APPENDIX F

GEOTECHNICAL REPORT

GEOTECHNICAL EXPLORATION

3666, 3672 AND 3682 MT. DIABLO BOULEVARD
LAFAYETTE, CALIFORNIA

Submitted to:
Mr. Chad Kiltz
Lennar Homes
6111 Bollinger Canyon Road #550
San Ramon, CA 94583

Prepared by:
ENGEO Incorporated

April 24, 2014

Project No:
10022.100.000

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GEOTECHNICAL
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WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
10022.100.000

April 24, 2014

Mr. Chad Kiltz
Lennar Homes
6111 Bollinger Canyon Road #550
San Ramon, CA 94583

Subject: 3666, 3672 and 3682 Mount Diablo Boulevard
Lafayette, California

GEOTECHNICAL REPORT

Dear Mr. Kiltz:

We prepared this geotechnical report for the proposed development at 3666, 3672 and 3682 Mount Diablo Boulevard as outlined in our proposal dated March 12, 2014. We characterized the subsurface conditions 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

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report is to provide recommendations regarding site grading, foundation design and site drainage for design of the proposed multi-family residential project.

Our scope of services included the following:

- Reviewing available literature, geologic maps and previous geotechnical reports pertinent to the site.

- Drilling four exploratory borings and excavating one test pit at the site.

- Laboratory testing of materials sampled during the field exploration.

- Geotechnical engineering analysis.

- Report preparation summarizing our conclusions and recommendations for the proposed development.

In preparation of this report, Lennar Homes provided us with preliminary architectural plans titled "3666, 3672 & 3682 Mt. Diablo Boulevard, Lafayette, California," dated February 25, 2014 by Studio T-SQ, Inc. This two page plan set shows the conceptual layout of the basement garage level and the first story (street level) for the project. We also received as-built plans of the adjacent East Bay Municipal Utility District Lafayette Aqueduct dated November 2, 1967.

This report was prepared for the exclusive use of Lennar Homes and their design team consultants. In the event that any changes are made in the character, design or layout of the development, the conclusions and recommendations contained in this report must be reviewed by ENGEO Incorporated to determine whether modifications to the report are necessary. 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 except where allowed by our contract with Lennar Homes.

1.2 SITE LOCATION

The site is located northwest of the intersection of Dolores Drive and Mount Diablo Boulevard in Lafayette, California, as shown in Figure 1. The approximately 2-acre site comprises three individual parcels. An EBMUD easement for the Lafayette Aqueduct is located adjacent to the site. Two of the parcels are adjacent to each other and front Mount Diablo Boulevard. The third parcel is located north of the other two parcels on the opposite side of the East Bay Municipal Utility easement.

1.3 PROPOSED DEVELOPMENT

According to preliminary architectural plans prepared by Studio T-SQ (dated February 25, 2014) the proposed project consists of a podium-style, multi-family residential with some retail development with wood-framed living levels over one level of subterranean parking. A post-tensioned slab will be constructed above the parking level to support the wood-framed construction. Building loads were not available at the time of this report preparation; however, based on our experience with this type of construction, we anticipate that column and wall loads will be moderate.

The approximate location of the proposed building on the site is depicted on the Site Plan, Figure 2. Based on preliminary information, we assume that the bottom of the subterranean parking may extend up to approximately 10 to 12 feet below existing grade to accommodate one level of below grade parking and foundation elements. This parking will require excavation below existing grade and a cut into the hillside adjacent to the Lafayette Aqueduct easement. The majority of the residential buildings will be constructed above the post-tensioned podium slab above the subterranean basement. The current site plan indicates that a pool and pool building are contemplated in the northwestern portion of the parcel. Given the layout of the proposed structure, temporary shoring and foundation retaining walls will be required around the entire perimeter of the subterranean parking.

We anticipate that site development will include removal of the existing buildings and associated improvements and excavation of the below grade portions to a firm, non-yielding surface for support of foundations and floor slabs. This is discussed in more detail in the recommendations section of this report.

2.0 FINDINGS

2.1 PREVIOUS STUDIES

Allwest Geoscience, Inc. previously completed a Geotechnical Engineering Investigation that included 3672 Mount Diablo Boulevard, dated March 10, 2009. The exploration included drilling four test borings to depths up to 15 feet below existing grade, collecting subsurface samples, performing laboratory testing on selected samples, analysis, and preparation of geotechnical recommendations for site development. The boring logs are presented in Appendix C.

The two borings (B-1 and B-2) drilled along the eastern boundary of the parcel encountered bedrock at a depth of 1½ and 9½ feet below the ground surface, respectively. Minor fill (approximately 2 feet thick) and alluvial soil, comprising stiff lean clay, were encountered overlying the bedrock. Bedrock is generally described as claystone and siltstone. The two borings (B-3 and B-4) drilled at the western portion of the parcel encountered minor fills (approximately 1 foot thick) overlying alluvial soil comprising stiff lean clay. Bedrock was not encountered within borings B-3 and B-4. The report provided recommendations for grading and foundations for the project proposed at the time.

2.2 GEOLOGY AND SEISMICITY

2.2.1 Geologic Setting

The site is located within the Coast Ranges physiographic province of California. The Coast Ranges physiographic province is typified by a system of northwest-trending, fault-bounded mountain ranges and intervening alluviated valleys. More specifically, the site is located along the northern edge of a narrow alluvial plane occupied by Lafayette Creek. The site is approximately 1,000 feet north of Lafayette Creek.

Bedrock in the Coast Ranges consists of igneous, metamorphic and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-order faults.

