APPENDIX B

Preliminary Geotechnical Exploration

790 PORTSWOOD DRIVE
SAN JOSE, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Justin P. Hu
Summerhill Homes
777 S. California Avenue,
Palo Alto, CA 94304

PREPARED BY
ENGEO Incorporated

November 21, 2019
Revised January 17, 2020

PROJECT NO.
16709.000.000

ENGEO
Expect Excellence

Project No.
16709.000.000

November 21, 2019
Revised January 17, 2020

Mr. Justin P. Hu
SummerHill Homes
777 S. California Avenue,
Palo Alto, CA 94304

Subject: 790 Portswood Drive
San Jose, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Hu:

We prepared this preliminary geotechnical report for the proposed residential development located at 790 Portswood Drive in San Jose, California as outlined in our agreement dated January 3, 2019. This report presents our geotechnical observations, as well as our preliminary conclusions and recommendations. We also provide preliminary site grading, drainage, and foundation recommendations for use during land planning.

Based upon our initial assessment, the proposed residential development at 790 Portswood Drive is feasible from a geotechnical perspective. Fault trenching and design-level exploration(s) should be conducted prior to site development once more detailed land plans have been prepared. 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

Hamish Foy

Robert H. Boeche, CEG

CERTIFIED ENGINEERING GEOLOGIST
ROBERT BOECHE
No. 2318
STATE OF CALIFORNIA

hf/rhb/dt

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical report for design planning of the proposed residential development at 790 Portswood Drive, San Jose, California. We prepared this report as outlined in our agreement dated January 3, 2019. SummerHill Homes authorized us to conduct the following scope of services:

- Review of published geologic maps and available data.
- Our scope comprised of conducting nine test pit excavations to a maximum depth of 15 feet.
- Prepare this preliminary geotechnical exploration report

This report was prepared for the exclusive use of our client and their consultants for evaluation 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 preliminary 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

Figure 1 displays a Site Vicinity Map. This site is located on a narrow corridor, oriented broadly in a north-south direction.

The site is bordered by single-family residential structures along the eastern and western boundaries, and is boarded to the north by Bret Harte Drive and to the south by Cahen Drive and Raich Drive in San Jose, California. The narrow corridor intersects Portswood Drive, the Almaden Expressway, and Hampswood Way. Figure 2 shows site boundaries, and our exploratory locations. The site is approximately 7.3 acres in area covering two parcels.

1.3 PROJECT DESCRIPTION

Based on our discussion with SummerHill Homes, we understand the following site improvements are proposed:

- We anticipate the development of 15 to 19 single-family housing units.
- Paved streets, parking and drive lanes.
- Utilities and other infrastructure improvements.
- Concrete flatwork.
- Water quality facilities.

Civil grading plans were not available for our review; however, based on the proposed development and site conditions, we anticipate minor cuts and fills. We anticipate building loads will be typical of the proposed structure type.

2.0 FINDINGS

2.1 SITE BACKGROUND

At the time of our evaluation, the site was occupied by 11 power poles with local distribution power cables between. The southernmost portion of the site is paved with asphalt while the narrow northern portion of the site is either grassed and vegetated or had baserock placed over the ground surface.

2.2 HISTORICAL TOPOGRAPHIC MAPS

As part of our study, we reviewed historical topographic maps to assess if improvements pertaining to the site had been recorded. We reviewed the topographic maps under the context of identifying general changes to the landform and history of site development.

TABLE 2.2-1: Historical Aerial Photography Summary

QUAD	YEAR	DESCRIPTION
New Almaden & Los Gatos	1916,1919	The site appears to have been developed with railway tracks and a railway station.
Los Gatos & Santa Teresa Hills	1940, 1943, 1947 & 1953	The railway tracks have since been removed from topographic map and orchards or agriculture has been developed adjacent to the site.
San Jose	1956, 1962, & 1966	No significant changes were observed from the 1953 topographic map.
San Jose	1978	No significant changes were observed from the 1953 topographic map.
Santa Teresa Hills	2012, 2015, & 2018	No significant changes were observed from the 1953 topographic map.

2.3 HISTORICAL AERIAL PHOTOGRAPHY

As part of our study, we reviewed historical aerial photographs, stereo-paired aerial photographs, and Google Earth images dating from 1948 to 2018. We viewed the photographs under the context of identifying general changes to the landform and history of site development.

TABLE 2.3-1: Historical Aerial Photography Summary

DATE	DESCRIPTION
1948 to 1968 Aerial Photograph Series	Used as an accessway for an orchard or agricultural purposes.
1980 Aerial Photograph Series	The southern portion of the site is being used as a storage area for either a business or the earthworks associated with the adjacent residential development. The narrow corridor on the northern portion of the site was being used as an accessway.
1982 Aerial Photograph Series	The southern portion of the site is vacant and has been sealed (likely with asphalt). The narrow corridor on the northern portion of the site was being used as an accessway.
1987 to 1993 Aerial Photograph Series	The southern portion of the site has various (unknown) materials stored onsite. The narrow corridor on the northern portion of the site was being used as an accessway.

DATE	DESCRIPTION
1998 to 2016 Aerial Photograph Series	The southern portion of the site is vacant and has had power lines and power poles constructed. The narrow corridor on the northern portion of the site was being used as a corridor for electrical services. No major changes were visible from 1998 to 2016.
2016 to 2018 Google Earth	No changes were visible from 2016 to 2018

Aside from the observed changes summarized in Table 2.3-1, and vegetation changes over time, no other significant or large-scale geomorphic changes were noted in the historical aerial photograph review.

2.4 REGIONAL GEOLOGY AND SEISMICITY

2.4.1 Geology

The study area is located within the Coast Ranges geomorphic province of California. The Coast Ranges are dominated by a series of northwest-trending mountain ranges that have been folded and faulted in a tectonic regime that involves both translational and compressional deformation.

Regional geologic maps locate the site in the broad, northwest-southeast trending, alluvial filled Santa Clara Valley. Regional geologic mapping prepared by Dibblee et. Al. (2005) indicates the site is underlain by alluvial fan deposits (Qa), submetamorphosed sedimentary rocks of the Franciscan Assemblage (fs and fg) and serpentinite of the Coast Range ophiolite complex as shown on Figure 3.

2.4.2 Seismicity

The San Francisco Bay Area contains numerous active earthquake faults. Nearby active faults are listed in Table 2.4.2-1. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Bryant and Hart, 2007). Figure 5 shows the approximate locations of these faults and significant historic earthquakes recorded within the San Francisco Bay Region.

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone. However, the northernmost portion of the site is located in the Santa Clara County Fault Rupture Hazard Zone. The active Monte Vista-Shannon fault is mapped intersecting the northern section of the site. The active faults mapped within 20 miles of the site are listed in Table 2.4.2-1 by proximity to the site with their estimated maximum moment magnitude.

TABLE 2.4.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site
Latitude: 37.203053 **Longitude:** -121.832973

FAULT NAME	DISTANCE FROM SITE (MILES)	MAXIMUM MOMENT MAGNITUDE
Monte Vista-Shannon	Onsite	6.5
North San Andreas	7.8	8.1
Calaveras	10.3	7.0
Hayward-Rodgers Creek	16.7	6.7

The Working Group on California Earthquake Probabilities (WGCEP, 2008) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area. The UCERF generated an overall probability of 63 percent for the Bay Area as a whole, and a probability of 31 percent for the Hayward fault, 21 percent for the Northern San Andreas fault, and 7 percent for the Calaveras fault.

