At Completion: None



Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
Ground Surface				
SILTY CLAY (CL/CH) Soft; dark brown/black moist, drills easily Stiff below 12 inches Hard and drills firmly below 1½ feet				
	103.9	14.4	■	66
With trace fine- to coarse-grained SAND below 5 feet	105.4	13.1	■	62
GRAVELLY CLAYEY SAND (SC) Very dense, fine- to coarse-grained; light brown, damp, drills hard				
End of Borehole				

Drill Method: Solid Flight

Drill Date: 1-23-20

Drill Rig: CME 45C-1

Krazan and Associates

Hole Size: 4½ Inches

Driller: Chris Wyneken

Elevation: 10 Feet

Sheet: 1 of 1

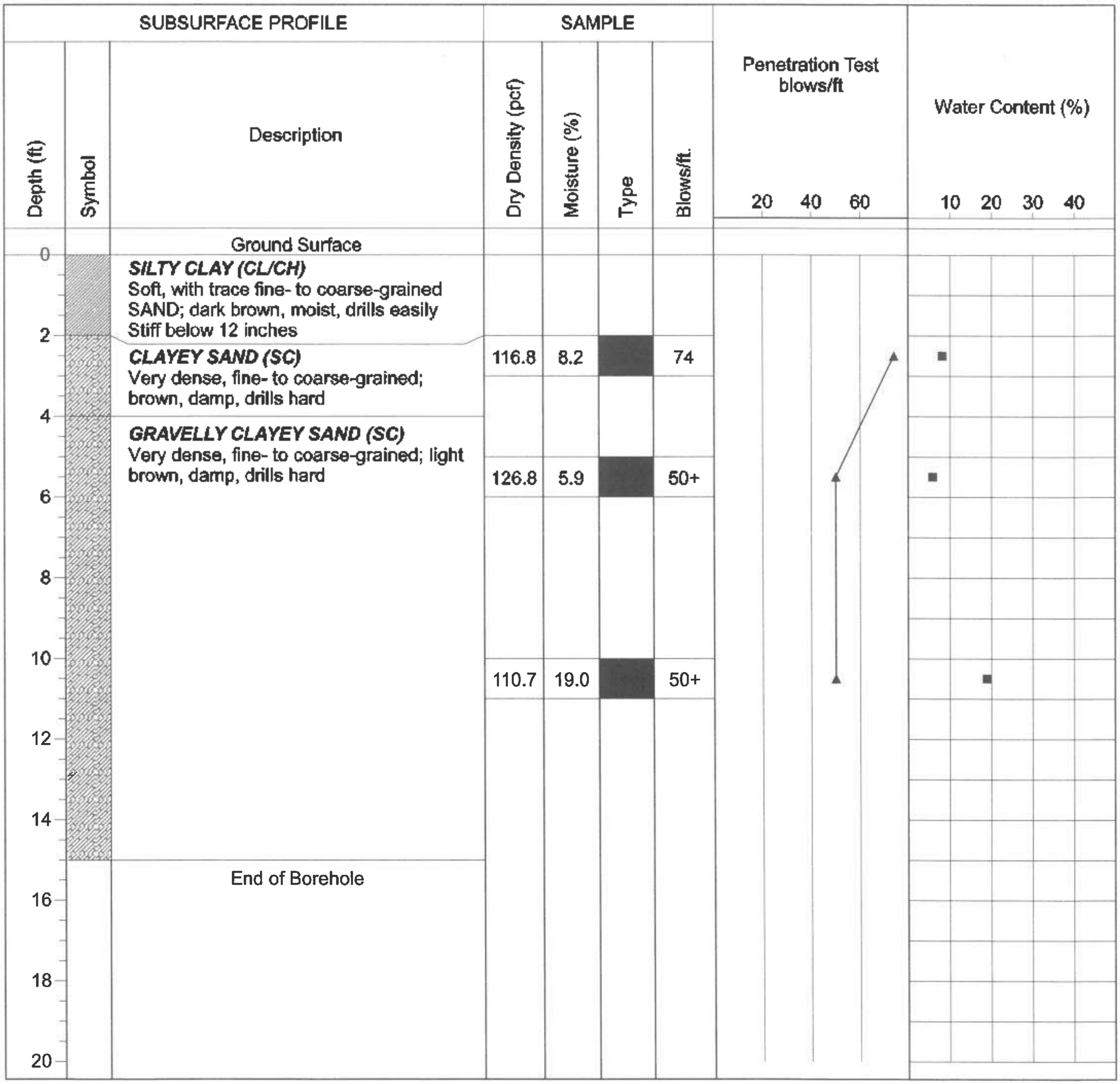
Log of Boring B6

Project: La Vista Residential Community
Project No: 042-19042

Client: Eden Housing
Figure No.: A-6

Location: East 26th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water> **Initial:** None **At Completion:** None



SUBSURFACE PROFILE		SAMPLE			
Depth (ft)	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
0	Ground Surface				
0	***SILTY CLAY (CL/CH)*** Soft, with trace fine- to coarse-grained SAND; dark brown, moist, drills easily Stiff below 12 inches				
2	***CLAYEY SAND (SC)*** Very dense, fine- to coarse-grained; brown, damp, drills hard	116.8	8.2	■	74
4	***GRAVELLY CLAYEY SAND (SC)*** Very dense, fine- to coarse-grained; light brown, damp, drills hard	126.8	5.9	■	50+
10		110.7	19.0	■	50+
15	End of Borehole				

Drill Method: Solid Flight
Drill Date: 1-22-20

Drill Rig: CME 45C-1
Krazan and Associates
Hole Size: 4½ Inches

Driller: Chris Wyneken
Elevation: 15 Feet

Sheet: 1 of 1

Log of Boring B7

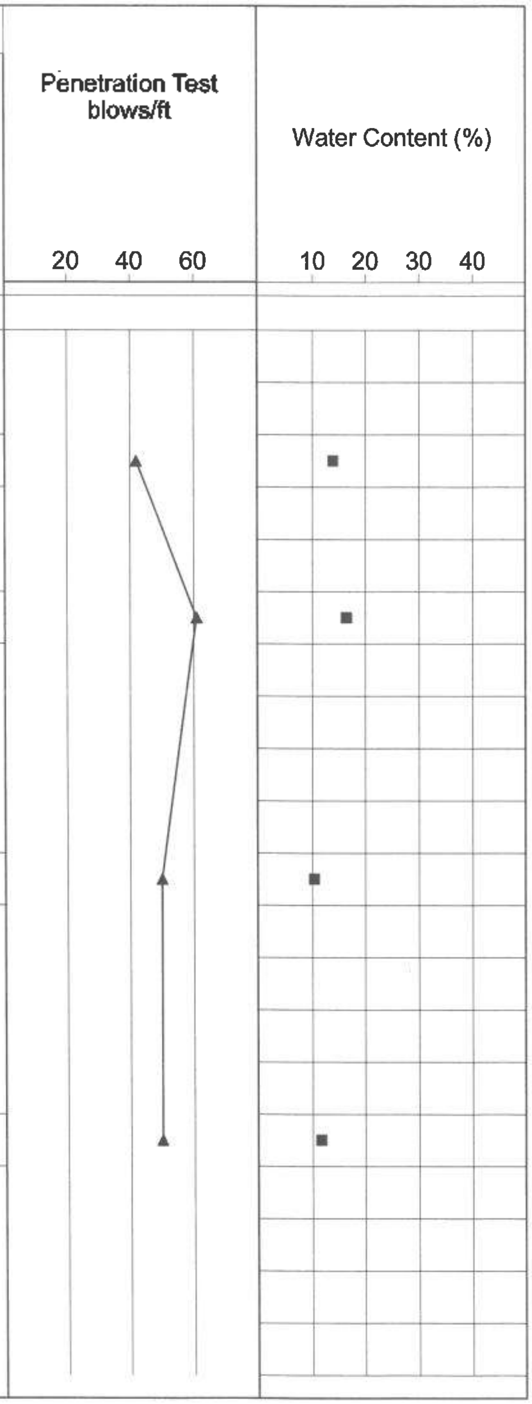
Project: La Vista Residential Community
Project No: 042-19042

Client: Eden Housing
Figure No.: A-7

Location: East 26th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water> **Initial:** None **At Completion:** None

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0		***CLAYEY SAND (SC)*** FILL, fine- to medium-grained; dark brown, moist, drills easily				
			110.6	14.0	■	42
		SILTY CLAY (CL/CH) Hard; dark brown, moist, drills firmly				
			102.4	16.6	■	61
		CLAYEY SAND/SANDY CLAY (SC/CL) Very dense, fine- to medium-grained with trace GRAVEL; olive-brown, moist, drills hard				
			117.7	10.4	■	50+
			125.0	11.7	■	50+
20						



Drill Method: Solid Flight
Drill Date: 1-22-20

Drill Rig: CME 45C-1
Hole Size: 4½ Inches

Krazan and Associates

Driller: Chris Wyneken
Elevation: 20 Feet

Sheet: 1 of 1

Log of Boring B8

Project: La Vista Residential Community

Project No: 042-19042

Client: Eden Housing

Figure No.: A-8

Location: East 26th Street and Tennyson Road, Hayward, California

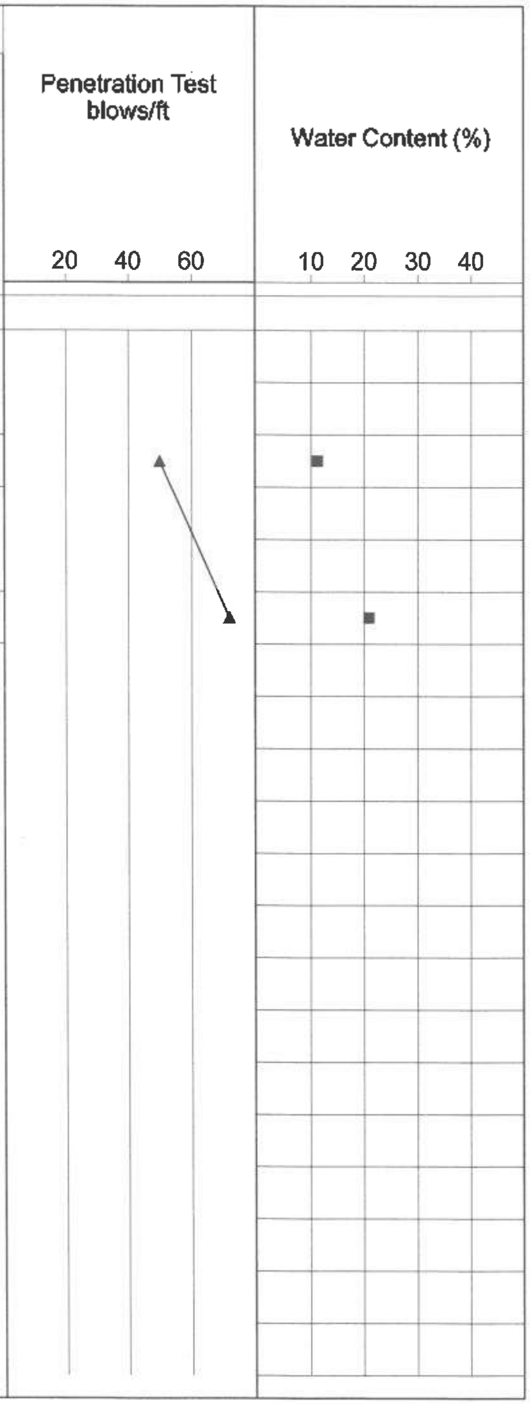
Logged By: Wayne Andrade

Depth to Water>

Initial: None

At Completion: None

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0	▨	***SILTY CLAY (CL/CH)*** Soft, fine- to medium-grained; dark brown, moist, drills easily Stiff below 12 inches Hard and drills hard below 2 feet				
2	▨		120.2	11.3	■	50+
4	▨					
	▨		106.9	21.0	■	72
6	▨					
8	▨	***CLAYEY SAND (SC)*** Very dense, fine- to medium-grained; olive-brown, damp, drills hard				
10		End of Borehole				
12						
14						
16						
18						
20						



Drill Method: Solid Flight

Drill Date: 1-22-20

Drill Rig: CME 45C-1

Krazan and Associates

Hole Size: 4½ Inches

Driller: Chris Wyneken

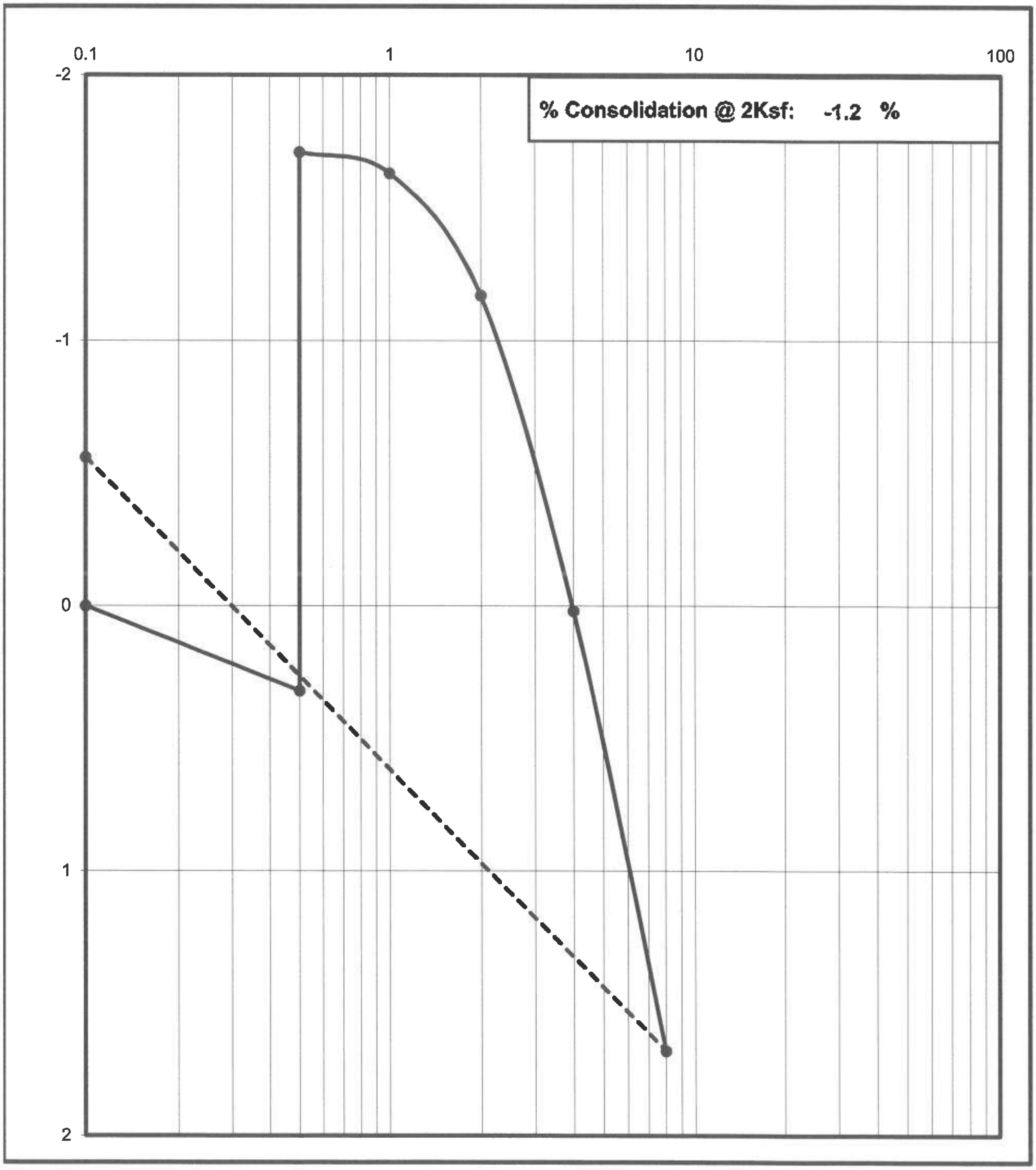
Elevation: 10 Feet

Sheet: 1 of 1

DRAFT

Consolidation Test

Project No	Boring No. & Depth	Date	Soil Classification
042-19042	B2 @ 2-3'	2/6/2020	CL/CH



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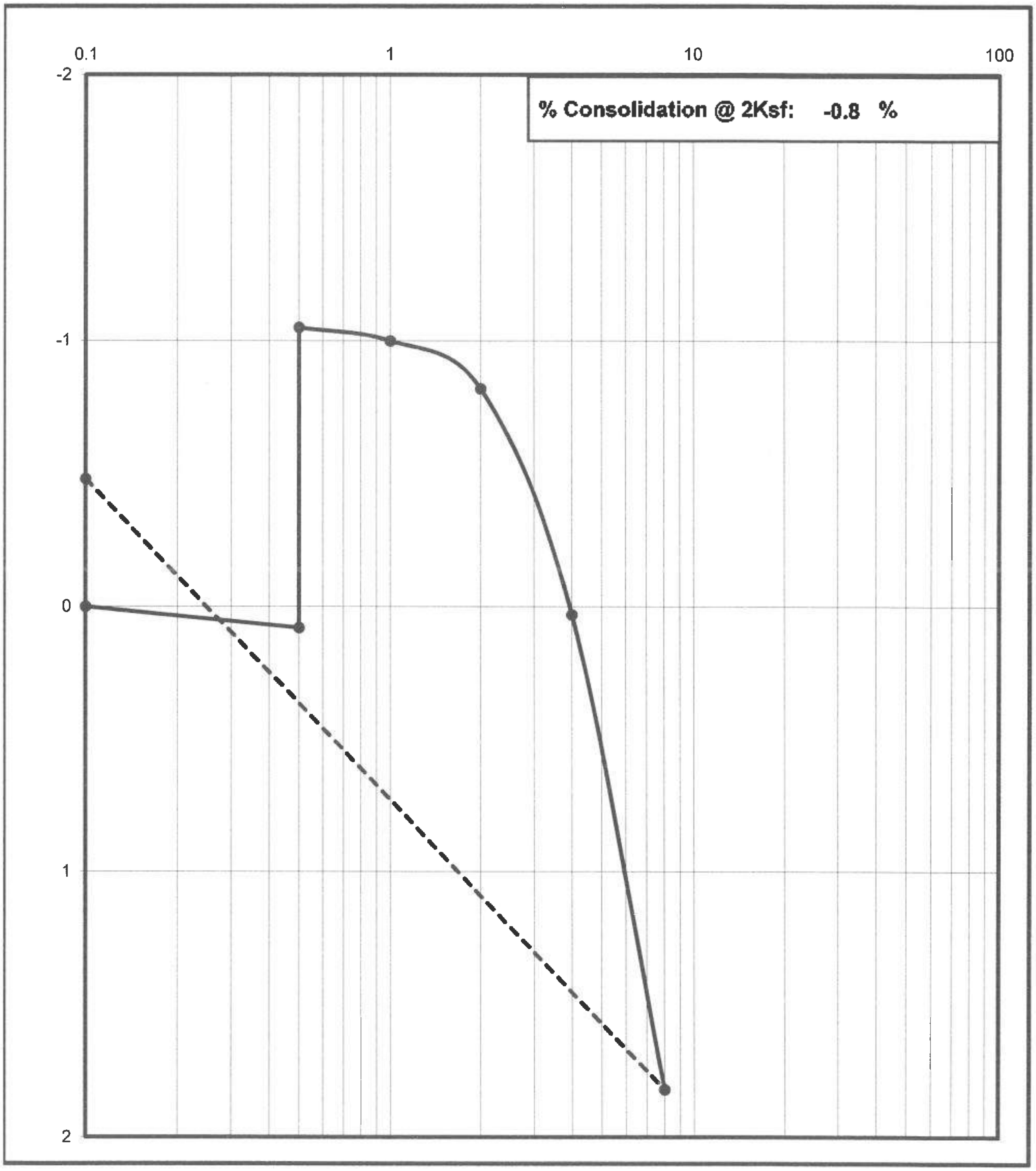
DRAFT

Consolidation Test

Project No	Boring No. & Depth	Date	Soil Classification
042-19042	B7 @ 2-3'	2/11/2020	CL/CH



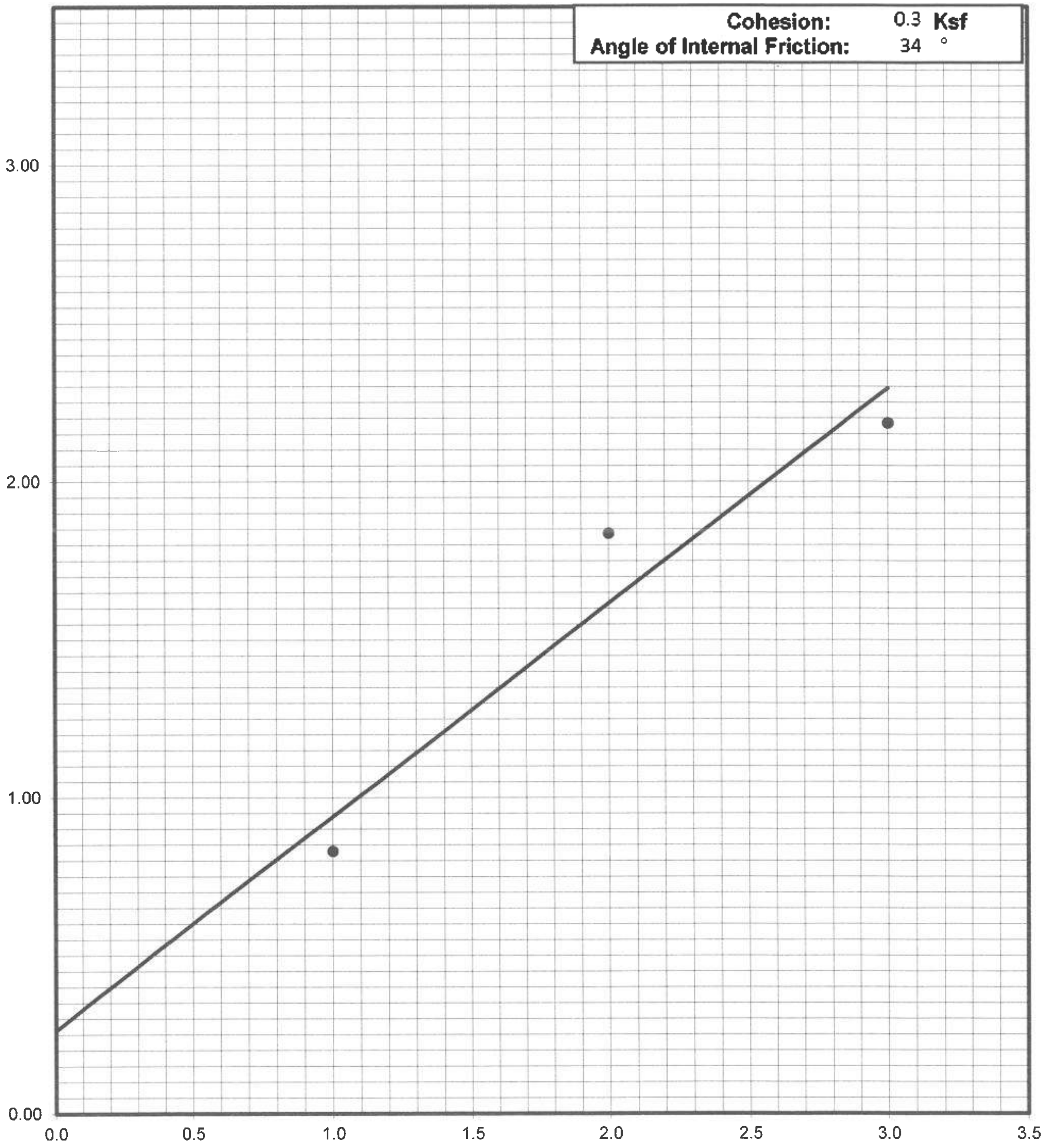
Krazan Testing Laboratory

DRAFT

Shear Strength Diagram (Direct Shear)

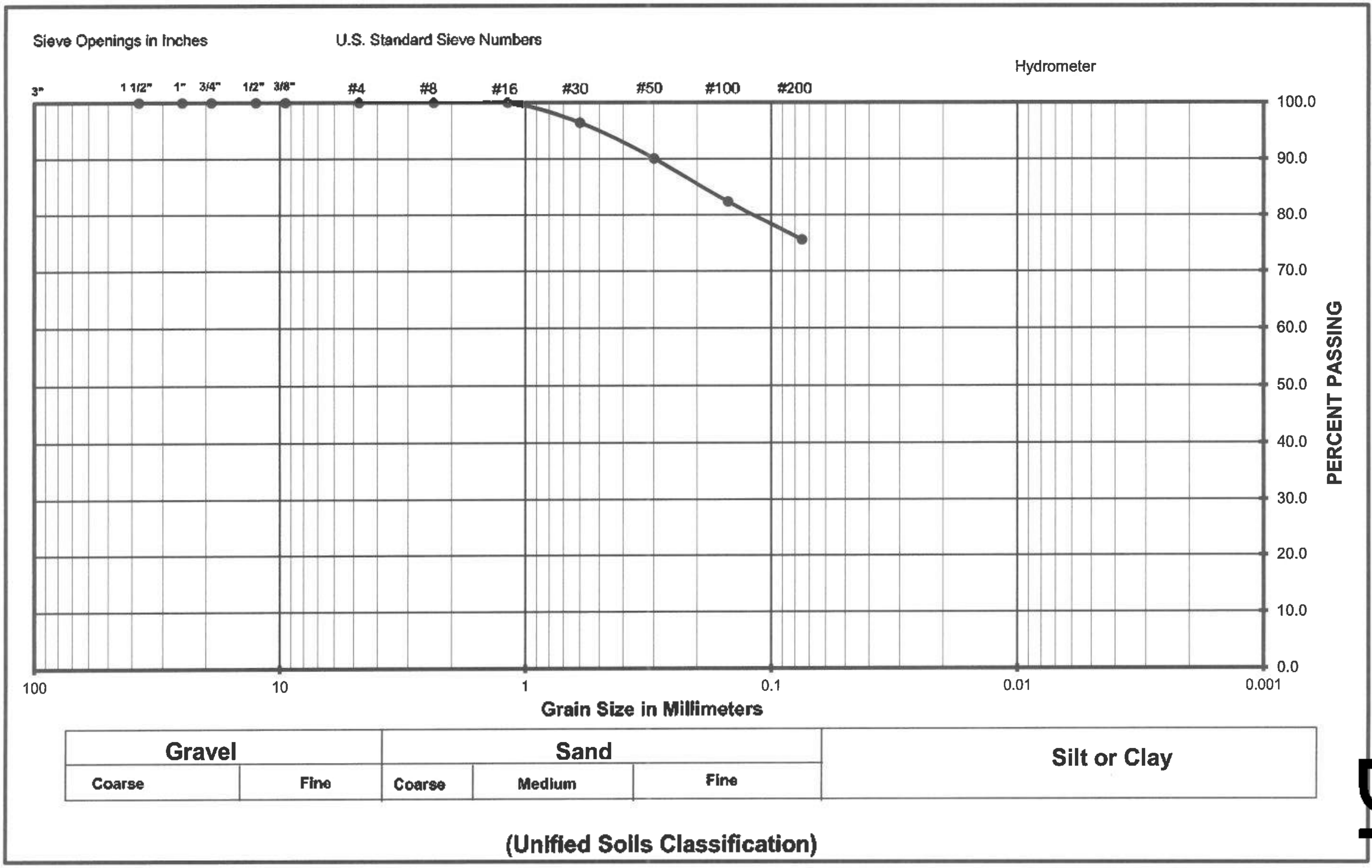
ASTM D - 3080 / AASHTO T - 236

Project Number	Boring No. & Depth	Soil Type	Date
042-19042	B3 @ 2-3'	SC w/ grvl	2/6/2020



Krazan Testing Laboratory

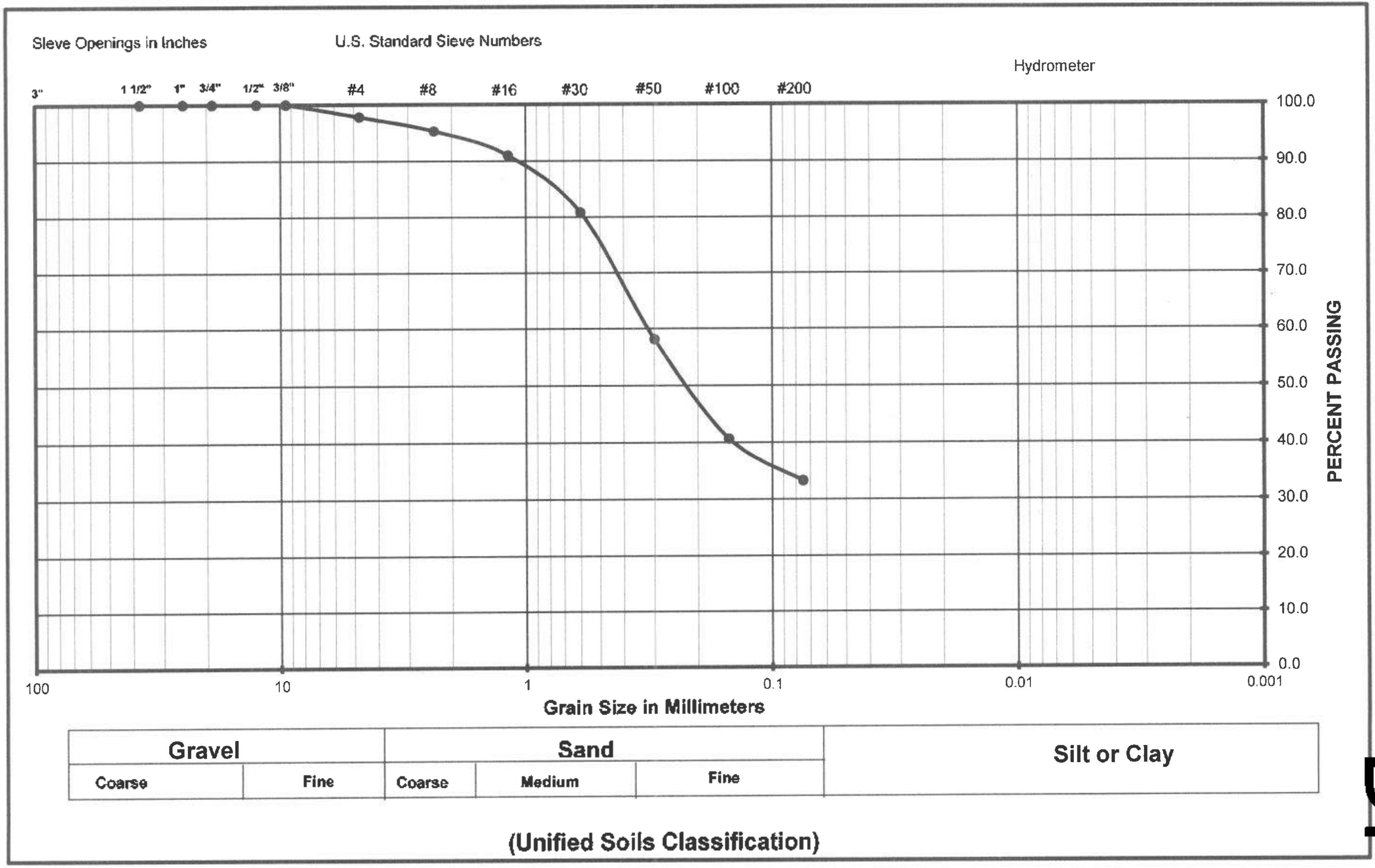
Grain Size Analysis



Project Name	La Vista Residential Community
Project Number	042-19042
Soil Classification	CL/CH
Sample Number	B2 @ 2-3'

Krazan Testing Laboratory

Grain Size Analysis



Project Name	La Vista Residential Community
Project Number	042-19042
Soil Classification	CL/CH
Sample Number	B7 @ 2-3'

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Expansion Index Test

ASTM D - 4829

Project Number : 042-19042
Project Name : La Vista Residential Community
Date : 2/6/2020
Sample location/ Depth : 0-4'
Sample Number : X1
Soil Classification : CH

Trial #	**1**	**2**	**3**
Weight of Soil & Mold, gms	722.4		
Weight of Mold, gms	367.8		
Weight of Soil, gms	354.6		
Wet Density, Lbs/cu.ft.	106.9		
Weight of Moisture Sample (Wet), gms	300.0		
Weight of Moisture Sample (Dry), gms	261.6		
Moisture Content, %	14.7		
Dry Density, Lbs/cu.ft.	93.3		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	49.1		

Time	Inital	30 min	1 hr	6hrs	12 hrs	24 hrs
Dial Reading	0	--	--	--	--	0.1418

Expansion Index $_{\text{measured}}$ = 141.8

Expansion Index = 142

Expansion Potential Table	
Exp. Index	Potential Exp.
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

Krazan Testing Laboratory

DRAFT

Expansion Index Test

ASTM D - 4829

Project Number : 042-19042
Project Name : La Vista Residential Community
Date : 2/6/2020
Sample location/ Depth : 4-7'
Sample Number : X2
Soil Classification : CL

Trial #	**1**	**2**	**3**
Weight of Soil & Mold, gms	729.0		
Weight of Mold, gms	367.5		
Weight of Soil, gms	361.5		
Wet Density, Lbs/cu.ft.	109.0		
Weight of Moisture Sample (Wet), gms	300.0		
Weight of Moisture Sample (Dry), gms	264.3		
Moisture Content, %	13.5		
Dry Density, Lbs/cu.ft.	96.0		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	48.4		

Time	Inital	30 min	1 hr	6hrs	12 hrs	24 hrs
Dial Reading	0	--	--	--	--	0.0862

Expansion Index $_{\text{measured}}$ = 86.2

Expansion Index = 86

Expansion Potential Table	
Exp. Index	Potential Exp.
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

Krazan Testing Laboratory

DRAFT

Expansion Index Test

ASTM D - 4829

Project Number : 042-19042
Project Name : La Vista Residential Community
Date : 2/11/2020
Sample location/ Depth : 4-8'
Sample Number : X3
Soil Classification : SC-CL

Trial #	1	2	3
Weight of Soil & Mold, gms	772.8		
Weight of Mold, gms	367.8		
Weight of Soil, gms	405.0		
Wet Density, Lbs/cu.ft.	122.1		
Weight of Moisture Sample (Wet), gms	300.0		
Weight of Moisture Sample (Dry), gms	274.0		
Moisture Content, %	9.5		
Dry Density, Lbs/cu.ft.	111.6		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	50.2		

Time	Inital	30 min	1 hr	6hrs	12 hrs	24 hrs
Dial Reading	0	--	--	--	--	0.044

Expansion Index $_{\text{measured}}$ = 44

Expansion Index = 44

Expansion Potential Table	
Exp. Index	Potential Exp.
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

Krazan Testing Laboratory

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

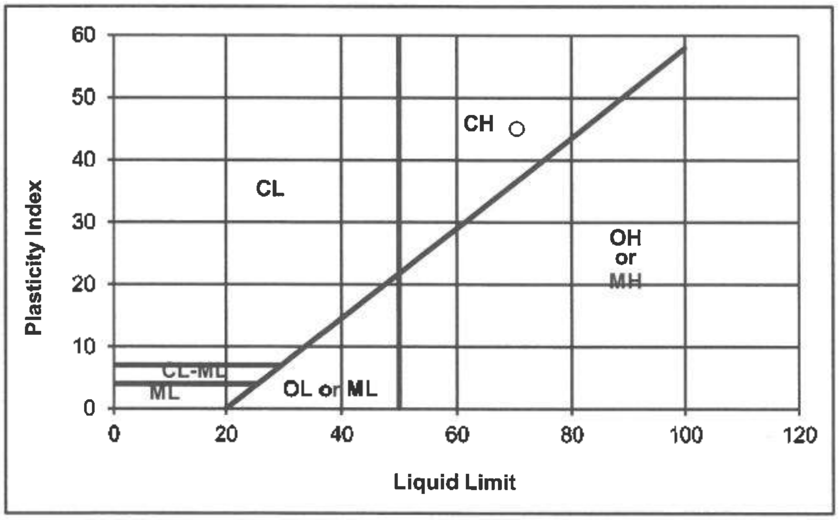
Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Sampled By: WA
Sample Number: X1
Sample Location: 0-4'
Sample Description: CH

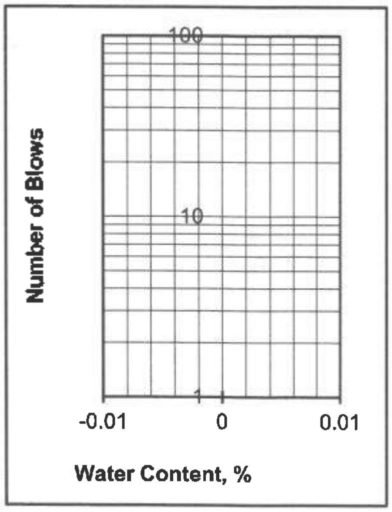
Date Tested: 2/5/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)	24.98	23.76		27.15	28.34	
Weight of Dry Soil & Tare (g)	22.96	21.60		21.53	23.69	
Weight of Tare (g)	14.72	12.86		13.56	17.05	
Weight of water (g)	2.02	2.16		5.62	4.65	
Weight of Dry Soil (g)	8.24	8.73		7.97	6.64	
Water Content (% of dry wt.)	24.6%	24.8%		70.4%	70.1%	
Number of Blows				25	26	

Plastic Limit : 25 **Liquid Limit : 70**

Plasticity Index : 45
Unified Soil Classification : CH
Requirement:
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