2.2.2 Site Geology

According to published maps covering the site by Dibblee (2005) and Graymer (1994), the southern portions of the project site are underlain by alluvial deposits, as shown in Figure 3 (Dibblee, 2005). According to mapping by Helley and Graymer (1997) the alluvial deposits are Pleistocene age deposits (Qpaf) that comprise dense gravely and clayey sand or clayey gravel that fines upward to sandy clay. Quaternary mapping by Witter et. al (2006) interpret the alluvial deposits (Qha) as Holocene in age and assign a moderate liquefaction susceptibility to them (Figure 4).

Both Dibblee (2005) and Graymer (1994) map bedrock in the vicinity of the project site as Pliocene terrestrial sedimentary rocks primarily consisting of interbedded sandstone, claystone and pebble conglomerate (Figure 3). Dibblee classifies bedrock in the vicinity of the site as belonging to the Orinda formation (Tor) and Graymer classifies it as unnamed sedimentary and volcanic rocks. Dibblee (2005) maps bedding within the Orinda formation in the site vicinity as striking northwest–southeast and dipping moderately towards the northeast from around 30 to 50 degrees. Exposures of this bedrock unit were generally observed to be very weak, closely fractured to crushed and highly weathered.

2.2.3 Faulting and Seismicity

Because of the presence of nearby active faults[1], the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (greater than Moment Magnitude 7) earthquakes have been recorded and can be expected to occur in the future. The site is not located within a State of California Earthquake Fault Zone. Figure 5 shows

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[1] An active fault is defined by the California Geological Survey as one that has had surface displacement within Holocene time (about the last 11,000 years) (SP42 CGS, 2007).

the approximate location of active and potentially active faults and significant historic earthquake epicenters mapped within the San Francisco Bay Region. Based on the 2008 update of the national seismic hazards maps, the table below shows the nearest known active faults capable of producing significant ground shaking at the site.

TABLE 2.2.3-1
Known Active Faults Capable of Producing Significant Ground Shaking at the Site

Fault Name	Distance from Site (miles)	Maximum Moment Magnitude (Elsworth)
Mount Diablo Thrust	4.9	6.7
Hayward-Rogers Creek	6.3	7.3
Concord-Green Valley	7.0	6.8
Calaveras	8.2	7.0
Greenville	16.1	7.0
Great Valley Pittsburg Kirby Hills	17.7	6.7
West Napa	19.9	6.7
San Andreas	24.4	7.9

The Uniform California Earthquake Rupture Forecast (UCERF, 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, including the Calaveras fault. The UCERF generated an overall probability of 63 percent for the Bay Area as a whole, a probability of 31 percent for the Hayward fault, 7 percent for the Calaveras fault, and 3 percent for the Concord-Green Valley fault.

2.3 EXISTING SITE CONDITIONS

The majority of the site is paved and currently occupied by several commercial buildings including a restaurant, automobile repair shop and other businesses and parking areas. Based on regional topographic mapping, and elevations shown on the previously referenced EBMUD plans, the site is relatively flat with the exception of an existing cut slope that lies between the site and the Lafayette Reservoir easement. Site grades range from a low of approximately 344 feet (NAVD88) to a high of approximately 370 feet at the cut slope adjacent to the EBMUD easement.

2.4 FIELD EXPLORATION

Our field exploration included drilling four borings and excavation of one test pit at the project site (Figure 2). We performed our field exploration on March 27, 2014.

The location and elevations of our explorations are shown on the attached Site Plan, Figure 2. Exploration locations are approximate and were determined using a hand held GPS device and should be considered accurate only to the degree implied by the method used.

2.4.1 Borings

We retained a subcontractor with a truck-mounted drill rig and crew to advance the borings using 4-inch diameter solid flight augers. The borings were advanced to depths ranging from approximately 10 to 45 feet below existing grade. We permitted and backfilled the borings in accordance with the requirements of Contra Costa County Department of Environmental Health.

An ENGEO engineering geologist collected soil samples using either a 3-inch outside-diameter (O.D.) California-type split-spoon sampler fitted with 6-inch-long brass liners, or a 2-inch O.D. Standard Penetration Test (SPT) split-spoon sampler. The samplers were driven with a 140-pound hammer falling a distance of 30 inches. A rope and cat head pulley was used to lift the safety hammer during our exploration. The penetration of the sampler was field recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs show the number of blows required for the last 1 foot of penetration, and the blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows.

2.4.2 Test Pit

The test pit was excavated into the existing slope at the north side of the existing restaurant parking lot using a Case 580 Super M rubber tire backhoe equipped with a 30-inch bucket. The test pit was excavated to a depth of approximately 6 feet below existing grade. An ENGEO engineering geologist logged subsurface conditions exposed in the test pit and collected bulk bedrock samples. The purpose of the test pit was to observe bedrock conditions at the location of the excavation into the existing slope at the northern portion of the proposed building along the Lafayette Reservoir easement.

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. In addition, stratification lines represent the approximate boundaries between soil types and the transitions may be gradual.

2.5 LABORATORY TESTING

Select samples recovered during our subsurface exploration were tested to determine various soil characteristics as presented on the following table.

TABLE 2.5-1
Laboratory Testing

Soil Characteristic	Testing Method	Location of Results
Moisture Content	ASTM D-2216	Appendix A
Plasticity Index	ASTM D-4318	Appendix B
Unconfined Compression Testing	ASTM 2166	Appendix B
ASTM Soil Corrossivity	ASTM D4327, D4972, D1498 & G57	Appendix B

The laboratory test results are shown on the borelogs (Appendix A), with individual test results presented in Appendix B.

2.6 GROUNDWATER CONDITIONS

We encountered groundwater during drilling within borings B-3 and B-4. The groundwater levels for the borings where groundwater was encountered are shown on the following table.