2.5 FIELD EXPLORATION – TEST PITS

Our field exploration included excavating nine test pits to a maximum depth of approximately 14½ feet using a 13t tracked excavator with a 2-foot-wide rock bucket. The test pits were backfilled in layers following field exploration activities using normal compactive effort by the bucket. We performed our field exploration on November 6 and November 7, 2019.

The location and elevations of our explorations are approximate. We estimated the locations of features shown on Figure 2; they should be considered accurate only to the degree implied by the method used. Test pit logs are presented in Appendix A.

2.6 SURFACE CONDITIONS

The site is currently an operating electrical services corridor and is occupied by power lines and power poles.

- In the southern portion of the site includes power poles and the ground surface is asphalted (which was proposed to be a substation area but was never constructed).
- In the northern portion of the site includes power poles and either vegetated, sparsely planted with trees, or the ground surface has been prepared with baserock.

2.7 SUBSURFACE CONDITIONS

In the southern portion of the site approximately 4 inches of asphalt was encountered at the surface in test pits 1, 2 and 3. The asphalt was over 4 to 6 inches of baserock aggregate. The test pits in this area generally encountered 1 to 3 feet of dry to moist re-worked gravelly silt or sandy silt (fill) followed by native alluvial sandy gravel with trace silt, cobbles and boulders. We estimated the density of this material from the test pit excavation and assessed the density varied, ranging from loose to dense. We did not encounter bedrock in the test pits excavated in the southern portion of the site (test pits 1, 2, and 3).

In the narrow corridor northern portion of the site, in test pits 7 and 8, we encountered 4 to 6 inches of baserock aggregate. The remaining test pits (4, 5, 6 and 9) did not encounter baserock. All of the test pits in the northern portion of the site encountered between 2 and 8 feet of dry to moist sandy silt, silty sand, or sandy gravel (fill). The density of the fill estimated during excavation, ranged from very loose to medium dense. This was generally followed by alluvial sandy gravel with trace silt, cobbles and boulders to a maximum depth of 14½ feet. In test pit 7, we observed sandy silt at 8 feet deep, followed by alluvial sandy gravel with trace silt, cobbles and boulders.

In test pit 6, we observed bedrock at approximately 7 feet deep below sandy gravel (fill). This was the only test pit that encountered bedrock in the explorations completed during our investigation onsite.

Given the previous use of the site, the near surface re-worked soil (up to 8 feet deep) was likely reworked or placed to provide a level site for the original use of a railway and interchange noted on the topographic maps between 1916 to 1919. While this near-surface soil is likely native to the site, due to the previous use and absence of grading records, it should be considered non-engineered fill from an engineering standpoint.

The Site Plan (Figure 2) provide the location of each test pit location.

2.8 GROUNDWATER CONDITIONS

Groundwater was not encountered in our test pit excavations. Groundwater mapping in the Seismic Hazard Zone Report for the Mountain View 7.5-Minute Quadrangle, Santa Clara, Alameda, and San Mateo Counties, CA (CGS, 2006) indicates groundwater may be encountered at approximately 30 to 50 feet bgs at the site depending on the site location. The groundwater levels at the site may fluctuate with time due to seasonal conditions, rainfall, and irrigation practices.

ENGEO also reviewed groundwater data from the Department of Water Resources and environmental cases in the site vicinity. Generally, it appears that depth to water west of Los Alamitos Creek (including the site area) has been reported at approximately 30 to 50 feet, and properties east of the Los Alamitos Creek have been reported at approximately 10 feet bgs. This corresponds with the creek bed generally being 20 feet lower than upland areas to the west.

3.0 PRELIMINARY CONCLUSIONS

From a geotechnical engineering standpoint, the site is suitable for the proposed development, provided the preliminary geotechnical recommendations in this report and future design-level geotechnical exploration studies are properly incorporated into the design plans and specifications.

A design-level geotechnical exploration should be performed as part of the design process. The exploration may include borings, additional test pits, and additional laboratory soil testing to provide data for preparation of specific recommendations regarding grading, foundation design, and drainage for the proposed development. The exploration will also allow for more detailed evaluations of the geotechnical issues, discussed below, and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

The primary geotechnical concerns that could affect development on the site are non-engineered fill and liquefaction hazards. We summarize our conclusions below.

3.1 NON-ENGINEERED FILL

Existing non-engineered fill was encountered in all of our test pits to various depths. We encountered non-engineered fill up to 4 feet deep in Test Pits TP01 to TP05, TP08 and TP09, up to 8 feet of fill in TP06, and TP07.

Disturbed native and non-engineered fills can undergo excessive settlement, especially under new fill or building loads. As non-engineered soil is prone to settlement under new structural loads or may exhibit volume loss when compacted during grading operations. To mitigate the effects of the disturbed near-surface materials, we recommend complete removal and recompaction of the fill observed onsite from any location to be graded, from areas to receive fill or structures, and from areas to serve as borrow. Section 4.6 provides recommendations for fill subgrade preparation to address this material.

3.2 EXPANSIVE SOIL

Our test pits encountered variable soil materials near the ground surface that predominantly consisted of coarse alluvial sandy and gravelly soils with some or trace clay and silt depending on the test location. With our experience with similar soils in the vicinity of the site, indicate that these soils are unlikely to be potentially expansive. However, as expansive soils change in volume with changes in moisture, they can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. The design-level geotechnical report should investigate any potentially expansive soil, and include suitable laboratory testing and provide mitigation alternatives (if required) based on the final development details and layout.

Based on the conditions encountered, and our experience with similar developments in the area, it is our opinion that post-tensioned mat foundations may be the preferred foundation system for the proposed structures. This foundation type is also generally suitable to mitigate expansive soil conditions if encountered during the design level investigation. Preliminary design criteria for this foundation type are presented in Section 5.2.

3.3 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, liquefaction, 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, landslides, tsunamis, flooding or seiches is considered low at the site.

3.3.1 Ground Rupture

As discussed in Section 2.4.2, the site is located in the Santa Clara County Fault Rupture Hazard Zone. The trace of the Monte Vista Shannon fault has been mapped traversing the northern portion of the site and is depicted as an undifferentiated Quaternary age reverse fault.

We did not observe any lineaments crossing the site during our site assessment or in the aerial photographs reviewed. Additionally, we did not observe the vegetation lineament mapped to the onsite or proximal to the site.

However, we recommend to trench perpendicular to where the fault is likely intersecting the northern side of the site, in an attempt to intercept the trace of the fault and assess if the fault trace is present within the subject site. Appropriate mitigation measures (as necessary) can be recommended following this investigation.

3.3.2 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, all 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).

3.3.3 Liquefaction

Liquefaction is the loss of strength to soil layers due to cyclic loading or seismic shaking. Generally, loose coarse-grained material will undergo liquefaction under a seismic event. Based on observations of soil behavior under seismic shaking and laboratory testing, some fine-grained material, such as silt and clay, can also undergo liquefaction or cyclic softening. In order for a soil to be potentially liquefiable, it must be saturated.