Plasticity Index of Soils

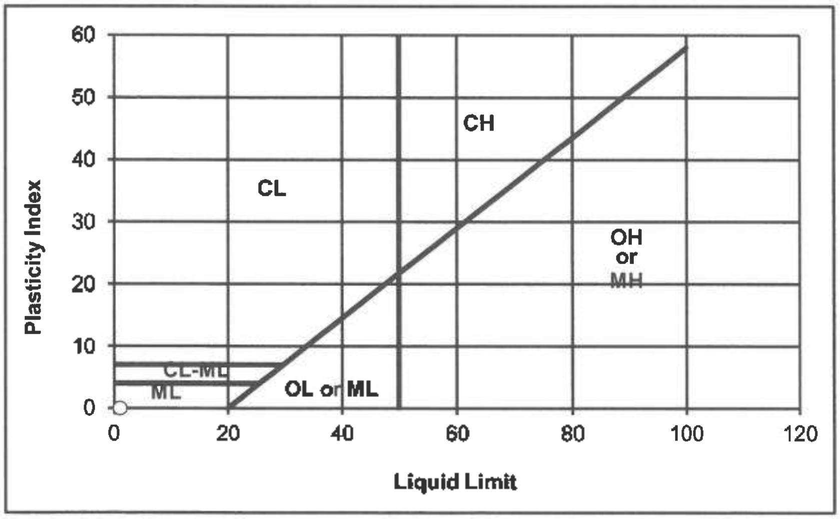
ASTM D4318/AASHTO T89 T90/CT 204

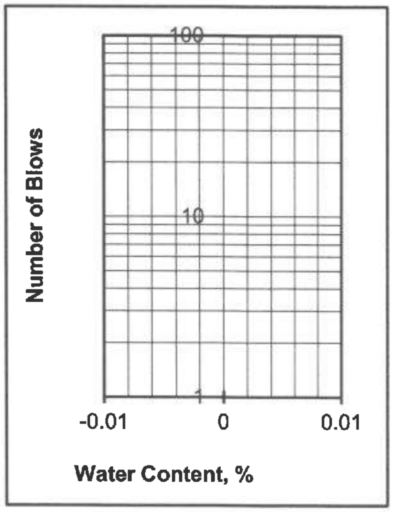
Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Date Tested: 2/5/2020
Sampled By: WA
Tested By: J Mitchell
Sample Number:
Verified By: J Gruszczynski
Sample Location: B1 @ 10-11'
Sample Description: SM w/ grvl

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D
Liquid Limit : N/D

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC
Requirement:
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Sampled By: WA
Sample Number:
Sample Location: B1 @ 15-16'
Sample Description: SM w/ grvl

Date Tested: 2/5/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D

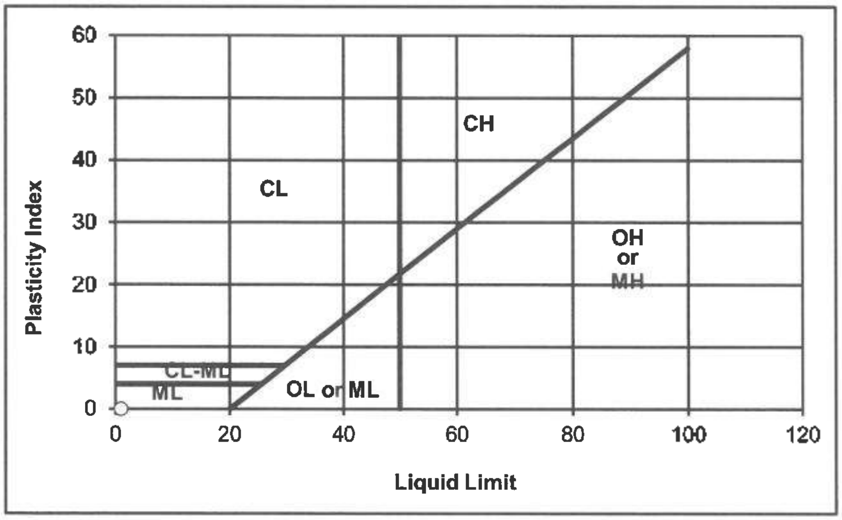
Liquid Limit : N/D

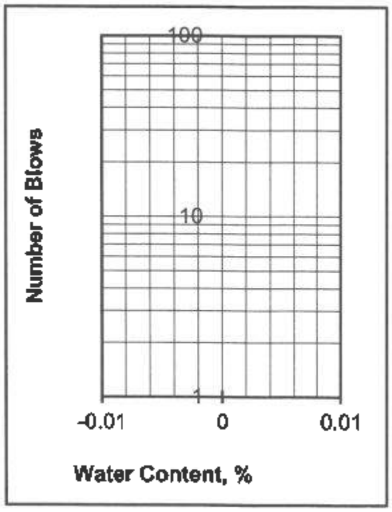
Plasticity Index : NON-PLASTIC

Unified Soil Classification : NON-PLASTIC

Requirement:

Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

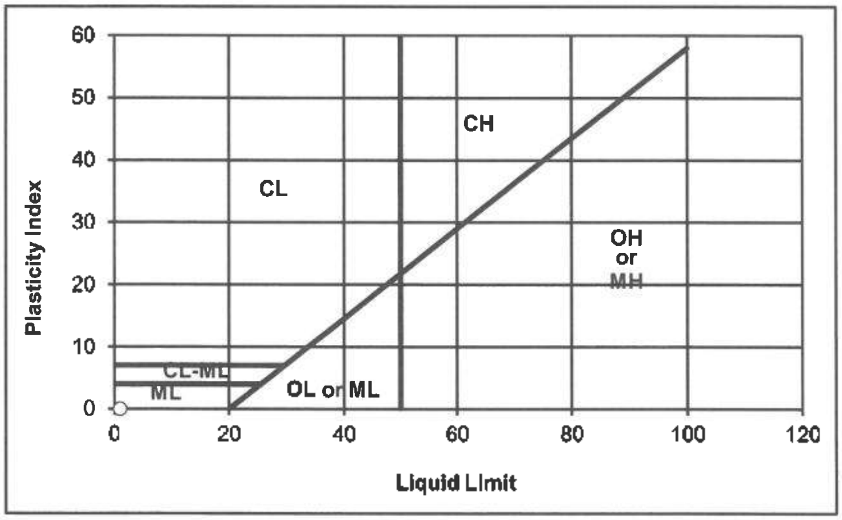
ASTM D4318/AASHTO T89 T90/CT 204

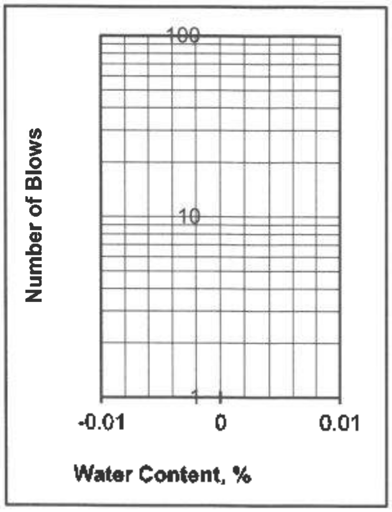
Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Date Tested: 2/5/2020
Sampled By: WA
Tested By: J Mitchell
Sample Number:
Verified By: J Gruszczynski
Sample Location: B1 @ 20-21'
Sample Description: SM w/ grvl

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D
Liquid Limit : N/D

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC
Requirement:
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Sampled By: WA
Sample Number:
Sample Location: B1 @ 25-26'
Sample Description: SC w/ grvl

Date Tested: 2/5/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)	32.65	32.26		29.05	26.32	
Weight of Dry Soil & Tare (g)	30.42	29.61		26.25	23.45	
Weight of Tare (g)	16.98	13.91		17.06	14.16	
Weight of water (g)	2.24	2.64		2.80	2.87	
Weight of Dry Soil (g)	13.43	15.70		9.18	9.30	
Water Content (% of dry wt.)	16.7%	16.8%		30.5%	30.8%	
Number of Blows				26	25	

Plastic Limit : 17

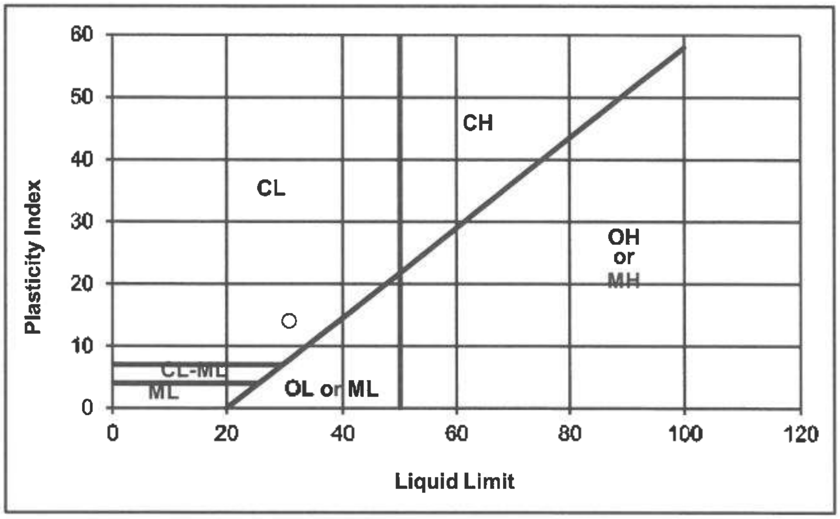
Liquid Limit : 31

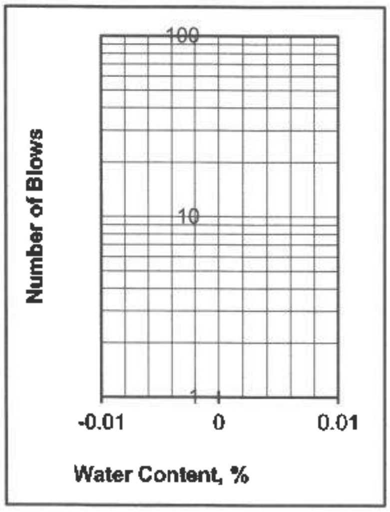
Plasticity Index : 14

Unified Soil Classification : CL

Requirement:

Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Date Tested: 2/5/2020
Sampled By: WA
Tested By: J Mitchell
Sample Number:
Verified By: J Gruszczynski
Sample Location: B1 @ 30-31'
Sample Description: SM w/ grvl

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

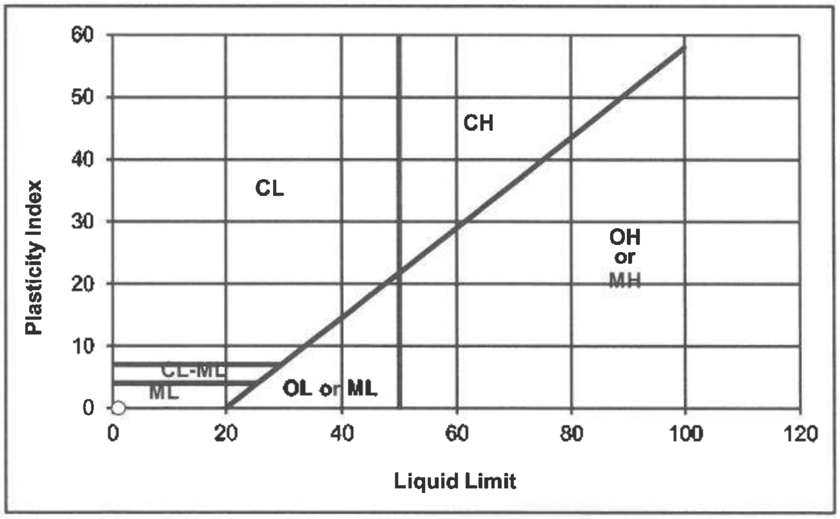
Plastic Limit : N/D

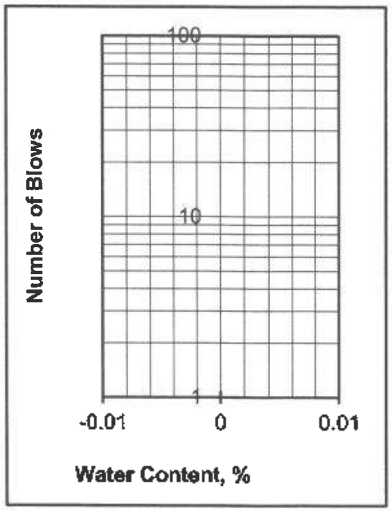
Liquid Limit : N/D

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC

Requirement:

Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

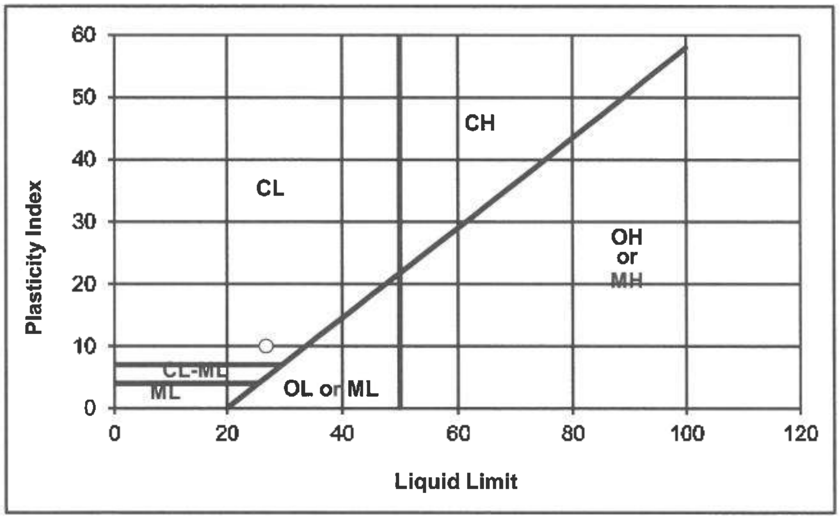
Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Sampled By: WA
Sample Number:
Sample Location: B1 @ 35-36'
Sample Description: SC w/ grvl

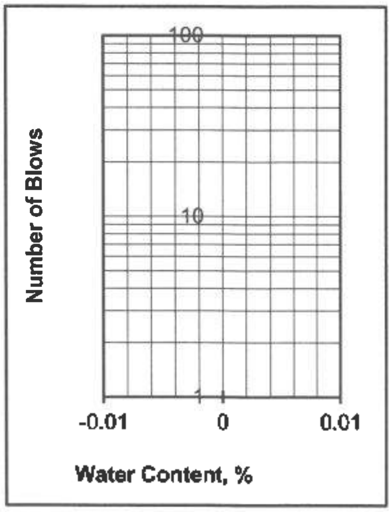
Date Tested: 2/5/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)	24.24	23.88		24.79		
Weight of Dry Soil & Tare (g)	22.74	22.45		22.26		
Weight of Tare (g)	13.96	14.14		12.84		
Weight of water (g)	1.50	1.43		2.53		
Weight of Dry Soil (g)	8.78	8.31		9.42		
Water Content (% of dry wt.)	17.1%	17.2%		26.8%		
Number of Blows				25		

Plastic Limit : 17
Liquid Limit : 27

Plasticity Index : 10
Unified Soil Classification : CL
Requirement:
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

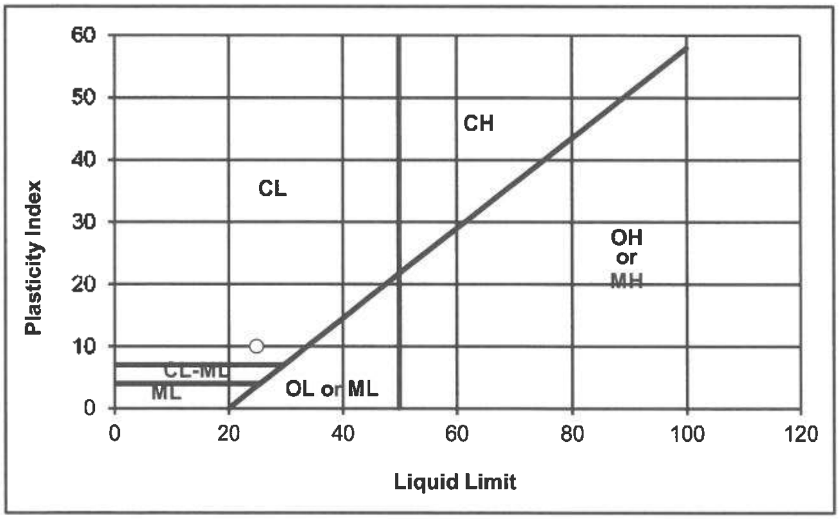
Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Sampled By: WA
Sample Number:
Sample Location: B1 @ 40-41'
Sample Description: SC w/ grvl

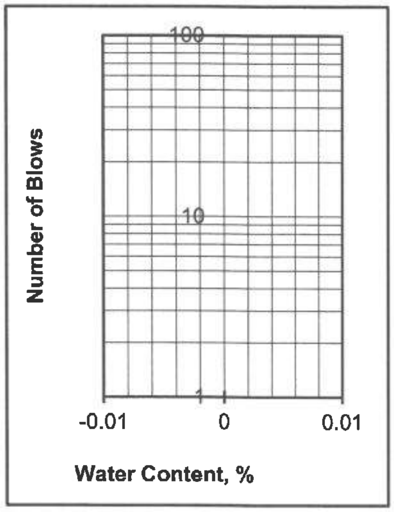
Date Tested: 2/5/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)	30.20	29.67		29.93	31.72	
Weight of Dry Soil & Tare (g)	28.42	28.00		26.80	28.55	
Weight of Tare (g)	17.01	17.05		14.29	15.66	
Weight of water (g)	1.78	1.67		3.13	3.18	
Weight of Dry Soil (g)	11.41	10.96		12.51	12.89	
Water Content (% of dry wt.)	15.6%	15.2%		25.0%	24.6%	
Number of Blows				25	26	

Plastic Limit : 15 **Liquid Limit : 25**

Plasticity Index : 10
Unified Soil Classification : CL
Requirement:
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

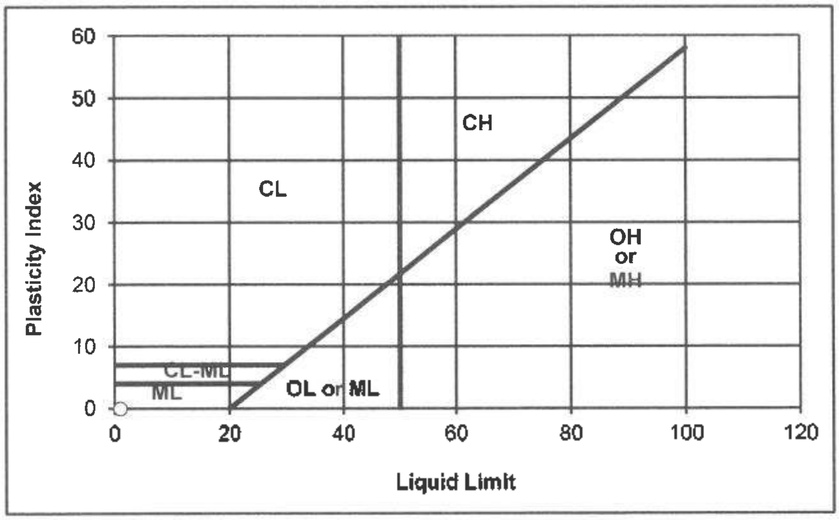
ASTM D4318/AASHTO T89 T90/CT 204

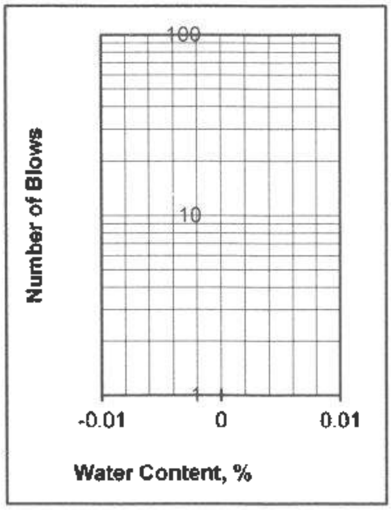
Project: **La Vista Residential Community**
Project Number: **042-19042**
Date Sampled: 1/23/2020
Date Tested: 2/5/2020
Sampled By: WA
Tested By: J Mitchell
Sample Number:
Verified By: J Gruszczynski
Sample Location: B1 @ 45-46'
Sample Description: SM-SP w/ grvl

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D **Liquid Limit : N/D**

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC **Requirement:**
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

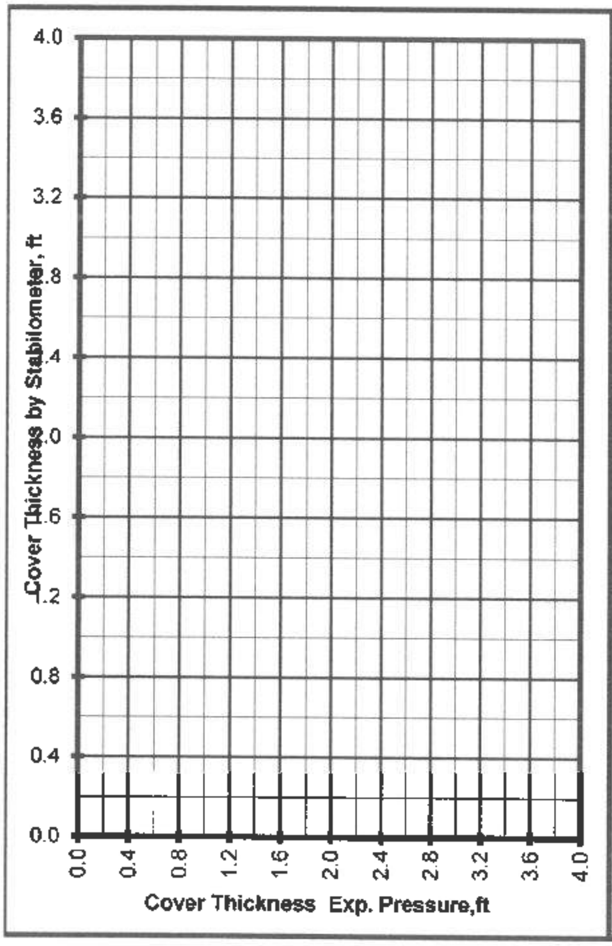
DRAFT

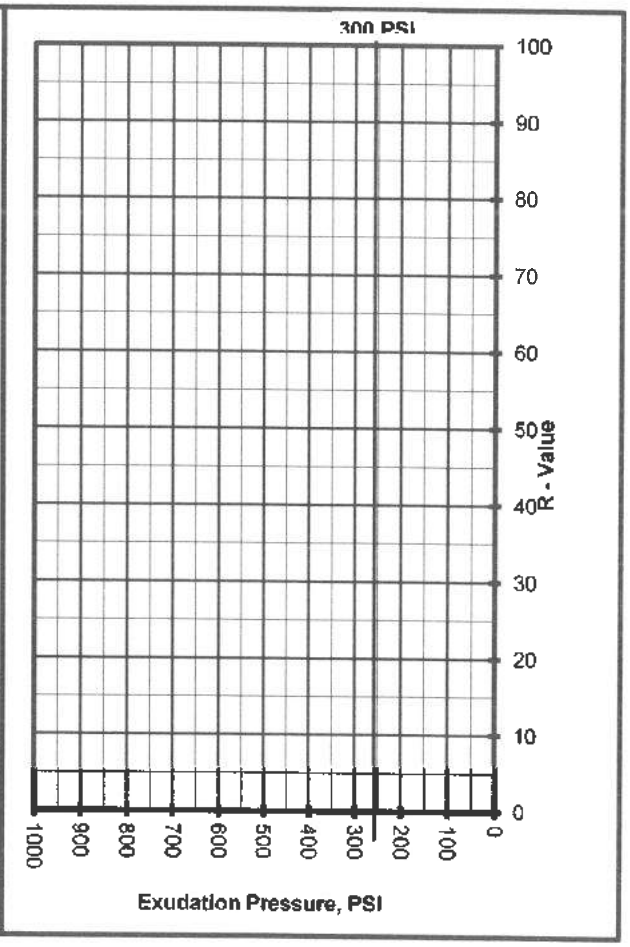
R - VALUE TEST
ASTM D - 2844 / CAL 301

Project Number : 042-19042
Project Name : La Vista Residential Community
Date : 1/28/2020
Sample Location/Curve Number : RV#1
Soil Classification : CL

TEST	A	B	C
Percent Moisture @ Compaction, %			
Dry Density, lbm/cu.ft.	R - Value less than 5 Sample Exuded from bottom of Mold During test		
Exudation Pressure, psi			
Expansion Pressure, (Dial Reading)			
Expansion Pressure, psf			
Resistance Value R			

R - Value at 300 PSI Exudation Pressure	< 5
R - Value by Expansion Pressure	





UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
		Clean Gravels (Less than 5% fines)
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (More than 12% fines)
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
		Clean Sands (Less than 5% fines)
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
		Sands with fines (More than 12% fines)
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)		
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of 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 50% or greater	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

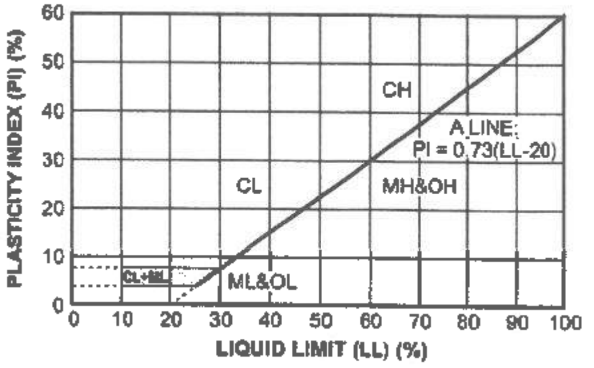
CONSISTENCY CLASSIFICATION

Description	**Blows per Foot**
Granular Soils	
Very Loose	< 5
Loose	5 – 15
Medium Dense	16 – 40
Dense	41 – 65
Very Dense	> 65
Cohesive Soils	
Very Soft	< 3
Soft	3 – 5
Firm	6 – 10
Stiff	11 – 20
Very Stiff	21 – 40
Hard	> 40

GRAIN SIZE CLASSIFICATION

Grain Type	*Standard Sieve Size*	*Grain Size in Millimeters*
Boulders	Above 12 inches	Above 305
Cobbles	12 to 13 inches	305 to 76.2
Gravel	3 inches to No. 4	76.2 to 4.76
Coarse-grained	3 to ¾ inches	76.2 to 19.1
Fine-grained	¾ inches to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
Coarse-grained	No. 4 to No. 10	4.76 to 2.00
Medium-grained	No. 10 to No. 40	2.00 to 0.042
Fine-grained	No. 40 to No. 200	0.042 to 0.074
Silt and Clay	Below No. 200	Below 0.074

PLASTICITY CHART



Log of Boring B1

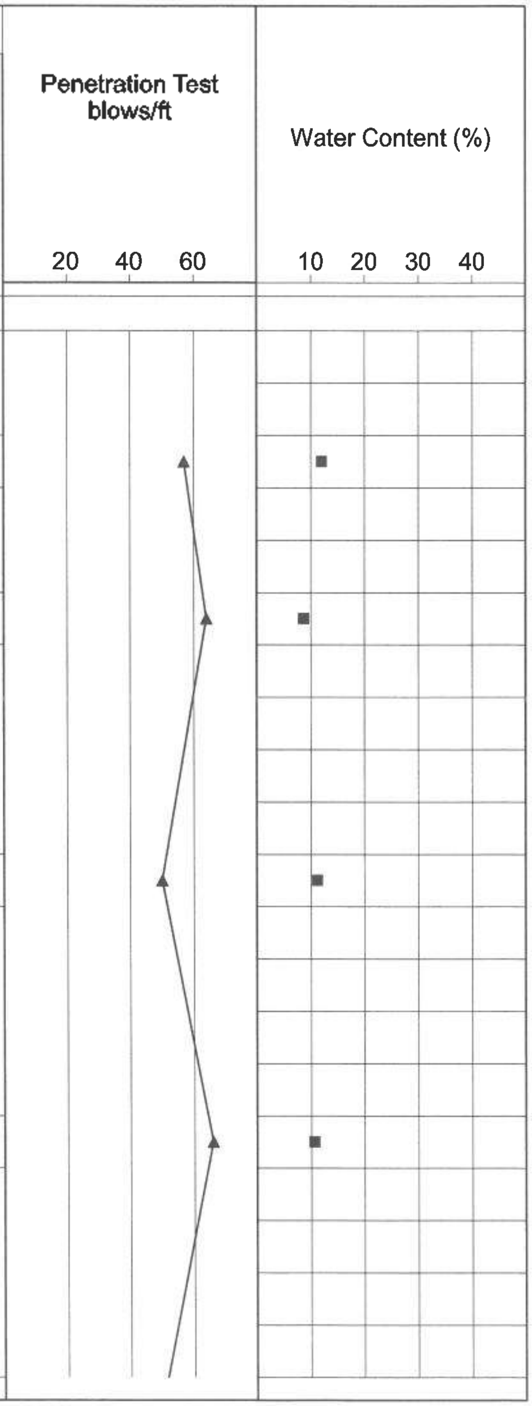
Project: New Charter School
Project No: 042-20001

Client: Pacific West Communities, Inc.
Figure No.: A-1

Location: East 16th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water>
Initial: None
At Completion: None

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0		***SILTY CLAY (CL/CH)*** Soft with trace fine- to coarse-grained SAND; black/dark brown, moist, drills easily Very stiff below 12 inches Hard below 2 feet				
2			110.0	12.1	■	57
4		***CLAYEY SAND (SC)*** Dense, fine- to coarse-grained; dark brown/black, damp, drills firmly				
6			120.0	8.7	■	64
8		Very dense and drills hard below 8 feet				
10			124.6	11.3	■	50+
12						
14		***GRAVELLY CLAYEY SAND (SC)*** Very dense, fine- to coarse-grained; brown, moist, drills hard				
16			123.5	10.7	■	66
18						
20		***SILTY SANDY GRAVEL (GM)*** Very dense, fine- to coarse-grained with trace CLAY; brown, damp, drills hard			■	



Drill Method: Solid Flight
Drill Date: 1-22-20

Drill Rig: CME 45C-1
Hole Size: 4½ Inches

Krazan and Associates

Driller: Chris Wyneken
Elevation: 25 Feet

Sheet: 1 of 2

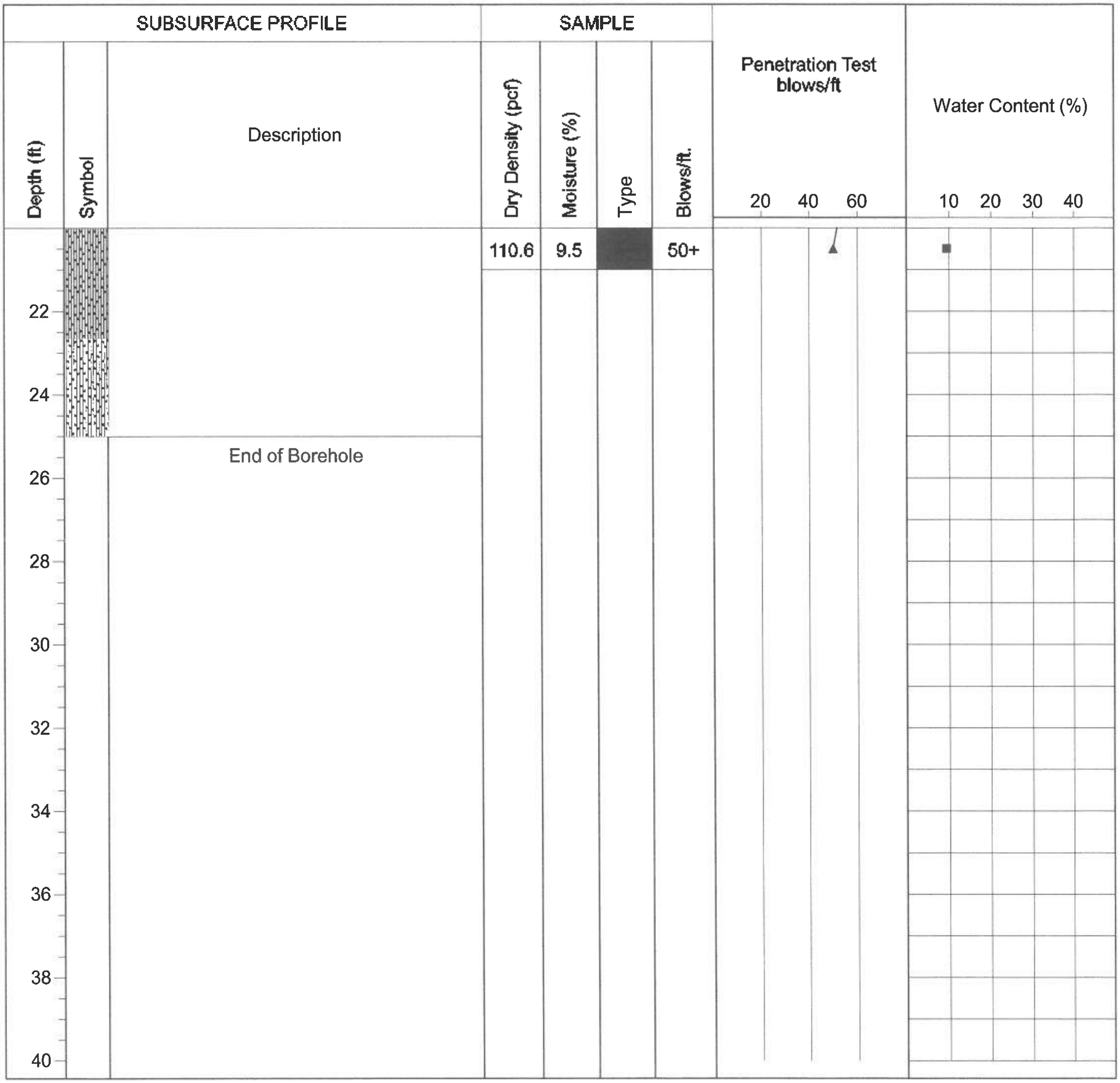
Log of Boring B1

Project: New Charter School
Project No: 042-20001

Client: Pacific West Communities, Inc.
Figure No.: A-1

Location: East 16th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water> **Initial:** None **At Completion:** None



Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
	110.6	9.5	■	50+
End of Borehole				

Drill Method: Solid Flight
Drill Date: 1-22-20

Drill Rig: CME 45C-1
Krazan and Associates
Hole Size: 4½ Inches

Driller: Chris Wyneken
Elevation: 25 Feet

Sheet: 2 of 2

Log of Boring B2

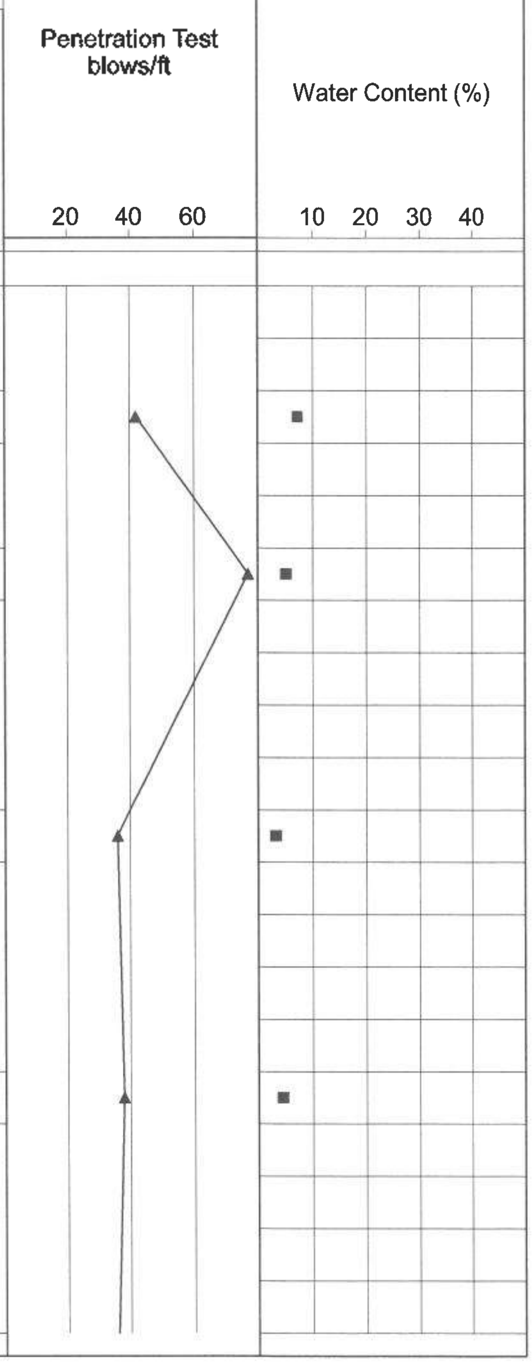
Project: New Charter School
Project No: 042-20001

Client: Pacific West Communities, Inc.
Figure No.: A-2

Location: East 16th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water> **Initial:** 36 Feet **At Completion:** 33 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0	▨	***SILTY CLAY (CL/CH)*** Soft with trace fine- to medium-grained SAND; black/dark brown, moist, drills easily Very stiff below 12 inches				
2			89.9	7.2	■	42
	▨	***CLAYEY SAND (SC)*** Dense, fine- to coarse-grained; dark brown, damp, drills firmly				
4	▨	***CLAYEY GRAVELLY SAND (SC)*** Very dense, fine- to coarse-grained; brown, damp, drills hard				
6			103.6	5.0	◢	77
8	▨	***SILTY SAND (SM)*** Dense, fine- to coarse-grained; brown, damp, drills firmly				
10			108.5	3.2	◢	36
12						
	▨	***SILTY SAND (SM)*** Dense, fine- to medium-grained with trace GRAVEL; brown, damp, drills firmly With increased GRAVEL below 14 feet				
14						
16			73.4	4.4	◢	38
18	▨	***SILTY SAND/CLAYEY SAND (SM/SC)*** Dense, fine- to medium-grained with trace GRAVEL; brown, damp, drills firmly				
20					◢	



Drill Method: Hollow Stem
Drill Date: 1-22-20

Drill Rig: CME 45C-1
Krazan and Associates
Hole Size: 6½ Inches

Driller: Chris Wyneken
Elevation: 50 Feet

DRAFT

Log of Boring B2

Project: New Charter School

Project No: 042-20001

Client: Pacific West Communities, Inc.