TABLE 2.6-1

Recorded Depths to Groundwater

Company	Exploration	Depth to GW (feet)	Date
ENGEO	B-3	19	March 27, 2014
ENGEO	B-4	23	March 27, 2014

Fluctuations in groundwater levels may occur daily, seasonally and over a period of years because of precipitation, changes in drainage patterns, fluctuations in nearby creeks, irrigation, and other factors.

2.7 SUBSURFACE CONDITIONS

Based on our subsurface exploration and site observations we compiled a geologic map of the site. Below are descriptions of the primary geologic units encountered during our exploration of the site (Figure 2). It should be noted that the site is paved with 3 to 4 inches of asphalt over 1 to 6 inches of aggregate base that are not described below.

2.7.1 Pleistocene-age Alluvial Deposits (Qal)

Pleistocene-age alluvial deposits (Qpaf) are present underlying the pavement in the west portion of the site (Figure 2). Borings B-3 and B-4 encountered alluvium to depths of 45 and 38 feet below the existing ground surface, respectively, in the west portion of the site. As previously mentioned, Allwest Geotechnical boring B-1 encountered alluvium to a depth of approximately 9½ feet below the existing ground surface in the southern central portion of the site. Two of our borings (B-3 and B-4) encountered alluvium extending to depths of approximately 45 and 38 feet below existing grade, respectively on the western side of the property. We did not encounter alluvium in our borings B-1 and B-2 on the western portion of the property. Subsurface exploration indicates that the alluvial deposits generally thicken towards the west and thin and pinch out towards the east. In general, the alluvium is fine grained in nature, comprising very dark brown to reddish brown, stiff to very stiff lean clay with minor amounts (less than 5 to 10%) of sand and gravel. Two PI tests were performed on soil from this unit that resulted in a PI range of 29 to 32. Unconfined compression testing of samples of the alluvium yielded results that range from approximately 1100 to 6300 psf. See Appendix A for exploratory logs and Appendix B for laboratory test result.

2.7.2 Pliocene to latest Miocene Orinda Formation (Tor)

Below the alluvium, or directly below the pavement in areas where alluvium was not encountered, each of our borings encountered bedrock. Two of the Allwest Geotechnical borings were not advanced deep enough to encounter bedrock. Based on our mapping and subsurface exploration, bedrock at the site comprises Pliocene to latest Miocene Orinda formation as described above (Dibblee, 2005). The Orinda formation exposed in the cut slope in along the northern portion of the site comprised interbedded sandstone and pebble conglomerate with thin siltstone and claystone interbeds. Sandstone encountered was yellowish brown, very weak, poorly cemented, poorly sorted, fine to medium grained, subrounded, moderately weathered and closely fractured and crushed in the upper 12 inches from the surface. The pebble conglomerate encountered was gray brown to yellowish brown, very weak, poorly sorted, fine to coarse-grained sand with gravel and small cobbles of Franciscan derived chert, quartzite, graywacke and greenstone. The siltstone and claystone encountered was generally reddish brown and mottled olive gray and reddish brown, very weak, closely fractured to crushed and laminated. Bedding measured at the site ranges from N60-65W and dips 50 degrees towards the northeast. Unconfined compression testing of samples of the bedrock yielded results of 3,500 to 4,000 psf. The blow counts and strength testing indicate that the bedrock is relatively weak for rock.

3.0 CONCLUSIONS

Based on the exploration and laboratory test results, it is our opinion, from a geotechnical standpoint, that the site is feasible for the proposed mixed use development. The site was evaluated with respect to known geologic and other hazards common to the greater San Francisco Bay Region. The primary hazards and the risks associated with these hazards with respect to the planned development are discussed in the following sections of this report.

3.1 DIFFERENTIAL FOUNDATION CONDITIONS

As discussed in the Geologic Mapping and Subsurface Conditions section of this report, the east portion of the site is underlain by bedrock of the Orinda formation and the west portion of the site is underlain by alluvial soil. Therefore, the proposed podium structure will have foundation elements that are bearing on rock in the east portion of the site and on alluvial soil in the west. The bedrock and alluvium have different engineering properties including, strength, bearing capacity, and settlement potential. To reduce the potential for damaging differential settlement between rock- and alluvium-supported foundations, we are recommending different bearing pressures for footings bearing in rock and alluvium. Recommendations for mitigation of the differential conditions underlying the proposed structure are provided in later sections of this report.

3.2 EXPANSIVE SOIL

As indicated in section 2.7, plasticity index testing of soil within the alluvium indicates a moderate to high expansion potential. Given the typical loading of the proposed podium structure and depth of excavation, expansive soil is not considered an impact on that structure. Surface improvements may be impacted by expansive soil. We were unable to access the location of the proposed pool and pool building due to easement considerations with EBMUD. Once the area is available for access and the building plan is better defined, we recommend performing an additional exploration in this area to determine if expansive soil is a consideration. The recommendations provided in the subsequent portions of this report have been prepared to address this moderately to highly expansive soil and reduce the impacts on surface improvements.

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, ground lurching, soil liquefaction, lateral spreading, and densification. Based on topographic and lithologic data, risk from earthquake-induced regional subsidence/uplift is considered negligible at the site, in our opinion. The following sections present a discussion of these hazards as they apply to the site.

3.3.1 Ground Surface Rupture

The site is not located within a State of California Earthquake Fault Zone. Based on our field mapping, review of aerial photographs and the results of our field exploration, it is our opinion that fault-related ground rupture is unlikely at the subject property.

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 2013 California Building Code (CBC) requirements, as a minimum.

3.3.3 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. 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, but based on the site location, it is our opinion that the offset is expected to be minor.

3.3.4 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded fine-grained sands. Empirical evidence indicates that loose to medium dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable.