While the Association of Bay Area Governments Resilience Program’s online Liquefaction Susceptibility Map shows the site is mapped adjacent to an area of moderate liquefaction susceptibility, clean, saturated sands were not encountered in our test pits. However, as the depth of our preliminary assessment was up to 14½ feet below ground surface, future design-level geotechnical explorations should further evaluate liquefaction potential onsite at depth.

3.3.4 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Generally, effects of lateral spreading are most significant at the free face or the crest of a slope and diminish with distance from the slope.

A roughly 5 to10-foot high break-in-slope descending at a gradient of approximately 2:1 (horizontal:vertical) is present along the eastern side of the northern portion of the site. We did not observe saturated or potentially liquefiable soils in the upper 14½ feet during our investigation. The potential for lateral spreading will be assessed during design-level study.

3.3.5 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area region, but based on the site location, the offset is expected to be minor.

3.4 LOOSE/COMPRESSIBLE SOILS

Although subsurface exploration was not performed as part of this study, due to past historic use and seasonal tilling/disking, the near-surface soils are anticipated to be loose/compressible and portions of the subsurface material located below groundwater levels may be potentially compressible as well. Compressible soils may be subject to load-induced settlement (compression) when subjected to new loads. Remedial grading to rework the near-surface soils as engineered fill can generally address this issue. Future design-level study can provide detailed assessment and recommendations associated with these soil type (if applicable).

3.4.1 Flooding

We reviewed the Federal Emergency Management Agency (FEMA) Flood Maps for the City of San Jose. The site is mapped as Zone D, as defined as an area with possible but undefined flood hazard.

The Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, as needed.

3.5 SOIL CORROSION POTENTIAL

Determination of soil corrosion potential was beyond the scope of this preliminary geotechnical report. Our experience with similar sites in the vicinity of this project indicate that site soils may be moderately to severely corrosive. We recommend that soil corrosion potential be addressed during a design-level geotechnical exploration report. At that time and as part of a design-level study, we recommend representative soil samples be collected and submitted to a qualified analytical lab for determination of pH, resistivity, sulfate, and chloride.

3.6 2019 CBC SEISMIC DESIGN PARAMETERS

If the proposed development is permitted in 2020, the design will be based on the 2019 California Building Code (CBC). The 2019 CBC utilizes design criteria set forth in the ASCE 7-16 Standard. Based on our local experience, we anticipate the site will be characterized as Site Class D in accordance with the 2019 CBC. We provide the 2019 CBC seismic design parameters in Table 3.6-1 below, which include design spectral response acceleration parameters based on the mapped Risk Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters. Note that ASCE 7-16 requires a site-specific seismic hazard analysis at Site Class D sites such as this with a mapped S_1 value greater than 0.2. However, Section 11.4.8 of ASCE 7-16 provides an exception to this requirement. If the structural engineer decides to take this exception, then a seismic hazard analysis is not required. If the structural engineer chooses not to take this exception, we can perform a seismic hazard analysis to develop a site-specific MCER and design acceleration response spectra.

TABLE 3.6-1: 2019 CBC Seismic Design Parameters, Latitude: 37.20335 **Longitude:** -121.83326

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	2.26
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.82
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.26
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.23
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.51
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.82
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.80
Site Coefficient, F_{PGA}	1.00
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.80
Long period transition-period, T_L	12 sec

4.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

The following preliminary recommendations are for initial land planning and preliminary estimating purposes. Final recommendations regarding site grading and foundation construction will be provided after additional design-level geotechnical exploration has been undertaken.

4.1 GENERAL SITE CLEARING AND STRIPPING

Grading operations should be observed and tested by our qualified field representative. We should be notified a minimum of three days prior to grading in order to coordinate our schedule with the grading contractor.

Site development will commence with the removal of existing improvements and their foundations, and buried structures, including abandoned utilities and their backfill. All debris or soft compressible soil should be removed from any location to be graded, from areas to receive fill or structures, and from those areas to serve as borrow. Because the site was previously used for railway and an accessway, we typically expect that a minimum of the upper 2 to 3 feet of soil (up to 8 feet) will need to be reworked to produce appropriately moisture conditioned and compacted material. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

Existing vegetation should be removed from areas to receive fill or structures, or those areas to serve for borrow. Tree roots should be removed (as required) down to a depth of at least 3 feet below existing grade. The actual depths of tree root removal should be determined by the Geotechnical Engineer's representative in the field. Subject to approval by the Landscape Architect, strippings and organically contaminated soils can be used in landscape areas. Otherwise, such soil should be removed from the study areas. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping is permitted.

4.2 SELECTION OF MATERIALS

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils (if any), we anticipate the site soil is suitable for use as engineered fill provided they are broken down to 6 inches or less in size. Other materials and debris, including trees with their root balls, should be removed from the study areas.

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

4.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible.

Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather;
2. Mixing with drier materials;
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated and approved by the Geotechnical Engineer prior to implementation.

4.4 GRADED SLOPES

In general and for preliminary purposes, graded slopes should be no steeper than 2:1 (horizontal:vertical).

4.5 DIFFERENTIAL FILL THICKNESS

Depending upon cuts associated with removal of undocumented fills, differential fill thickness conditions could possibly arise.

For subexcavation activities that create a differential fill thickness across the building footprint, mitigation to achieve a similar fill thickness across the pad is beneficial for the performance of a shallow foundation system. We recommend that a differential fill thickness of up to 5 feet is acceptable across the building footprint. For a differential fill thickness exceeding 5 feet across the footprint, we recommend performing subexcavation activities to bring this vertical distance to within the 5-foot tolerance and that the material be replaced as engineered fill. As a minimum, the subexcavation area should include the entire structure footprint plus 5 feet beyond the edges of the building footprint.

4.6 FILL COMPACTION

4.6.1 Grading in Structural Areas

The contractor should perform the following compaction control requirements for subgrade preparation and fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 12 inches.
2. Moisture condition soil to at up to 3 percentage points over the optimum moisture content; **and**
3. Compact the soil to 90 percent relative compaction.

The contractor should compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to a minimum moisture content of optimum prior to compaction.

4.6.2 Landscape Fill

The contractor should process, place and compact fill in accordance with the recommendations in Section 4.0 except compact to at least 85 percent relative compaction (ASTM D1557).

4.7 SITE DRAINAGE

4.7.1 Surface 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 1804.3 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.

4.4 STORMWATER INFILTRATION AND SELECT PROJECT RISK LEVEL FACTORS

Due to the granular soil generally encountered onsite, the near-surface site soil is expected to have a low to moderate permeability value for stormwater infiltration in grassy swales or permeable pavers. Therefore, Best Management Practices should assume that low to moderate stormwater infiltration will occur at the site.

4.5 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 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.

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.

4.6 LANDSCAPING CONSIDERATION

To minimize degradation and potential loss of strength to near surface soils due to the effects of excess moisture, 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.

5.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

We developed preliminary foundation recommendations using data obtained from our field exploration and engineering assessment. The following preliminary recommended foundation options address the effects of the native expansive soil and differential soil movement:

1. Post-tensioned mat foundation.
2. Structural mat foundation.

For design purposes, we recommend obtaining subsurface geotechnical data below the proposed foundation once the building layout and type are known to develop design-level foundation recommendations.