Figure No.: A-2

Location: East 16th Street and Tennyson Road, Hayward, California

Logged By: Wayne Andrade

Depth to Water> **Initial:** 36 Feet **At Completion:** 33 Feet

SUBSURFACE PROFILE

Depth (ft): 22, 24, 26, 28, 30, 32, 34, 36, 38, 40

SILTY SAND (SM)
Very dense, fine- to medium-grained with trace GRAVEL; brown, damp, drills hard

SILTY SAND/CLAYEY SAND (SM/SC)
Dense, fine- to medium-grained with trace GRAVEL; brown, damp, drills firmly

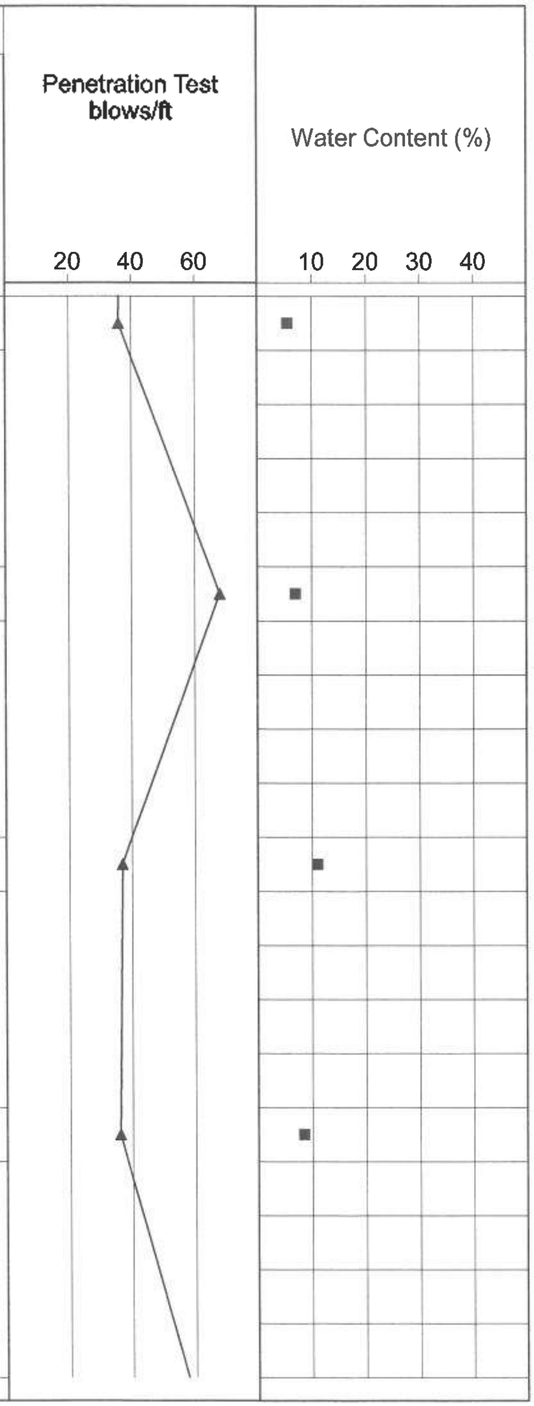
▼ Saturated below 33 feet

With increased GRAVEL below 35 feet ▽

Very dense below 40 feet

SAMPLE

Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
120.3	5.5	◢	36
126.2	7.0	◢	68
124.8	11.1	◢	37
125.6	8.5	◢	36



Drill Method: Hollow Stem

Drill Rig: CME 45C-1

Driller: Chris Wyneken

Krazan and Associates

Drill Date: 1-22-20

Hole Size: 6½ Inches

Elevation: 50 Feet

Sheet: 2 of 3

DRAFT

Log of Boring B2

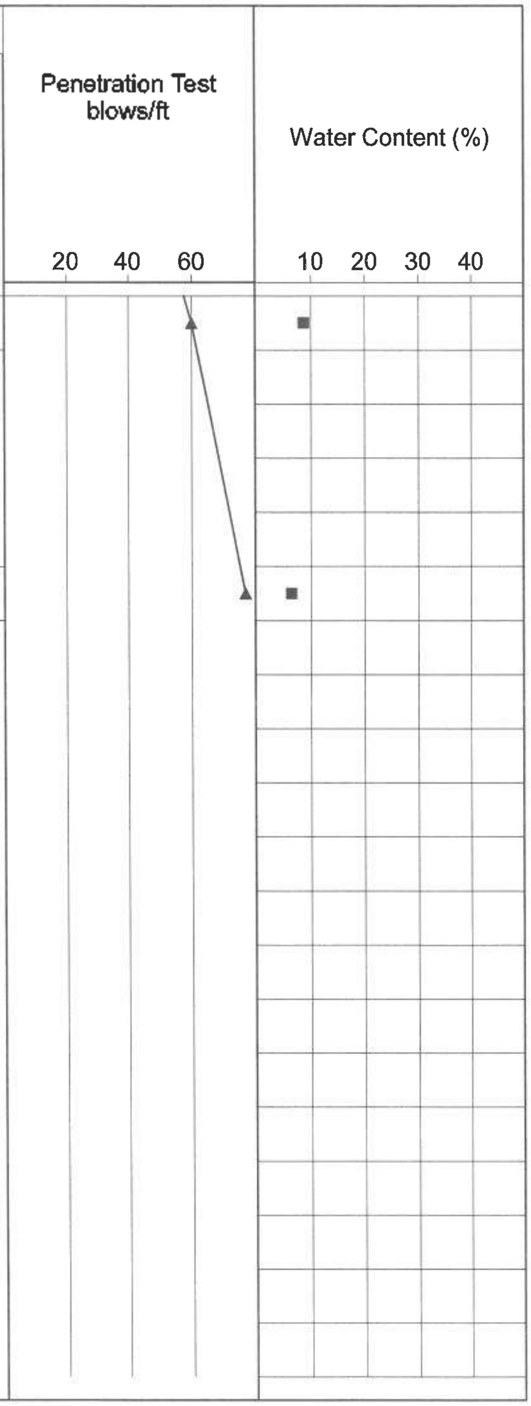
Project: New Charter School
Project No: 042-20001

Client: Pacific West Communities, Inc.
Figure No.: A-2

Location: East 16th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water> **Initial:** 36 Feet **At Completion:** 33 Feet

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
	▨		127.2	8.8	◢	60
	▨		108.2	6.5	◢	77
50		End of Borehole				



Drill Method: Hollow Stem
Drill Date: 1-22-20

Drill Rig: CME 45C-1
Krazan and Associates
Hole Size: 6½ Inches

Driller: Chris Wyneken
Elevation: 50 Feet

Sheet: 3 of 3

DRAFT

Log of Boring B3

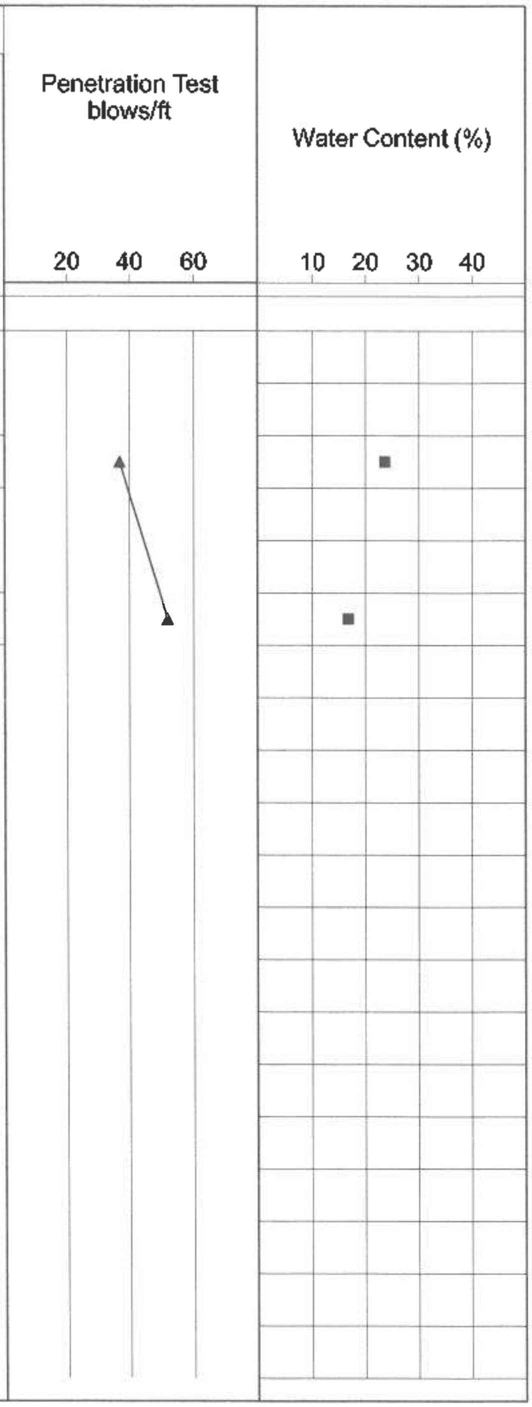
Project: New Charter School
Project No: 042-20001

Client: Pacific West Communities, Inc.
Figure No.: A-3

Location: East 16th Street and Tennyson Road, Hayward, California
Logged By: Wayne Andrade

Depth to Water>
Initial: None
At Completion: None

SUBSURFACE PROFILE			SAMPLE			
Depth (ft)	Symbol	Description	Dry Density (pcf)	Moisture (%)	Type	Blows/ft.
		Ground Surface				
0 2 4	▨	***SILTY CLAY (CL/CH)*** Soft; black/dark brown, moist, drills easily Stiff below 12 inches Very stiff with trace fine- to coarse-grained SAND below 2 feet	101.2	23.8	■	37
6 8	▨	***CLAYEY SAND (SC)*** Dense, fine- to coarse-grained; brown, moist, drills firmly	114.2	16.9	■	52
10		End of Borehole				
12						
14						
16						
18						
20						



Drill Method: Solid Flight
Drill Date: 1-22-20

Drill Rig: CME 45C-1
Krazan and Associates
Hole Size: 4½ Inches

Driller: Chris Wyneken
Elevation: 10 Feet

Sheet: 1 of 1

DRAFT

Consolidation Test

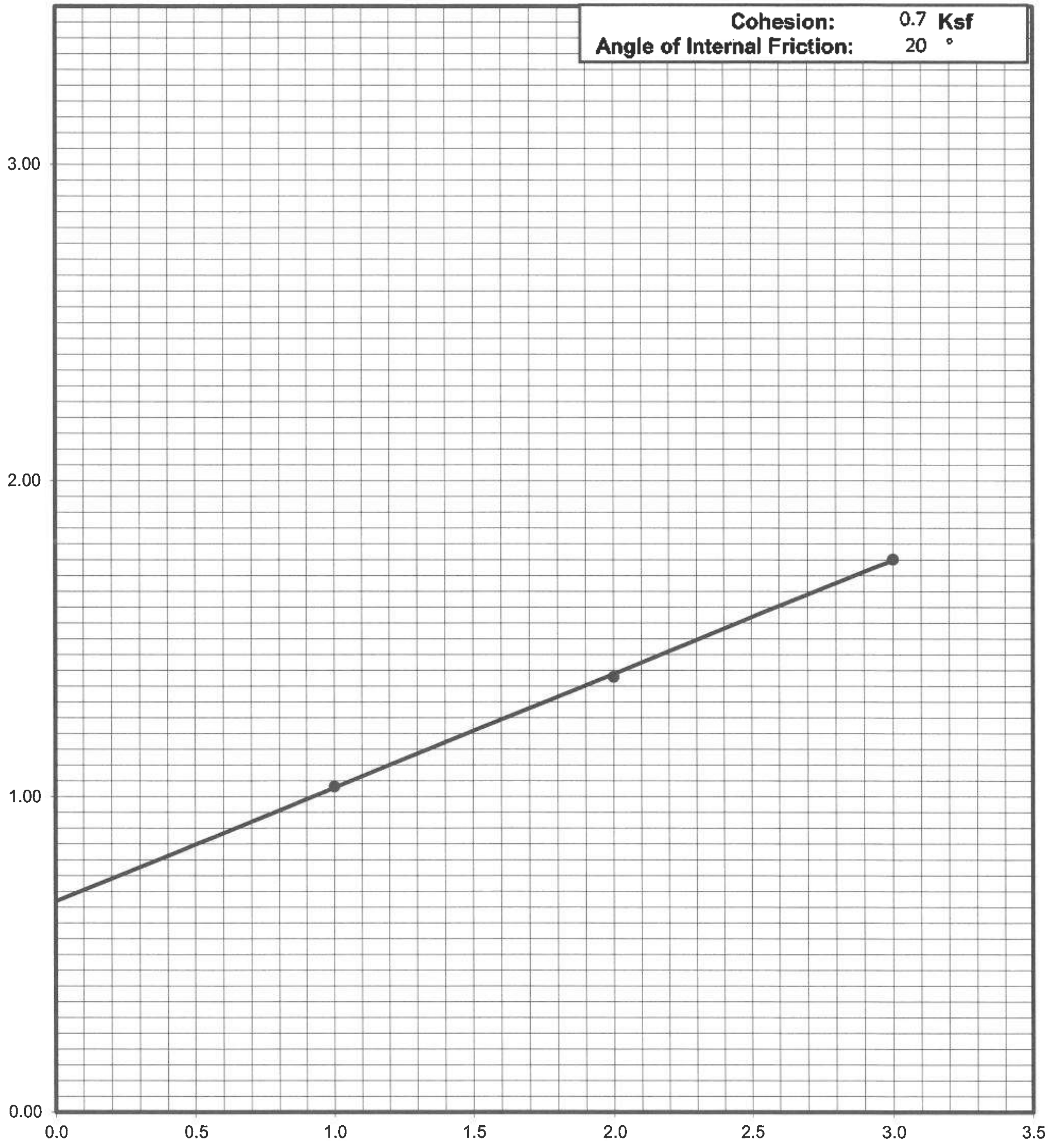
Project No	Boring No. & Depth	Date	Soil Classification
042-20001	B3 @ 2-3'	2/11/2020	CL/CH

Krazan Testing Laboratory

DRAFT

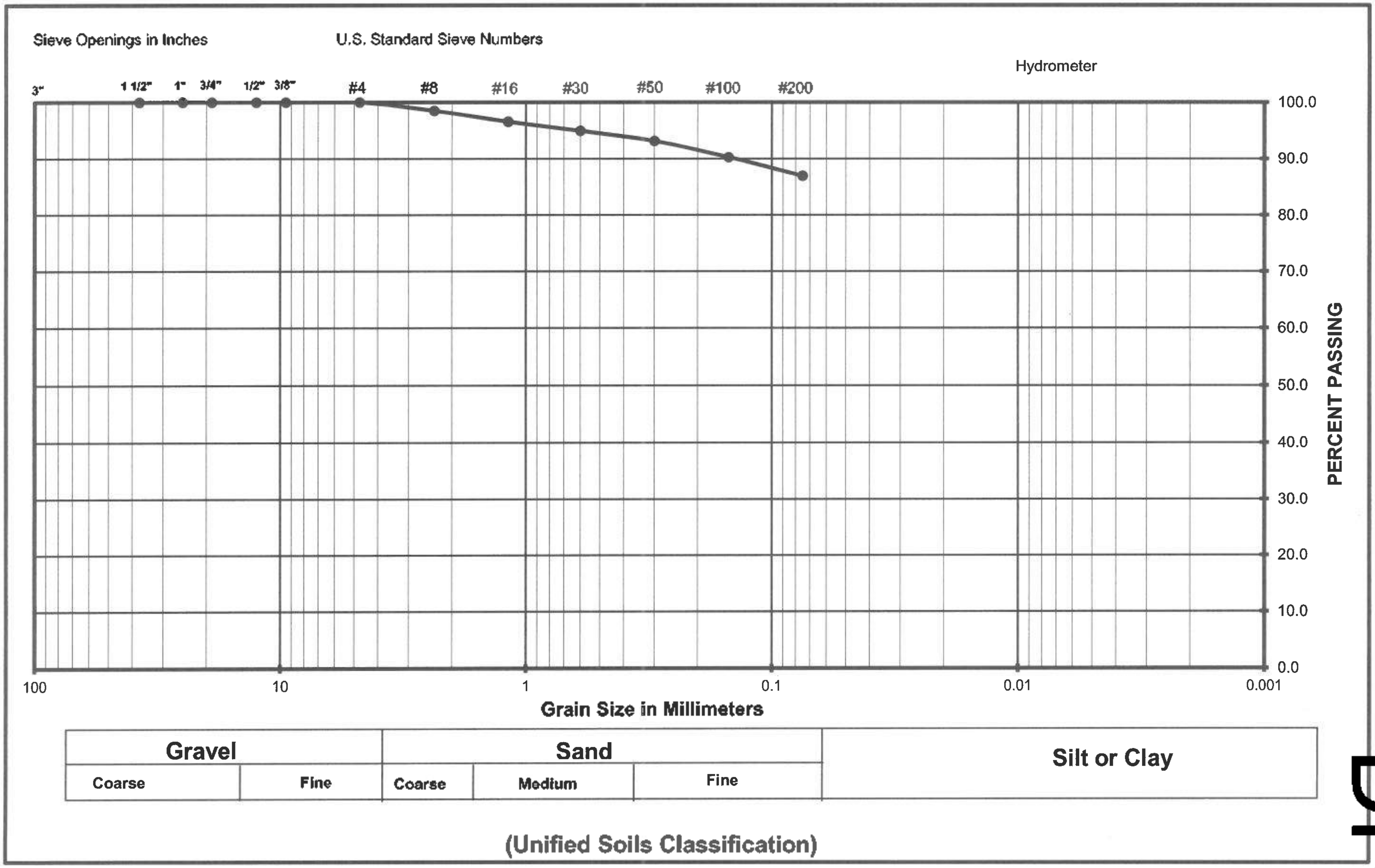
Shear Strength Diagram (Direct Shear)
ASTM D - 3080 / AASHTO T - 236

Project Number	Boring No. & Depth	Soil Type	Date
042-20001	B1 @ 2-3'	CL/CH	2/11/2020



Krazan Testing Laboratory

Grain Size Analysis



Project Name	New Charter School
Project Number	042-20001
Soil Classification	CL/CH
Sample Number	B3 @ 2-3'

Krazan Testing Laboratory

DRAFT

Expansion Index Test

ASTM D - 4829

Project Number : 042-20001
Project Name : New Charter School
Date : 2/11/2020
Sample location/ Depth : 0-4'
Sample Number : X1
Soil Classification : CH

Trial #	**1**	**2**	**3**
Weight of Soil & Mold, gms	716.8		
Weight of Mold, gms	368.4		
Weight of Soil, gms	348.4		
Wet Density, Lbs/cu.ft.	105.1		
Weight of Moisture Sample (Wet), gms	300.0		
Weight of Moisture Sample (Dry), gms	260.6		
Moisture Content, %	15.1		
Dry Density, Lbs/cu.ft.	91.3		
Specific Gravity of Soil	2.7		
Degree of Saturation, %	48.3		

Time	Inital	30 min	1 hr	6hrs	12 hrs	24 hrs
Dial Reading	0	--	--	--	--	0.1265

Expansion Index $_{\text{measured}}$ = 126.5

Expansion Index = 127

Expansion Potential Table	
Exp. Index	Potential Exp.
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

Krazan Testing Laboratory

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

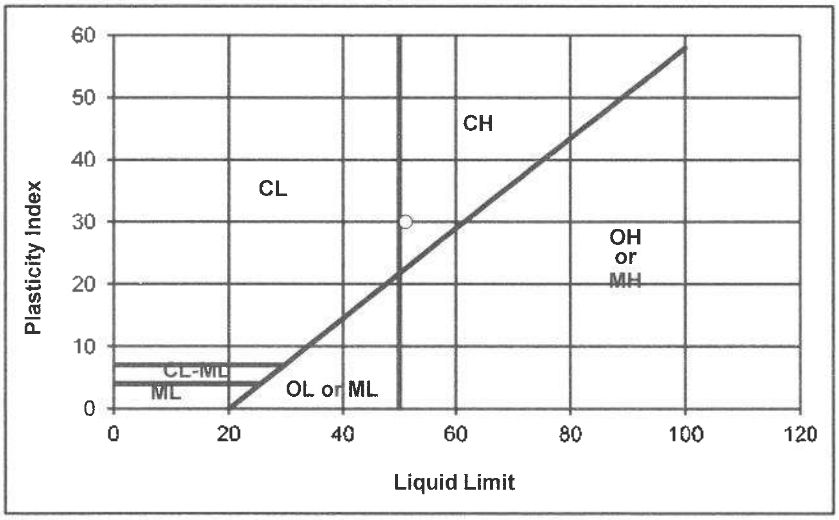
Project: **New Charter School**
Project Number: **042-20001**
Date Sampled: 1/22/2020
Sampled By: WA
Sample Number: X1
Sample Location: 0-4'
Sample Description: CL/CH

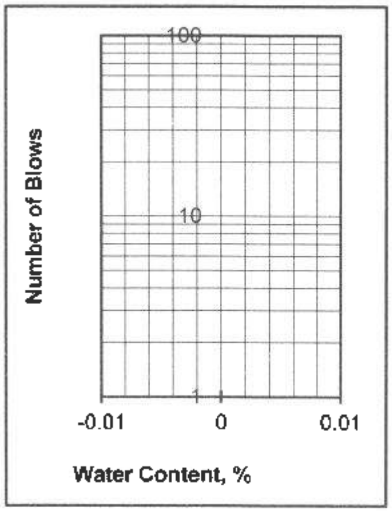
Date Tested: 2/10/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)	26.86	26.30		28.00	27.82	
Weight of Dry Soil & Tare (g)	24.90	24.75		24.31	22.74	
Weight of Tare (g)	15.66	17.00		17.03	12.89	
Weight of water (g)	1.97	1.56		3.69	5.08	
Weight of Dry Soil (g)	9.24	7.75		7.28	9.86	
Water Content (% of dry wt.)	21.3%	20.1%		50.7%	51.5%	
Number of Blows				25	25	

Plastic Limit : 21 **Liquid Limit : 51**

Plasticity Index : 30
Unified Soil Classification : CH **Requirement:**
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

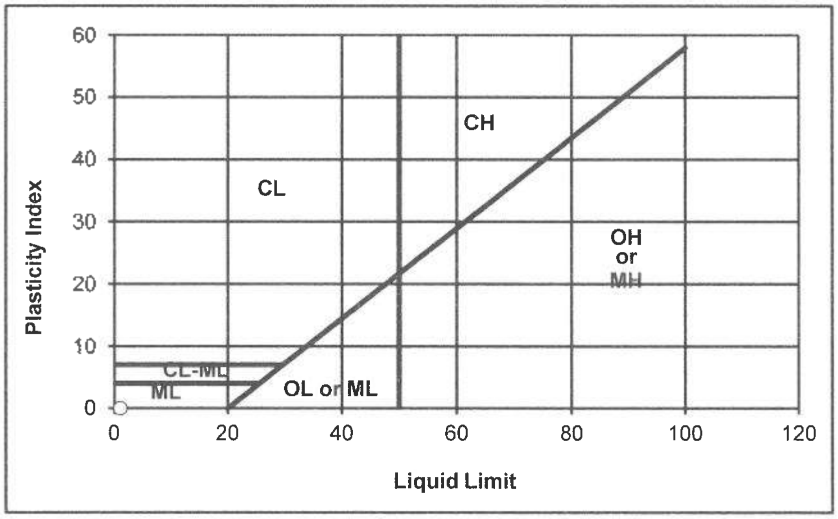
Project: **New Charter School**
Project Number: **042-20001**
Date Sampled: 1/22/2020
Sampled By: WA
Sample Number:
Sample Location: B2 @ 20-21
Sample Description: SM/SC w/ grvl

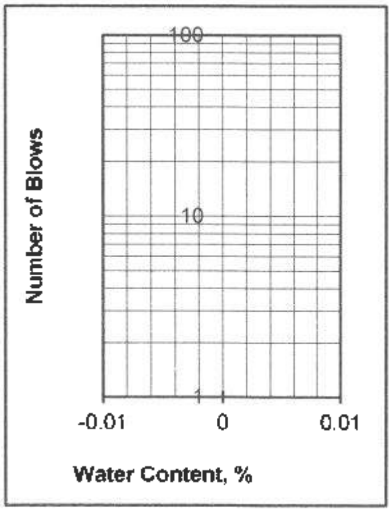
Date Tested: 2/10/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D **Liquid Limit : N/D**

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC **Requirement:**
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

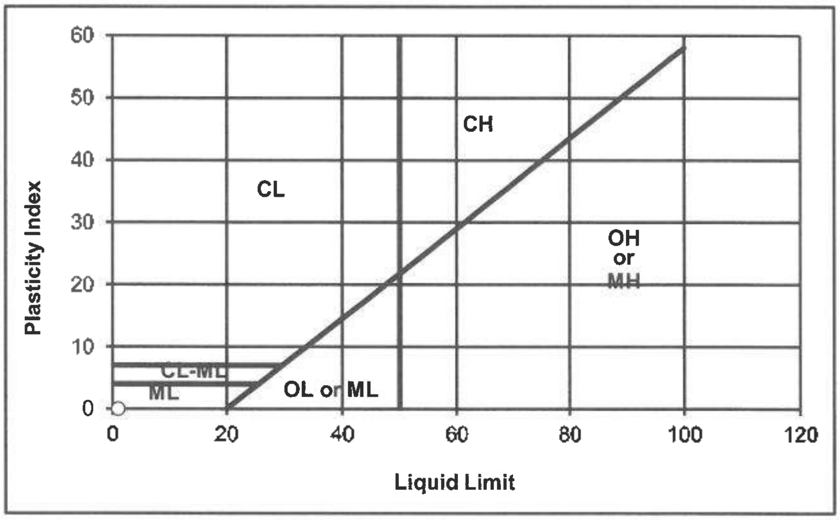
ASTM D4318/AASHTO T89 T90/CT 204

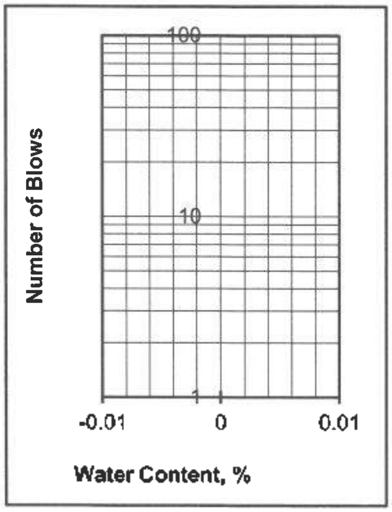
Project:	**New Charter School**		
Project Number:	**042-20001**		
Date Sampled:	1/22/2020	Date Tested:	2/10/2020
Sampled By:	WA	Tested By:	J Mitchell
Sample Number:		Verified By:	J Gruszczynski
Sample Location:	B2 @ 25-26'		
Sample Description:	SM w/ grvl		

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D **Liquid Limit : N/D**

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC **Requirement:**
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

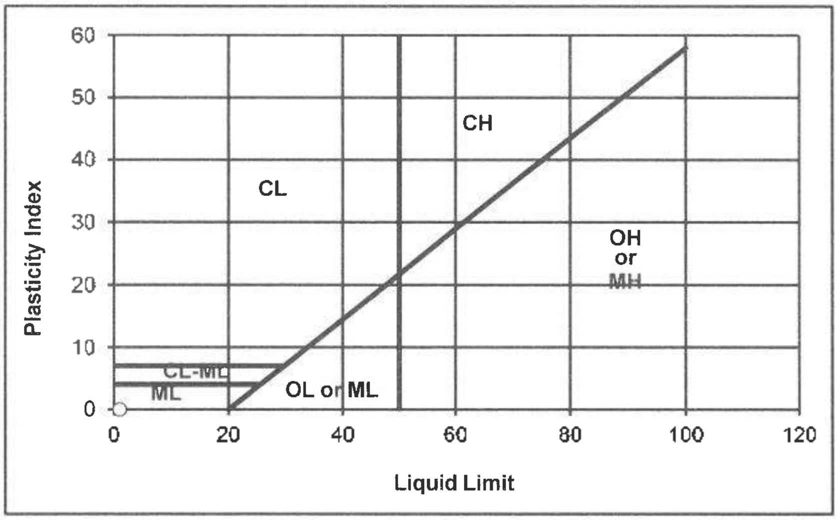
Project: **New Charter School**
Project Number: **042-20001**
Date Sampled: 1/22/2020
Sampled By: WA
Sample Number:
Sample Location: B2 @ 30-31'
Sample Description: SM/SC w/ grvl

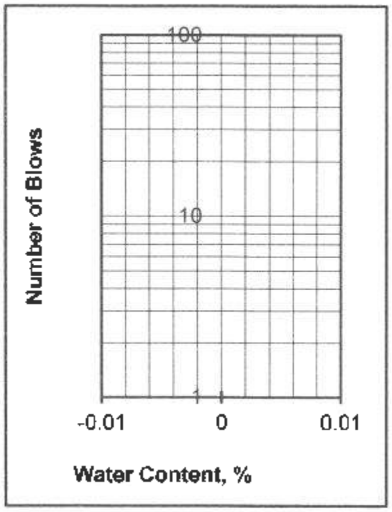
Date Tested: 2/10/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D **Liquid Limit : N/D**

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC
Requirement:
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

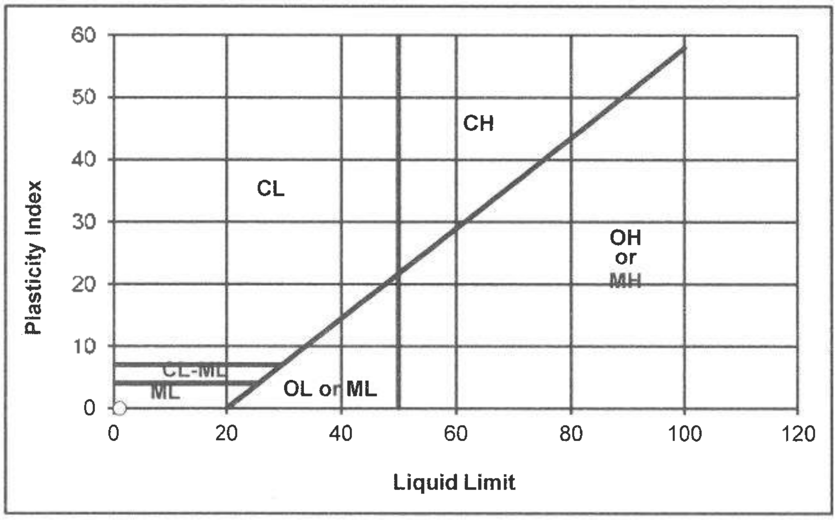
Project: **New Charter School**
Project Number: **042-20001**
Date Sampled: 1/22/2020
Sampled By: WA
Sample Number:
Sample Location: B2 @ 35-36'
Sample Description: SM/SC w/ grvl

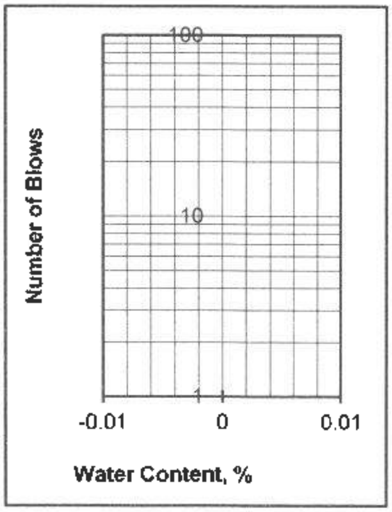
Date Tested: 2/10/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D **Liquid Limit : N/D**

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC **Requirement:**
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

Project: **New Charter School**
Project Number: **042-20001**
Date Sampled: 1/22/2020
Sampled By: WA
Sample Number:
Sample Location: B2 @ 40-41'
Sample Description: SM/SC w/ grvl

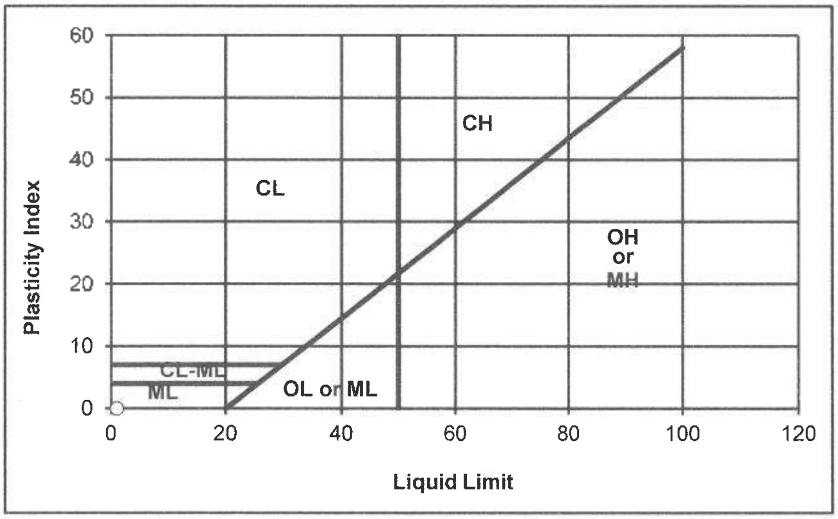
Date Tested: 2/10/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

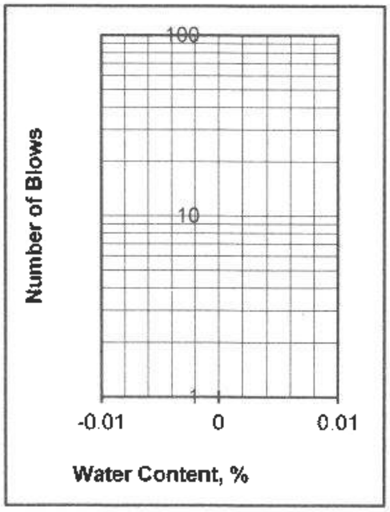
	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D

Liquid Limit : N/D

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC
Requirement:
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

DRAFT

Plasticity Index of Soils

ASTM D4318/AASHTO T89 T90/CT 204

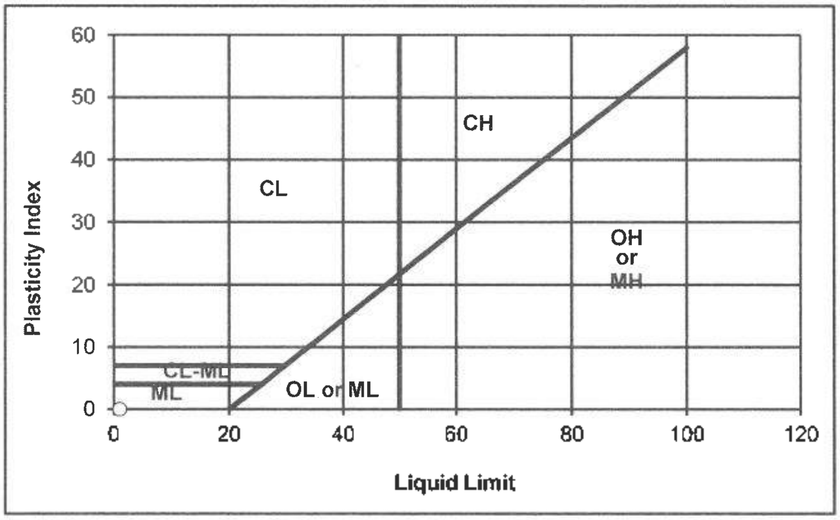
Project: **New Charter School**
Project Number: **042-20001**
Date Sampled: 1/22/2020
Sampled By: WA
Sample Number:
Sample Location: B2 @ 45-46'
Sample Description: SM/SC w/ grvl

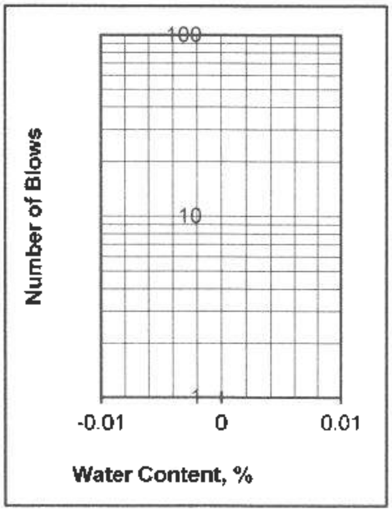
Date Tested: 2/10/2020
Tested By: J Mitchell
Verified By: J Gruszczynski

	Plastic Limit			Liquid Limit		
Trial Number	1	2	3	1	2	3
Weight of Wet Soil & Tare (g)						
Weight of Dry Soil & Tare (g)						
Weight of Tare (g)						
Weight of water (g)						
Weight of Dry Soil (g)						
Water Content (% of dry wt.)						
Number of Blows						

Plastic Limit : N/D **Liquid Limit : N/D**

Plasticity Index : NON-PLASTIC
Unified Soil Classification : NON-PLASTIC **Requirement:**
Approx. % of Material Retained on # 40 Sieve:





Departures from Outlined Procedure:

Unusual Conditions, Other Notes:

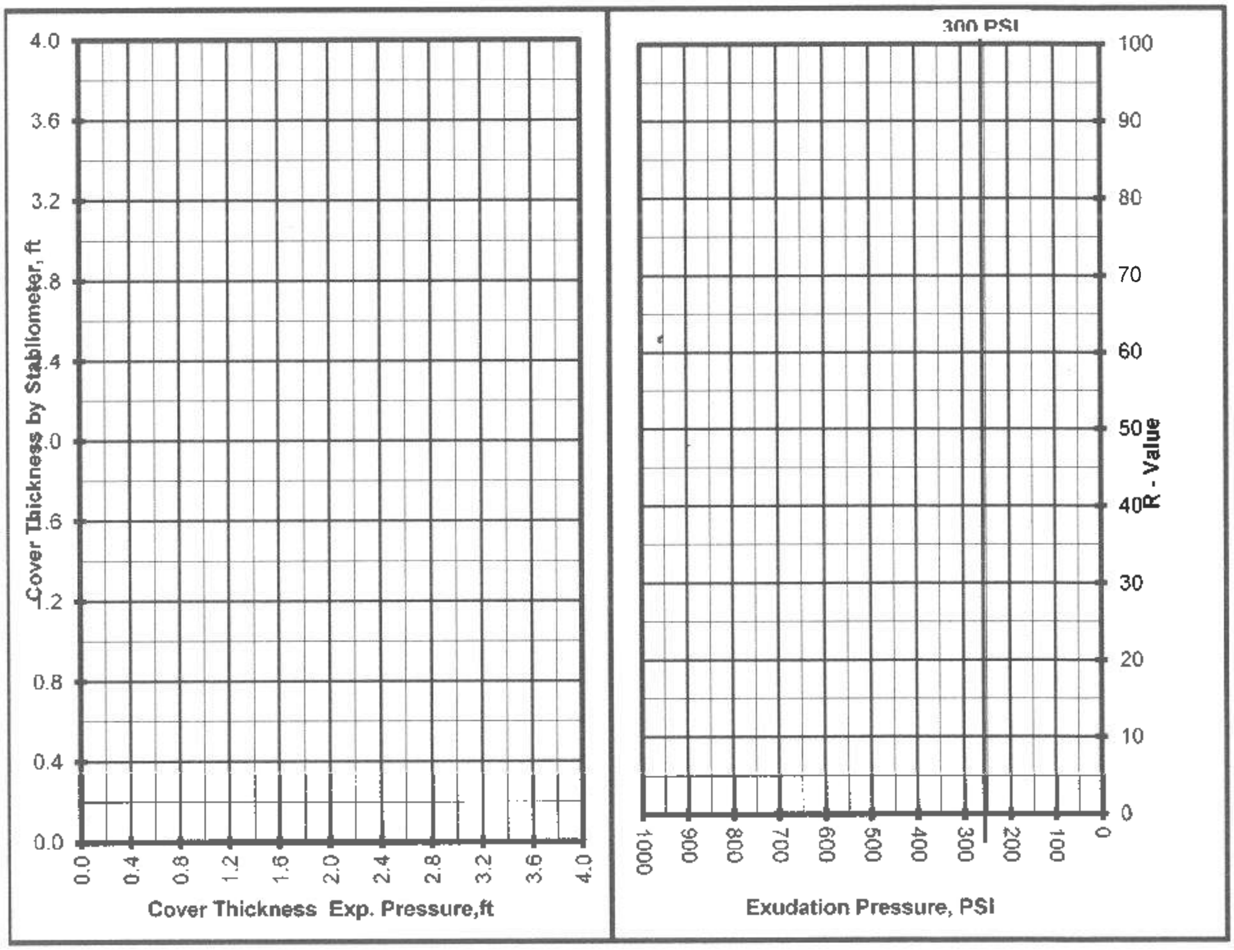
DRAFT

R - VALUE TEST
ASTM D - 2844 / CAL 301

Project Number : 042-20001
Project Name : New Charter School
Date : 1/28/2020
Sample Location/Curve Number : RV#1
Soil Classification : CL

TEST	A	B	C
Percent Moisture @ Compaction, %			
Dry Density, lbm/cu.ft.	R - Value less than 5 Sample Exuded from bottom of Mold During test		
Exudation Pressure, psi			
Expansion Pressure, (Dial Reading)			
Expansion Pressure, psf			
Resistance Value R			

R - Value at 300 PSI Exudation Pressure	< 5
R - Value by Expansion Pressure	



APPENDIX D

SEISMIC HAZARD DISAGGREGATION

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

37.63804875435438

Longitude

Decimal degrees, negative values for western longitudes

-122.05263934034456

Site Class

360 m/s (C/D boundary)

Spectral Period

Peak Ground Acceleration

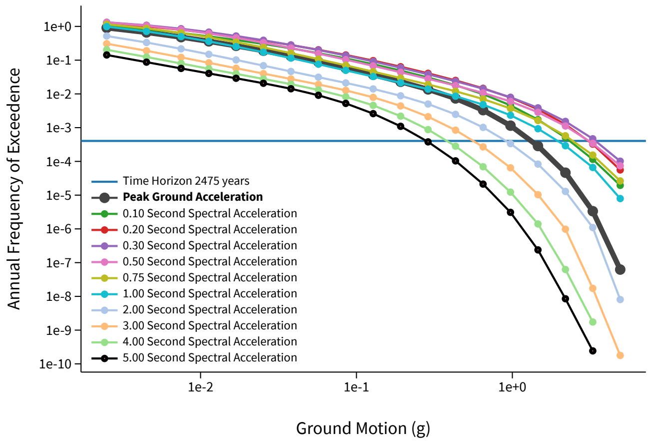
Time Horizon

Return period in years

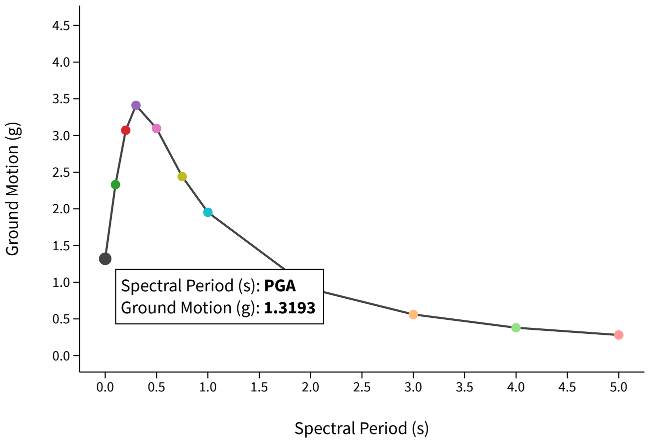
2475

Hazard Curve

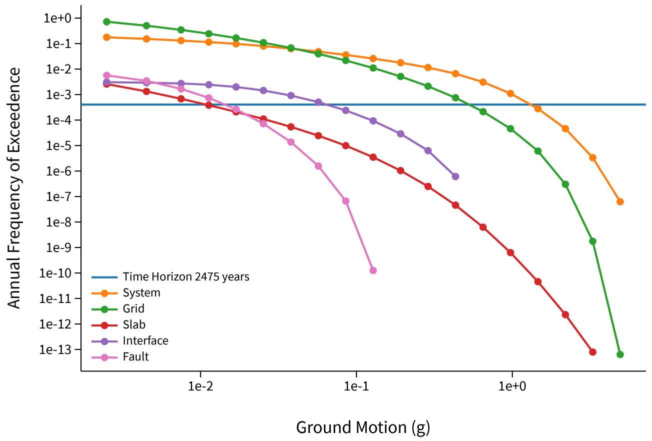
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

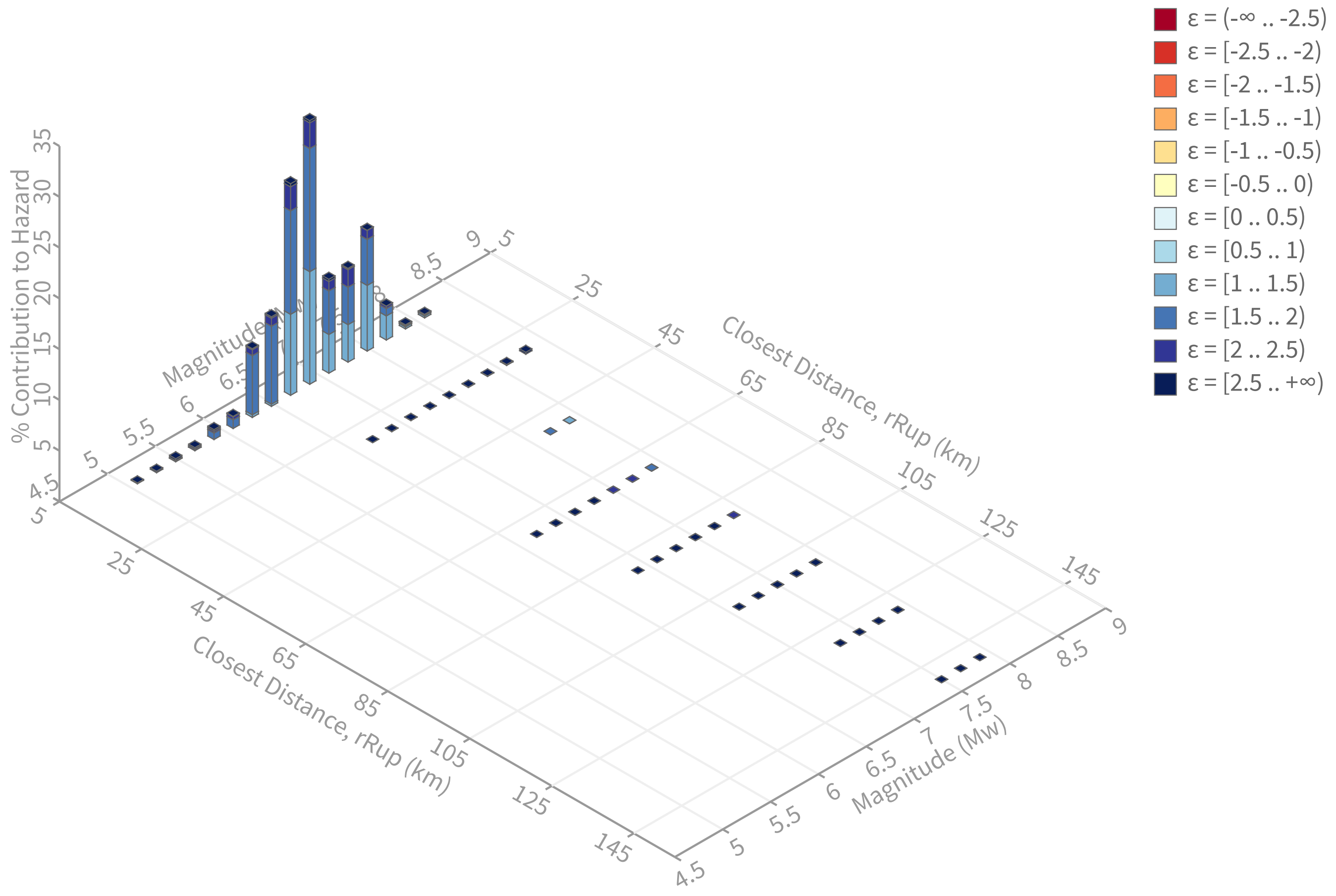


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 1.3192885 g

Recovered targets

Return period: 2959.5834 yrs
Exceedance rate: 0.0003378854 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.02 %

Mean (over all sources)

m: 6.91
r: 3.94 km
ε_0: 1.68 σ

Mode (largest m-r bin)

m: 6.87
r: 3.75 km
ε_0: 1.67 σ
Contribution: 25.95 %

Mode (largest m-r-ε_0 bin)

m: 6.87
r: 3.5 km
ε_0: 1.72 σ
Contribution: 12.12 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							49.46
Hayward (So) [5]		3.52	7.00	1.61	122.043°W	37.644°N	49.84	32.78
Hayward (So) [4]		3.65	6.74	1.69	122.036°W	37.638°N	91.88	7.80
Mission (connected) [0]		1.13	6.72	1.51	122.043°W	37.633°N	125.08	5.00
Calaveras (No) [3]		12.69	7.21	2.34	121.940°W	37.681°N	64.15	1.58
Hayward (So) [3]		8.49	6.76	2.33	121.989°W	37.590°N	133.70	1.11
UC33brAvg_FM32	System							48.86
Hayward (So) [5]		3.52	6.99	1.61	122.043°W	37.644°N	49.84	33.13
Hayward (So) [4]		3.65	6.74	1.69	122.036°W	37.638°N	91.88	7.69
Mission (connected) [0]		1.13	6.71	1.51	122.043°W	37.633°N	125.08	4.28
Calaveras (No) [3]		12.69	7.20	2.34	121.940°W	37.681°N	64.15	1.61
Hayward (So) [3]		8.49	6.75	2.33	121.989°W	37.590°N	133.70	1.04

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

⌃ Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

37.63804875435438

Longitude

Decimal degrees, negative values for western longitudes

-122.05263934034456

Site Class

360 m/s (C/D boundary)

Spectral Period

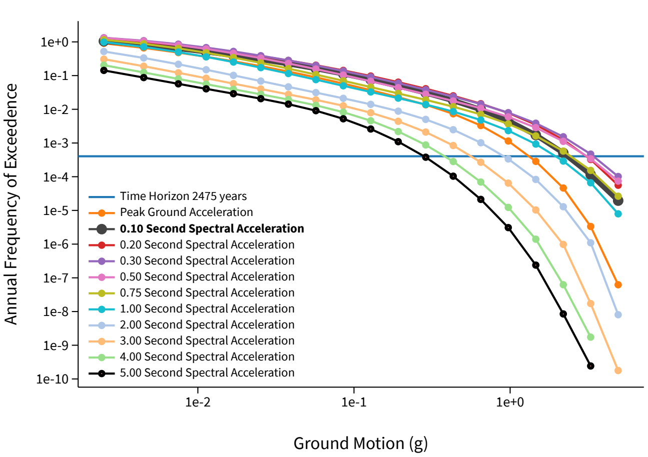
0.10 Second Spectral Acceleration

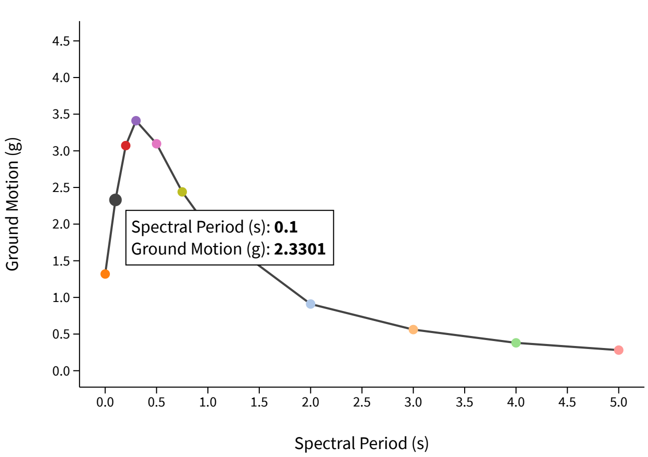
Time Horizon

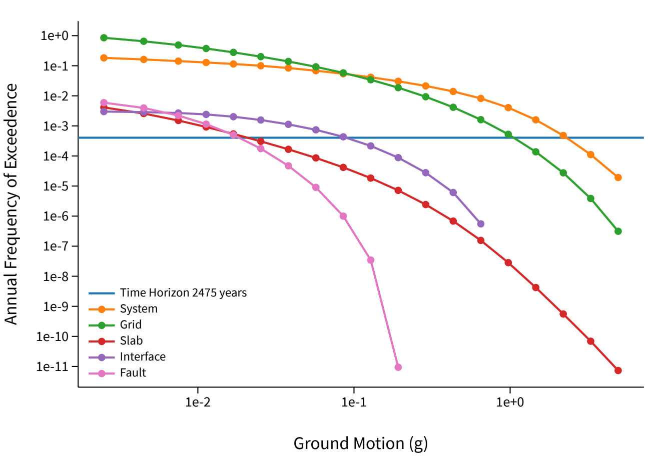
Return period in years

2475

Hazard Curve


Hazard Curves


Uniform Hazard Response Spectrum

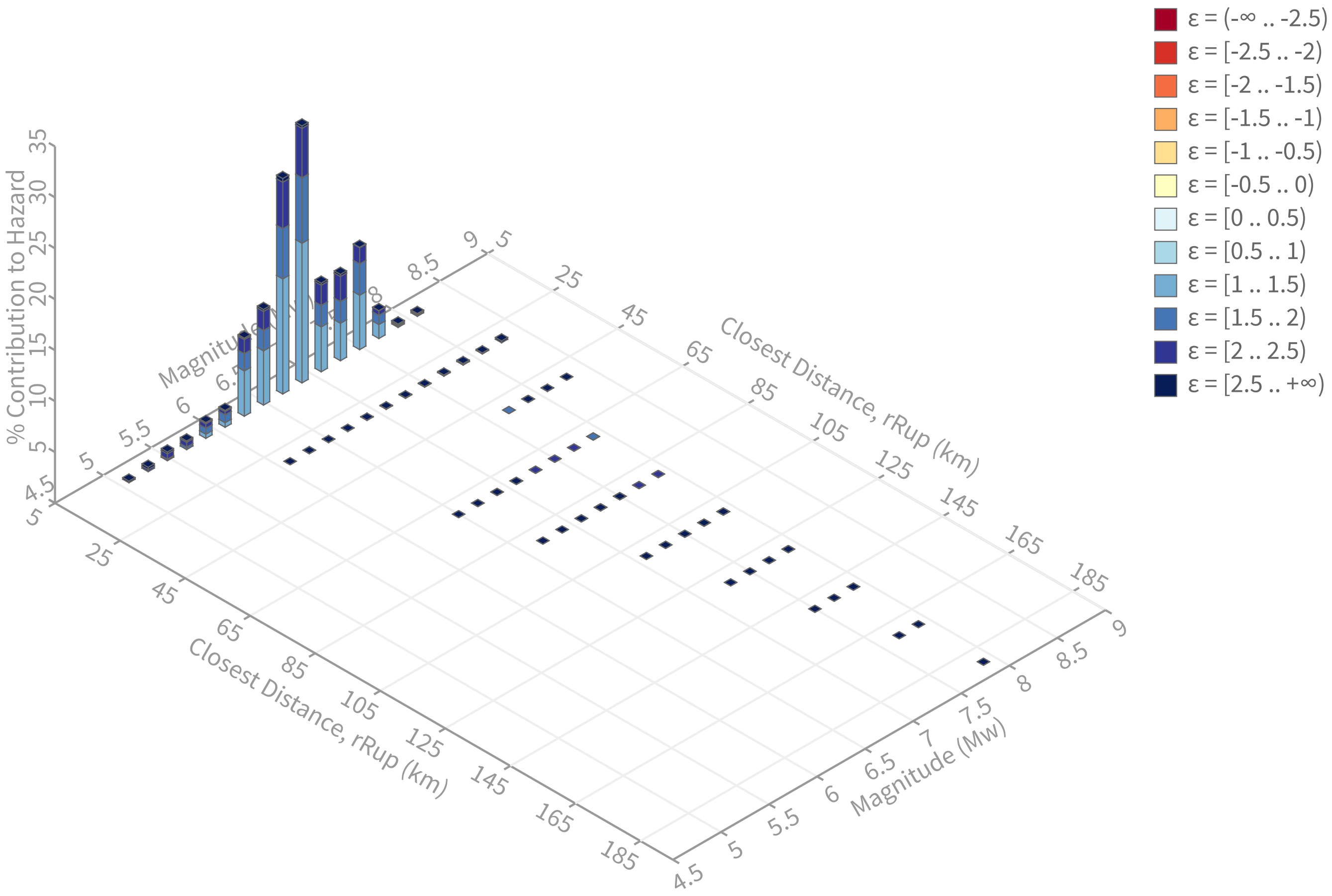

Component Curves for 0.10 Second Spectral Acceleration

View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
0.1 s SA ground motion: 2.3301167 g

Recovered targets

Return period: 2953.0613 yrs
Exceedance rate: 0.00033863165 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.02 %

Mean (over all sources)

m: 6.86
r: 4.38 km
ε_0: 1.66 σ

Mode (largest m-r bin)

m: 6.87
r: 3.94 km
ε_0: 1.62 σ
Contribution: 25.21 %

Mode (largest m-r-ε_0 bin)

m: 6.87
r: 3.37 km
ε_0: 1.34 σ
Contribution: 13.71 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							48.26
Hayward (So) [5]		3.52	6.97	1.57	122.043°W	37.644°N	49.84	30.52
Hayward (So) [4]		3.65	6.73	1.60	122.036°W	37.638°N	91.88	7.80
Mission (connected) [0]		1.13	6.66	1.51	122.043°W	37.633°N	125.08	4.54
Calaveras (No) [3]		12.69	7.14	2.19	121.940°W	37.681°N	64.15	2.13
Hayward (So) [3]		8.49	6.73	2.16	121.989°W	37.590°N	133.70	1.60
UC33brAvg_FM32	System							47.72
Hayward (So) [5]		3.52	6.97	1.57	122.043°W	37.644°N	49.84	30.87
Hayward (So) [4]		3.65	6.72	1.60	122.036°W	37.638°N	91.88	7.71
Mission (connected) [0]		1.13	6.66	1.51	122.043°W	37.633°N	125.08	3.89
Calaveras (No) [3]		12.69	7.12	2.19	121.940°W	37.681°N	64.15	2.19
Hayward (So) [3]		8.49	6.72	2.16	121.989°W	37.590°N	133.70	1.51
UC33brAvg_FM32 (opt)	Grid							2.01
UC33brAvg_FM31 (opt)	Grid							2.01

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

37.63804875435438

Longitude

Decimal degrees, negative values for western longitudes

-122.05263934034456

Site Class

360 m/s (C/D boundary)

Spectral Period

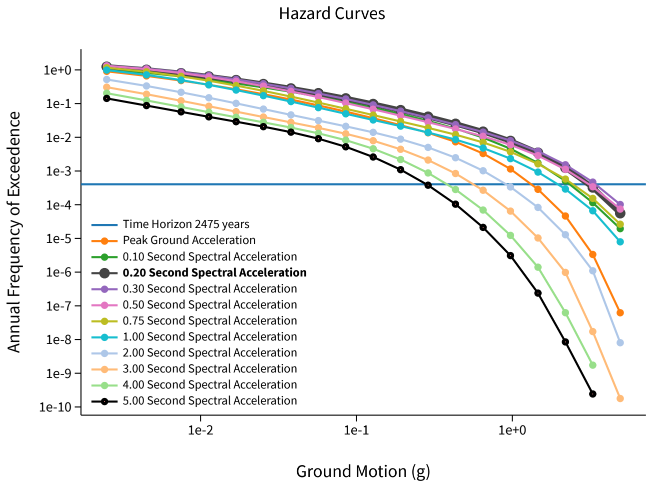
0.20 Second Spectral Acceleration

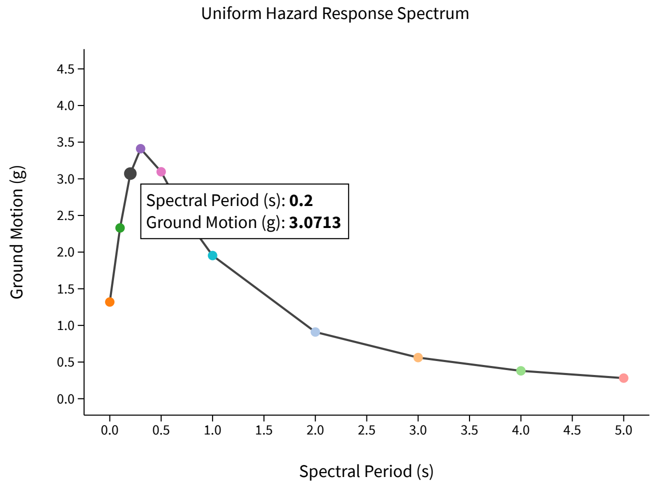
Time Horizon

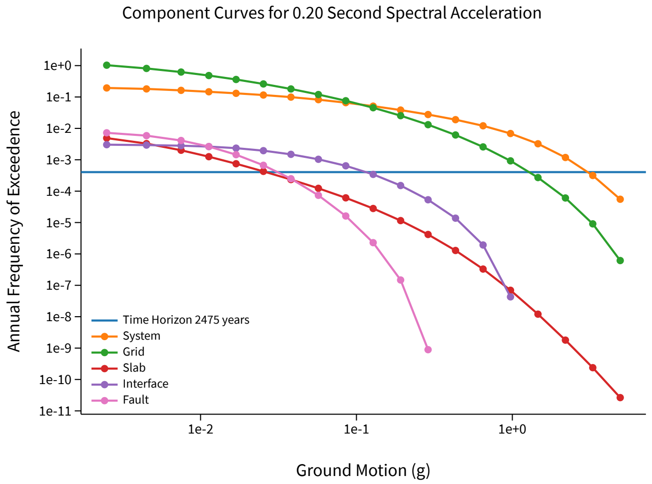
Return period in years

2475

Hazard Curve


Hazard Curves


Uniform Hazard Response Spectrum

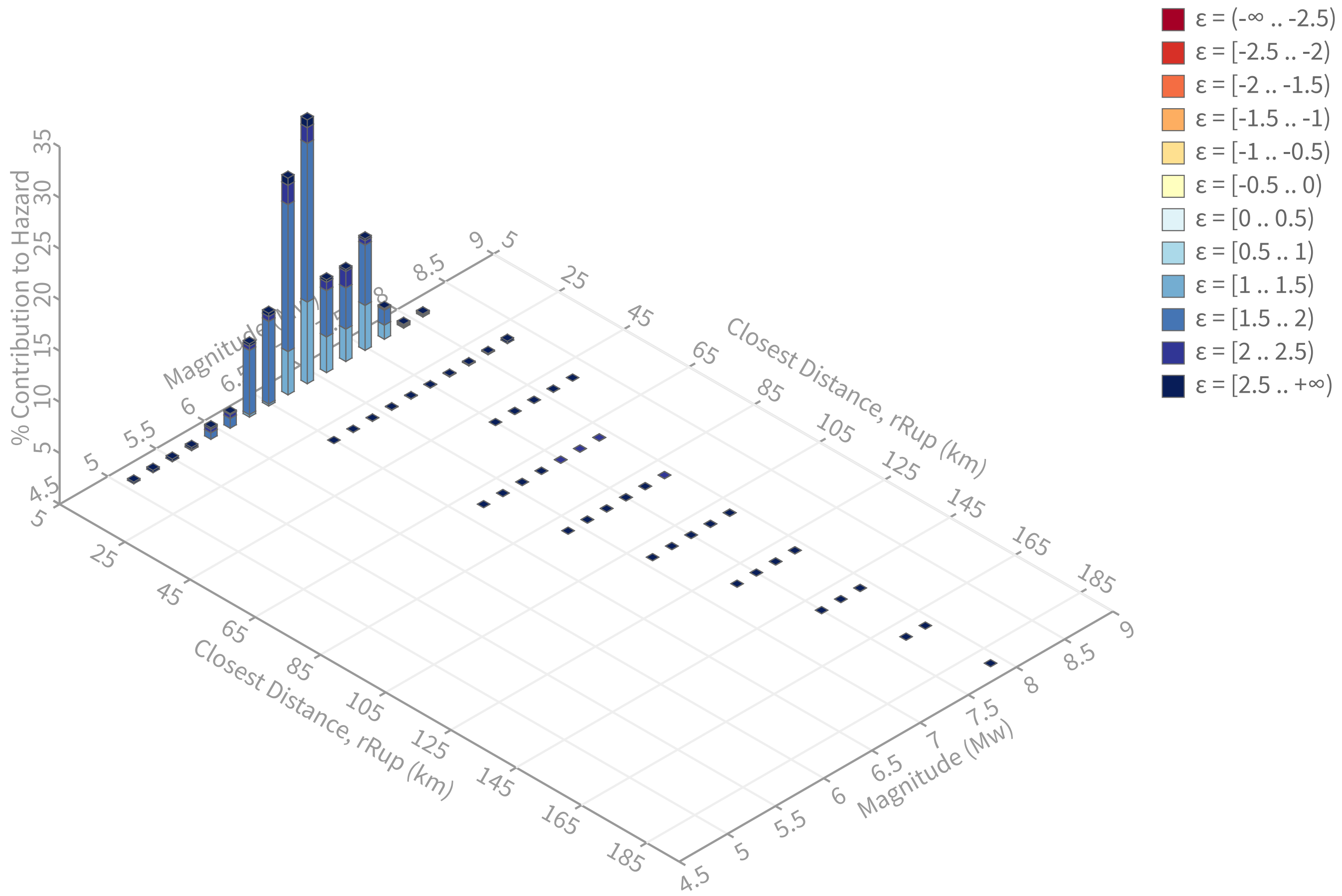

Component Curves for 0.20 Second Spectral Acceleration

View Raw Data

^ Deaggregation

Component

Total ⌄



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
0.2 s SA ground motion: 3.0712941 g

Recovered targets

Return period: 2973.8967 yrs
Exceedance rate: 0.00033625915 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.03 %

Mean (over all sources)

m: 6.9
r: 4.2 km
ε_0: 1.69 σ

Mode (largest m-r bin)

m: 6.87
r: 3.88 km
ε_0: 1.67 σ
Contribution: 25.84 %

Mode (largest m-r-ε_0 bin)

m: 6.87
r: 3.59 km
ε_0: 1.66 σ
Contribution: 15.53 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							49.16
Hayward (So) [5]		3.52	6.98	1.61	122.043°W	37.644°N	49.84	31.63
Hayward (So) [4]		3.65	6.74	1.67	122.036°W	37.638°N	91.88	7.80
Mission (connected) [0]		1.13	6.70	1.52	122.043°W	37.633°N	125.08	4.71
Calaveras (No) [3]		12.69	7.16	2.32	121.940°W	37.681°N	64.15	2.00
Hayward (So) [3]		8.49	6.74	2.26	121.989°W	37.590°N	133.70	1.49
UC33brAvg_FM32	System							48.59
Hayward (So) [5]		3.52	6.98	1.61	122.043°W	37.644°N	49.84	31.97
Hayward (So) [4]		3.65	6.73	1.67	122.036°W	37.638°N	91.88	7.70
Mission (connected) [0]		1.13	6.70	1.52	122.043°W	37.633°N	125.08	4.03
Calaveras (No) [3]		12.69	7.15	2.32	121.940°W	37.681°N	64.15	2.05
Hayward (So) [3]		8.49	6.73	2.26	121.989°W	37.590°N	133.70	1.40
UC33brAvg_FM32 (opt)	Grid							1.13
UC33brAvg_FM31 (opt)	Grid							1.13

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

37.63804875435438

Longitude

Decimal degrees, negative values for western longitudes

-122.05263934034456

Site Class

360 m/s (C/D boundary)

Spectral Period

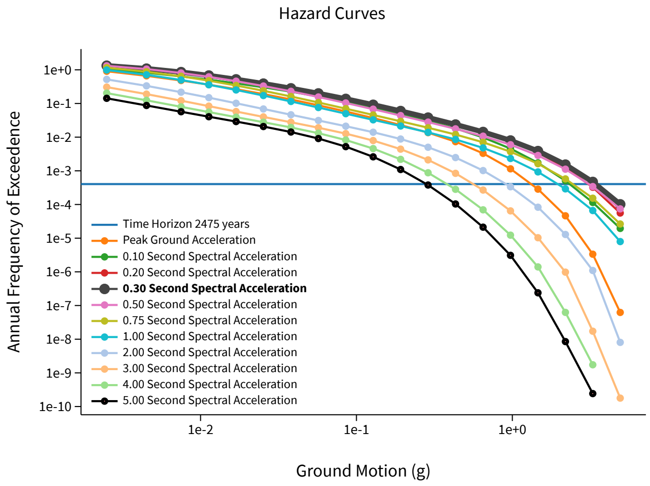
0.30 Second Spectral Acceleration

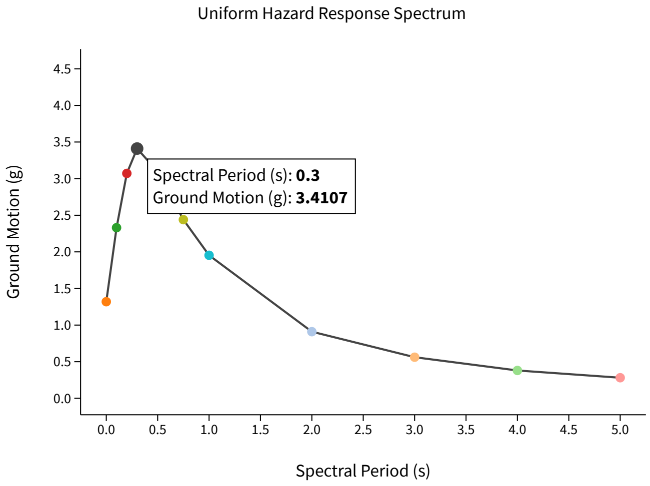
Time Horizon

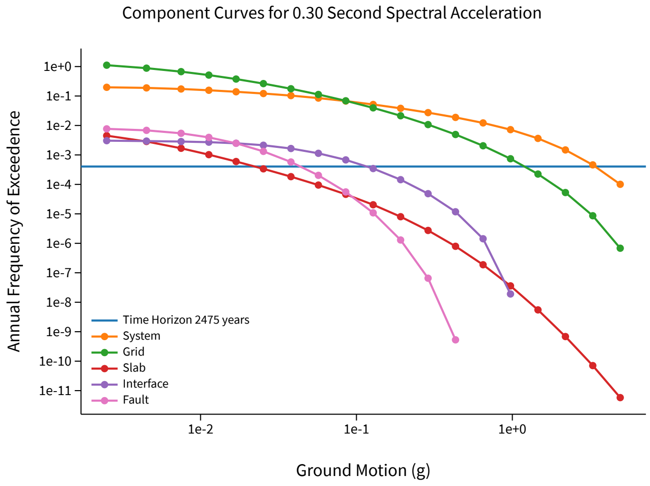
Return period in years

2475

Hazard Curve


Hazard Curves


Uniform Hazard Response Spectrum

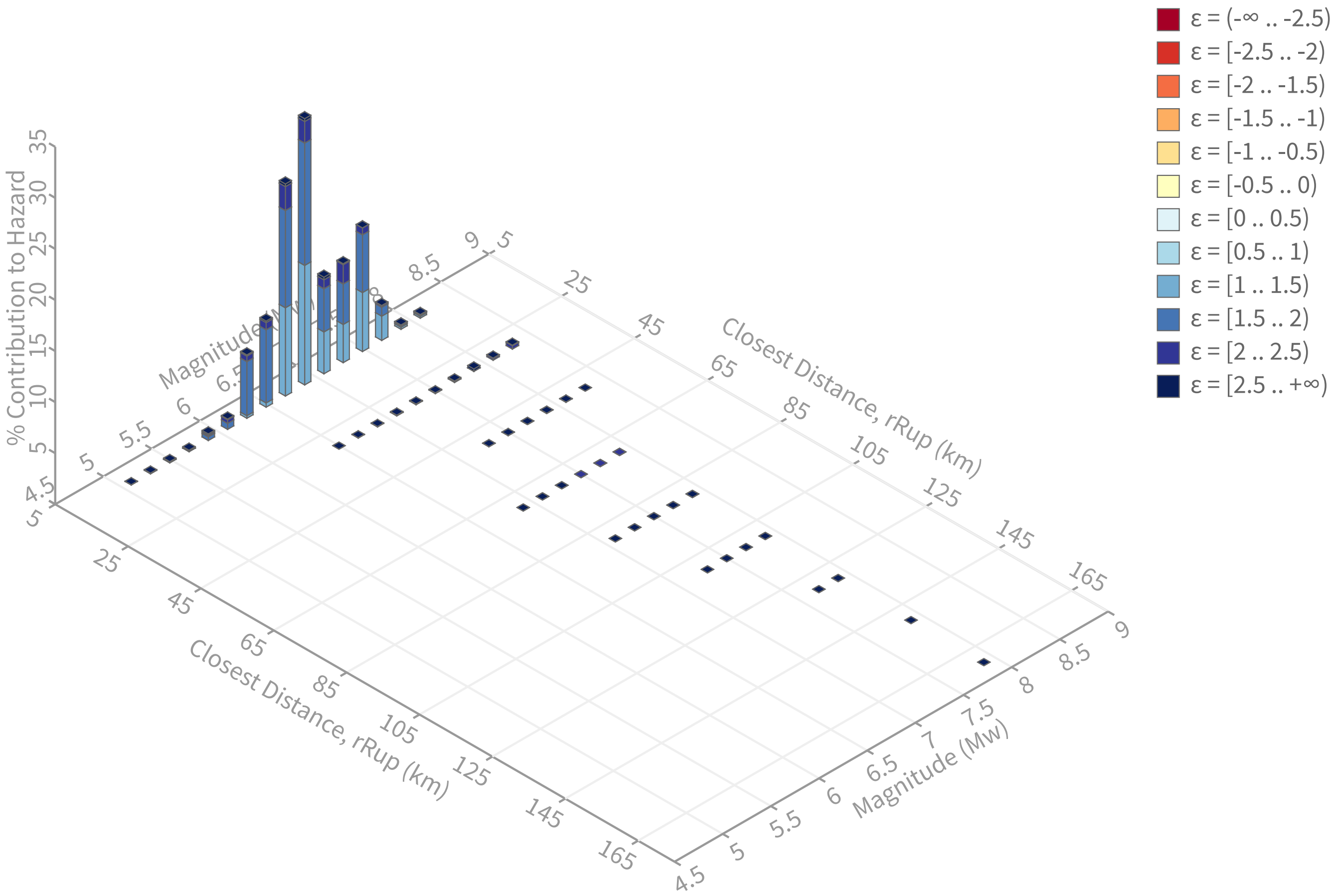

Component Curves for 0.30 Second Spectral Acceleration

View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
0.3 s SA ground motion: 3.4106759 g

Recovered targets

Return period: 2918.4896 yrs
Exceedance rate: 0.00034264299 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.02 %

Mean (over all sources)

m: 6.94
r: 4.24 km
ε_0: 1.66 σ

Mode (largest m-r bin)

m: 6.87
r: 3.82 km
ε_0: 1.65 σ
Contribution: 26.06 %

Mode (largest m-r-ε_0 bin)

m: 6.87
r: 3.65 km
ε_0: 1.74 σ
Contribution: 11.98 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							49.74
Hayward (So) [5]		3.52	7.00	1.59	122.043°W	37.644°N	49.84	32.44
Hayward (So) [4]		3.65	6.75	1.68	122.036°W	37.638°N	91.88	7.57
Mission (connected) [0]		1.13	6.75	1.47	122.043°W	37.633°N	125.08	4.78
Calaveras (No) [3]		12.69	7.20	2.27	121.940°W	37.681°N	64.15	1.87
Hayward (So) [3]		8.49	6.76	2.24	121.989°W	37.590°N	133.70	1.33
UC33brAvg_FM32	System							49.12
Hayward (So) [5]		3.52	7.00	1.59	122.043°W	37.644°N	49.84	32.77
Hayward (So) [4]		3.65	6.75	1.69	122.036°W	37.638°N	91.88	7.46
Mission (connected) [0]		1.13	6.75	1.47	122.043°W	37.633°N	125.08	4.08
Calaveras (No) [3]		12.69	7.19	2.27	121.940°W	37.681°N	64.15	1.91
Hayward (So) [3]		8.49	6.76	2.24	121.989°W	37.590°N	133.70	1.24

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

37.63804875435438

Longitude

Decimal degrees, negative values for western longitudes

-122.05263934034456

Site Class

360 m/s (C/D boundary)