As discussed in the Site Geology section of this report, the site is mapped as moderately susceptible to liquefaction. We evaluated the liquefaction potential of the subsurface soils encountered in the borings drilled at the site. As described in the Geologic Mapping and Subsurface Conditions section of this report, Borings B-3 and B-4 encountered stiff to very stiff lean clay with minor amounts (less than 5 to 10%) of sand and gravel. The results of our laboratory testing on samples of the alluvium collected from our test borings indicate Plasticity Indexes of 29 and 32. Based on the soil type and stiffness, the potential for liquefaction at the site is low.

3.3.5 Lateral Spreading

Lateral spreading involves lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied or weak soils. Due to the low potential for liquefaction at the site, the potential for lateral spreading at the site is considered negligible.

3.4 2013 CBC SEISMIC DESIGN PARAMETERS

The 2013 California Building Code (CBC) utilizes design criteria set forth in the 2010 ASCE 7 Standard. Based on the subsurface conditions encountered, we characterized the site as Site Class D in accordance with the 2013 CBC. We provide the 2013 CBC seismic design parameters in the table below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 3.4-1
2013 CBC Seismic Design Parameters

Parameter	Design Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.50
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.60
Site Coefficient, F_A	1.0
Site Coefficient, F_V	1.5
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.50
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.90
Design Spectral Response Acceleration at Short Period, S_{DS} (g)	1.00
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.60

3.5 CORROSION POTENTIAL

Because much of the surface soil will be removed through site grading to construct the planned below-grade parking garage, and final site improvement plans have not been developed, we have not performed corrosion testing for the site. Prior to site construction, we recommend performing corrosion testing to determine, at a minimum the sulfate content of the soil to determine structural concrete requirements as well as other testing, as appropriate, to evaluate potential of corrosion impacting buried utilities.

4.0 GRADING RECOMMENDATIONS

4.1 GENERAL

Grading at the site will consist of removal of the existing facilities, pavement, retaining walls and concrete slabs. An excavation up to 12 feet in depth for the parking garage of the podium structure is anticipated. Beyond the footprint of the parking garage, grading is anticipated to establish a building pad for the remainder of the structures and a firm subgrade for other exterior elements such as sidewalks, garage ramp, etc. The relative compaction and optimum moisture content of soil, rock, and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as determined by an ENGEO representative. As used in this report, 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” in Section 4 of this report 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.

Ponding of stormwater must not be allowed at the site and on the building pad during construction stoppage for rainy weather. Before the grading is halted by rain, positive slopes should be provided to carry surface runoff in a controlled manner to a discharge point approved by the Civil Engineer.

4.2 DEMOLITION AND STRIPPING

Following the demolition of existing improvements, site development should include removal of debris, loose soil, and soft compressible materials in any location to be graded. Any soft compressible soil should be removed from areas to receive fill or structures.

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 or stripping is permitted.

4.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during any subsurface excavation due to the potential for relatively shallow groundwater as well as following wet weather periods. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated utilizing the following options:

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

We should review for approval the contractor's planned procedure should they elect options 3 or 4 prior to implementation.

4.4 SELECTION OF MATERIALS

The site soils are suitable for use as engineered fill provided they do not contain deleterious material, debris and high organic content (soil that contains more than 3 percent organics).

The excavated aggregate base from the existing parking lot can be used as fill material. From a geotechnical perspective, recycled asphalt concrete or material deriving from on-site concrete may be used as engineered fill below pavement, concrete walkways, parking areas, and the parking garage but should be avoided near the surface of landscaped areas.

We should be informed when import materials are planned for the site. Import materials should be submitted and approved by the Geotechnical Engineer prior to delivery at the site.

4.5 FILL PLACEMENT

After removal of any loose soil, the exposed non-yielding surface of areas to receive fill should be scarified to a depth of 12 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. The lift thickness should not exceed 10 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction control requirements should be applied to all fill including backfill but excepting landscape areas:

Test Procedures:	ASTM D-1557, latest edition.
Required Moisture Content:	Not less than 2 percentage points above optimum moisture content.
Minimum Relative Compaction:	Not less than 90 percent.

We recommend that all site preparation, including demolition and stripping be performed under the observation of the Geotechnical Engineer's qualified field representative.

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

4.7 UTILITIES

It is recommended that utility trench backfilling be done under the observation of a Geotechnical Engineer. Pipe zone backfill (i.e. material beneath and immediately surrounding the pipe) may consist of a well-graded import or native material less than ¾ inch in maximum dimension compacted in accordance with recommendations provided above for engineered fill. Trench zone backfill (i.e. material placed between the pipe zone backfill and the ground surface) may consist of native soil compacted in accordance with recommendations for engineered fill.

Where import material is used for pipe zone backfill, we recommend it consist of fine to medium grained sand or a well-graded mixture of sand and gravel, and that this material not be used within 2 feet of finish grades. In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of (1) soil into the relatively large void spaces present in this type of material, and (2) water along trenches backfilled with this type of material. All utility trenches entering buildings and paved areas must be provided with an impervious seal consisting of native materials or concrete where the trenches pass under the building perimeter or curb lines. The impervious plug should extend at least 4 feet to either side of the crossing. This is to prevent surface water percolation into the sands under foundations and pavements where such water would remain trapped in a perched condition.

Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

Utility trenches in areas to be paved should be constructed in accordance with City of Lafayette requirements. Compaction of trench backfill by jetting should not be allowed at this site. If there appears to be a conflict between the City or other agency requirements and the recommendations contained in this report, this should be brought to the Owner's attention for resolution prior to submitting bids.

4.8 LANDSCAPE IRRIGATION

If planting adjacent to the building is desired, the use of drought-tolerant plants that require very little moisture is recommended. Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils. Such ponding or saturation could result in undesirable loss of compaction and consequent foundation and slab movements.