5.1 STRUCTURAL MAT FOUNDATIONS

The proposed residential structures may be supported on structural mat foundation systems. If found, following laboratory testing during the design-level geotechnical investigation, structural mats may need to be stiffened to reduce differential movements due to swelling/shrinkage to a value compatible with the type of superstructure that will be constructed on them. The structural engineer should be consulted on this matter. We recommend that it be designed for an edge cantilever length of 8 feet with a random, interior unsupported span of 25 feet. Additionally, foundations should be designed for 1 inch of differential movement over a distance of 30 feet for the seismic case.

The perimeter should be thickened by 2 inches, and the minimum soil backfill height against the slab at the perimeter should be 6 inches. For preliminary planning purposes, structural mat foundations should be designed for a uniform bearing pressure of 1,000 pounds per square foot (psf) for dead-plus-live load. This value may be increased to 1,500 psf under individual columns or walls to accommodate stress concentrations at those locations. These values can be increased by one-third for seismic loading.

The thickness of the structural mat will be driven by the structural design. The structural mat should be underlain by a water vapor transmission reduction system as in Section 5.3.

5.2 POST-TENSIONED MAT FOUNDATIONS

The proposed residential structures may also be supported on post-tensioned (PT) mat foundations bearing on prepared native soil or compacted engineered fill.

For preliminary planning purposes, PT mats should be designed for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead-plus-live loads, with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for all loads including wind or seismic. In addition to the parameters below, foundations should be designed for 1 inch of differential movement over a distance of 30 feet for the seismic case.

5.3 SLAB MOISTURE VAPOR REDUCTION

When buildings are constructed with mats, water vapor from beneath the mat will migrate through the foundation and into the building. This water vapor can be reduced but not eliminated. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. Where water vapor migrating through the mat would be undesirable, we recommend the following measures to reduce water vapor transmission upward through the mat foundations.

1. Install a vapor retarder membrane directly beneath the mat. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders should conform to Class A vapor retarder in accordance with ASTM E 1745-11 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs.”
2. Concrete should have a concrete water-cement ratio of no more than 0.5.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.
4. Consider and implement adequate moist cure procedures for mat foundations.
5. Protect foundation subgrade soils from seepage by providing impermeable plugs within utility trenches.

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.

5.4 SUBGRADE TREATMENT FOR MAT FOUNDATIONS

The subgrade material under structural mats should be uniform. The upper 12 inches of pad subgrade should be moisture conditioned to a moisture content of at least 2 percentage points above optimum. The subgrade should be thoroughly soaked prior to placing the concrete. The subgrade should not be allowed to dry prior to concrete placement.

6.0 PRELIMINARY PAVEMENT DESIGN

6.1 FLEXIBLE PAVEMENTS

Based on the site soil, a Resistance (R-Value) of 5 is appropriate for design. The design sections may be reduced based on R-Value testing of samples collected from actual pavement subgrade. Using the traffic indices provided by the civil engineer, we developed the following recommended pavement sections using Chapter 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in Table 6.1-1 below.

TABLE 6.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION BASED ON R-VALUE 5	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	3.0	10.0
6	3.5	13.0
7	4.0	16.0
9	5.5	20.5
11	7	25.0

Notes: AC is asphalt concrete
AB is Class 2 aggregate base material with a minimum R-value of 78

Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, Civil Engineer, and appropriate public agency.

6.2 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 4 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

6.3 SUBGRADE AND AGGREGATE BASE COMPACTION

The contractor should compact finish subgrade and aggregate base in accordance with the design-level geotechnical report. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

6.4 CUT-OFF CURBS

Saturated 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.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the proposed residential development at 790 Portswood Drive, San Jose, California. 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 geotechnical engineering principles and practices currently employed in the area; no warranty is expressed 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 or provide insurance; 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 is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, 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, notify ENGEO 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, notify the proper regulatory officials 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.

SELECTED REFERENCES

American Concrete Institute, 2005, Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05).

Bray, J.D. and Sancio, R.B., 2006, "Assessment of the liquefaction susceptibility of fine-grained soils," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 132, No. 9, pp. 1165-1177.

Bryant, W. and Hart, E., 2007, Special Publication 42, "Fault-Rupture Hazard Zones in California", Interim Revision 2007, California Department of Conservation.

California Building Standards Commission, 2016 California Building Code, Volumes 1 and 2. Sacramento, California.

California Department of Transportation, 2016, 6th Edition Highway Design Manual.

California Division of Mines and Geology, by Hart, E.W et. al.; Proceedings Conference on Earthquake Hazards in the Eastern San Francisco Bay Area; Special Publication 62, 1982.

California Division of Mines and Geology, 2006, Seismic Hazard Zone Report for the Mountain View 7.5-Minute Quadrangle, Santa Clara, Alameda, and San Mateo Counties, CA.

California Geologic Survey, 2008, Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

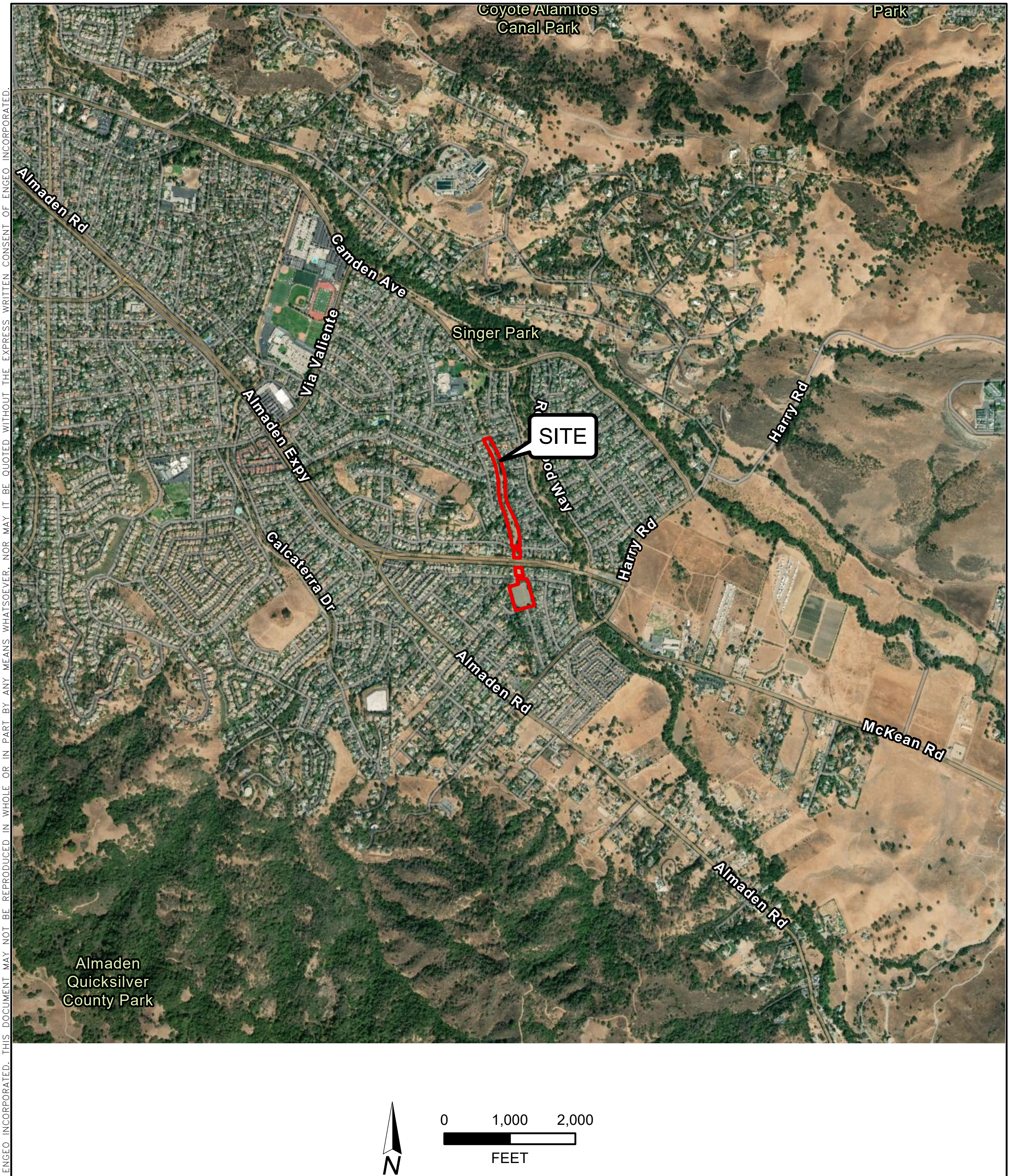
Division of Mines and Geology, 1997, Special Publication 117, Guidelines for Evaluation and Mitigating Seismic Hazards in California, Adopted March 13.