Spectral Period

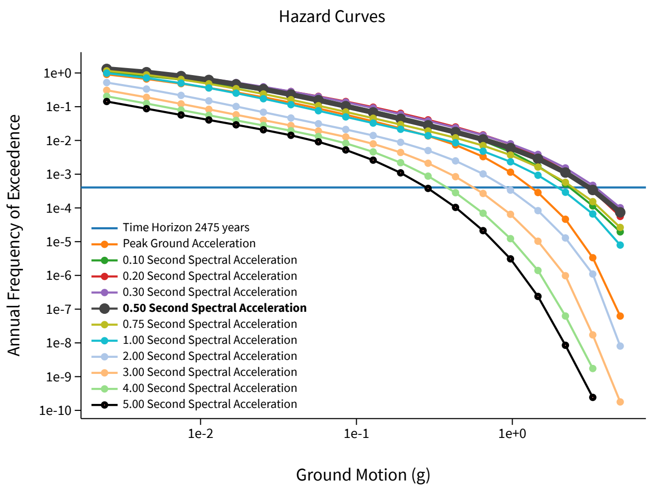
0.50 Second Spectral Acceleration

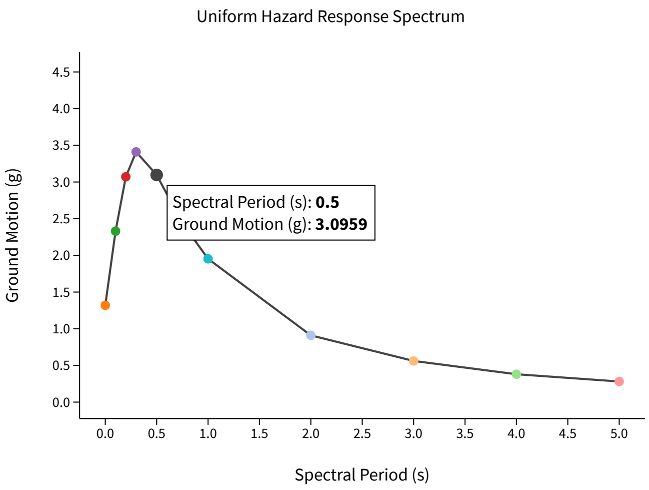
Time Horizon

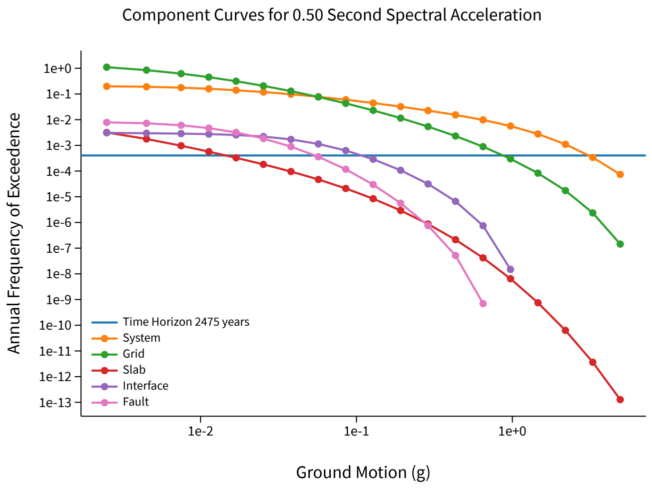
Return period in years

2475

Hazard Curve





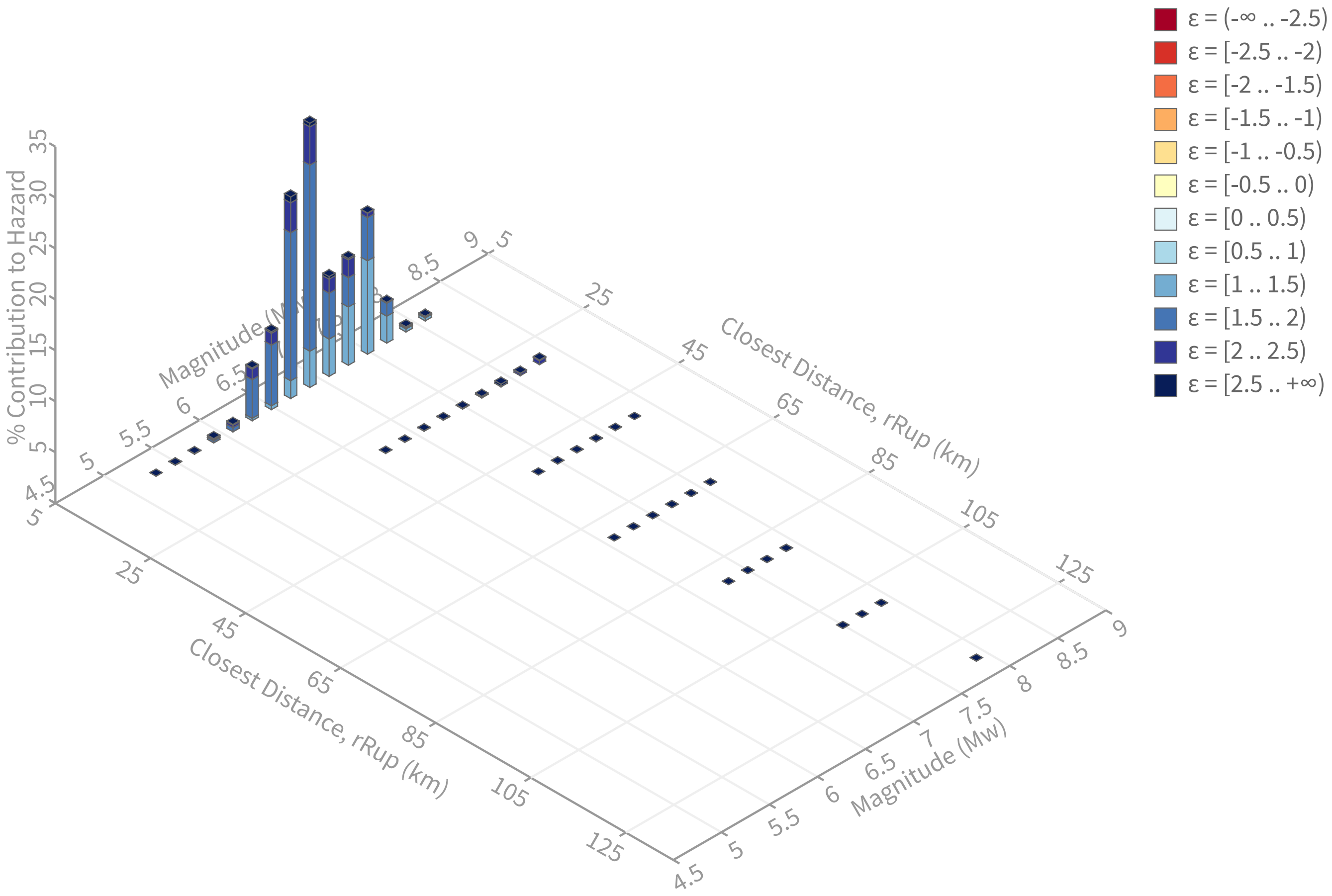


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
0.5 s SA ground motion: 3.0959132 g

Recovered targets

Return period: 2856.4747 yrs
Exceedance rate: 0.00035008186 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.03 %

Mean (over all sources)

m: 6.99
r: 4.19 km
ε_0: 1.69 σ

Mode (largest m-r bin)

m: 6.88
r: 3.77 km
ε_0: 1.7 σ
Contribution: 25.92 %

Mode (largest m-r-ε_0 bin)

m: 6.87
r: 3.53 km
ε_0: 1.65 σ
Contribution: 18.26 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							50.04
Hayward (So) [5]		3.52	7.04	1.62	122.043°W	37.644°N	49.84	33.29
Hayward (So) [4]		3.65	6.77	1.74	122.036°W	37.638°N	91.88	7.28
Mission (connected) [0]		1.13	6.80	1.48	122.043°W	37.633°N	125.08	4.79
Calaveras (No) [3]		12.69	7.25	2.33	121.940°W	37.681°N	64.15	1.76
Hayward (So) [3]		8.49	6.78	2.33	121.989°W	37.590°N	133.70	1.19
UC33brAvg_FM32	System							49.41
Hayward (So) [5]		3.52	7.04	1.62	122.043°W	37.644°N	49.84	33.63
Hayward (So) [4]		3.65	6.77	1.75	122.036°W	37.638°N	91.88	7.17
Mission (connected) [0]		1.13	6.80	1.48	122.043°W	37.633°N	125.08	4.09
Calaveras (No) [3]		12.69	7.24	2.34	121.940°W	37.681°N	64.15	1.78
Hayward (So) [3]		8.49	6.78	2.33	121.989°W	37.590°N	133.70	1.11

Unified Hazard Tool

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Input

Edition: Dynamic: Conterminous U.S. 2014 (u…

Spectral Period: 0.75 Second Spectral Acceleration

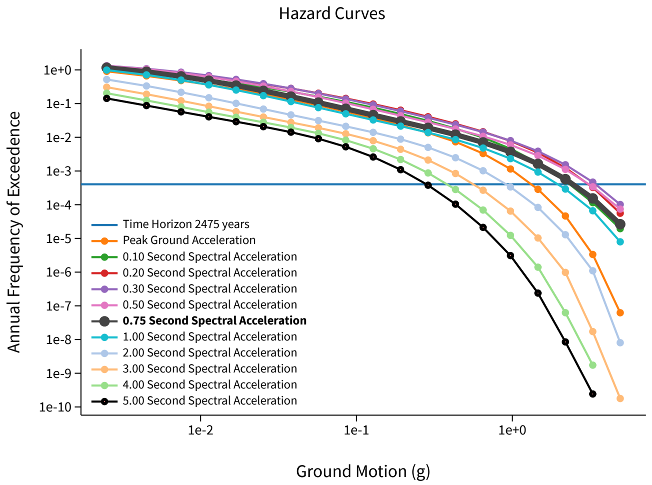
Latitude
Decimal degrees
37.63804875435438

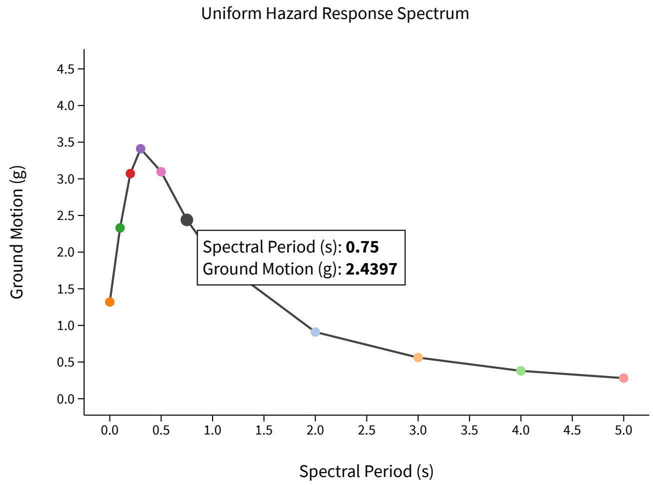
Time Horizon
Return period in years
2475

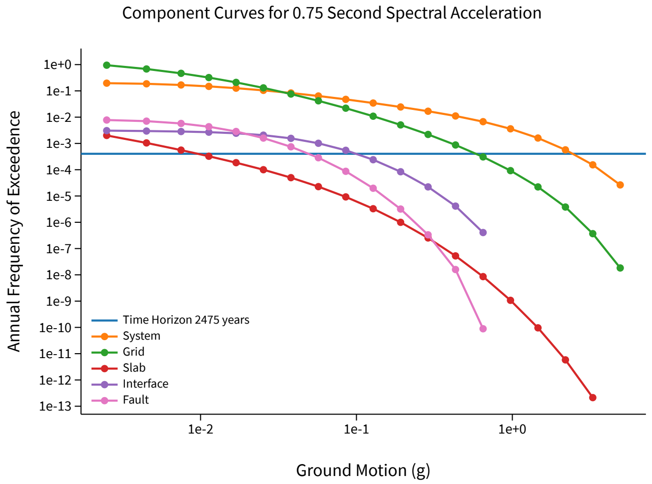
Longitude
Decimal degrees, negative values for western longitudes
-122.05263934034456

Site Class: 360 m/s (C/D boundary)

Hazard Curve


Hazard Curves


Uniform Hazard Response Spectrum

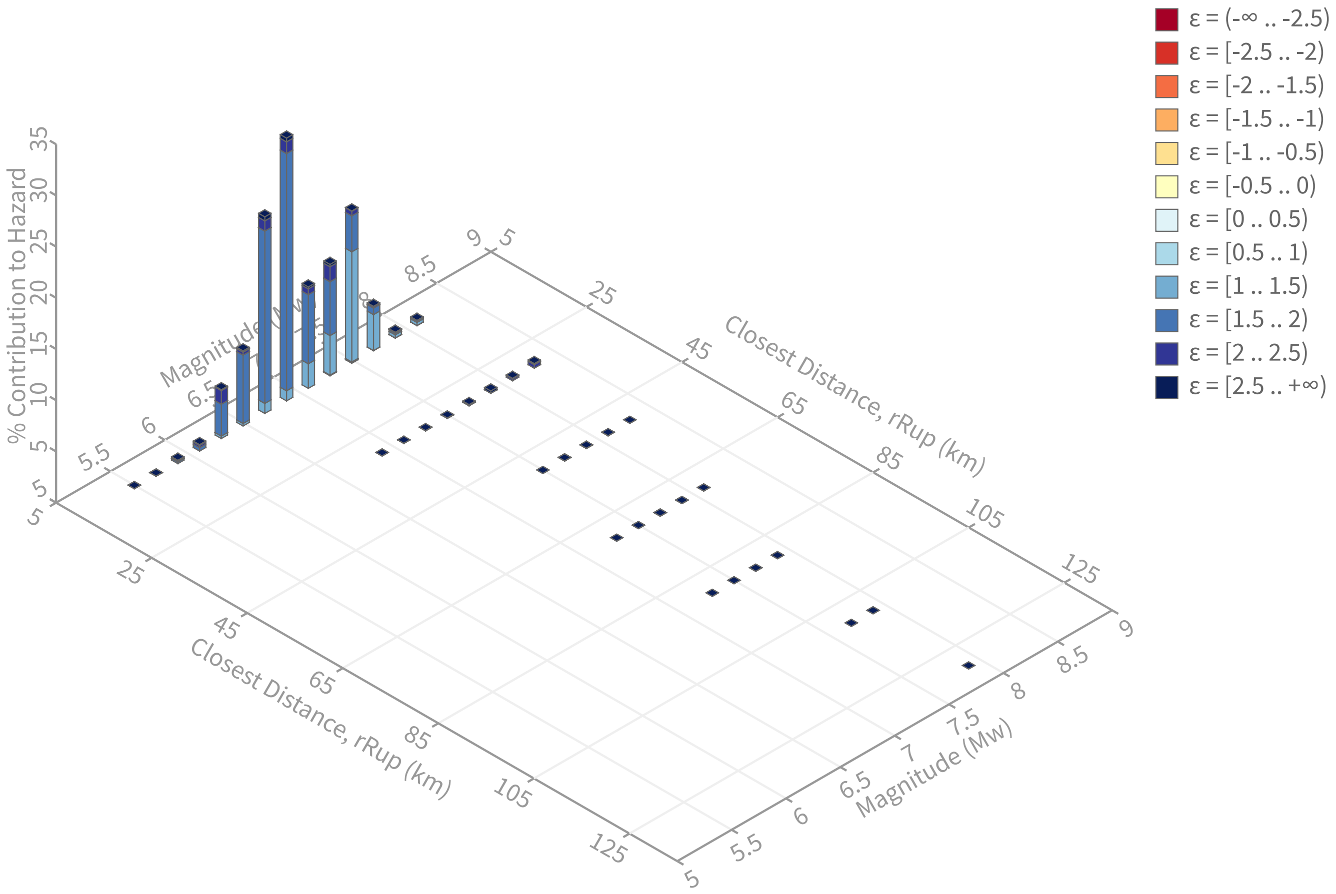

Component Curves for 0.75 Second Spectral Acceleration

View Raw Data

⌃ Deaggregation

Component

Total ⌄



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
0.75 s SA ground motion: 2.4396711 g

Recovered targets

Return period: 2784.5141 yrs
Exceedance rate: 0.00035912908 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.03 %

Mean (over all sources)

m: 7
r: 4.12 km
$ε_0$: 1.7 σ

Mode (largest m-r bin)

m: 6.88
r: 3.74 km
$ε_0$: 1.72 σ
Contribution: 25.65 %

Mode (largest m-r-$ε_0$ bin)

m: 6.87
r: 3.49 km
$ε_0$: 1.7 σ
Contribution: 23.16 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							50.11
Hayward (So) [5]		3.52	7.05	1.62	122.043°W	37.644°N	49.84	33.73
Hayward (So) [4]		3.65	6.78	1.77	122.036°W	37.638°N	91.88	7.14
Mission (connected) [0]		1.13	6.82	1.48	122.043°W	37.633°N	125.08	4.83
Calaveras (No) [3]		12.69	7.27	2.37	121.940°W	37.681°N	64.15	1.70
Hayward (So) [3]		8.49	6.79	2.37	121.989°W	37.590°N	133.70	1.10
UC33brAvg_FM32	System							49.50
Hayward (So) [5]		3.52	7.05	1.63	122.043°W	37.644°N	49.84	34.08
Hayward (So) [4]		3.65	6.77	1.77	122.036°W	37.638°N	91.88	7.02
Mission (connected) [0]		1.13	6.82	1.48	122.043°W	37.633°N	125.08	4.12
Calaveras (No) [3]		12.69	7.26	2.37	121.940°W	37.681°N	64.15	1.73
Hayward (So) [3]		8.49	6.79	2.38	121.989°W	37.590°N	133.70	1.03

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

1.00 Second Spectral Acceleration

Latitude

Decimal degrees

37.63804875435438

Time Horizon

Return period in years

2475

Longitude

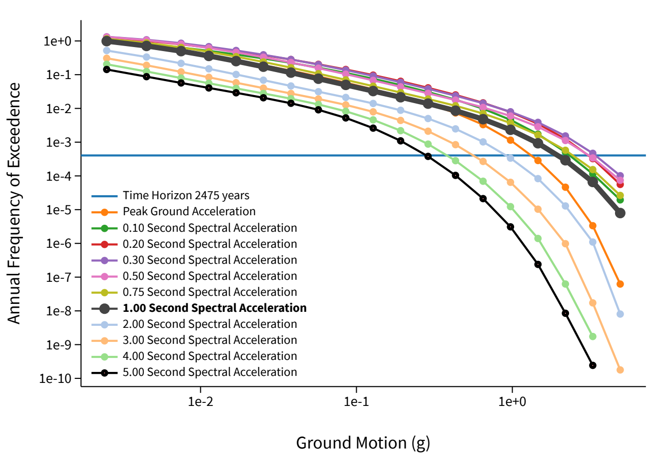
Decimal degrees, negative values for western longitudes

-122.05263934034456

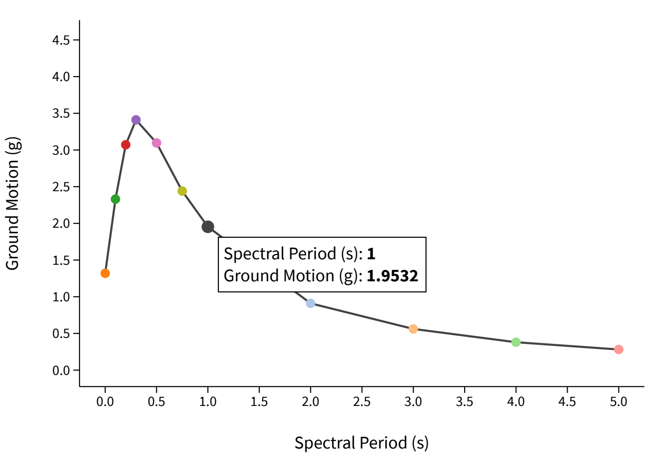
Site Class

360 m/s (C/D boundary)

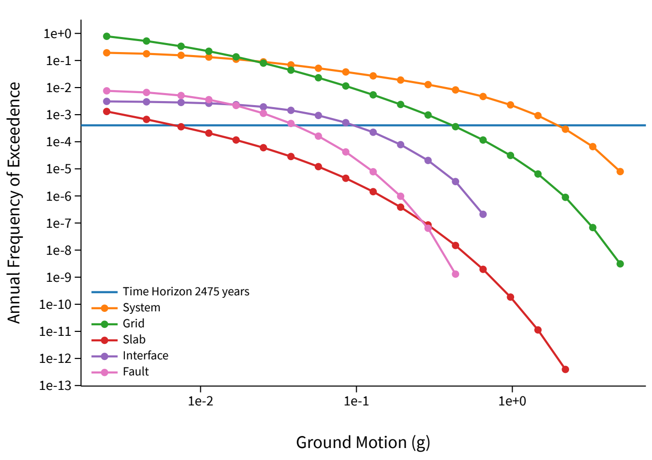
^ Hazard Curve



Hazard Curves



Uniform Hazard Response Spectrum



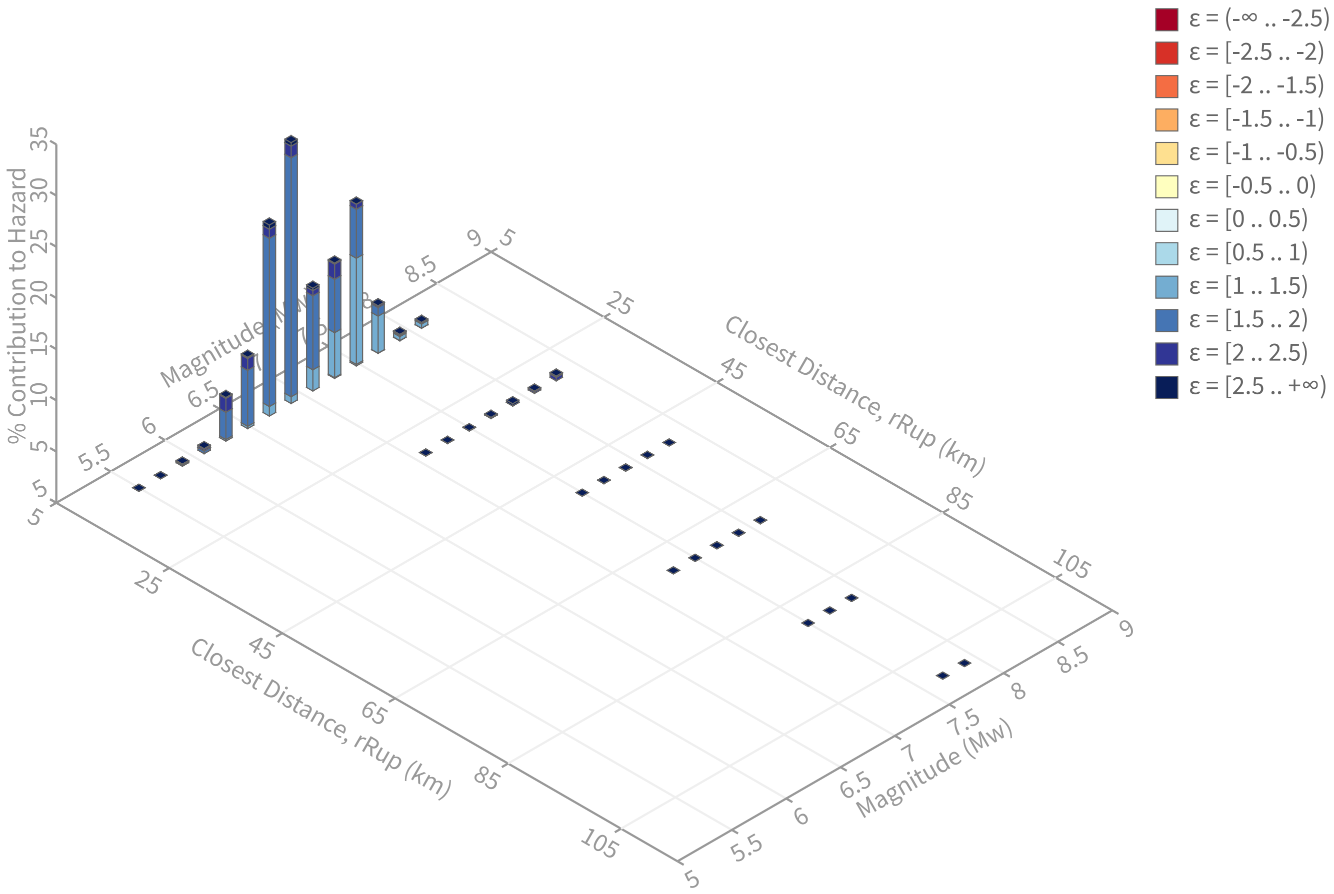
Component Curves for 1.00 Second Spectral Acceleration

View Raw Data

⌃ Deaggregation

Component

Total ⌄



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
1.0 s SA ground motion: 1.953162 g

Recovered targets

Return period: 2776.6076 yrs
Exceedance rate: 0.00036015172 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.02 %

Mean (over all sources)

m: 7.02
r: 4.11 km
ε_0: 1.69 σ

Mode (largest m-r bin)

m: 6.88
r: 3.73 km
ε_0: 1.73 σ
Contribution: 25.47 %

Mode (largest m-r-ε_0 bin)

m: 6.87
r: 3.46 km
ε_0: 1.7 σ
Contribution: 23.33 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							50.13
Hayward (So) [5]		3.52	7.07	1.61	122.043°W	37.644°N	49.84	34.12
Hayward (So) [4]		3.65	6.79	1.77	122.036°W	37.638°N	91.88	6.98
Mission (connected) [0]		1.13	6.85	1.48	122.043°W	37.633°N	125.08	4.68
Calaveras (No) [3]		12.69	7.28	2.37	121.940°W	37.681°N	64.15	1.71
Hayward (So) [3]		8.49	6.80	2.38	121.989°W	37.590°N	133.70	1.06
UC33brAvg_FM32	System							49.55
Hayward (So) [5]		3.52	7.07	1.61	122.043°W	37.644°N	49.84	34.48
Hayward (So) [4]		3.65	6.78	1.77	122.036°W	37.638°N	91.88	6.87
Mission (connected) [0]		1.13	6.85	1.49	122.043°W	37.633°N	125.08	3.99
Calaveras (No) [3]		12.69	7.28	2.37	121.940°W	37.681°N	64.15	1.73

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

2.00 Second Spectral Acceleration

Latitude

Decimal degrees

37.63804875435438

Time Horizon

Return period in years

2475

Longitude

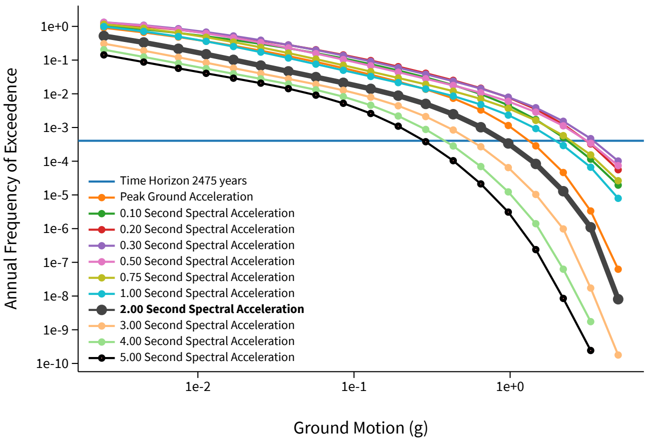
Decimal degrees, negative values for western longitudes

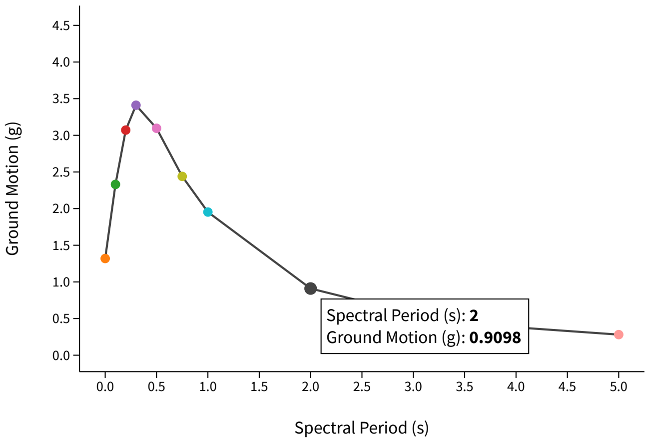
-122.05263934034456

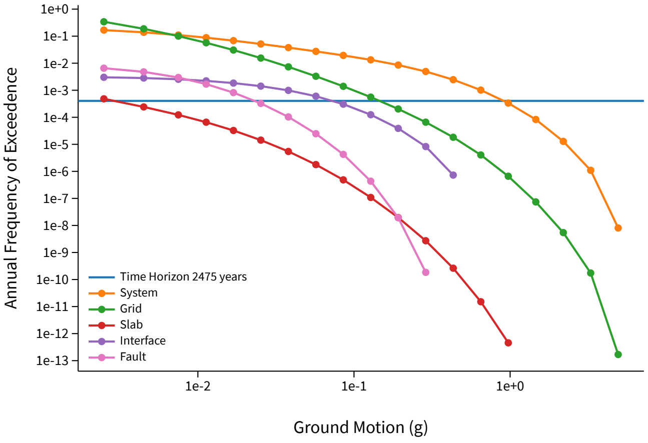
Site Class

360 m/s (C/D boundary)

Hazard Curve


Hazard Curves


Uniform Hazard Response Spectrum

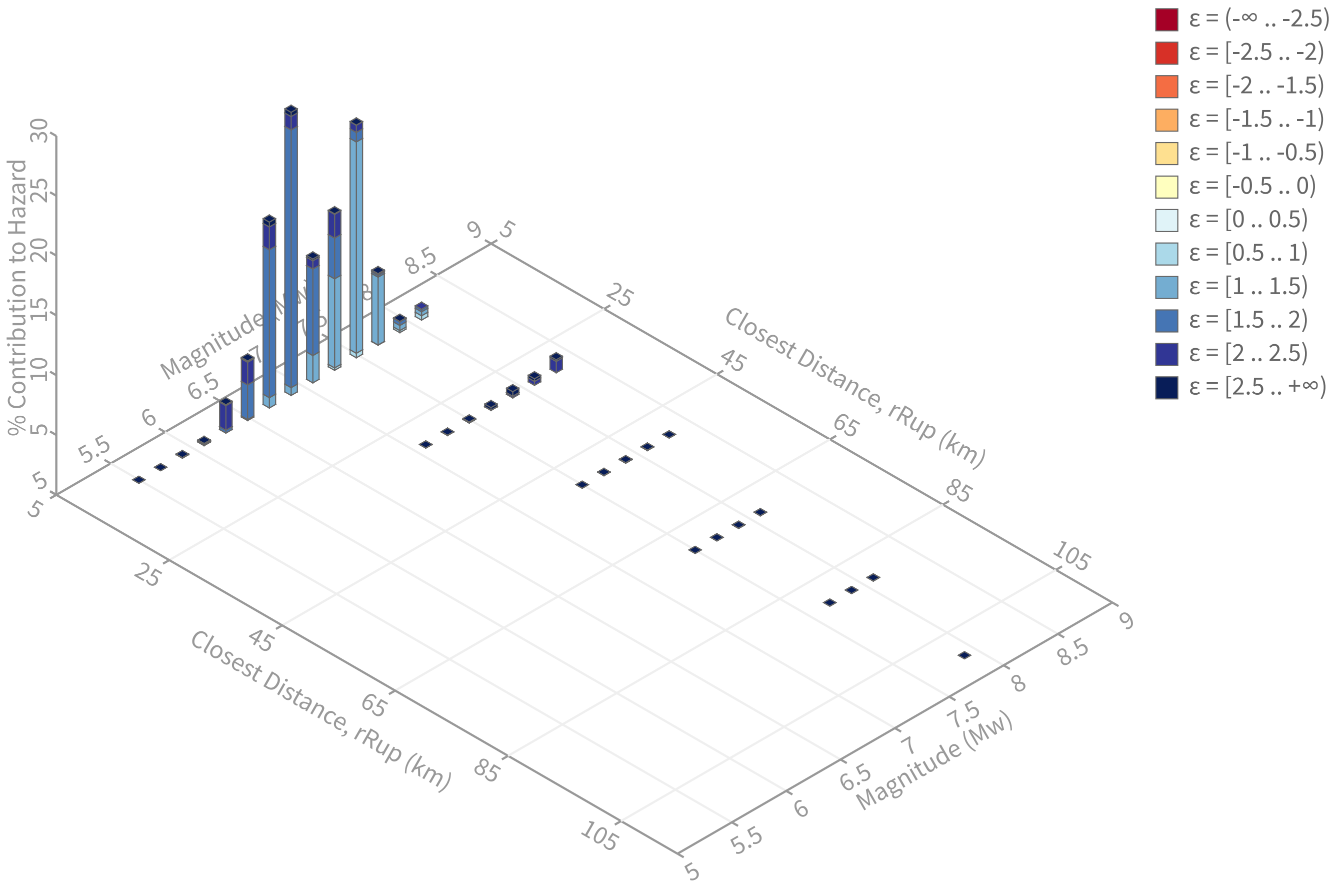

Component Curves for 2.00 Second Spectral Acceleration

View Raw Data

^ Deaggregation

Component

Total ⌄



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
2.0 s SA ground motion: 0.90979782 g

Recovered targets

Return period: 2810.5626 yrs
Exceedance rate: 0.00035580065 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 7.12
r: 4.61 km
ε_0: 1.66 σ

Mode (largest m-r bin)

m: 6.88
r: 3.76 km
ε_0: 1.76 σ
Contribution: 23.66 %

Mode (largest m-r-ε_0 bin)

m: 6.88
r: 3.46 km
ε_0: 1.74 σ
Contribution: 21.54 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							50.17
Hayward (So) [5]		3.52	7.14	1.57	122.043°W	37.644°N	49.84	34.41
Hayward (So) [4]		3.65	6.83	1.80	122.036°W	37.638°N	91.88	6.04
Mission (connected) [0]		1.13	6.95	1.46	122.043°W	37.633°N	125.08	4.17
Calaveras (No) [3]		12.69	7.34	2.24	121.940°W	37.681°N	64.15	2.17
San Andreas (Peninsula) [7]		29.51	8.11	2.43	122.317°W	37.476°N	232.35	1.05
UC33brAvg_FM32	System							49.64
Hayward (So) [5]		3.52	7.14	1.57	122.043°W	37.644°N	49.84	34.76
Hayward (So) [4]		3.65	6.83	1.80	122.036°W	37.638°N	91.88	5.92
Mission (connected) [0]		1.13	6.96	1.46	122.043°W	37.633°N	125.08	3.56
Calaveras (No) [3]		12.69	7.34	2.24	121.940°W	37.681°N	64.15	2.18
San Andreas (Peninsula) [7]		29.51	8.11	2.43	122.317°W	37.476°N	232.35	1.06

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Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

37.63804875435438

Longitude

Decimal degrees, negative values for western longitudes

-122.05263934034456

Site Class

360 m/s (C/D boundary)

Spectral Period

3.00 Second Spectral Acceleration

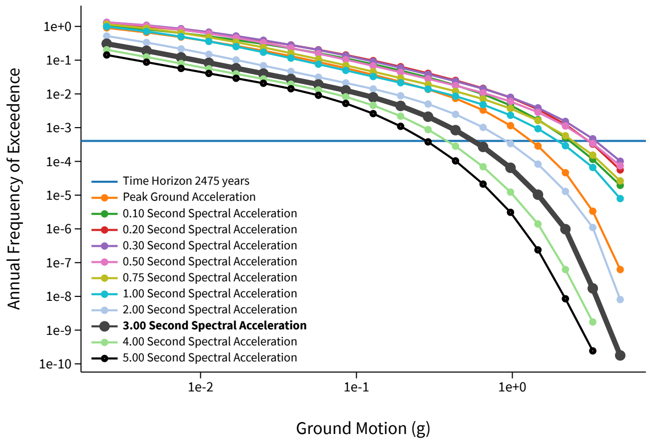
Time Horizon

Return period in years

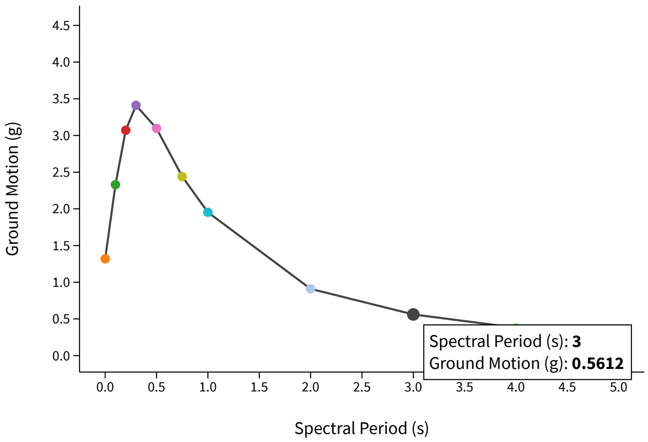
2475

Hazard Curve

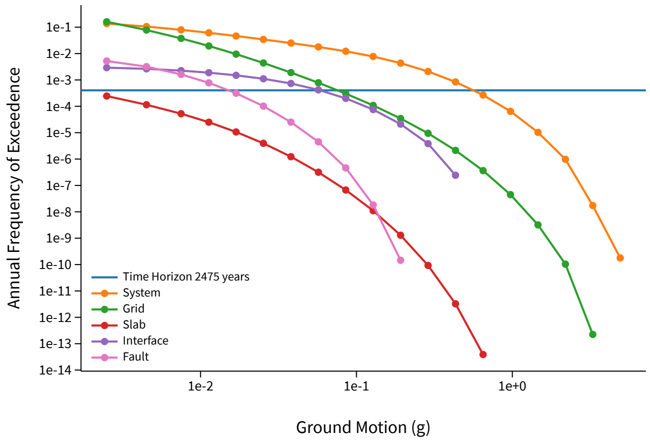
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for 3.00 Second Spectral Acceleration