Irrigation of landscaped areas should be strictly limited to that necessary to sustain vegetation. Excessive irrigation could result in saturation and weakening of foundation soils. The Landscape Architect and prospective owners should be informed of the surface drainage requirements included in this report.

5.0 FOUNDATION RECOMMENDATIONS

Based on the building type and subsurface conditions, the podium building may be supported on a foundation system consisting of isolated interior and perimeter spread footings bearing in native soil or bedrock. We provide recommendations for different bearing capacities for the differing soil conditions across the proposed building footprint. Figure 2 shows the estimated bedrock contact with the alluvium at the depth of the basement subgrade. To the west of this line, we recommend using the foundation parameters for alluvium and to the east, we recommend using the parameters for bedrock. Isolated footings that fall on the contact should be designed for the alluvium parameters. The actual location of this contact should be confirmed in the field during excavation and foundation sizing modified if the actual contact is east of where shown on Figure 2.

The pool building, which will be constructed at about existing grade, may be impacted by expansive soil if the soil below the building is similar to the alluvium on-site. Because of access issues, we were unable to perform exploration in this area to confirm subsurface conditions. For planning purposes, we recommend that either the building be constructed with a post-tensioned mat foundation or the upper 3 feet of building pad be excavated and replaced with low-expansion potential soil and founded on spread footings designed for the alluvium parameters in the following sections. Exploration should eventually be performed in this area of the site to confirm building pad conditions and refine the pool building parameters.

5.1 SPREAD FOOTINGS

We recommend that footings be designed with the minimum footing dimensions indicated in the table below.

TABLE 5.1
Minimum Footing Dimensions

Footing Type	*Minimum Depth (inches)	Minimum Width (inches)
Continuous	24	18
Isolated	24	24

*below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent pad grade. Where footings are designed for alluvium soil conditions, a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) may be used for dead-plus-live loads. For footings on bedrock, an allowable bearing capacity of 6,000 psf can be used for design. The designer may increase this bearing capacity 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. All 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.

5.1.1 Settlement

Based on the loads typical for this type of construction, we estimate that post-construction settlement will be on the order of ¾ inches or less with differential settlement less than ½ inch between adjacent columns and over a lineal distance of 30 feet along continuous wall footings.

5.1.2 Foundation Lateral Resistance

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of footings. 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: 400 pcf
- Coefficient of Friction: 0.35

The above allowable values include a factor of safety of 1.5. A combination of both friction and passive pressure may be used if one of the values is reduced by 50 percent.

5.2 POOL BUILDING POST-TENSIONED MAT

As mentioned, the pool building area was inaccessible for exploration. For planning purposes, and based on the moderate to highly expansive soil encountered within the alluvium, the building could be constructed with a post-tensioned (PT) mat to mitigate the potentially damaging expansive soil conditions. For planning purposes, we recommend assuming that the post-tensioned mat will have a minimum thickness of 10 inches. The mat may 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. The PT mat can 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. This allowable bearing pressure can be assumed to be 1/3 higher when evaluating transient loading conditions including wind or seismic.

Parameters for PT mat design can be provided once the building area is accessible for exploration.

When buildings are constructed with concrete slab-on-grade, 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. Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall conform to Class A vapor retarder per ASTM E 1745-11 “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs”.

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

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specific by the structural engineer.

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 layer is used, the edges of the PT slab should be thickened to limit the moisture intrusion into the sand layer. The thickened edge, if necessary, should be 2 inches thicker than the slab thickness and at least 12 inches wide.

5.3 BASEMENT FOUNDATION WALLS AND NON-BUILDING WALLS

The basement walls for the garage will act as retaining walls. Basement walls should be designed for at-rest lateral loading conditions. Some miscellaneous cantilever retaining walls may be required and can be designed for active lateral loading conditions provided they are free to move at the top of the wall. Section 5.7.1 includes wall drainage recommendations; where these recommendations can not be met, the walls should be designed for undrained conditions. The following table summarizes our recommendations for at-rest and active pressures in both drained and undrained conditions for level backfill conditions:

TABLE 5.3-1
Earth Pressures for Wall Design

Loading Condition	Drained	Undrained
Active	40 pcf	80 pcf
At-Rest	60 pcf	100 pcf

A pressure equal to one-half of any surcharge load within 10 feet of the wall should be added to the basement wall lateral loads.

Foundations for retaining walls may be designed using the parameters provided above for building foundations.

All backfill should be placed in accordance with recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to minimize possible overstressing of the walls.

5.3.1 Wall Drainage

Wall drainage for any drained walls may be provided using a 4-inch-diameter perforated pipe embedded in Class 2 permeable material, or free-draining gravel surrounded by synthetic filter fabric. The width of the drain blanket should be at least 12 inches. The drain blanket should extend to about 1 foot below the finished grades. As an alternative, prefabricated synthetic wall drain panels can be used. The upper 1 foot of wall backfill should consist of clayey soils. Drainage should be collected by perforated pipes and directed to an outlet approved by the Civil Engineer. Synthetic filter fabric should meet the minimum requirement.

5.3.2 Seismic Design Considerations

Seismic lateral earth pressures should be considered in the design of retaining walls. Under seismic conditions, the incremental seismic force along the face of a retaining wall should be calculated as follows:

$$\Delta P = 10H^2$$

H is the design height of the wall (in feet) and ΔP is the incremental seismic force in pounds per lineal foot of wall. This force has a horizontal direction and should be applied at the bottom one-third point of the wall. This seismic increment should be added to active pressures for evaluation of the seismic condition. Therefore, regardless of wall type, this force should be combined with an equivalent fluid pressure of 40 pcf in drained conditions and 80 pcf when walls lack drainage.

5.4 EXCAVATIONS AND TEMPORARY SHORING SYSTEMS

Grading and construction activities of the proposed structure will require excavations and the need for lateral support. Based on available plans from the architect, we understand that excavations extending up to approximately 12 feet deep are proposed at the majority of the development for the construction of the podium garage.