Ellsworth, W.L., 2003, Magnitude and area data for strike slip earthquakes, U.S. Geological Survey Open File Report, 03-214 Appendix D.

Graymer, R.W., 2000, Geologic Map and Map Database of the Palo Alto 30' x 60' Quadrangle, California.

Structural Engineers Association of California (SEAOC), 1996, Recommended Lateral Force Requirements and Tentative Commentary.

FIGURES



BASEMAP SOURCE: ESRI MAPPING SERVICE 2017

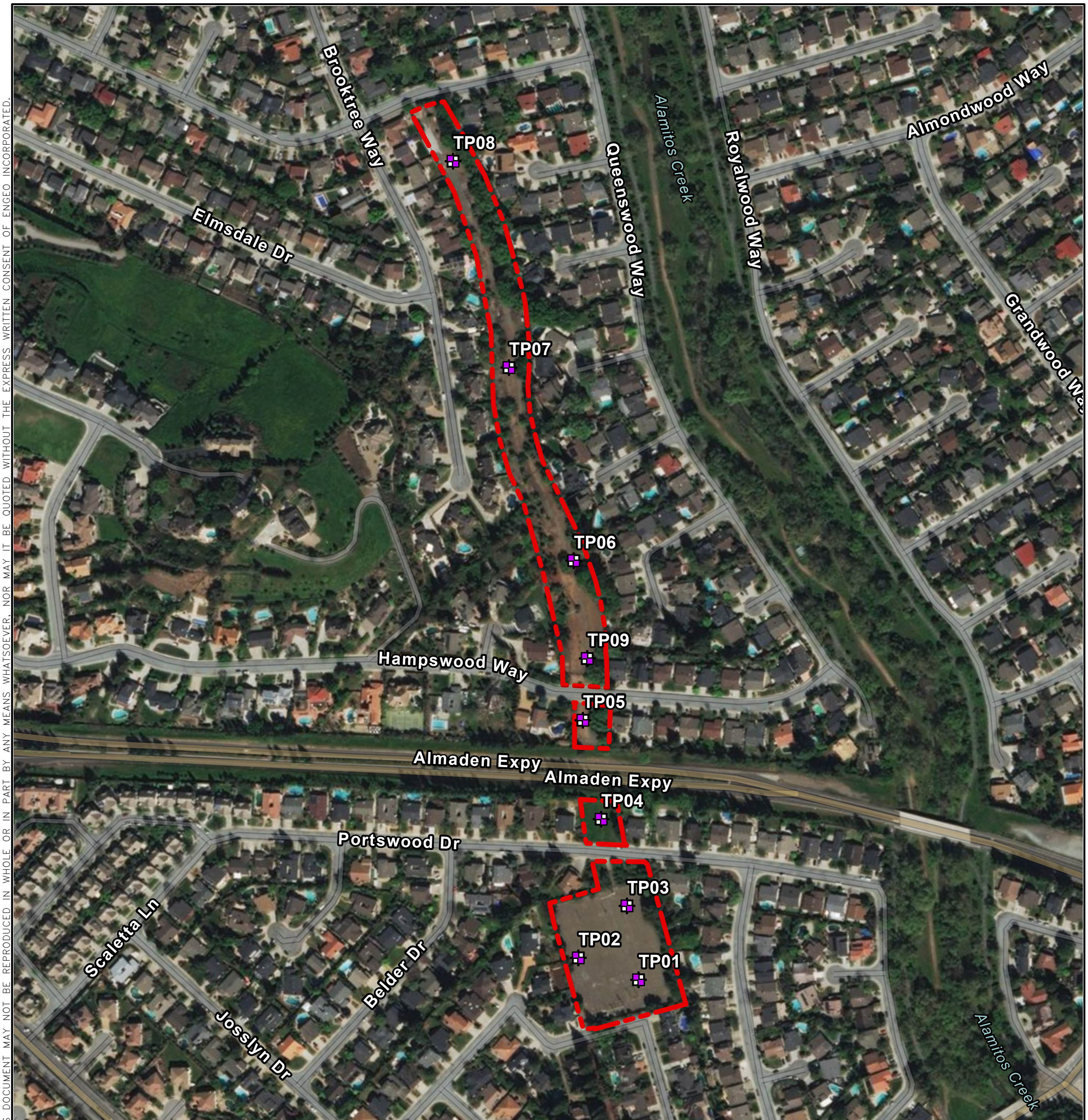
ENGEO Expect Excellence

VICINITY MAP
790 PORTSWOOD DRIVE
SAN JOSE, CALIFORNIA

PROJECT NO. : 16709.000.000	FIGURE NO. 1
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: RHB

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16709\16709000000\GEOTECHNICAL\PGEX\790 PORTSWOOD DRIVE.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PROJECT SITE

TP09 TEST PIT (ENGEO, 2019)