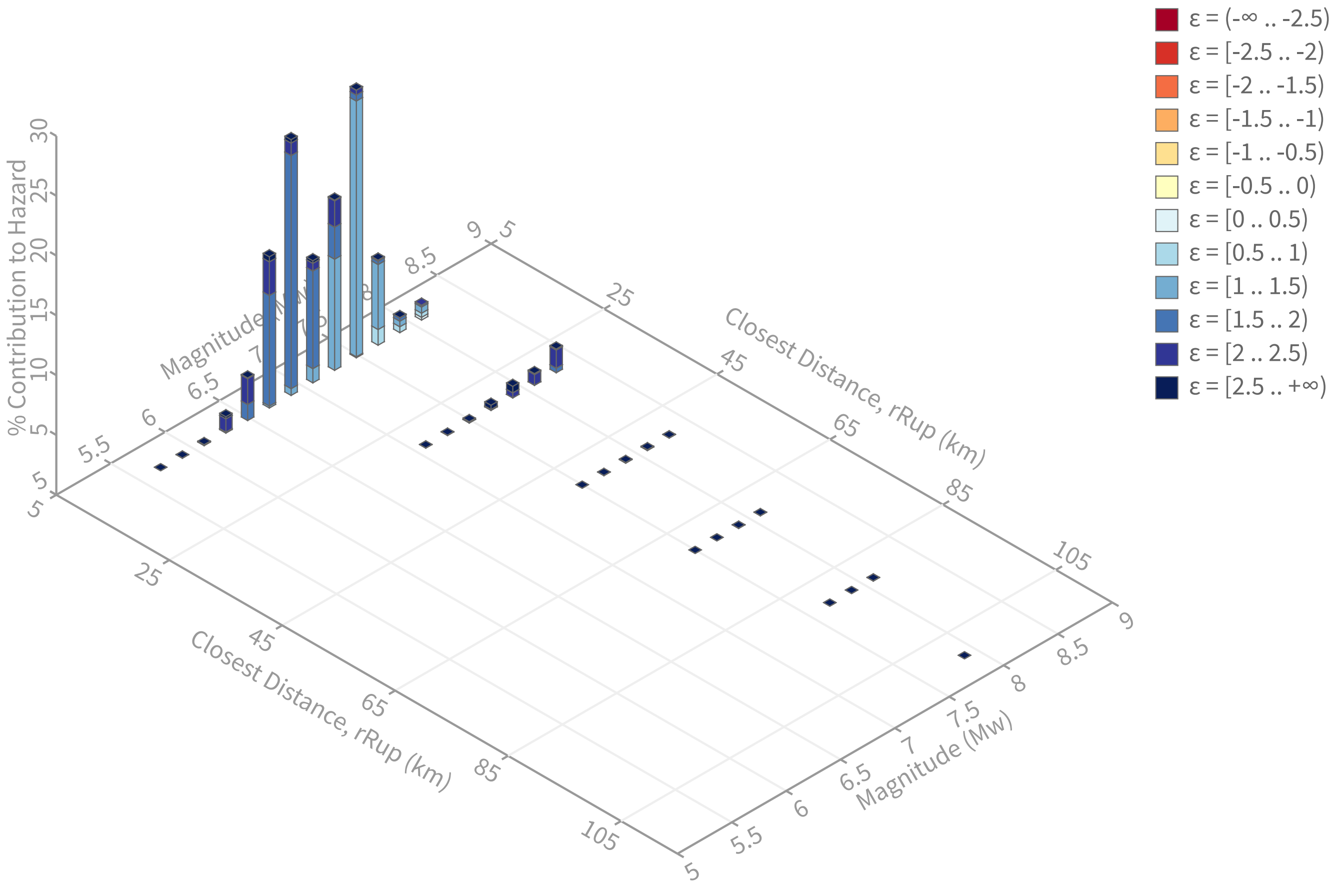


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
3.0 s SA ground motion: 0.5611693 g

Recovered targets

Return period: 2782.44 yrs
Exceedance rate: 0.00035939679 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 7.2
r: 5.23 km
ε_0: 1.62 σ

Mode (largest m-r bin)

m: 7.51
r: 3.77 km
ε_0: 1.26 σ
Contribution: 22.36 %

Mode (largest m-r-ε_0 bin)

m: 7.51
r: 3.4 km
ε_0: 1.23 σ
Contribution: 21.38 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							50.14
Hayward (So) [5]		3.52	7.20	1.51	122.043°W	37.644°N	49.84	34.25
Hayward (So) [4]		3.65	6.87	1.82	122.036°W	37.638°N	91.88	5.26
Mission (connected) [0]		1.13	7.01	1.46	122.043°W	37.633°N	125.08	3.78
Calaveras (No) [3]		12.69	7.39	2.11	121.940°W	37.681°N	64.15	2.51
San Andreas (Peninsula) [7]		29.51	8.11	2.27	122.317°W	37.476°N	232.35	1.93
UC33brAvg_FM32	System							49.71
Hayward (So) [5]		3.52	7.20	1.51	122.043°W	37.644°N	49.84	34.61
Hayward (So) [4]		3.65	6.87	1.82	122.036°W	37.638°N	91.88	5.14
Mission (connected) [0]		1.13	7.03	1.45	122.043°W	37.633°N	125.08	3.24
Calaveras (No) [3]		12.69	7.39	2.10	121.940°W	37.681°N	64.15	2.53
San Andreas (Peninsula) [7]		29.51	8.11	2.28	122.317°W	37.476°N	232.35	1.95

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

4.00 Second Spectral Acceleration

Latitude

Decimal degrees

37.63804875435438

Time Horizon

Return period in years

2475

Longitude

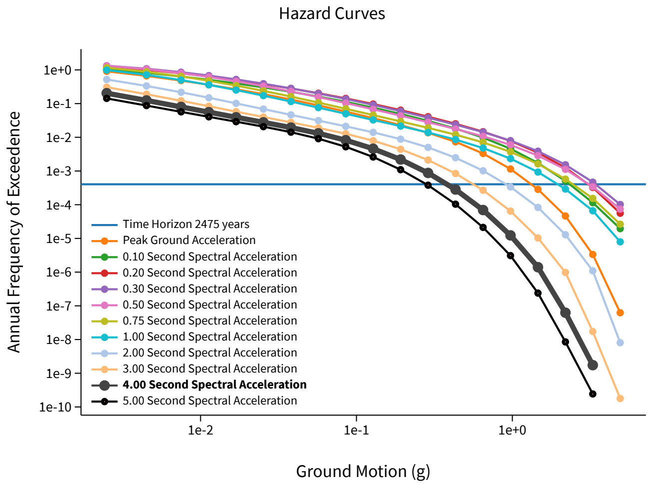
Decimal degrees, negative values for western longitudes

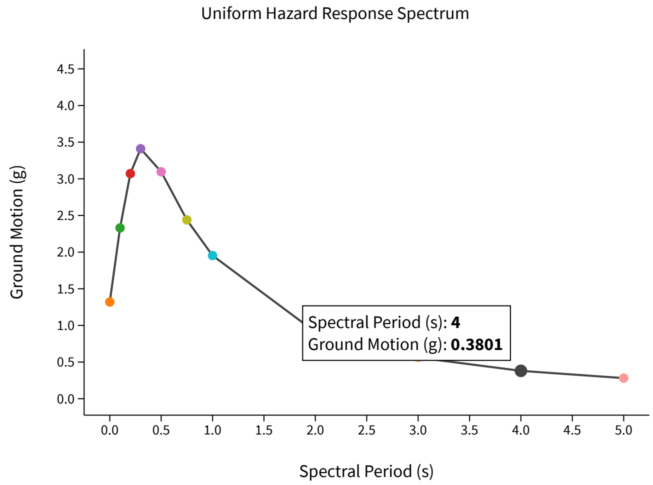
-122.05263934034456

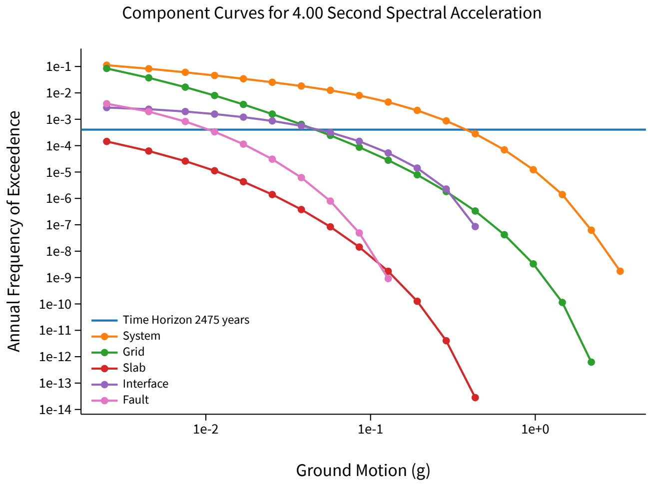
Site Class

360 m/s (C/D boundary)

Hazard Curve


Hazard Curves


Uniform Hazard Response Spectrum

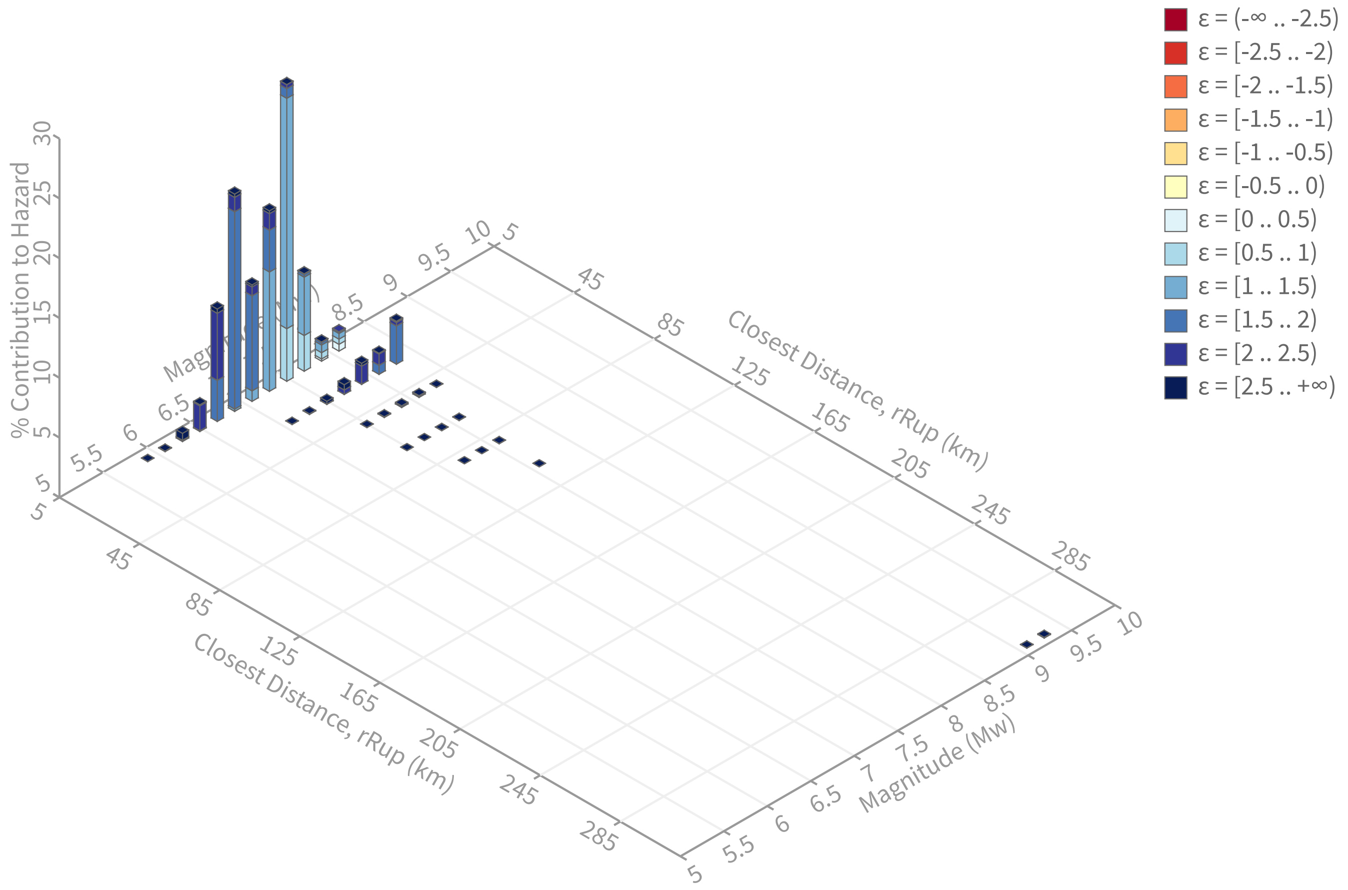

Component Curves for 4.00 Second Spectral Acceleration

View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
4.0 s SA ground motion: 0.38006575 g

Recovered targets

Return period: 2808.6477 yrs
Exceedance rate: 0.00035604324 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 7.29
r: 6.61 km
ε_0: 1.59 σ

Mode (largest m-r bin)

m: 7.51
r: 3.82 km
ε_0: 1.2 σ
Contribution: 24.79 %

Mode (largest m-r-ε_0 bin)

m: 7.5
r: 3.38 km
ε_0: 1.22 σ
Contribution: 19.26 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							50.05
Hayward (So) [5]		3.52	7.25	1.46	122.043°W	37.644°N	49.84	33.15
Hayward (So) [4]		3.65	6.91	1.85	122.036°W	37.638°N	91.88	4.34
San Andreas (Peninsula) [7]		29.51	8.10	2.03	122.317°W	37.476°N	232.35	3.54
Mission (connected) [0]		1.13	7.08	1.46	122.043°W	37.633°N	125.08	3.33
Calaveras (No) [3]		12.69	7.42	1.95	121.940°W	37.681°N	64.15	2.99
UC33brAvg_FM32	System							49.76
Hayward (So) [5]		3.52	7.25	1.45	122.043°W	37.644°N	49.84	33.51
Hayward (So) [4]		3.65	6.91	1.85	122.036°W	37.638°N	91.88	4.25
San Andreas (Peninsula) [7]		29.51	8.10	2.04	122.317°W	37.476°N	232.35	3.58
Calaveras (No) [3]		12.69	7.43	1.94	121.940°W	37.681°N	64.15	3.01
Mission (connected) [0]		1.13	7.10	1.45	122.043°W	37.633°N	125.08	2.87

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

5.00 Second Spectral Acceleration

Latitude

Decimal degrees

37.63804875435438

Time Horizon

Return period in years

2475

Longitude

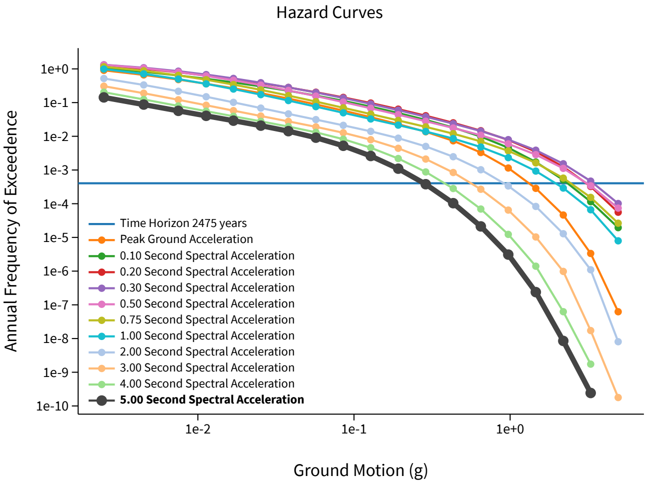
Decimal degrees, negative values for western longitudes

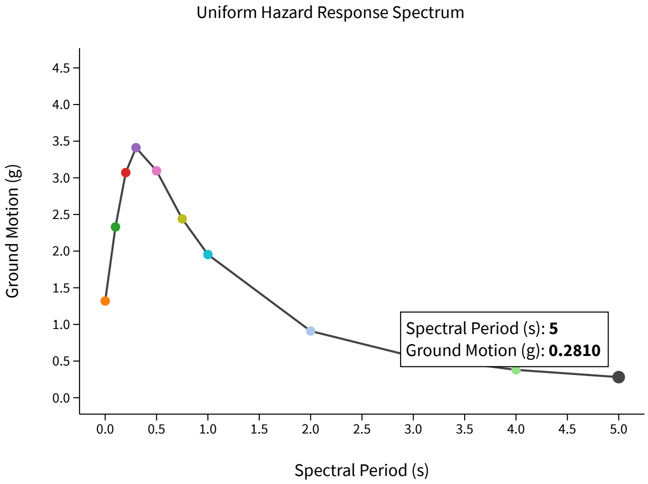
-122.05263934034456

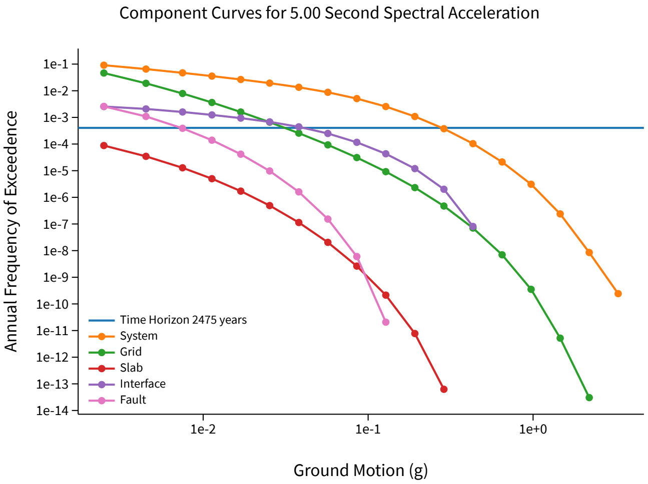
Site Class

360 m/s (C/D boundary)

Hazard Curve





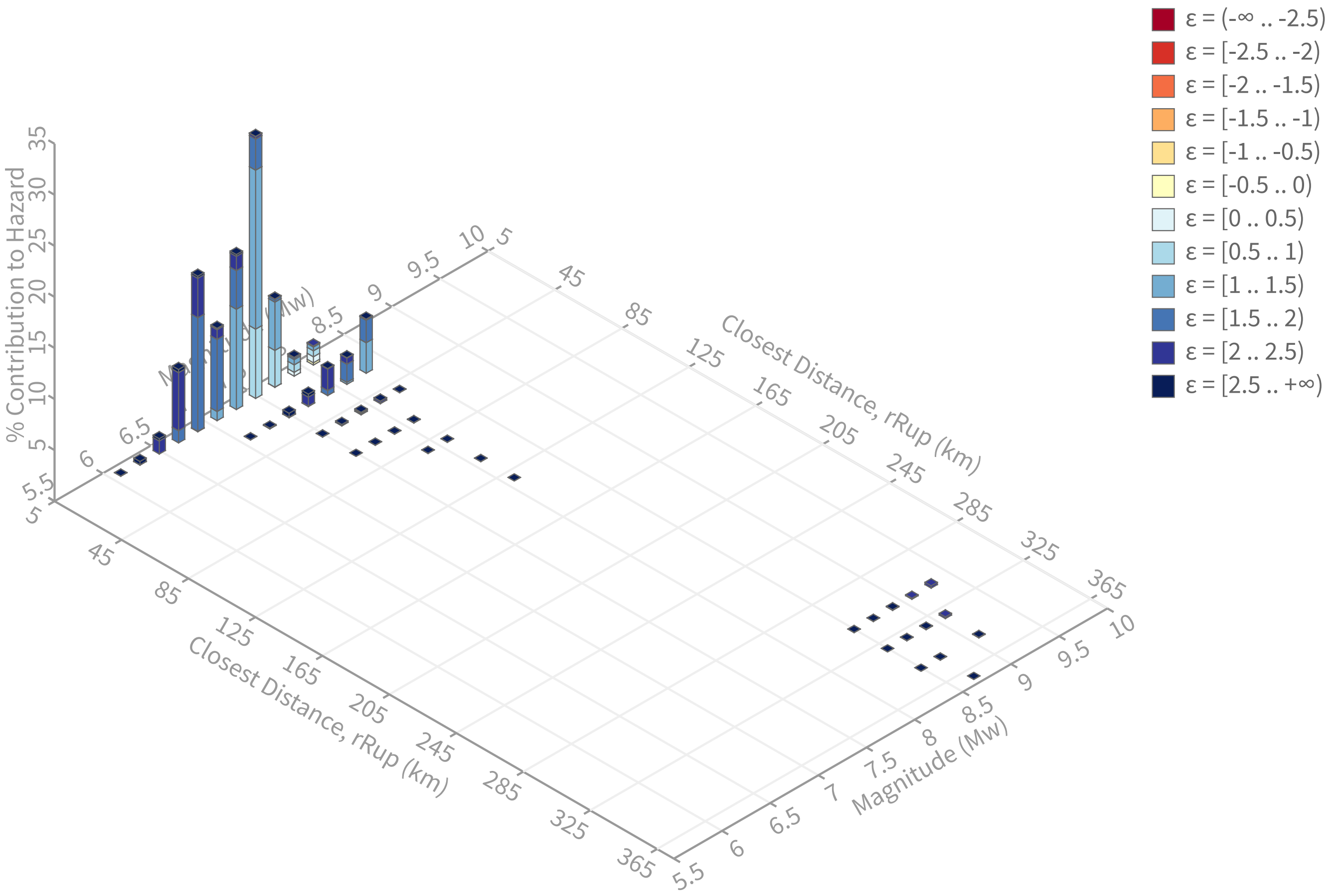


View Raw Data

^ Deaggregation

Component

Total ⌄



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
5.0 s SA ground motion: 0.28103274 g

Recovered targets

Return period: 2859.9413 yrs
Exceedance rate: 0.00034965753 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 7.38
r: 9.81 km
ε_0: 1.56 σ

Mode (largest m-r bin)

m: 7.51
r: 3.88 km
ε_0: 1.17 σ
Contribution: 25.64 %

Mode (largest m-r-ε_0 bin)

m: 7.5
r: 3.56 km
ε_0: 1.19 σ
Contribution: 15.57 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							49.70
Hayward (So) [5]		3.52	7.29	1.41	122.043°W	37.644°N	49.84	31.42
San Andreas (Peninsula) [7]		29.51	8.09	1.84	122.317°W	37.476°N	232.35	5.29
Hayward (So) [4]		3.65	6.94	1.86	122.036°W	37.638°N	91.88	3.64
Calaveras (No) [3]		12.69	7.44	1.83	121.940°W	37.681°N	64.15	3.34
Mission (connected) [0]		1.13	7.13	1.47	122.043°W	37.633°N	125.08	2.90
UC33brAvg_FM32	System							49.53
Hayward (So) [5]		3.52	7.29	1.41	122.043°W	37.644°N	49.84	31.77
San Andreas (Peninsula) [7]		29.51	8.09	1.84	122.317°W	37.476°N	232.35	5.35
Hayward (So) [4]		3.65	6.95	1.86	122.036°W	37.638°N	91.88	3.56
Calaveras (No) [3]		12.69	7.45	1.82	121.940°W	37.681°N	64.15	3.37
Mission (connected) [0]		1.13	7.15	1.45	122.043°W	37.633°N	125.08	2.53

APPENDIX E

GEOPHYSICAL TESTING REPORT

REPORT

SURFACE WAVE MEASUREMENTS

HAYWARD PARCEL 3
HAYWARD, CALIFORNIA

GEO*Vision* Project No. 21110

Prepared for

Engeo, Inc.
2010 Crow Canyon Pl., Suite 250
San Ramon, California 94583
(925) 866-9000

Prepared by

GEO*Vision*, Inc.
1124 Olympic Drive
Corona, California 92881
(951) 549-1234

Report 21110-01 Rev 0

April 28, 2021

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1 INTRODUCTION

In-situ seismic measurements using active-source surface wave techniques were performed at the Hayward Parcel 3 site located in Hayward, California on April 11, 2021. The purpose of this investigation was to provide a shear (S) wave velocity profile to a depth of 30 m (100 ft) and estimate the average S-wave velocity of the upper 100 ft (V_{S100ft}). The active-source surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The location of the MASW testing location (Array 1) is shown on Figure 1.

V_{S30} is used in the NEHRP provisions and the Uniform Building Code (UBC) to separate sites into classes for earthquake engineering design (BSSC, 2009). V_{S100ft} is used in the International Building Code (IBC) for site classification. These site classes are as follows:

Class A – hard rock – $V_{S30} > 1500$ m/s (UBC) or $V_{S100ft} > 5,000$ ft/s (IBC)
Class B – rock – $760 < V_{S30} \leq 1500$ m/s (UBC) or $2,500 < V_{S100ft} \leq 5,000$ ft/s (IBC)
Class C – very dense soil and soft rock – $360 < V_{S30} \leq 760$ m/s (UBC)
or $1,200 < V_{S100ft} \leq 2,500$ ft/s (IBC)
Class D – stiff soil – $180 < V_{S30} \leq 360$ m/s (UBC) or $600 < V_{S100ft} \leq 1,200$ ft/s (IBC)
Class E – soft soil – $V_{S30} < 180$ m/s (UBC) or $V_{S100ft} < 600$ ft/s (IBC)
Class F – soils requiring site-specific evaluation

An overview of the surface wave method is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Data modeling is presented in Section 5 and interpretation and results are presented in Section 6. References and our professional certification are presented in Sections 7 and 8, respectively.

▬ Active Surface Wave Array (MASW)

NOTES:
1. Coordinate System: California State Plane, NAD83, Zone III(0403), US Survey Feet
2. Base map source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

GEOVision
geophysical services

Date:	4/16/2021
GV Project:	21110
Developed by:	A Martin
Drawn by:	T Rodriguez
Approved by:	A Martin
File Name:	21110-1.MXD

FIGURE 1
SITE MAP

SITE LOCATED AT
HAYWARD PARCEL 3
HAYWARD, CALIFORNIA

PREPARED FOR
ENGEO, INC.

2 OVERVIEW OF SURFACE WAVE TECHNIQUES

2.1 Introduction

Active- and passive-source (ambient vibration) surface wave techniques are routinely utilized for site characterization. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the horizontal over vertical spectral ratio (HVSR) technique and the array and refraction microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh and Love waves when propagating in a layered medium. Surface waves of different wavelengths (λ) or frequencies (f) sample different depth. As a result of the variance in the shear stiffness of the distinct layers, waves with different wavelengths propagate at different phase velocities; hence, dispersion. A surface wave dispersion curve is the variation of V_R or V_L with λ or f. The Rayleigh wave phase velocity (V_R) depends primarily on the material properties (V_S, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. The Love wave phase velocity (V_L) depends primarily on V_S and mass density. Rayleigh and Love wave propagation are also affected by damping or seismic quality factor (Q). Rayleigh wave techniques are utilized to measure vertically polarized S-waves (S_V-wave); whereas Love wave techniques are utilized to measure horizontally polarized S-waves (S_H-wave).

2.2 MASW Technique

A description of the MASW method is given by Park, 1999a and 1999b and Foti, 2000. Ground motions are typically recorded by 24, or more, geophones typically spaced 1 to 3 m apart along a linear array and connected to a seismograph. Energy sources for shallow investigations include various sized hammers and vehicle mounted weight drops. When applying the MASW technique to develop a one-dimensional (1-D) V_S model, it is preferable to use multiple-source offsets from both ends of the array. The most commonly applied MASW technique is the Rayleigh-wave based MASW method, which we refer to as MAS_RW to distinguish from Love-wave based MASW (MAS_LW). MAS_RW and MAS_LW acquisition can easily be combined with P- and S-wave seismic refraction acquisition, respectively. MAS_RW data are generally recorded using a vertical source and vertical geophone but may also be recorded using a horizontal geophone with radial (in-line) orientation. MAS_LW data are recorded using transversely orientated horizontal source and transverse horizontal geophone.

A wavefield transform is applied to the time-history data to convert the seismic record from time-offset space to frequency-wavenumber (f-k) space in which the fundamental or higher surface-wave modes can be easily identified as energy maxima and picked. Frequency and/or wavenumber can easily be mapped to phase velocity, slowness, or wavelength using the following properties: $k = 2\pi/\lambda$, $\lambda = v/f$. Common wave-field transforms include: the f-k transform (a 2D fast Fourier transform), slant-stack transform (also referred to as intercept-slowness or τ-p transform and equivalent to linear Radon transform), frequency domain beamformer, and phase-shift transform. The minimum wavelength that can be recovered from MASW data set without spatial aliasing is equal to the minimum receiver spacing. Occasionally, SASW analysis procedures are used to extract surface wave dispersion data, from fixed receiver pairs, at smaller wavelengths than can be recovered by wavefield transformation. Construction of a dispersion curve over the wide frequency/wavelength range necessary to develop a robust V_S model while also limiting the maximum wavelength based on an established near-field criterion (e.g. Yoon and Rix, 2009; Li and Rosenblad, 2011), generally requires multiple source offsets.

Although the clear majority of MASW surveys record Rayleigh waves, it has been shown that Love wave techniques can be more effective in some environments, particularly shallow rock sites and sites with a highly attenuative, low velocity surface layer (Xia, et al., 2012; **GEO***Vision*, 2012; Yong, et al., 2013; Martin, et al., 2014). Rayleigh wave techniques, however, are generally more effective at sites where velocity gradually increases with depth because larger energy sources are readily available for the generation of Rayleigh waves. Rayleigh wave techniques are also more applicable to sites with high velocity layers and/or velocity inversions because the presence of such structures is more apparent in the Rayleigh wave dispersion curves than in Love wave dispersion curves. Rayleigh wave techniques are preferable at sites with a high velocity surface layer because Love waves do not theoretically exist in such environments. Occasionally, the horizontal radial component of a Rayleigh wave may yield higher quality dispersion data than the vertical component because different modes of propagation may have more energy in one component than the other. Recording both the vertical and horizontal components of the Rayleigh wave is particularly useful at sites with complex modes of propagation or when attempting to recover multiple Rayleigh wave modes for multi-mode modeling as demonstrated in Dal Moro, et al, 2015. Joint inversion of Rayleigh and Love wave data may yield more accurate V_S models and offers a means to investigate anisotropy, where S_V- and S_H-wave velocity are not equal, as shown in Dal Moro and Ferigo, 2011.

2.3 Surface Wave Dispersion Curve Modeling

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled using iterative forward and inverse modeling routines. The final model profile is assumed to represent actual site conditions. The theoretical model used to interpret the dispersion curve assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of active and/or passive surface wave testing provide a good “global” estimate of the material properties along the array. The results may be more representative of the site than a borehole “point” estimate.

The surface wave forward problem is typically solved using the Thomson-Haskell transfer-matrix (Thomson, 1950; Haskell, 1953) later modified by Dunkin (1965) and Knopoff (1964), dynamic stiffness matrix (Kausel and Roësset, 1981), or reflection and transmission coefficient (Kennett, 1974) methods. All of these methods can determine fundamental- and higher-mode phase velocities, which correspond to plane waves in 2-D space. The transfer-matrix method is often used in MASW and passive surface-wave software packages, whereas the dynamic stiffness matrix is utilized in many SASW software packages. MAS_RW and/or passive surface-wave modeling may involve modeling of the fundamental mode, some form of effective mode, or multiple individual modes (multi-mode). As outlined in Roësset et al. (1991), several options exist for forward modeling of Rayleigh wave SASW data. One formulation takes into account only fundamental mode plane Rayleigh-wave motion (called the 2-D solution), whereas another includes all stress waves (e.g. body, fundamental, and higher mode surface waves) and incorporates a generalized receiver geometry (3-D global solution) or actual receiver geometry (3-D array solution).

The fundamental mode assumption is generally applicable to modeling Rayleigh-wave dispersion data collected at normally dispersive sites, providing there are not abrupt increases in velocity or steep velocity gradients. Effective-mode or multi-mode approaches are often required for irregularly dispersive sites and sites with steep velocity gradients at shallow depth. If active and passive surface wave data are combined or MAS_RW data are combined from multiple seismic records with different source offsets and receiver gathers, then effective-mode computations are limited to algorithms that assume far-field plane Rayleigh wave propagation. Local search (e.g. linearized matrix inversion methods) or global search methods (e.g., Monte Carlo approaches such as simulated annealing, generic algorithms and neighborhood algorithm) are typically used to solve the inverse problem.

The maximum wavelength (λ_{max}) recovered from a surface wave data set is typically used to estimate depth of investigation although a sensitivity analysis of the V_S models would be a more robust means to estimate depth of investigation. For normally dispersive velocity profiles with a gradual increase in V_S with depth, the maximum depth of investigation is on the order of $\lambda_{max}/2$ for both Rayleigh and Love wave dispersion data. For velocity profiles with an abrupt increase in V_S at depth, the maximum depth of investigation is on the order of $\lambda_{max}/3$ for Rayleigh wave dispersion data but less than $\lambda_{max}/3$ for Love wave dispersion data. The depth of investigation can be highly variable for sites with complex velocity structure (e.g. high velocity layers).

As with all surface geophysical methods, the inversion of surface wave dispersion data does not yield a unique V_S model and multiple possible solutions may equally fit the experimental data. Based on experience at other sites, the shear wave velocity models (V_S and layer thicknesses) determined by surface wave testing are within 20% of the velocities and layer thicknesses that would be determined by other seismic methods (Brown, 1998). The average velocity of the upper 30 m, however, is much more accurate, often to better than 5%, because it is not sensitive to the layering in the model. Because V_{S30} is not significantly affected by the non-uniqueness inherent in V_S models derived from surface wave dispersion curves (Martin et al., 2006, Comina et al., 2011), a single V_S model is considered adequate for estimating V_{S30}.

It may not always be possible to develop a coherent, fundamental mode dispersion curve over sufficient frequency range for modeling due to dominant higher modes with the higher modes not clearly identifiable for multi-mode modeling. It may, however, be possible to identify the Rayleigh wave phase velocity of the fundamental mode at 40 m wavelength (V_{R40}) in which case V_{S30} can at least be estimated using the Brown et al., 2000 relationship:

$$V_{S30} = 1.045V_{R40}$$

This relationship was established based on a statistical analysis of a large number of surface wave data sets from sites with control by velocities measured in nearby boreholes and has been further evaluated by Martin and Diehl, 2004, and Albarello and Gargani, 2010. Further investigation of this approach has revealed that V_{S30} is generally between V_{R40} and V_{R45} with V_{R40} often being most appropriate for shallow groundwater sites and V_{R45} for deep ground water sites. A detailed study of such an approach for Love wave dispersion data has not been conducted; however, preliminary analysis demonstrates that V_{S30} is generally between V_{L50} and

V_{L55}. Although we do not recommend that these empirical V_{S30} estimates replace modeling of surface wave dispersion data, they do offer a means of cost effectively evaluating V_{S30} over a large area. V_{R40} or V_{L55} can also be used to quantify error in V_{S30} by evaluating the scatter in the dispersion data at these wavelengths.

3 FIELD PROCEDURES

The MASW sounding location was established by Engeo and **GEO***Vision* personnel and is shown in Figure 1.

MASW equipment used during this investigation consisted of a Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, seismic cable, a 4 lb hammer, and 12 lb sledgehammer, and 240 lb accelerated weight drop (AWD). MASW data were acquired along a linear array of 48 geophones spaced 3 m (9.8 ft) apart. Shot points were located between 3 and 30 m (9.8 and 98 ft) from the end geophone locations, as possible, and at 18 m (59 ft) intervals in the interior of the array. The 4 lb hammer and 12 lb sledgehammer were used for the near offset source locations and interior source locations. The AWD was used for all source locations offset from the ends of the array and all interior source locations. Data from the transient impacts (hammers) were generally averaged 5 to 10 times to improve the signal-to-noise ratio. All field data were saved to hard disk and documented on field data acquisition forms.

4 DATA REDUCTION

The MASW data were reduced using the software Seismic Pro Surface Plus developed by Geogiga and multiple in-house scripts for various data extraction and formatting tasks, with all data reduction documented in a Microsoft Excel spreadsheet.

The following steps were used for data reduction:

- Input seismic records to be used for analysis into software package.
- Check and correct source and receiver geometry as necessary.
- Select offset range used for analysis (multiple offset ranges utilized for each seismic record as discussed below) and document in spreadsheet.
- Apply phase shift transform to seismic record to convert the data from time – offset to frequency – phase velocity space.
- Identify, pick, save, and document dispersion curve.
- Change the receiver offset range and repeat process.
- Repeat process for all seismic records.
- Use in-house script to apply near-field criteria with maximum wavelength set equal to 1.0 times the source to midpoint of receiver array distance.
- Use in-house script to merge multiple dispersion curves extracted from the MASW data collected along each seismic line for a specific source type (different source locations, different receiver offset ranges, etc.).
- Edit dispersion data, as necessary (e.g. delete poor quality curves and outliers).
- Calculate a representative dispersion curve at equal log-frequency or log-wavelength spacing for the MASW dispersion data using a moving average, polynomial curve fitting routine.

This unique data reduction strategy, which can involve combination of over 50 dispersion curves for a 1D sounding, is designed for characterizing sites with complex velocity structure that do not yield surface wave dispersion data over a wide frequency range from a single source type or source location. The data reduction strategy ensures that the dispersion curve selected for modeling is representative of average conditions beneath the array and spans as broad a frequency/wavelength range as possible while considering near field effects.

5 DATA MODELING

Surface wave data were modeled using the fundamental mode routine in WinSASW V3 software package. During this process, an initial velocity model was generated based on general characteristics of the dispersion curve and the inverse modeling routine utilized to adjust the layer V_S until an acceptable agreement with the observed data was obtained. Layer thicknesses were adjusted, and the inversion process repeated until a V_S model was developed with low RMS error between the observed and calculated dispersion curves. In many cases, once an acceptable V_S model is developed, layer thicknesses are again adjusted, and the inversion process repeated to develop an ensemble of V_S models with similar RMS error to quantify non-uniqueness. The primary purpose of this investigation was to estimate V_{S30} and, therefore, it was not considered necessary to develop multiple V_S models. Data inputs into the modeling software include layer thickness, S-wave velocity, P-wave velocity or Poisson's ratio, and mass density. P-wave velocity and mass density only have a very small influence (i.e., less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is significantly impacted by the location of the saturated zone, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 1.73 to 1.98 gm/cm^3 (108 to 124 lb/ft^3) were used in the velocity profiles for subsurface soils/rock depending on P- and S-wave velocity. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible (±2%) effect on the estimated V_S from surface wave dispersion data. During modeling of Rayleigh wave dispersion data, the compression wave velocity, V_P, for unsaturated sediments was estimated using a Poisson's ratio, v, of 0.3 and the relationship:

$$V_P = V_S\,[(2(1-v))/(1-2v)]^{0.5}$$

Poisson's ratio has a larger effect than density on the estimated V_S from Rayleigh wave dispersion data. Achenbach (1973) provides approximate relationship between Rayleigh wave velocity (V_R), V_S and v:

$$V_R = V_S\,[(0.862 + 1.14\,v)/(1+ v)]$$

Using this relationship, it can be shown that V_S derived from V_R only varies by about 10% over possible 0 to 0.5 range for Poisson's ratio where:

$$V_S = 1.16V_R \text{ for } v = 0$$
$$V_S = 1.05V_R \text{ for } v = 0.5$$

The realistic range for Poisson's ratio of typical unsaturated sediments is about 0.25 to 0.35. Over this range, V_S derived from modeling of Rayleigh wave dispersion data will vary by about 5%. An intermediate Poisson's ratio of 0.3 was selected for modeling to minimize any error associated with the assumed Poisson's ratio.