It is the responsibility of the contractor to establish and maintain stable excavation slopes in accordance with OSHA requirements. For planning purposes, the relatively soft bedrock and stiff alluvium would generally be classified as a Type B soil by OSHA criteria. The soil may be classified as Type A during construction where appropriately stiff soil and bedrock are encountered.

Along the majority of the project perimeter, adjacent improvements are too close to the excavation to allow the garage excavation to be made without shoring.

5.4.1 Secant Pile Temporary Shoring

Temporary shoring will be required to facilitate site construction. Due to the stiffness of the existing soil and bedrock, and the adjacent improvements, we understand you are currently planning on constructing a shoring system consisting of overlapping secant piles. We recommend that shoring be designed in accordance with the current Caltrans Trenching and Shoring Manual or equivalent design methodology. For wall design, the earth pressures in Section 5.7 may be used.

5.4.2 Tie-Back Anchors

Due to the proximity of the excavation to the property boundaries, we have assumed that tie-back anchors will not be implemented due to encroachment on other properties. Should this type of anchoring be required, we should be contacted to provide appropriate design recommendations.

5.5 SECONDARY SLAB-ON-GRADE CONSTRUCTION

Secondary slabs include exterior walkways, driveways and steps. Secondary slabs-on-grade should be designed specifically for their intended use and loading requirements. Cracking of the exterior flatwork is normal as it is part of the concrete curing process and should be expected. Frequent control joints should be provided during slab construction for control of cracking.

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. The Structural Engineer should design the slab reinforcement if required.

Exterior slabs should slope away from the building to prevent water from flowing toward the foundations. Consideration should be given to lightly moistening the site soils just prior to concrete placement.

6.0 PAVEMENT DESIGN

6.1 FLEXIBLE PAVEMENTS

Based on on-site soil, we assumed an R-value of 15 was appropriate for flexible pavement design. Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Procedure 608 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

TABLE 6.1-1

Recommended Asphalt Concrete Pavement Sections

	Section	
Traffic Index	**Asphalt Concrete (inches)**	**Class 2 Aggregate Base (inches)**
5	3	8
6	3.5	11
7	4	13

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

6.2 RIGID PAVEMENTS

Use concrete pavement sections 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.

- 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 to a relative compaction of 95 percent (ASTM D 1557). Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB per Section 26 of the latest Caltrans Standard Specifications.

7.0 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 determine whether our recommendations have been implemented, and to provide additional or modified recommendations, if necessary. 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. Excavation and construction will take place adjacent to existing slopes, roadways and underground utilities. We recommend that a pre-construction survey (e.g. crack survey) and monitoring program for the surrounding roadways, utilities, etc. which may be affected by construction activities be performed before and during construction. This will form a basis for any damage claims and also assist the contractor in assessing the performance of the shoring or excavation slopes. The pre-construction survey should record the elevation and horizontal position of all existing improvements within 50 feet minimum and may consist of photographs, video tapes, topographic survey, etc. We also recommend that survey points be installed along the adjacent slope in the Lafayette Reservoir Easement.

3. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. All 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 essential.

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).

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

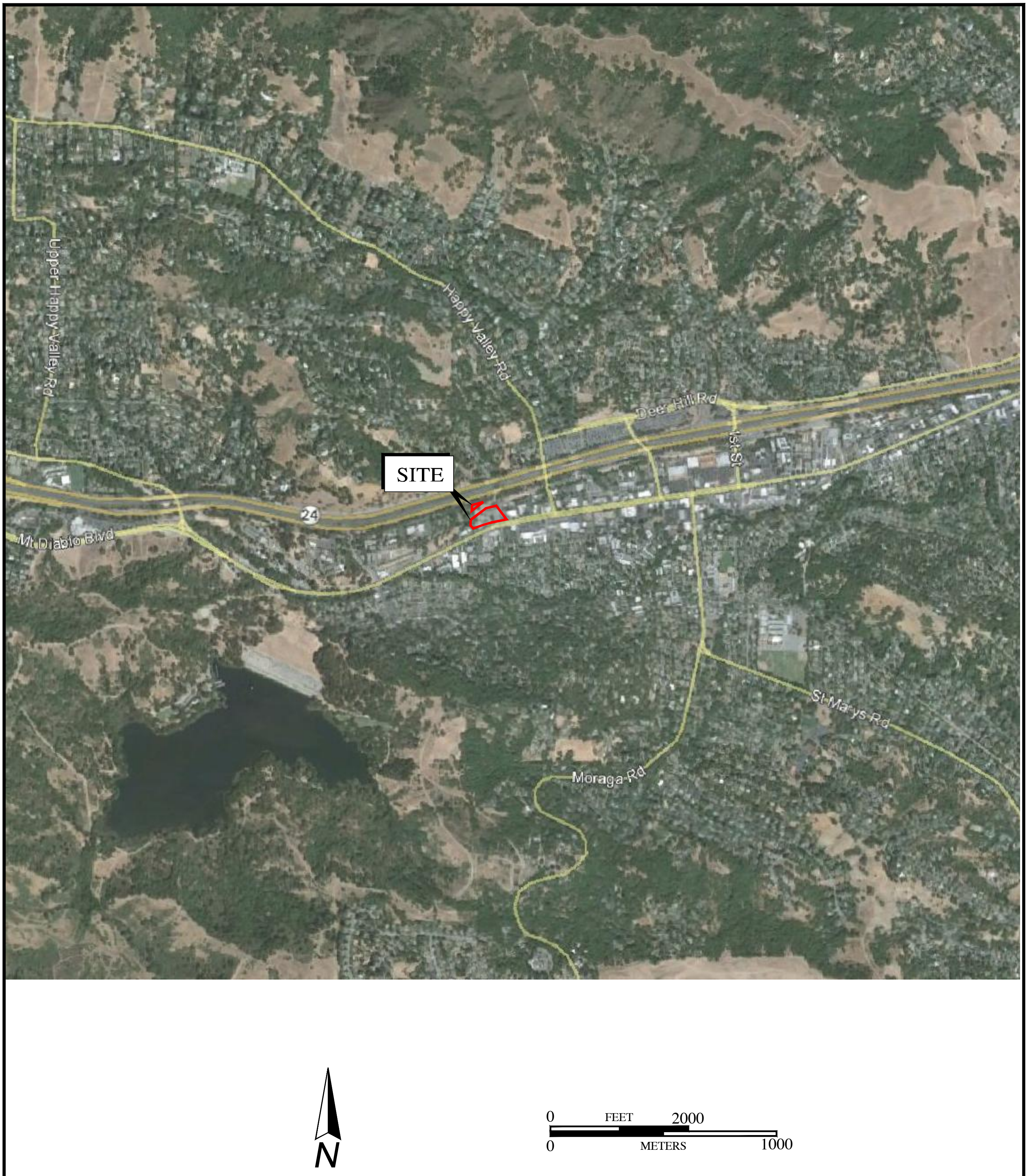
This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