N

0 200 400

FEET

BASEMAP SOURCE: ESRI MAPPING SERVICE 2017

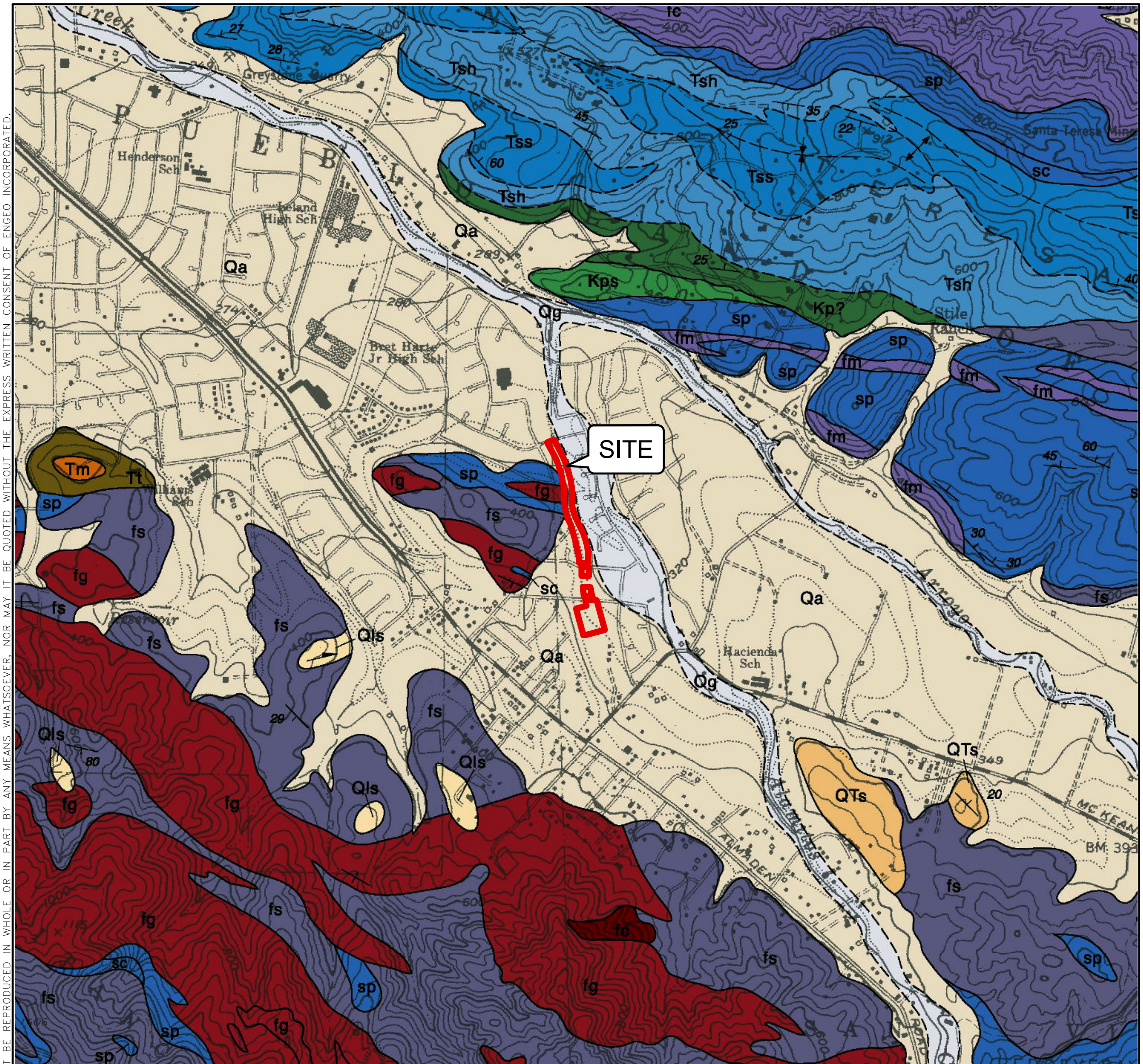
ENGEO
Expect Excellence

SITE PLAN
790 PORTSWOOD DRIVE
SAN JOSE, CALIFORNIA

PROJECT NO.: 16709.000.000
SCALE: AS SHOWN
DRAWN BY: QRL
CHECKED BY: RHB
FIGURE NO. 2

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16709\16709000000\GEOTECHNICAL\PGEX\790 PORTSWOOD DRIVE.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



N

0 1,000 2,000
FEET

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qg	SAND AND GRAVEL OF MAJOR STREAM CHANNELS
Qa	ALLUVIAL GRAVEL, SAND AND CLAY OF VALLEYS
Qls	RUBBLE DERIVED FROM ROCKS UPSLOPE
Qts	GRAVEL/CONGLOMERATE OF PEBBLES OF FRANCISCIAN AND SERPENTINITE DETRITUS
Tm	SILICEOUS SHALE
Tt	SANDSTONE, LIGHT GRAY TO TAN, THICK BEDDED, ARKOSIC
Tss	SANDSTONE, LIGHT GRAY TO TAN, FINE TO MEDIUM GRAINED
Tsh	CLAY SHALE
Kp	CLAY SHALE OR MUDSTONE
Kps	SANDSTONE, LIGHT GRAY TO BROWN, CRUDELY BEDDED
sp	SERPENTINITE
fm	MELANGE (MIXTURE) OF GRAYWACKE AND CLAY SHALE

BASEMAP SOURCE: DIBBLEE 2005

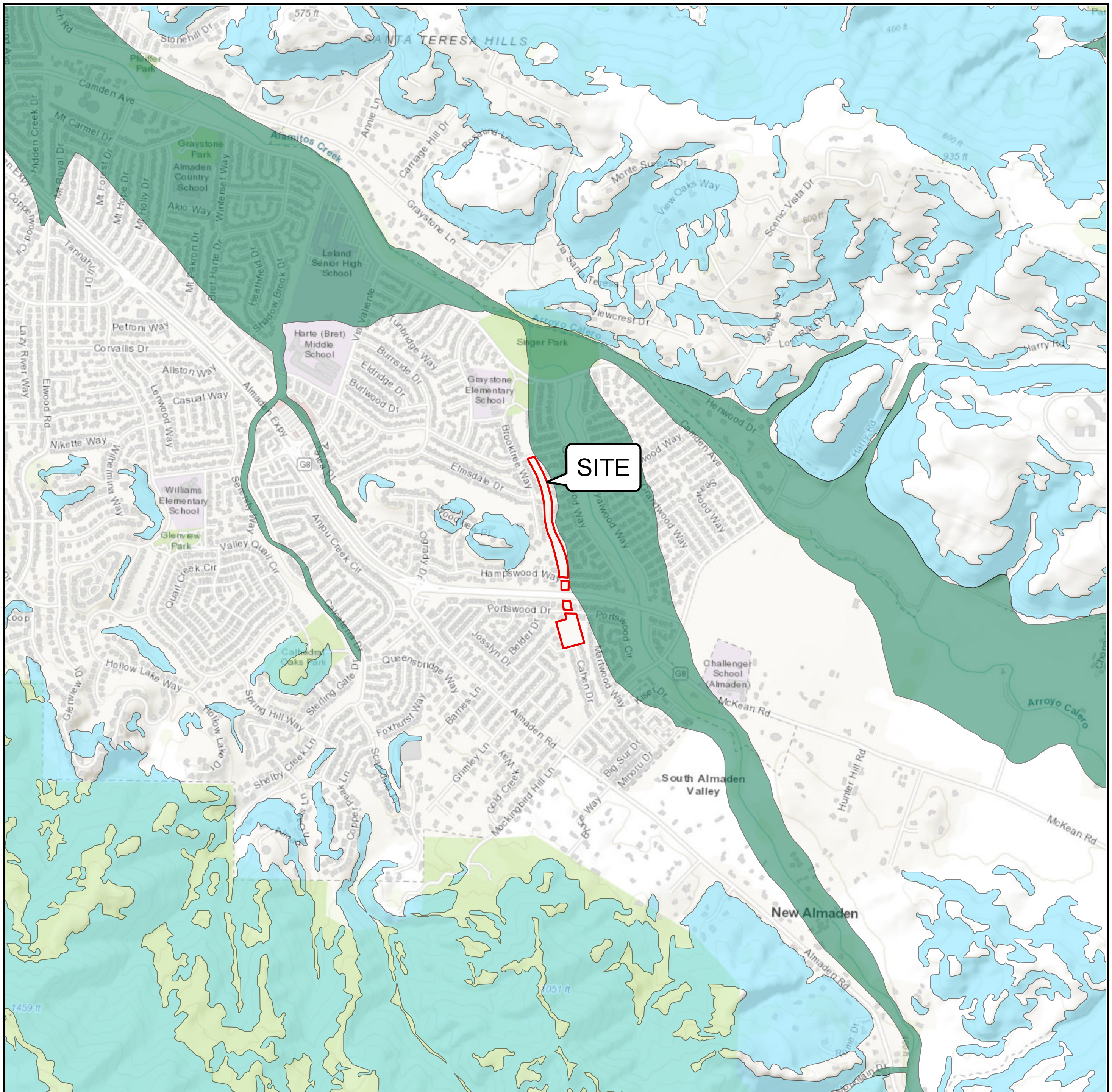
ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
790 PORTSWOOD DRIVE
SAN JOSE, CALIFORNIA