High Poisson's ratio saturated sediments with $V_P > 1{,}500$ m/s (4,921 ft/s) were constrained at an approximate depth of 8 m (26 ft) based on interactive analysis of seismic refraction first arrival data.

6 INTERPRETATION AND RESULTS

The fit of the calculated fundamental mode dispersion curve to the experimental data collected along Array 1 and the modeled V_S profile for the surface wave sounding are presented as Figure 2. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in V_S at greater depth. Scatter in the MASW dispersion data is expected to be primarily associated with lateral velocity variability beneath the array. The V_S profile used to match the field data is provided in tabular form in both metric and Imperial units as Tables 1 and 2.

The estimated depth of investigation for the active-source surface wave sounding is about 35 m (115 ft). The V_S model indicates that V_S gradually increases with depth from about 150 m/s (500 ft/s) immediately below the surface to about 425 m/s (1,390 ft/s) at a depth of about 27 m (89 ft).

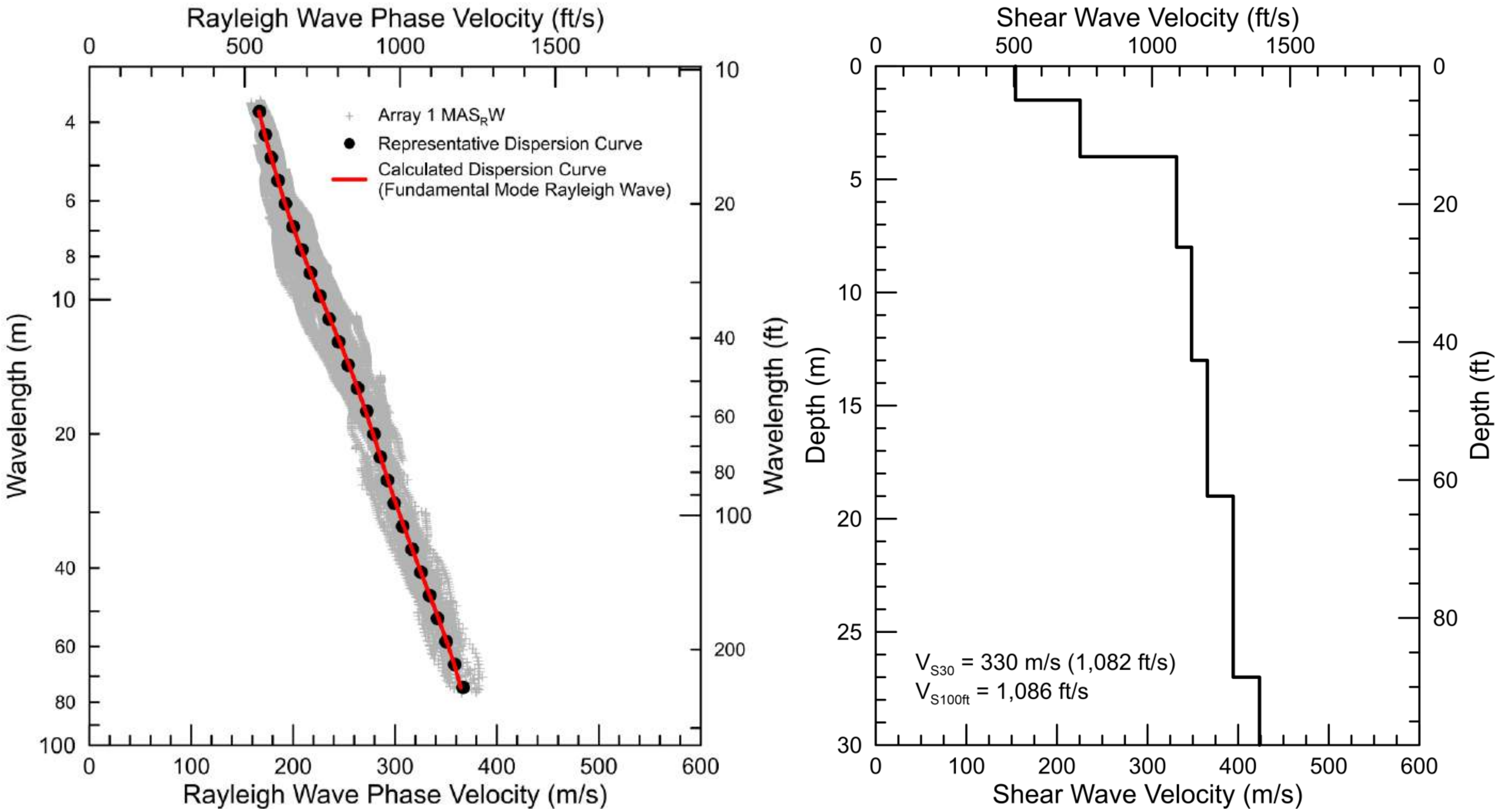
The average shear wave velocity to a depth of 30 m (V_{S30}) is 330 m/s for the V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,086 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of Array 1 is classified as Site Class D, stiff soil.

Table 1 Array 1 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Inferred Density (g/cm^3)
0	1.5	154	289	0.300	1.73
1.5	2.5	226	422	0.300	1.87
4	4	332	621	0.300	1.94
8	5	349	1750	0.479	1.95
13	6	366	1800	0.478	1.96
19	8	395	1850	0.476	1.97
27	Half Space	424	1900	0.474	1.98

Table 2 Array 1 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Inferred Density (lb/ft^3)
0.0	4.9	506	947	0.300	108
4.9	8.2	740	1384	0.300	117
13.1	13.1	1089	2038	0.300	121
26.2	16.4	1144	5741	0.479	122
42.7	19.7	1201	5906	0.478	122
62.3	26.2	1294	6070	0.476	123
88.6	Half Space	1389	6234	0.474	124



V_{S30} = 330 m/s (1,082 ft/s)
V_{S100ft} = 1,086 ft/s

GEOVision geophysical services

Project No: 21110
Date: APR 27, 2021
Drawn By: A MARTIN
Approved By: Antony J Martin

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FIGURE 2
SURFACE WAVE MODEL - ARRAY 1

HAYWARD PARCEL 3
HAYWARD, CALIFORNIA

PREPARED FOR
ENGEO, INC.

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8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Reviewed and approved by,

Antony J Martin

ANTONY JEFFREY MARTIN
GP 989

04/28/2021

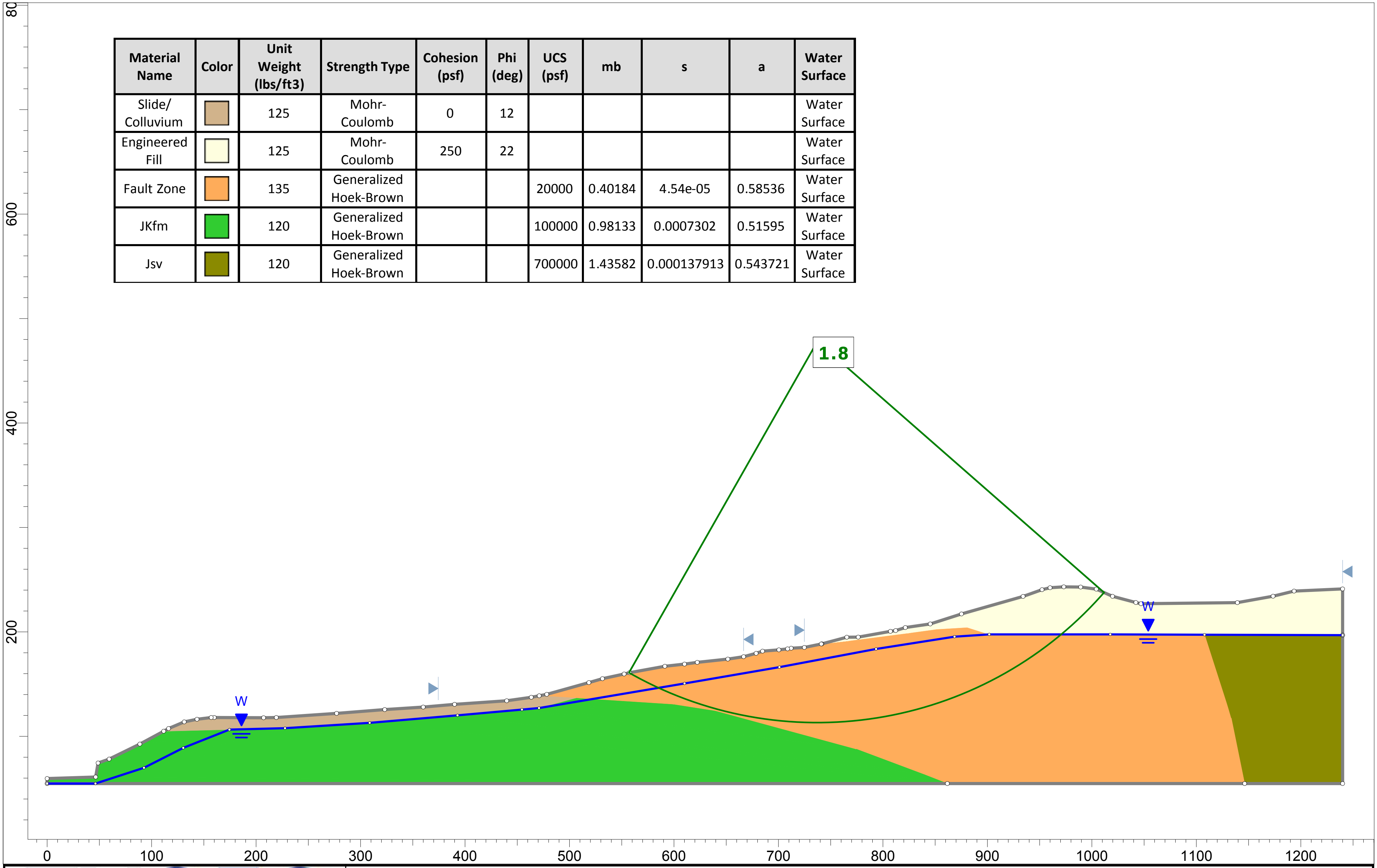
Antony J. Martin — Date
California Professional Geophysicist, P. Gp.
GEO*Vision* Geophysical Services

∗ This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations, or ordinances.

APPENDIX F

SLOPE STABILITY ANALYSIS



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Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12					Water Surface
Engineered Fill	⬜	125	Mohr-Coulomb	250	22					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown			20000	0.40184	4.54e-05	0.58536	Water Surface
JKfm	🟩	120	Generalized Hoek-Brown			100000	0.98133	0.0007302	0.51595	Water Surface
Jsv	🟫	120	Generalized Hoek-Brown			700000	1.43582	0.000137913	0.543721	Water Surface

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Project	Parcel Group 3 La Vista		
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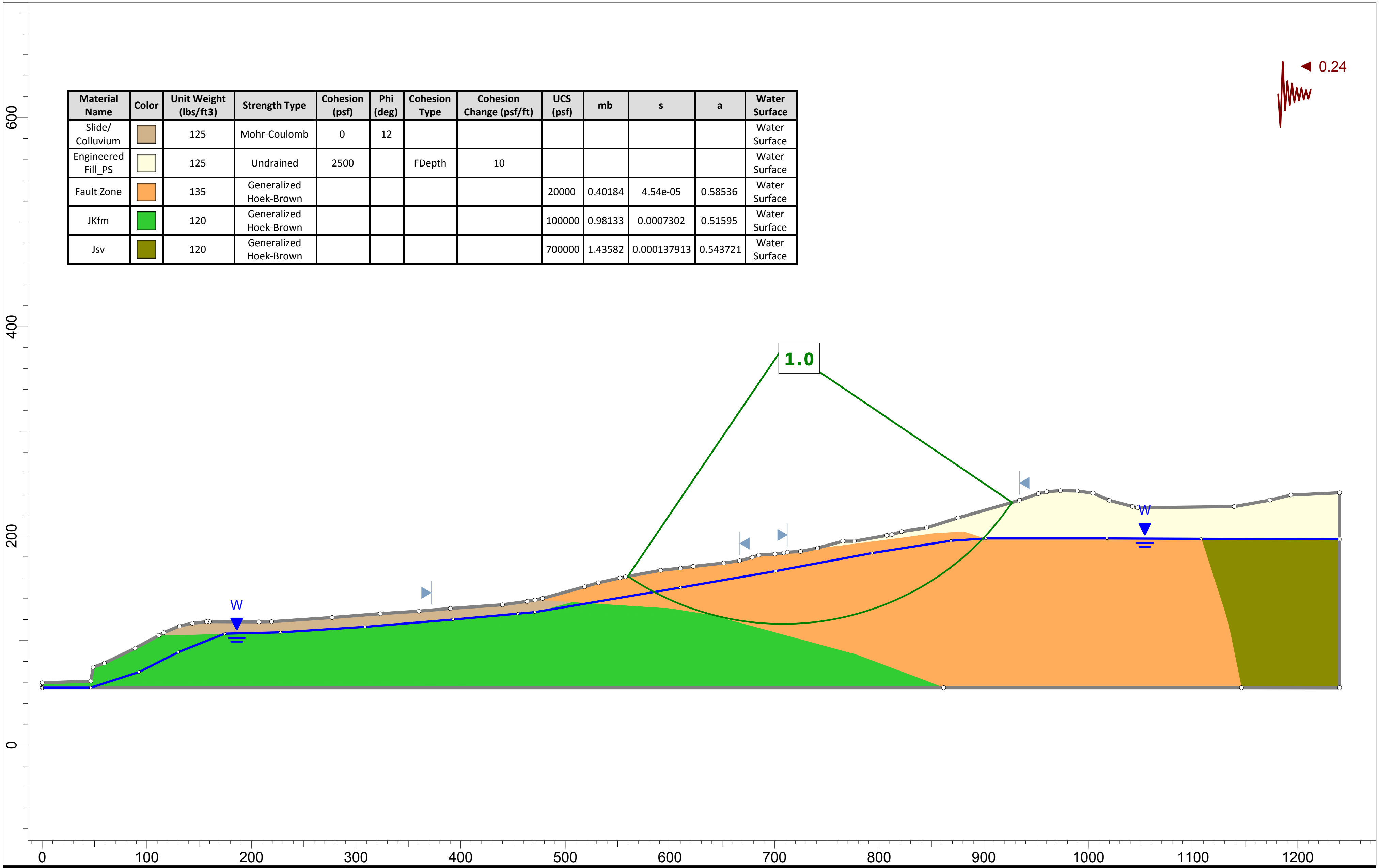
Date	June 1, 2021	*Job Number*	15577.000.001
Condition	Section 1 Existing - Static		



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Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12							Water Surface
Engineered Fill_PS	⬜	125	Undrained	2500		FDepth	10					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown					20000	0.40184	4.54e-05	0.58536	Water Surface
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Author APL
Date June 1, 2021
Job Number 15577.000.001

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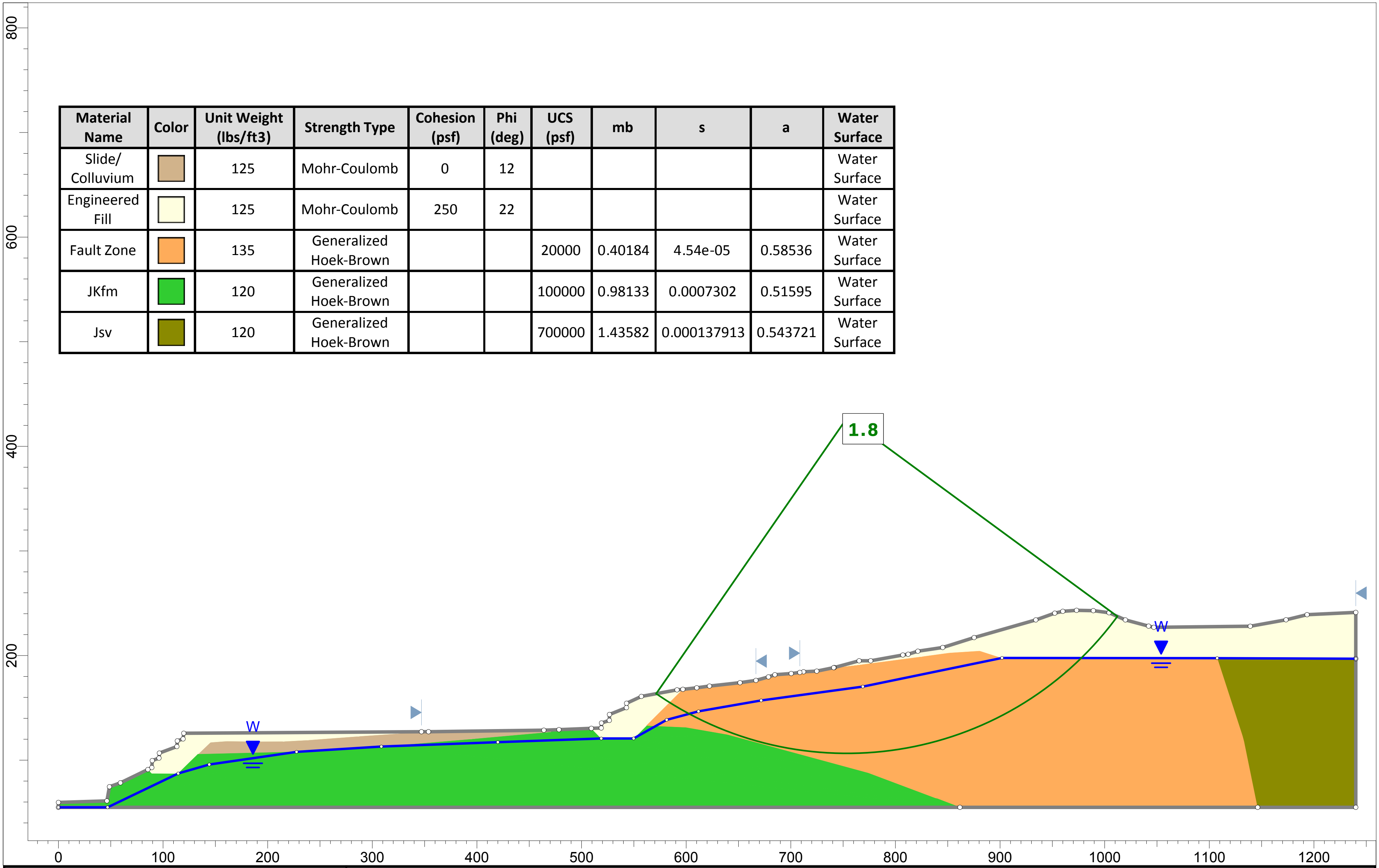

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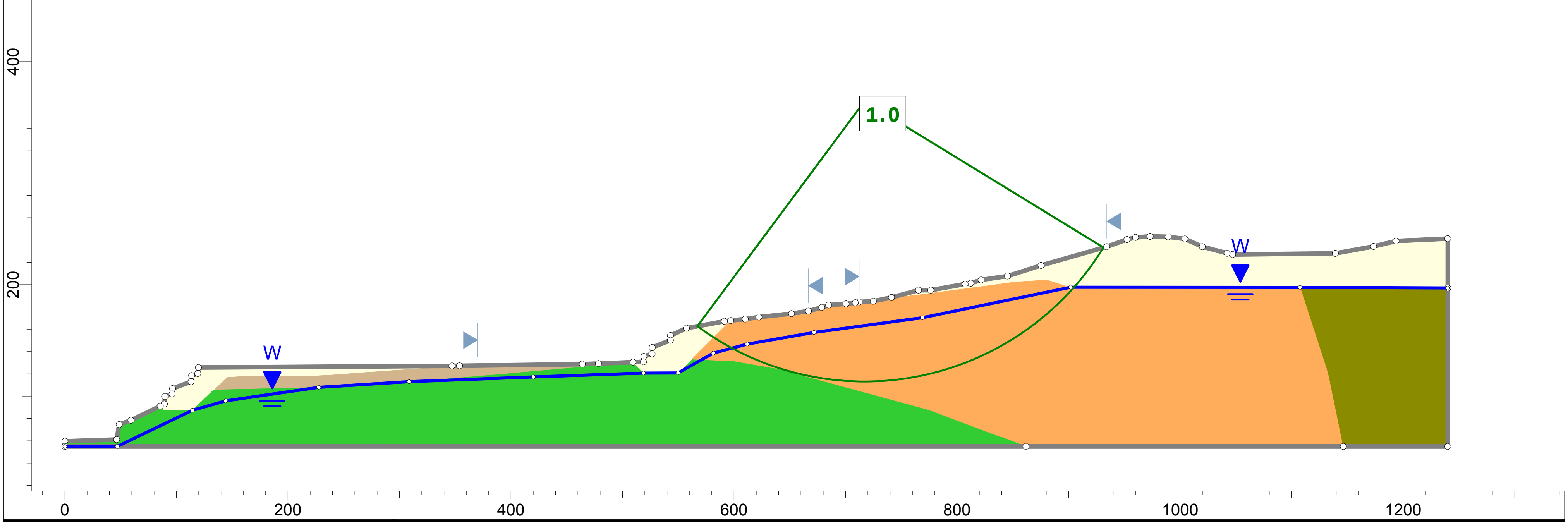

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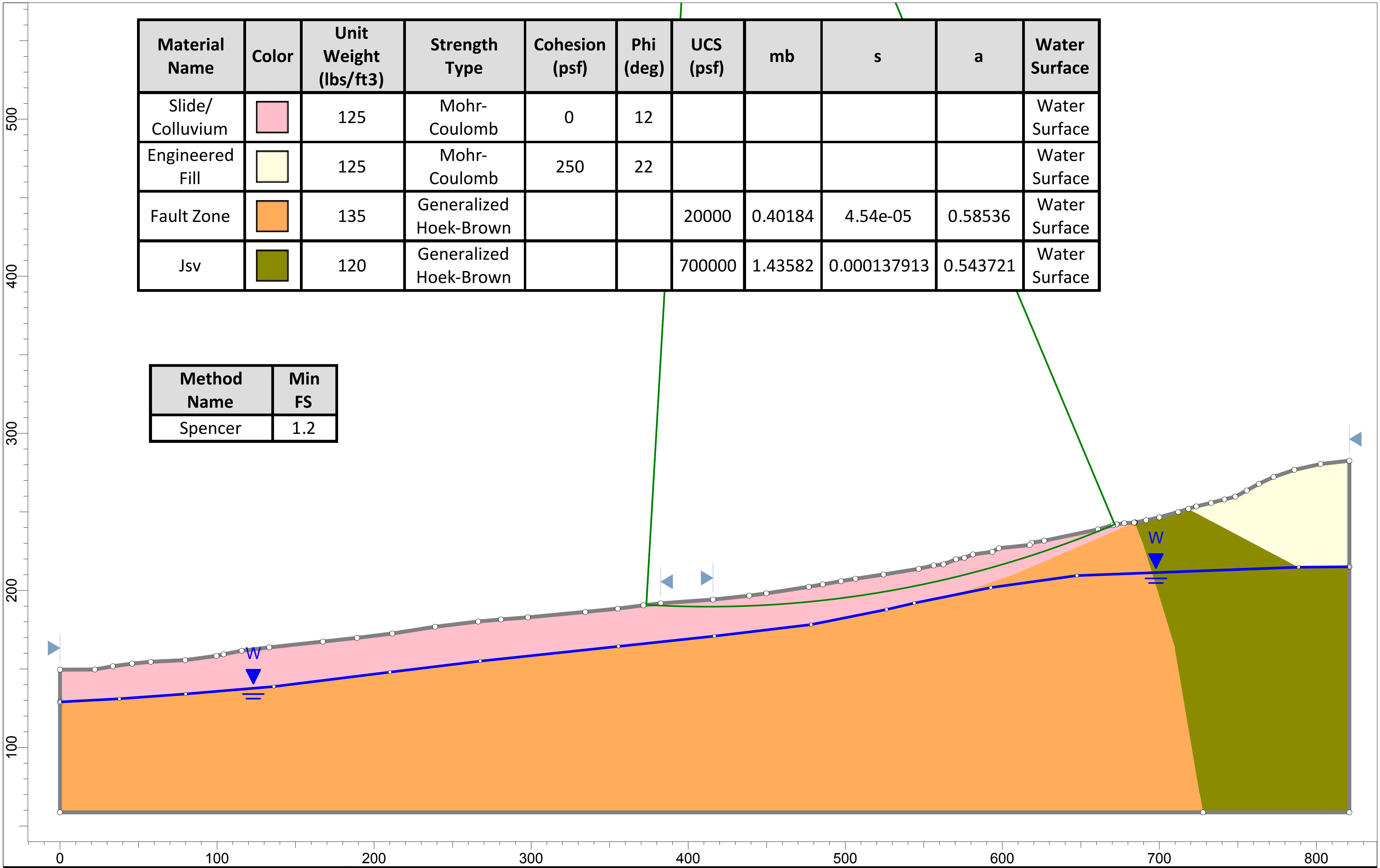
Date June 1, 2021
Job Number 15577.000.001



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟥	125	Mohr-Coulomb	0	12					Water Surface
Engineered Fill	⬜	125	Mohr-Coulomb	250	22					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown			20000	0.40184	4.54e-05	0.58536	Water Surface
Jsv	🟩	120	Generalized Hoek-Brown			700000	1.43582	0.000137913	0.543721	Water Surface

Method Name	Min FS
Spencer	1.2

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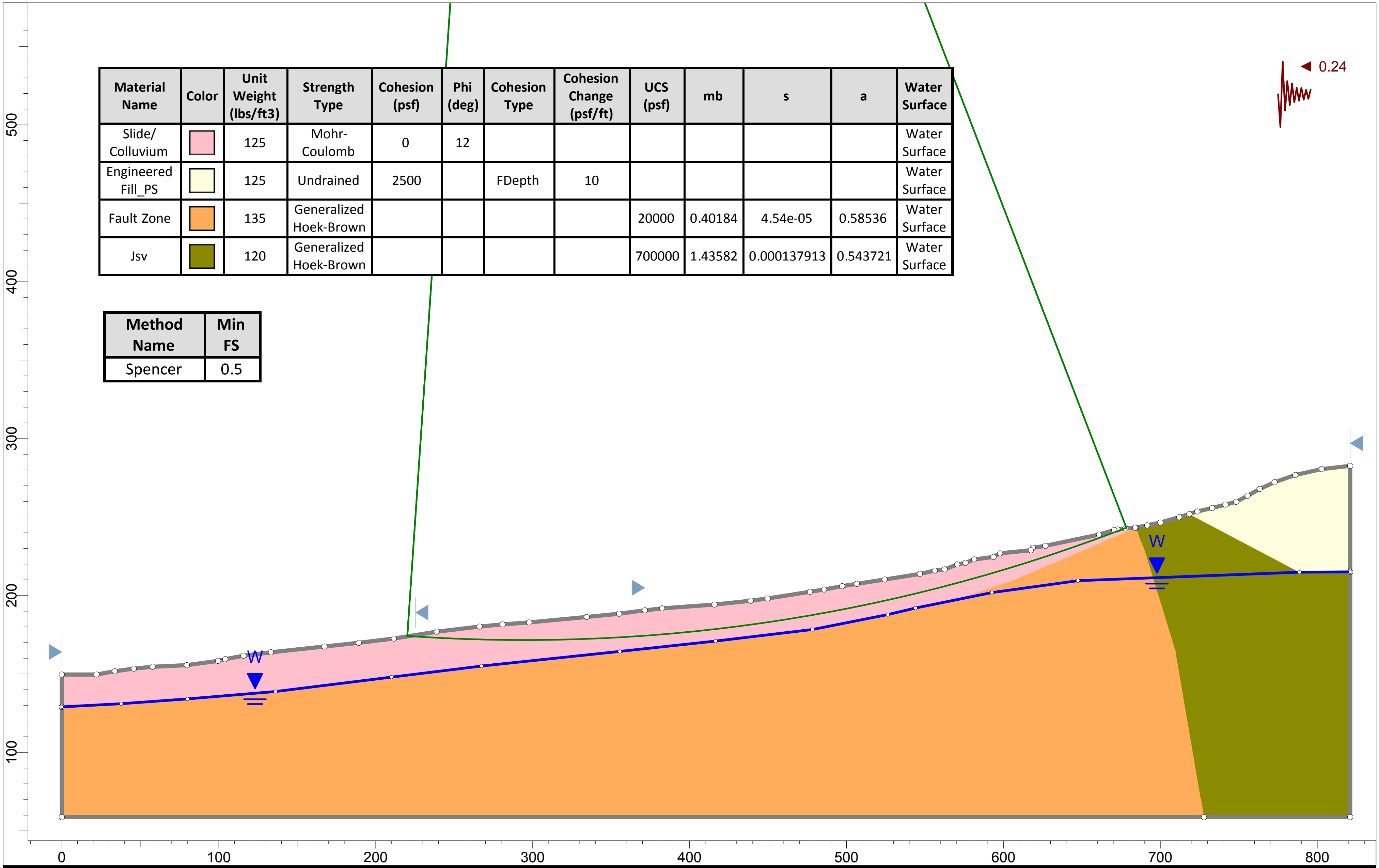
Project	Parcel Group 3 La Vista		
Scale 1:1000	*Author* APL	*Condition*	Section 2 Existing - Static
Date June 1, 2021	*Job Number* 15577.000.001		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟥	125	Mohr-Coulomb	0	12							Water Surface
Engineered Fill_PS	🟨	125	Undrained	2500		FDepth	10					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown					20000	0.40184	4.54e-05	0.58536	Water Surface
Jsv	🟩	120	Generalized Hoek-Brown					700000	1.43582	0.000137913	0.543721	Water Surface

Method Name	Min FS
Spencer	0.5

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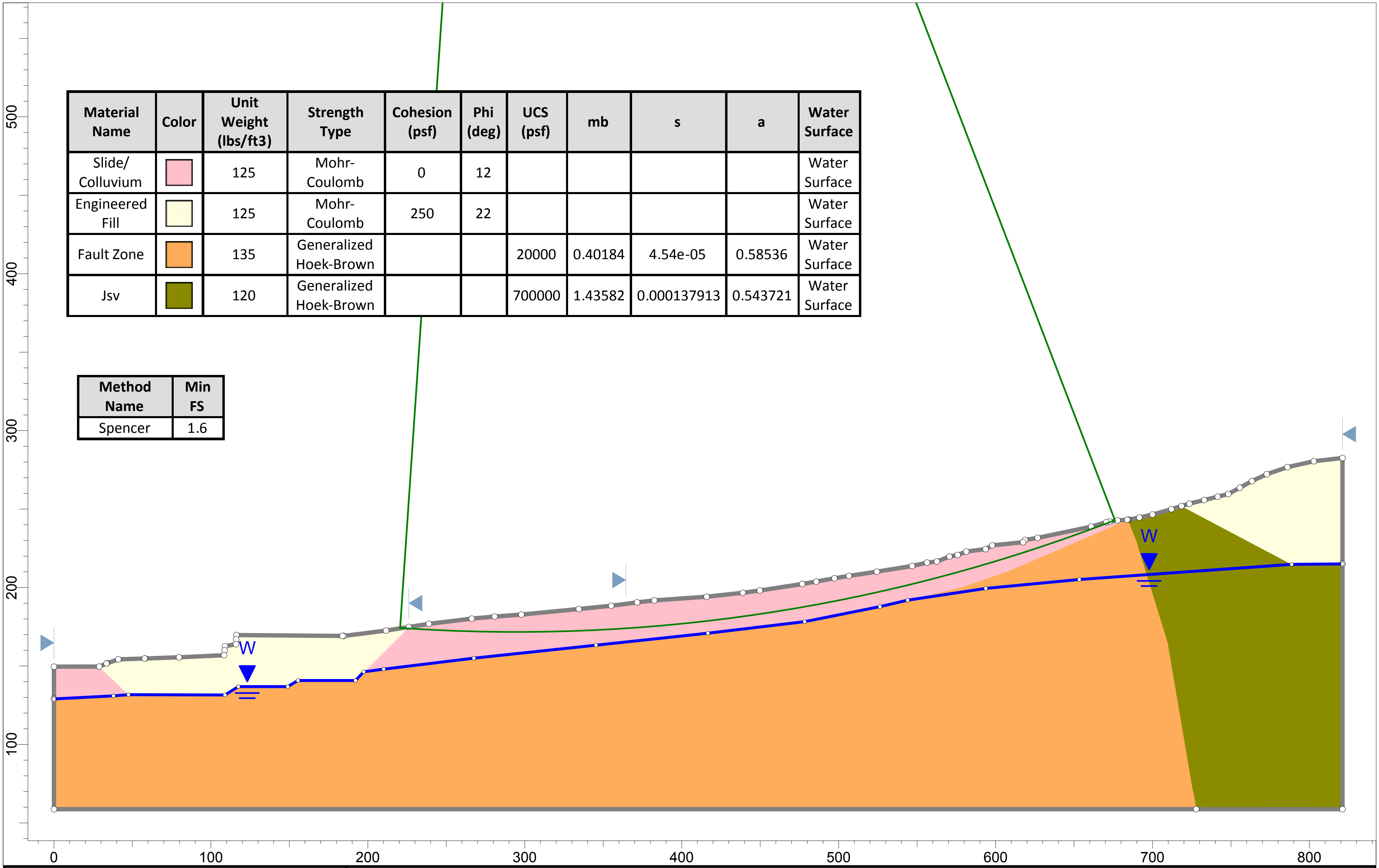
Project
Parcel Group 3
La Vista

Scale 1:1000
Author APL
Condition
Section 2
Existing - Pseudo-static
Date June 1, 2021
Job Number 15577.000.001



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟥	125	Mohr-Coulomb	0	12					Water Surface
Engineered Fill	⬜	125	Mohr-Coulomb	250	22					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown			20000	0.40184	4.54e-05	0.58536	Water Surface
Jsv	🟩	120	Generalized Hoek-Brown			700000	1.43582	0.000137913	0.543721	Water Surface

Method Name	Min FS
Spencer	1.6

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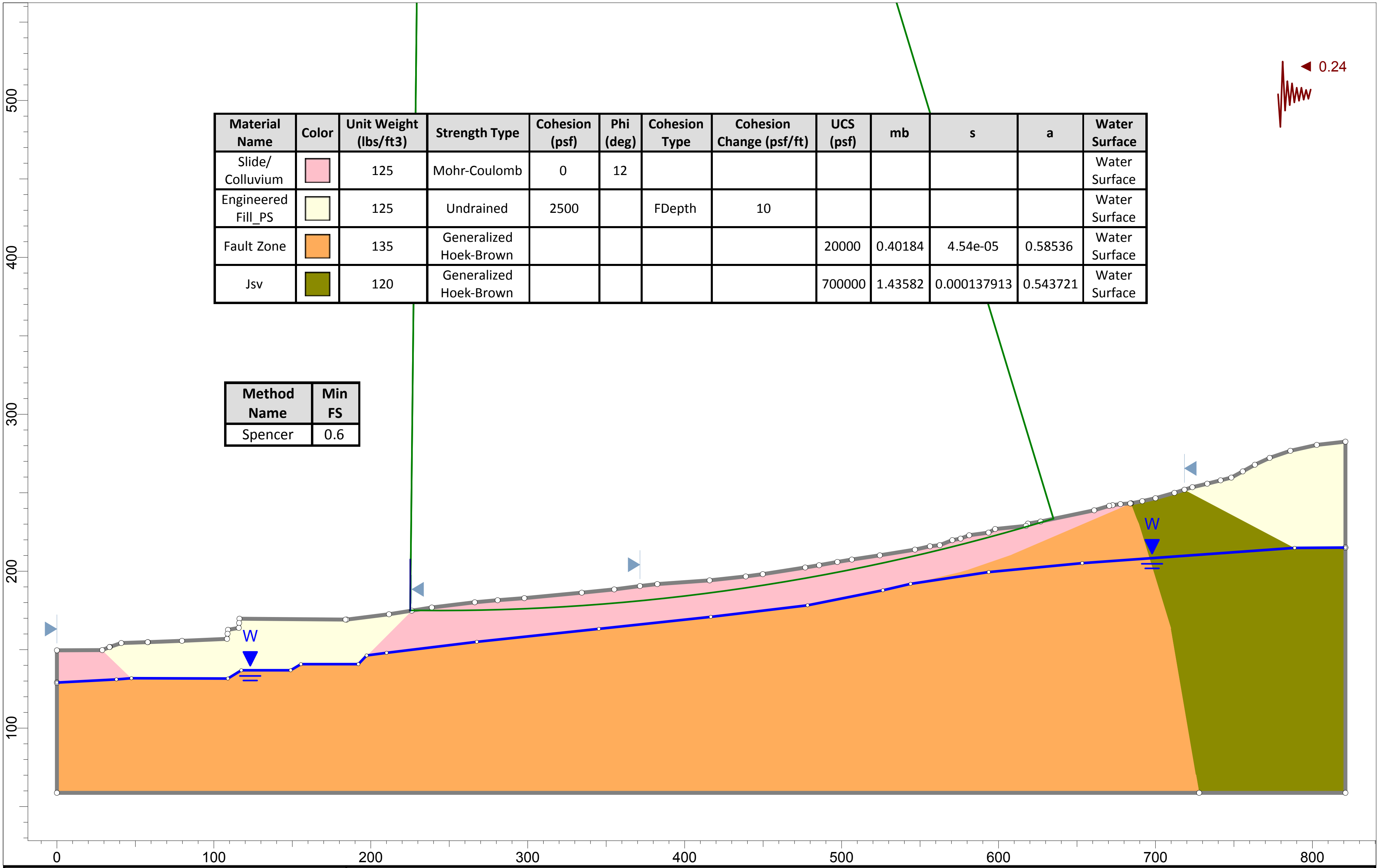
Project	Parcel Group 3 La Vista		
Scale	1:1000	*Author*	APL
Date	June 1, 2021	*Job Number*	15577.000.001
Condition	Section 2 Proposed - Static		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟥	125	Mohr-Coulomb	0	12							Water Surface
Engineered Fill_PS	🟨	125	Undrained	2500		FDepth	10					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown					20000	0.40184	4.54e-05	0.58536	Water Surface
Jsv	🟩	120	Generalized Hoek-Brown					700000	1.43582	0.000137913	0.543721	Water Surface