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 land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our work.

This report is based upon field and other conditions discovered at the time of preparation of our work. This document must not be subject to unauthorized reuse, that is, reuse without our written authorization. Such authorization is essential because it requires us 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 our work. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If our scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, we cannot be held responsible for any or all claims, including, but not limited to claims arising from or resulting from the performance of such services by other persons or entities, and any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

FIGURES

FIGURES



BASE MAP SOURCE: GOOGLE EARTH PRO

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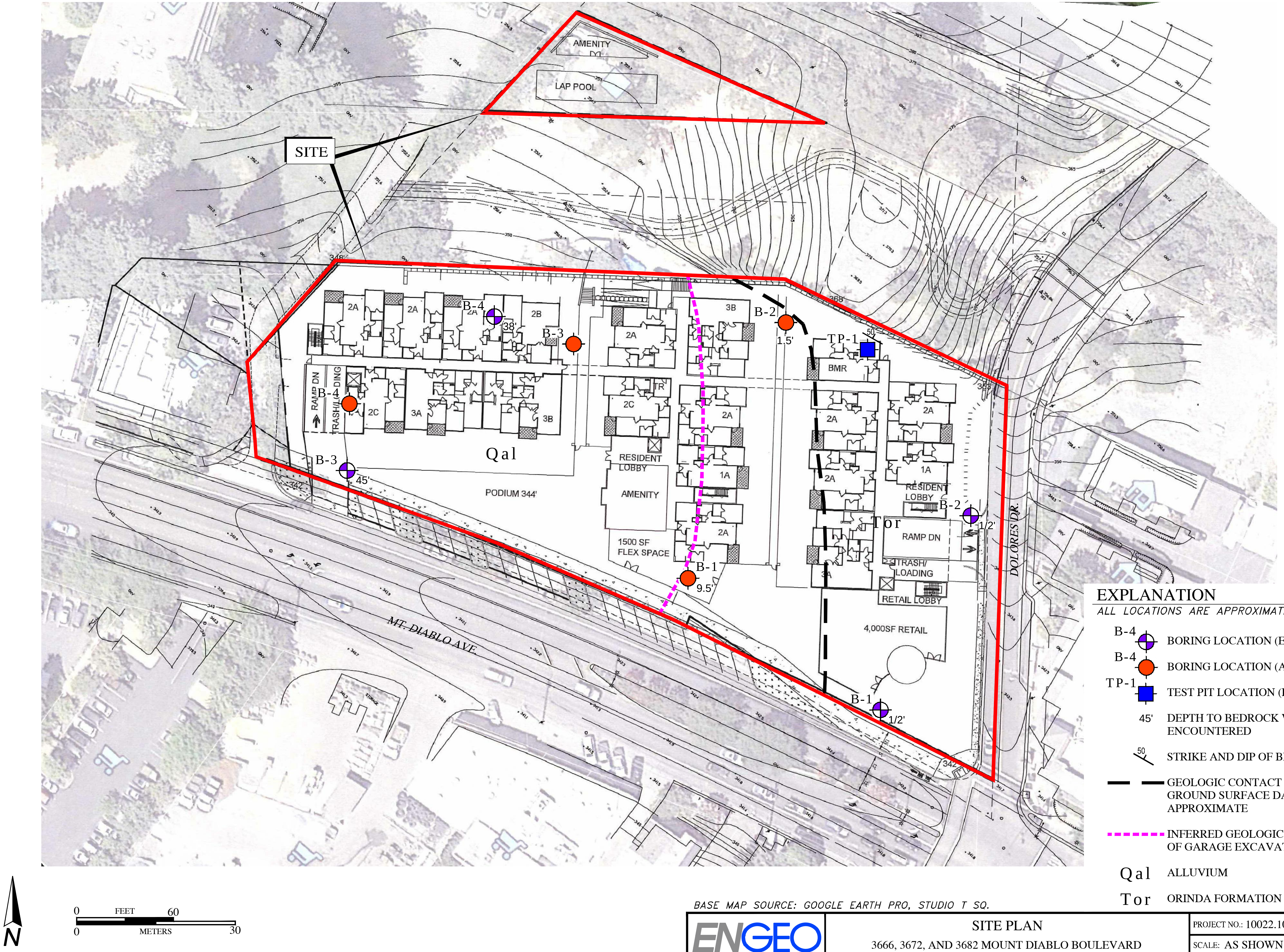
VICINITY MAP
3666, 3672, AND 3682 MOUNT DIABLO BOULEVARD
LAFAYETTE, CALIFORNIA