PROJECT NO. : 16709.000.000		FIGURE NO.
SCALE: AS SHOWN		3
DRAWN BY: QRL	CHECKED BY: RHB	

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16709\16709000000\GEOTECHNICAL\PGEX\790 PORTSWOOD DRIVE.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

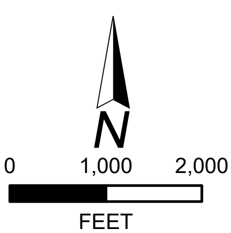


EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Liquefaction Zone
Areas where historical occurrence of liquefaction, or local geological,geotechnical and ground water 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 Landslide Zones
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.

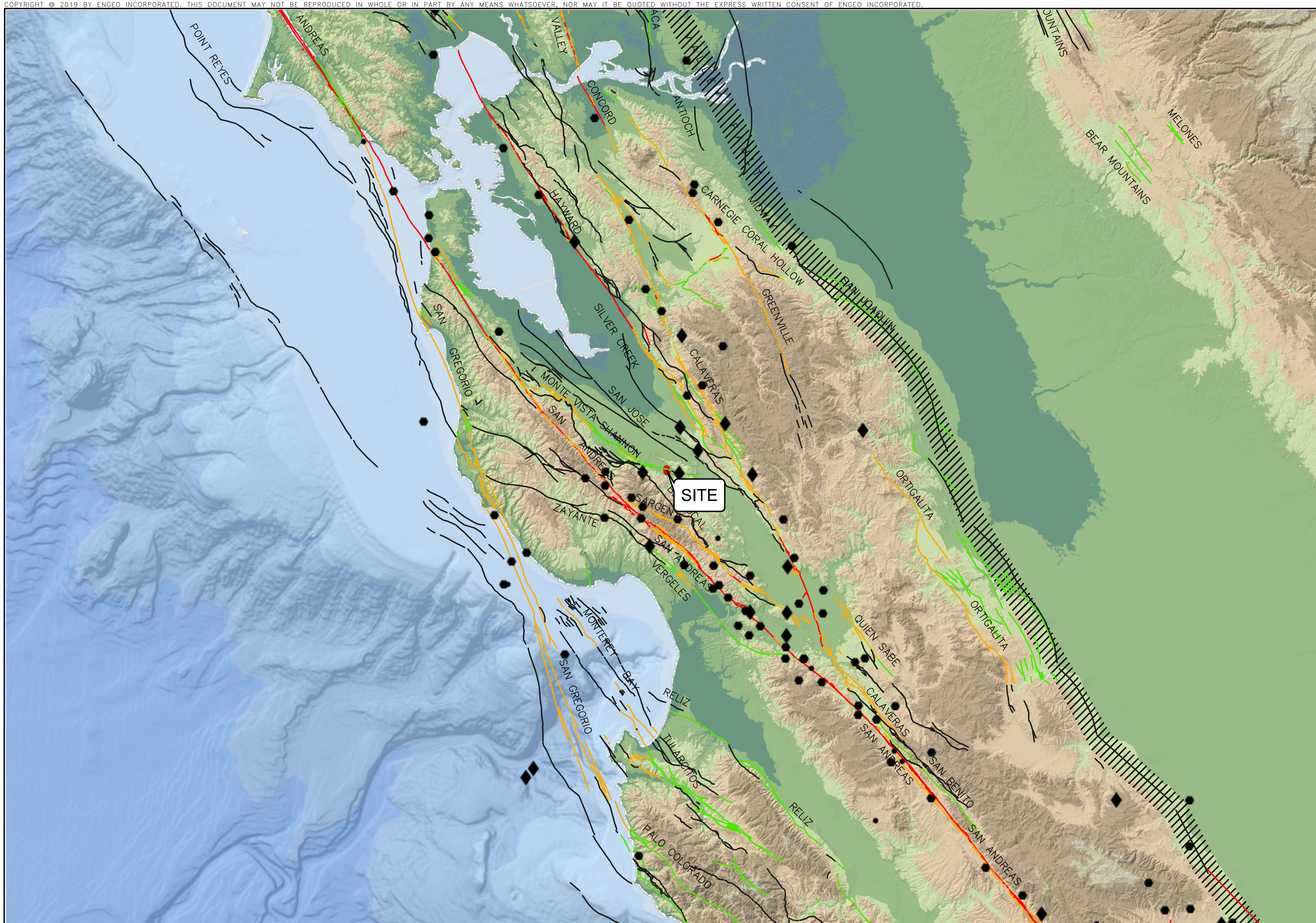


BASEMAP SOURCE: ESRI MAPPING SERVICE
CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY

ENGEO Expect Excellence	SEISMIC HAZARDS ZONE MAP 790 PORTSWOOD DRIVE SAN JOSE, CALIFORNIA	PROJECT NO. : 16709.000.000 SCALE: AS SHOWN DRAWN BY: QRL / CHECKED BY: RHB	FIGURE NO. 4

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16709\16709000000\GEOTECHNICAL\PGEX\790 PORTSWOOD DRIVE.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR



N

0 7.5 15

Miles

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EARTHQUAKE

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

USGS QUATERNARY FAULTS

- HISTORICAL
- LATEST QUATERNARY
- LATE QUATERNARY
- UNDIFFERENTIATED QUATERNARY
- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE

ESRI, GARMIN, GEBCO, NOAA NGDC, AND OTHER CONTRIBUTORS
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY
790 PORTSWOOD DRIVE
SAN JOSE, CALIFORNIA

PROJECT NO.: 16709.000.000		FIGURE NO. 5
SCALE: AS SHOWN		
DRAWN BY: QRL	CHECKED BY: RHB	

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16709\16709000000\GEOTECHNICAL\PGEX\790 PORTSWOOD DRIVE.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