Method Name	Min FS
Spencer	0.6

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Project
Parcel Group 3
La Vista

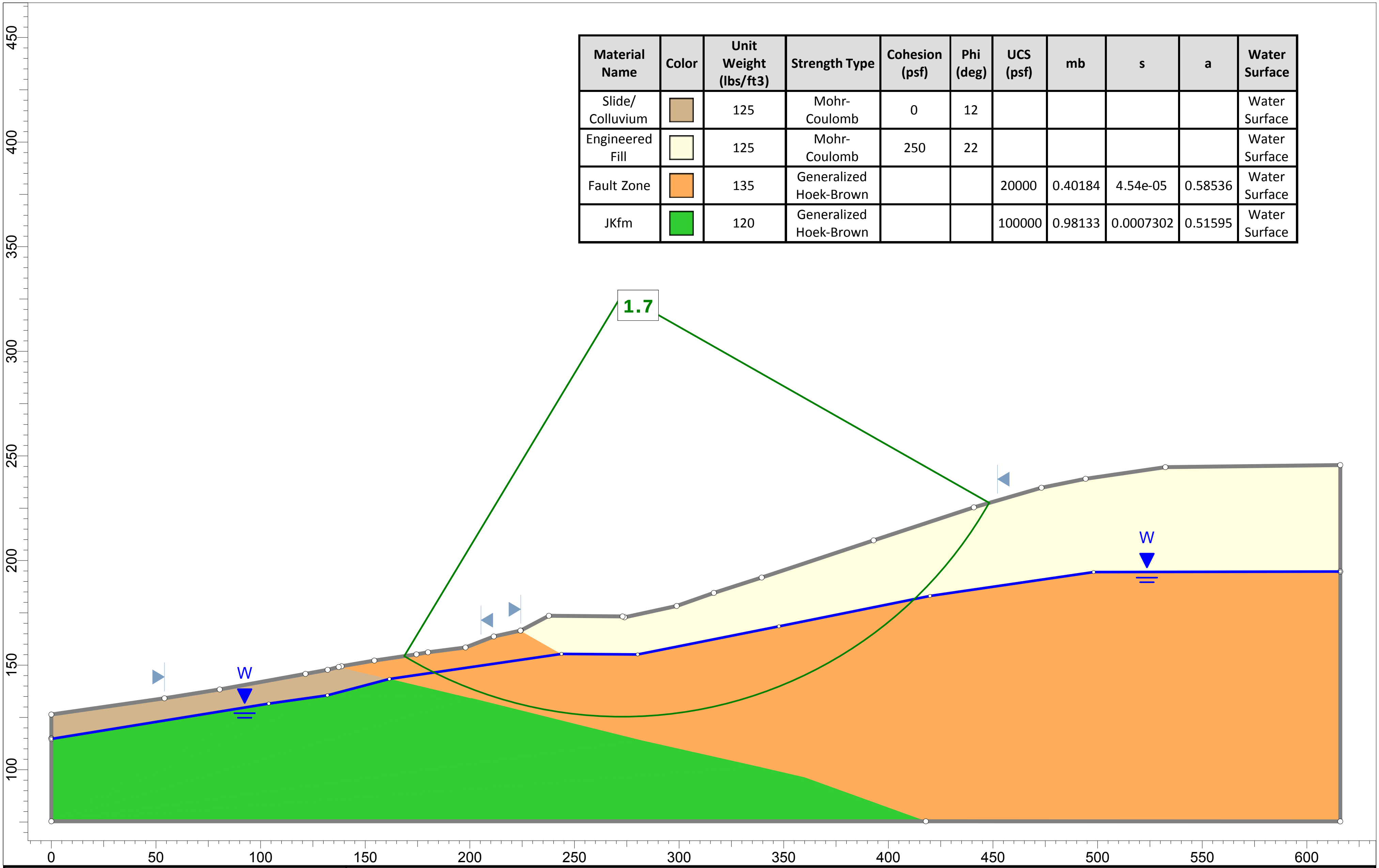
Scale 1:1000

Author APL

Condition Section 2
Proposed - Pseudo-static

Date June 1, 2021

Job Number 15577.000.001



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12					Water Surface
Engineered Fill	🟨	125	Mohr-Coulomb	250	22					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown			20000	0.40184	4.54e-05	0.58536	Water Surface
JKfm	🟩	120	Generalized Hoek-Brown			100000	0.98133	0.0007302	0.51595	Water Surface

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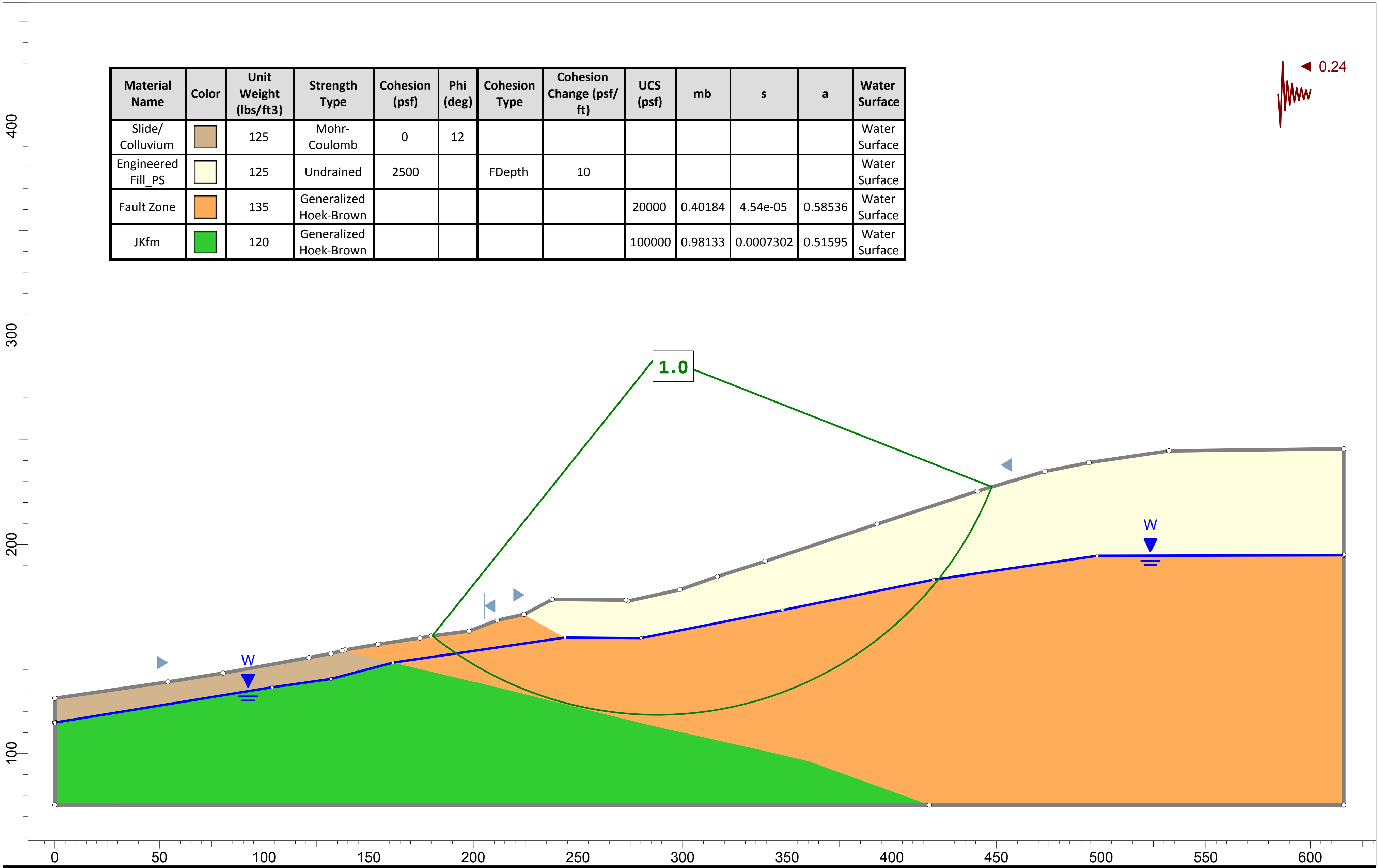
Project
Parcel Group 3
La Vista

Scale 1:750
Author APL
Condition Section 3
Existing - Static
Date June 1, 2021
Job Number 15577.000.001



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12							Water Surface
Engineered Fill_PS	⬜	125	Undrained	2500		FDepth	10					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown					20000	0.40184	4.54e-05	0.58536	Water Surface
JKfm	🟩	120	Generalized Hoek-Brown					100000	0.98133	0.0007302	0.51595	Water Surface

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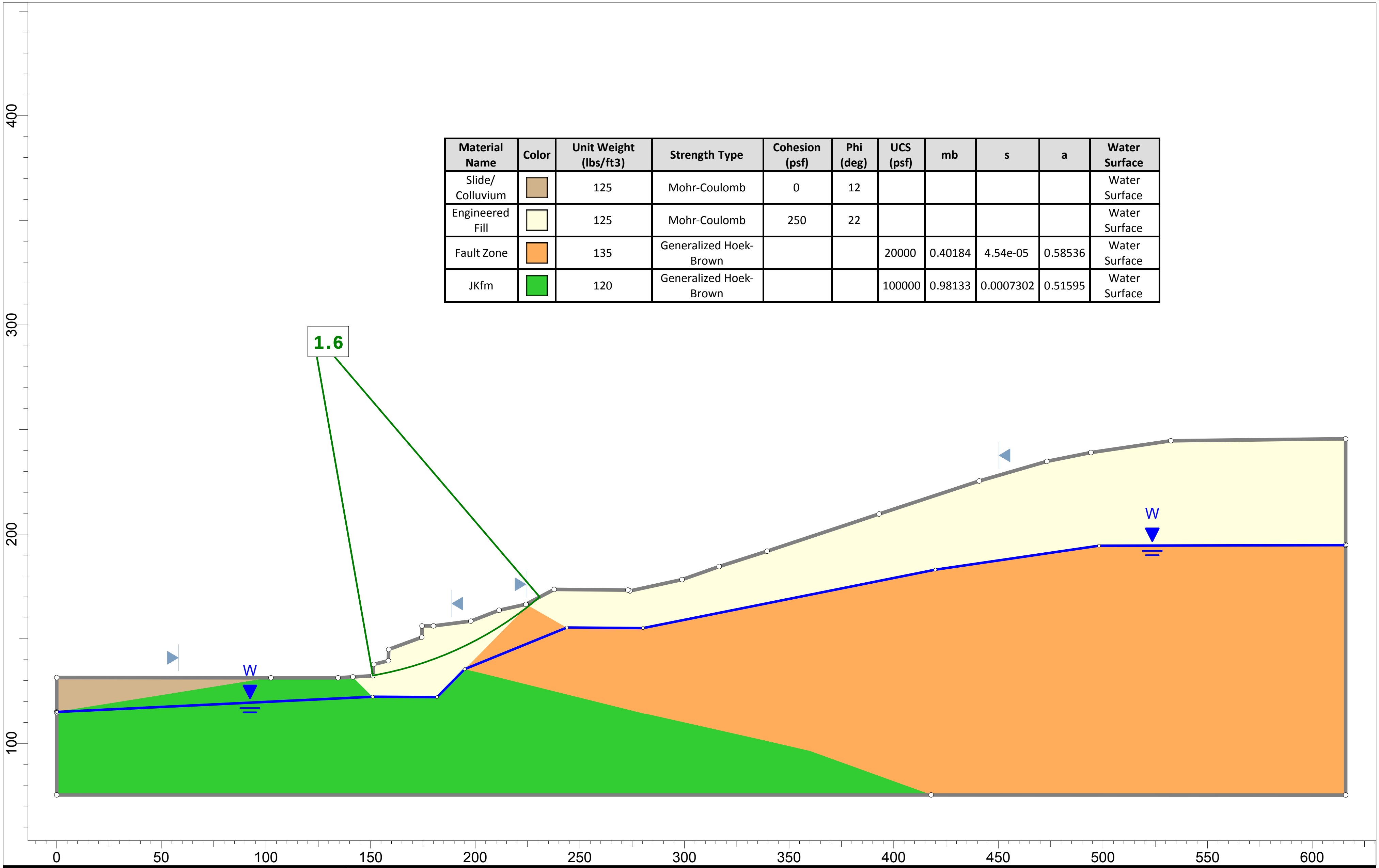
Project	Parcel Group 3 La Vista
Scale	1:750
Author	APL
Condition	Section 3 Existing - Pseudo-static
Date	June 1, 2021
Job Number	15577.000.001



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12					Water Surface
Engineered Fill	🟨	125	Mohr-Coulomb	250	22					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown			20000	0.40184	4.54e-05	0.58536	Water Surface
JKfm	🟩	120	Generalized Hoek-Brown			100000	0.98133	0.0007302	0.51595	Water Surface

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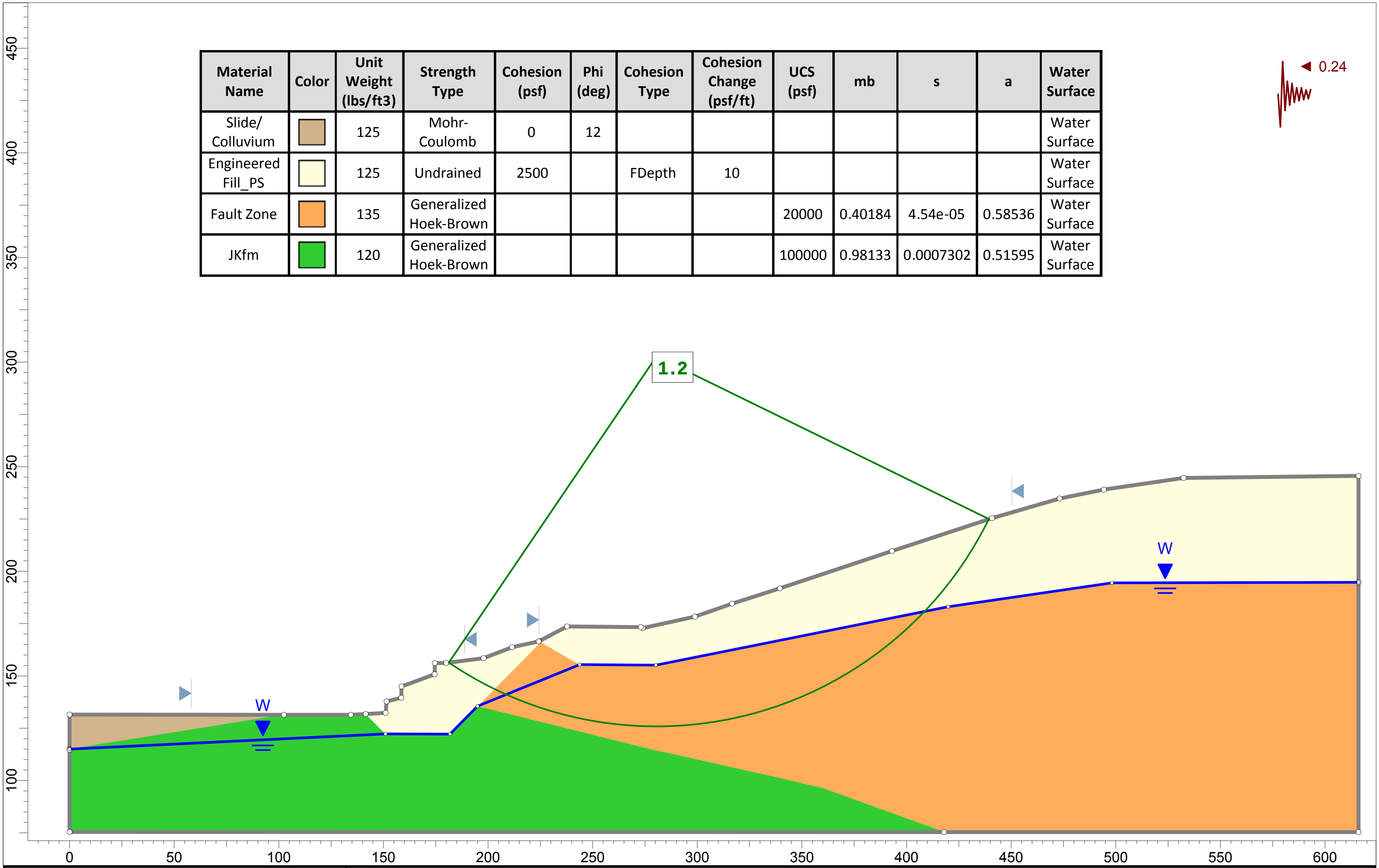
Project Parcel Group 3
La Vista

Scale 1:750
Author APL
Condition Section 3
Proposed - Static

Date June 1, 2021
Job Number 15577.000.001

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12							Water Surface
Engineered Fill_PS	🟨	125	Undrained	2500		FDepth	10					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown					20000	0.40184	4.54e-05	0.58536	Water Surface
JKfm	🟩	120	Generalized Hoek-Brown					100000	0.98133	0.0007302	0.51595	Water Surface



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Project

Parcel Group 3
La Vista

Scale 1:750

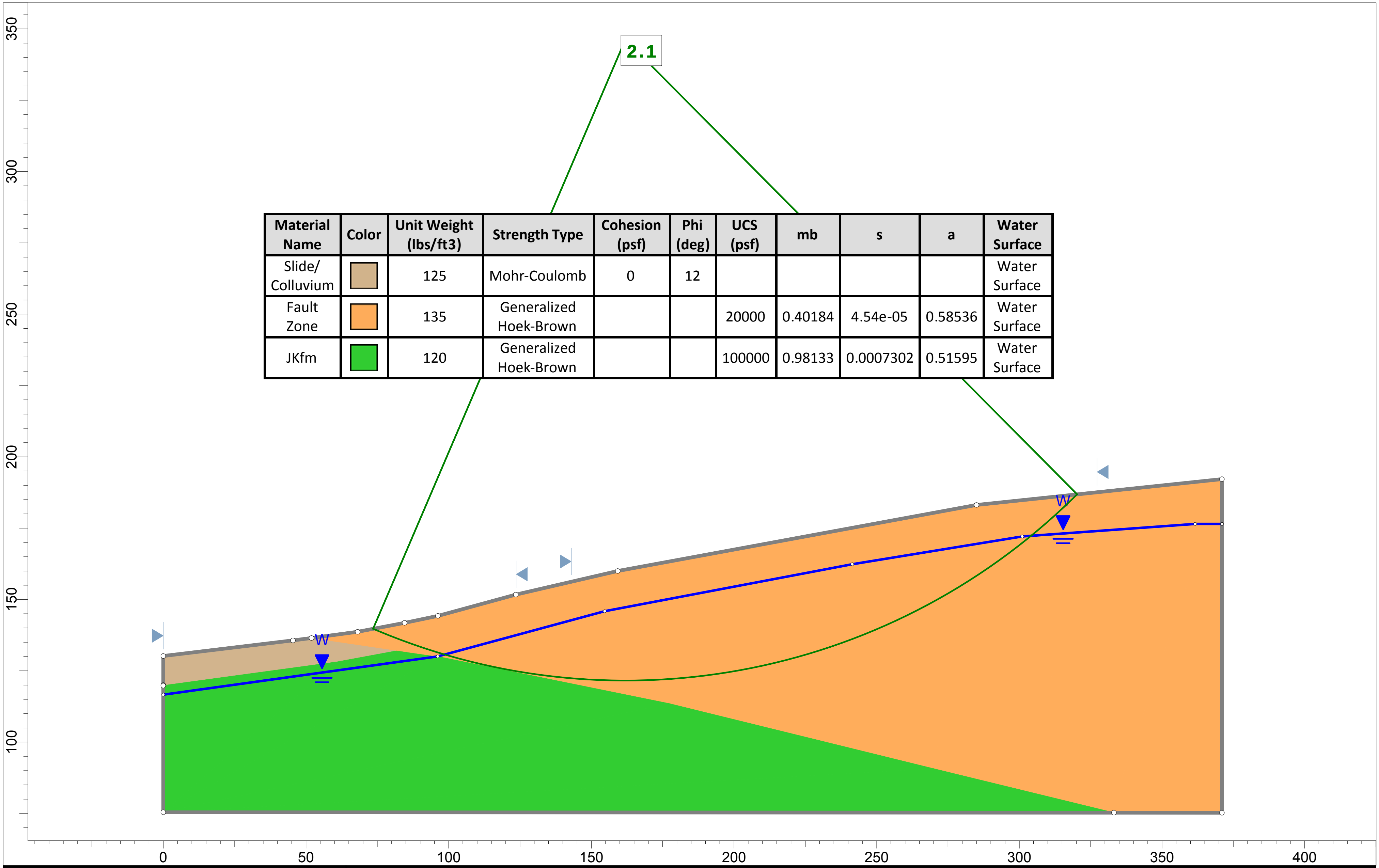
Author APL

Condition

Section 3
Proposed - Pseudo-static

Date June 1, 2021

Job Number 15577.000.001



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown			20000	0.40184	4.54e-05	0.58536	Water Surface
JKfm	🟩	120	Generalized Hoek-Brown			100000	0.98133	0.0007302	0.51595	Water Surface

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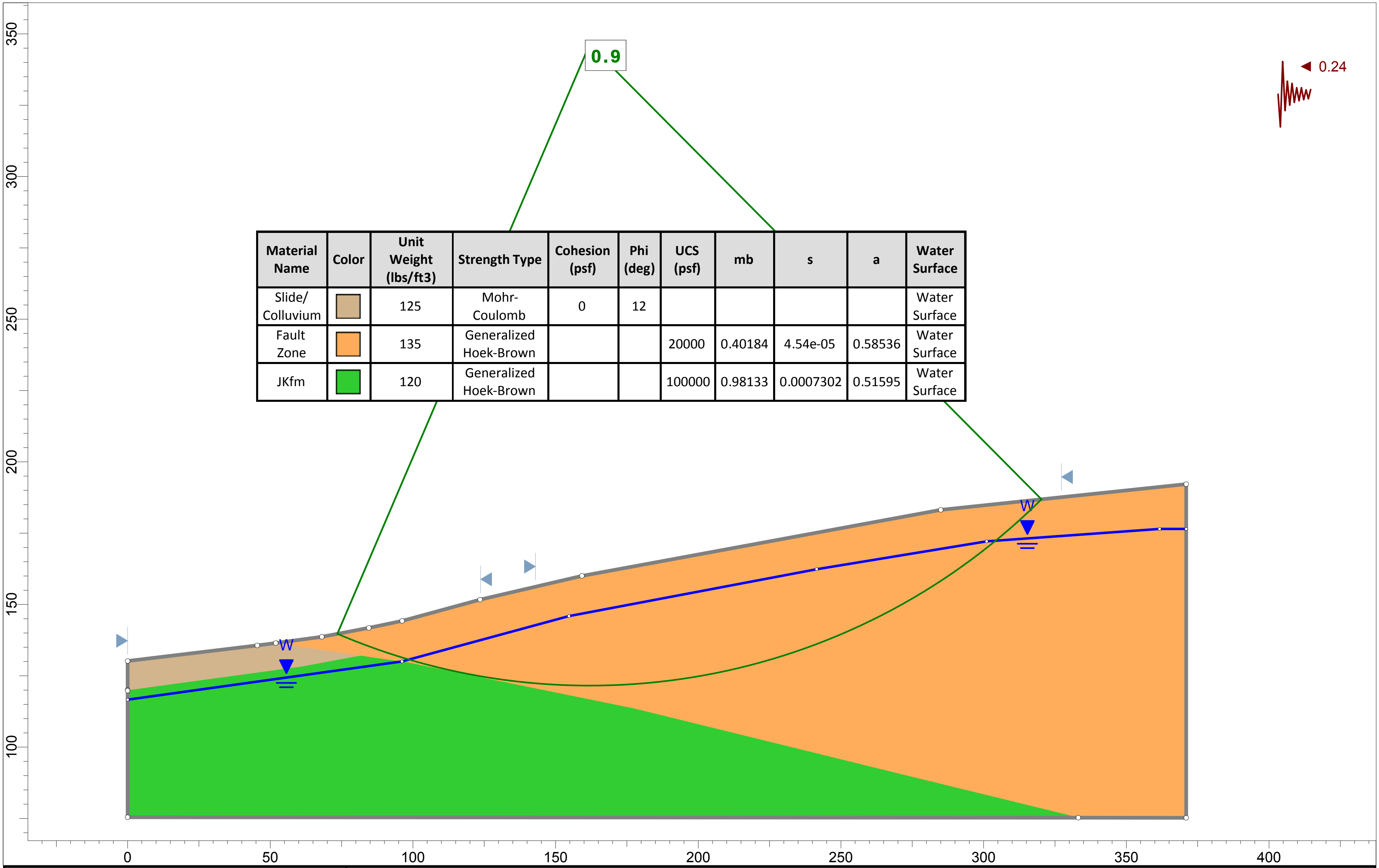
Project
Parcel Group 3
La Vista

Scale	1:550	*Author*	APL	*Condition*	Section 4 Existing - Static
Date	June 1, 2021	*Job Number*	15577.000.001		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown			20000	0.40184	4.54e-05	0.58536	Water Surface
JKfm	🟩	120	Generalized Hoek-Brown			100000	0.98133	0.0007302	0.51595	Water Surface

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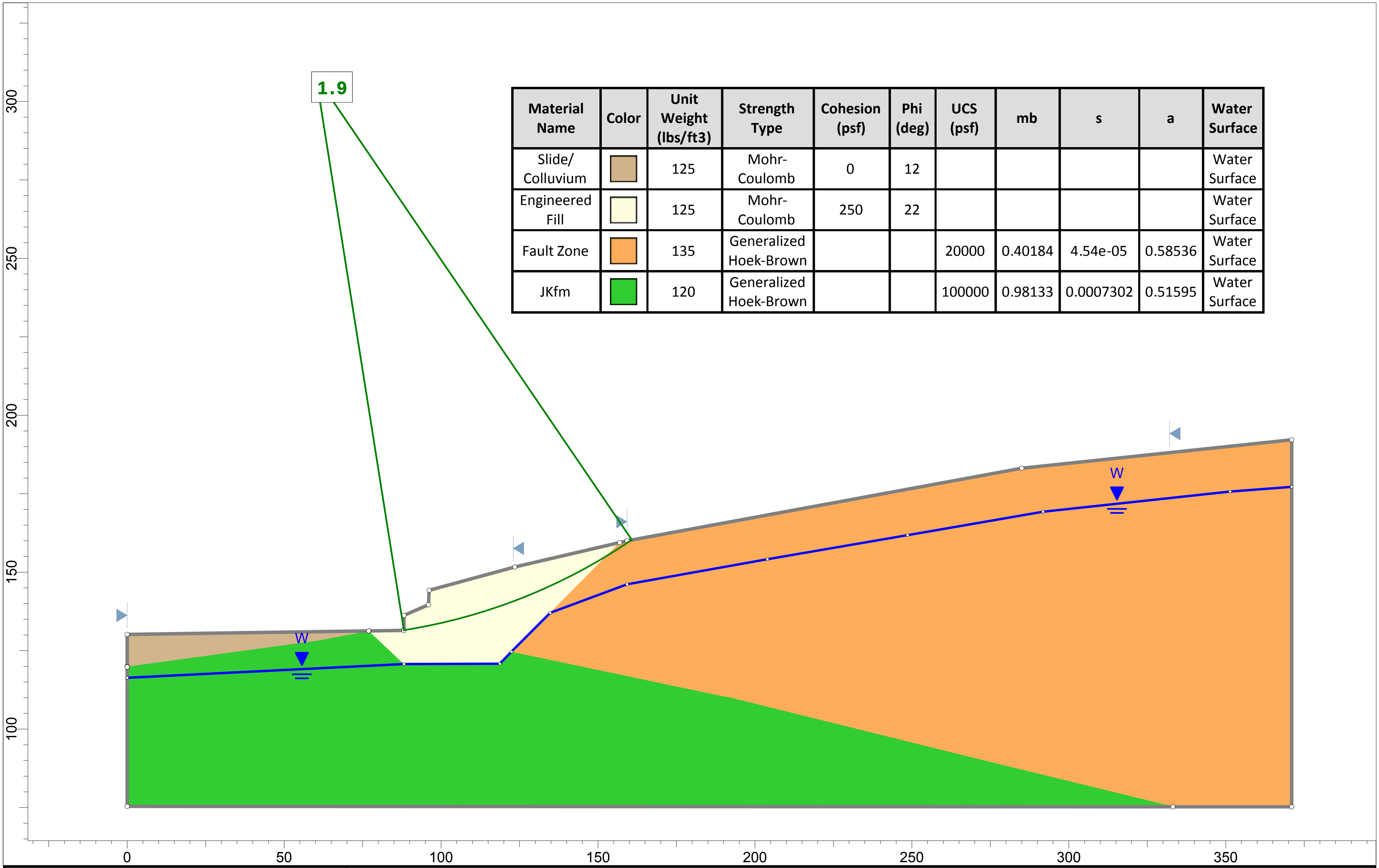
Project	Parcel Group 3 La Vista		
Scale	1:550	*Author*	APL
Date	June 1, 2021	*Job Number*	15577.000.001
Condition	Section 4 Existing - Pseudo-static		



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	UCS (psf)	mb	s	a	Water Surface
Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12					Water Surface
Engineered Fill	🟨	125	Mohr-Coulomb	250	22					Water Surface
Fault Zone	🟧	135	Generalized Hoek-Brown			20000	0.40184	4.54e-05	0.58536	Water Surface
JKfm	🟩	120	Generalized Hoek-Brown			100000	0.98133	0.0007302	0.51595	Water Surface

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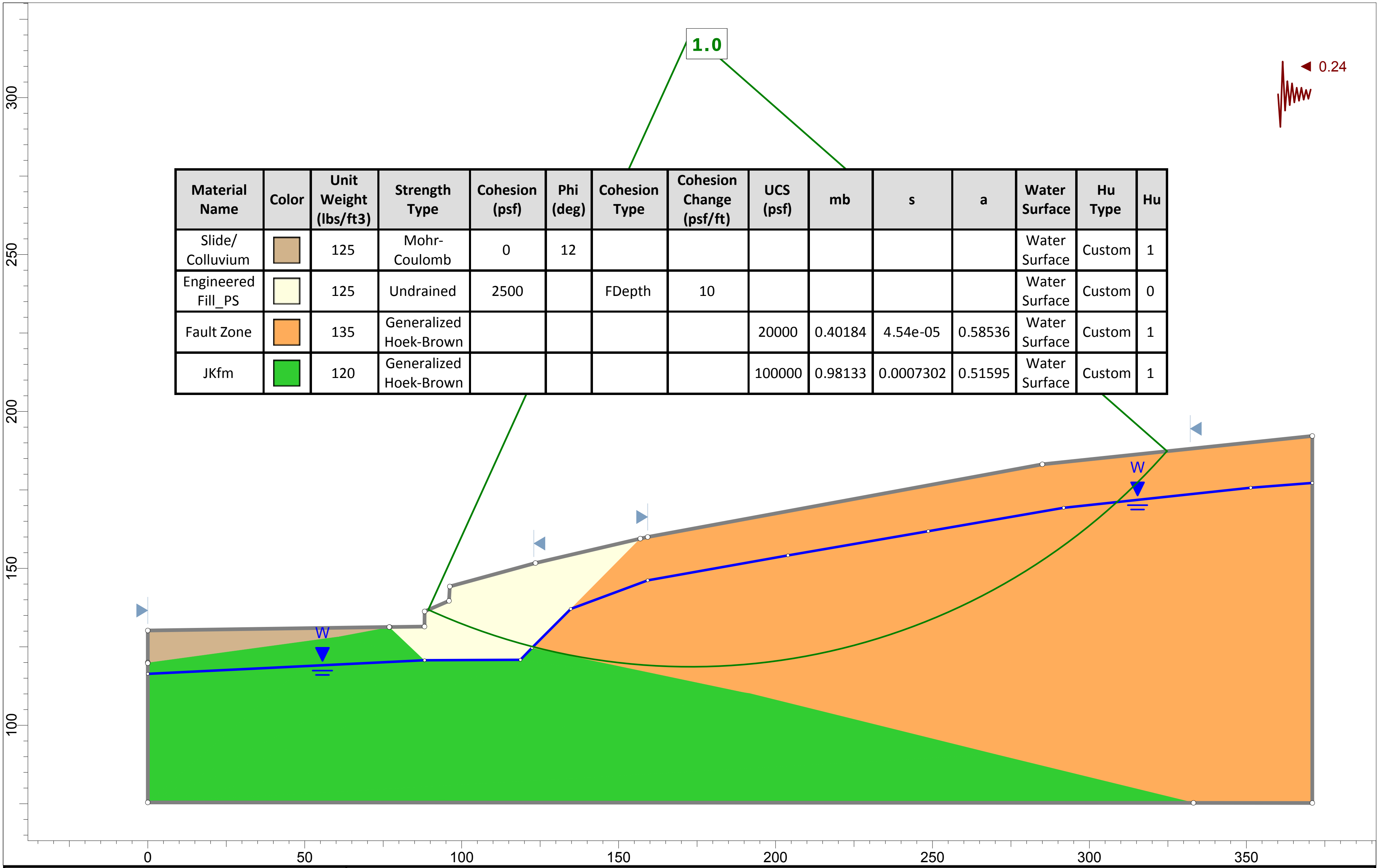
Project
Parcel Group 3
La Vista

Scale 1:500
Author APL
Condition Section 4
Proposed - Static
Date June 1, 2021
Job Number 15577.000.001



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)	UCS (psf)	mb	s	a	Water Surface	Hu Type	Hu
Slide/ Colluvium	🟫	125	Mohr-Coulomb	0	12							Water Surface	Custom	1
Engineered Fill_PS	🟨	125	Undrained	2500		FDepth	10					Water Surface	Custom	0
Fault Zone	🟧	135	Generalized Hoek-Brown					20000	0.40184	4.54e-05	0.58536	Water Surface	Custom	1
JKfm	🟩	120	Generalized Hoek-Brown					100000	0.98133	0.0007302	0.51595	Water Surface	Custom	1

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Project
Parcel Group 3
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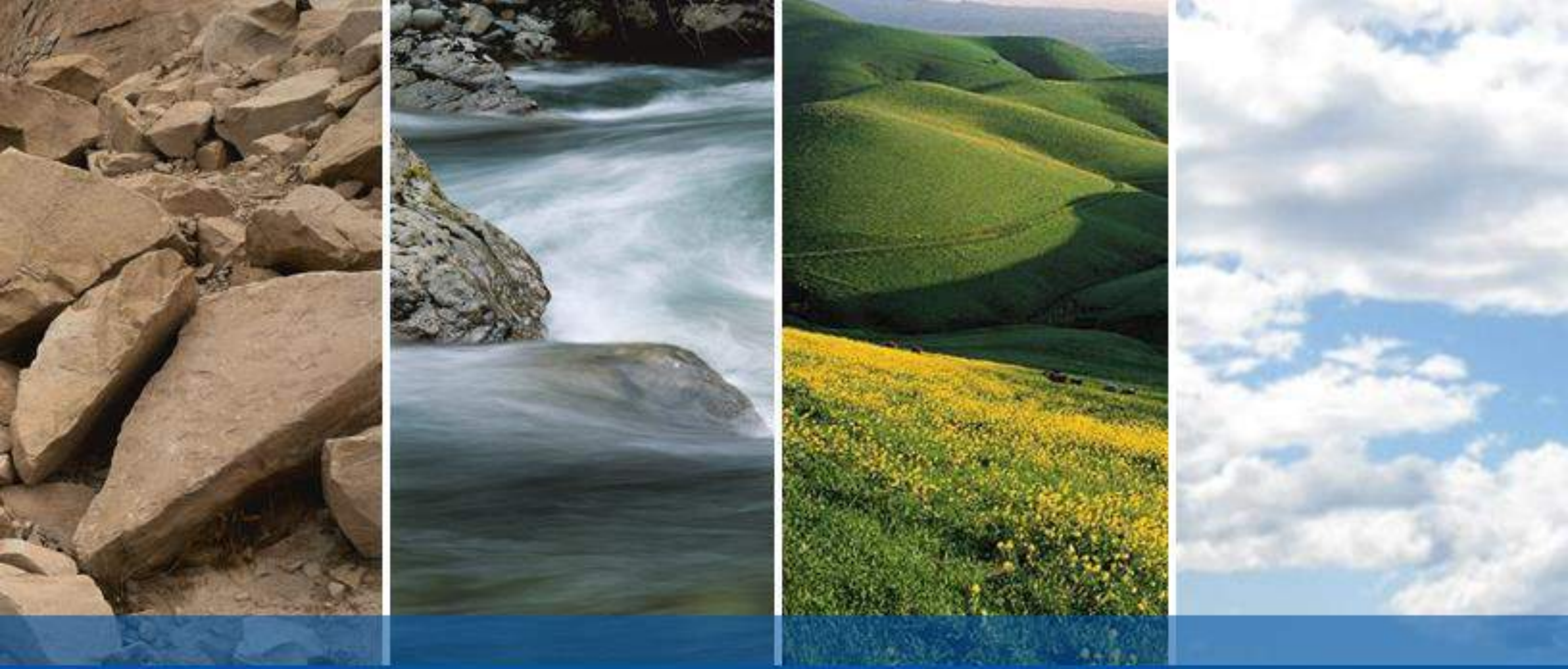
Scale 1:500

Author APL

Condition
Section 4
Proposed - Pseudo-static

Date June 1, 2021

Job Number 15577.000.001



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