EXPLORATION LOGS

ENGEO Expect Excellence		TEST PIT LOG
16709 790 Portswood Drive San Jose, California 16709.000.000		Logged By: H. Foy Logged Date: 11/06/2019
Test Pit Number	Depth (feet)	Description
1-TP1	0 – ¼	Asphalt with trace gravel, black [5YR 2.5/1], dry [FILL].
	¼ – ½	Sandy GRAVEL (GP), gray [5Y 6/1], dry, poorly graded. Gravel is fine-grained and angular [FILL].
	½ – 2	Gravelly SILT (ML) with trace clay and sand, reddish brown [2.5YR 4/4], moist, low plasticity. Gravel fine to medium and subrounded to rounded.
	2 – 12½	Sandy GRAVEL (GW) with some silt, trace clay, cobbles and boulders. Brownish yellow [2.5YR 5/3], moist, well graded. Gravel is subrounded to rounded.
		Test Pit terminated at 12½ feet below the ground surface (bgs) and is approximately 120' long. Contacts dip at less than 5°. No free water was encountered.
1-TP2	0 – ¼	Asphalt with gravel, black [5YR 2.5/1], dry, [FILL].
	¼ – ½	Sandy GRAVEL (GP), gray [5Y 6/1], dry, poorly graded. Gravel is fine-grained and angular [FILL].
	½ – 2	Gravelly SILT (ML) with trace clay and sand, reddish brown [2.5YR 4/4], moist, low plasticity. Gravel fine to medium and subrounded to rounded.
	2 – 11	Sandy GRAVEL (GW) with some silt, trace clay, cobbles and boulders. Brownish yellow [2.5YR 5/3], moist, well graded. Gravel is subrounded to rounded.
		Test Pit terminated at 11 feet below the ground surface (bgs) and is approximately 120' long. Contacts dip at less than 5°. No free water was encountered.

ENGEO Expect Excellence	TEST PIT LOG	
16709 790 Portswood Drive San Jose, California 16709.000.000	Logged By: H. Foy Logged Date: 11/06/2019	
Test Pit Number	Depth (feet)	Description
1-TP3	0 – ¼	Asphalt with gravel, black [5YR 2.5/1], dry, [FILL].
	¼ – ½	Sandy GRAVEL (GP), gray [5Y 6/1], dry, poorly graded. Gravel is fine-grained and angular [FILL].
	½ – 2½	Gravelly SILT (ML) with trace clay and sand, reddish brown [2.5YR 4/4], moist, low plasticity. Gravel fine to medium and subrounded to rounded.
	2½ – 13	Sandy GRAVEL (GW) with some silt, trace clay, cobbles and boulders. Reddish brown [2.5YR 4/3], moist, well graded. Gravel is subrounded to rounded
		Test Pit terminated at 13 feet below the ground surface (bgs) and is approximately 120' long. Contacts dip at less than 5°. No free water was encountered.
1-TP4	0 – 1	SILT (ML-SC) with trace sand and gravel, dark brown [7.5YR 3/2], dry, some rootlets [TOPSOIL].
	1 – 1½	Gravelly SILT (ML) with trace clay, sand and gravel, light reddish brown [5YR 6/4], dry, some rootlets. Gravel is angular to subrounded [FILL].
	1½ – 5	Gravelly SAND (SW) with trace silt, reddish brown [5YR 5/4], dry, hard, fine-grained gravel. Gravel is subrounded to rounded.
	5 – 11	Sandy GRAVEL (GW) with trace clay, silt, cobbles and boulders, dark reddish brown [5YR 3/3], dry. Gravel is subrounded to rounded.
		Test Pit terminated at 11 feet bgs and is 120' long. Contacts dip at less than 5°. No free water was encountered.

TEST PIT LOG

16709 790 Portswood Drive San Jose, California 16709.000.000	Logged By: H. Foy Logged Date: 11/06/2019

Test Pit Number	Depth (feet)	Description
1-TP5	0 – 2	Gravelly SILT (ML) with trace clay and sand, reddish brown [2.5YR 4/4], moist, low plasticity. Gravel fine to medium. Gravel is angular to subrounded [FILL].
	2 – 11	Sandy GRAVEL (GW) with some silt, trace clay, cobbles and boulders. Reddish brown [2.5YR 4/3], moist, well graded. Gravel is subrounded to rounded.
		Test Pit terminated at 11 feet below the ground surface (bgs) and is approximately 120' long. Contacts dip at less than 5°. No free water was encountered.
1-TP6	0 – 7	Sandy GRAVEL (GW) with trace silt and clay, cobbles and boulders. Reddish brown [2.5YR 4/3], dry, well graded. Gravel is angular to rounded [FILL].
	7 – 7¼	Slightly weathered SANDSTONE. Dark gray [7.5YR 4/1] [BEDROCK].
		Test Pit terminated at 7¼ feet below the ground surface (bgs) on assumed bedrock and is approximately 120' long. Contacts dip at less than 5°. No free water was encountered.

ENGEO Expect Excellence	TEST PIT LOG	
16709 790 Portswood Drive San Jose, California 16709.000.000	Logged By: H. Foy Logged Date: 11/06/2019	
Test Pit Number	Depth (feet)	Description
1-TP7	0 – 1	Sandy GRAVEL (GP), gray [5Y 6/1], dry, poorly graded. Gravel is fine-grained and angular [FILL].
	1 – 8	Sandy GRAVEL (GW) with trace silt and clay, cobbles and boulders. Reddish brown [2.5YR 4/3], dry, well graded. Gravel is angular to rounded [FILL].
	8 – 12	Sandy SILT (ML) with trace rootlets and clay. Yellowish brown [10 YR 5/6], moist, stiff to hard, low plasticity.
	12 – 14½	Sandy GRAVEL (GW) with some silt, trace clay, cobbles and boulders. Reddish brown [2.5YR 4/3], moist, well graded. Gravel is subrounded to rounded.
		Test Pit terminated at 14½ feet below the ground surface (bgs) and is approximately 120' long. Contacts dip at less than 5°. No free water was encountered.
1-TP8	0 – 1	Sandy GRAVEL (GP), gray [5Y 6/1], dry, poorly graded. Gravel is fine-grained and angular [FILL].
	1 – 3	Sandy GRAVEL (GW) with trace silt and clay, cobbles and boulders. Reddish brown [2.5YR 4/3], dry, well graded. Gravel is angular to rounded [FILL].
	3 – 11½	Sandy GRAVEL (GW) with some silt, trace clay, cobbles and boulders. Reddish brown [2.5YR 4/3], moist, well graded. Gravel is subrounded to rounded.
		Test Pit terminated at 11½ feet below the ground surface (bgs) and is approximately 120' long. Contacts dip at less than 5°. No free water was encountered.

	TEST PIT LOG	
16709 790 Portswood Drive San Jose, California 16709.000.000		Logged By: H. Foy Logged Date: 11/06/2019
Test Pit Number	Depth (feet)	Description
1-TP9	0 – 4	Sandy GRAVEL (GW) with trace silt and clay, cobbles and boulders. Reddish brown [2.5YR 4/3], dry, well graded. Gravel is angular to rounded [FILL].
	4 – 11	Sandy GRAVEL (GW) with some silt, trace clay, cobbles and boulders. Reddish brown [2.5YR 4/3], moist, well graded. Gravel is subrounded to rounded.
		Test Pit terminated at 11 feet below the ground surface (bgs) and is approximately 120' long. Contacts dip at less than 5°. No free water was encountered.

ENGEO
Expect Excellence