PROJECT NO.: 10022.100.000
SCALE: AS SHOWN
DRAWN BY: SRP
CHECKED BY: BF
FIGURE NO.
1

C:\Users\spatters\appdata\local\temp\AcPublish_4684\10022100000-GEX-1-VicinityMap-0414.dwg Plot Date: 4-10-14 spatters

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

B-4 ⊕	BORING LOCATION (ENGEO, 2013)
B-4 ●	BORING LOCATION (ALL WEST, 2008)
TP-1 ■	TEST PIT LOCATION (ENGEO)
45'	DEPTH TO BEDROCK WHERE ENCOUNTERED
50	STRIKE AND DIP OF BEDDING
— —	GEOLOGIC CONTACT AT EXISTING GROUND SURFACE DASHED WHERE APPROXIMATE
- - - -	INFERRED GEOLOGIC CONTACT AT BASE OF GARAGE EXCAVATION
Qal	ALLUVIUM
Tor	ORINDA FORMATION

BASE MAP SOURCE: GOOGLE EARTH PRO, STUDIO T SQ.

ENGEO
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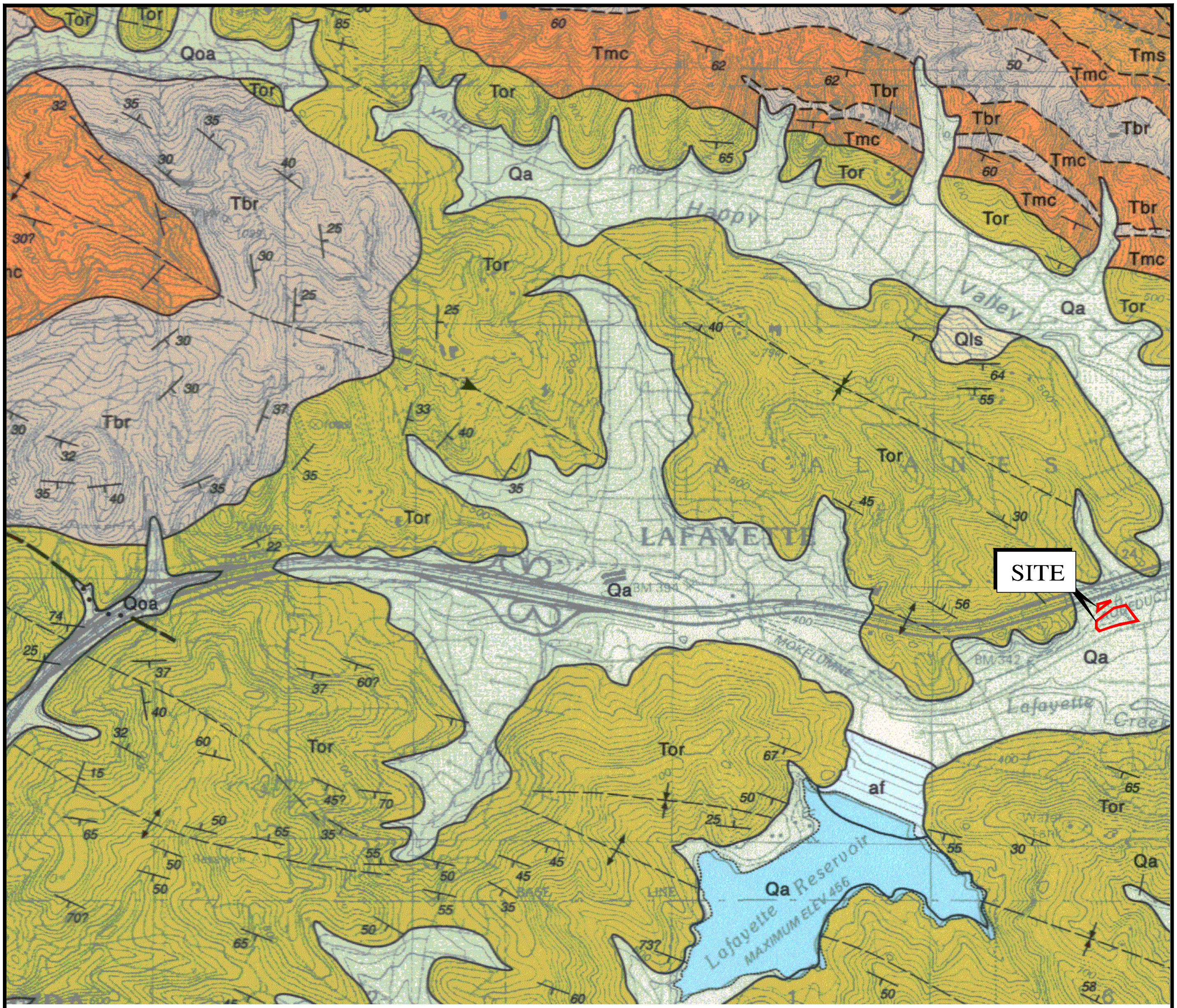
SITE PLAN
3666, 3672, AND 3682 MOUNT DIABLO BOULEVARD
LAFAYETTE, CALIFORNIA

PROJECT NO.: 10022.100.000
SCALE: AS SHOWN
DRAWN BY: SRP
CHECKED BY: BF
FIGURE NO. 2

C:\Users\spotters\appdata\local\temp\AcPublish_6168\10022100000-GEX-2-SitePlan-0414.dwg Plot Date: 4-24-14 spotters

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

———— – – – – GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

——▲–▲– – – · · FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT.

Qa	ALLUVIAL GRAVEL
af	ARTIFICIAL FILL
Tor	ORINDA FORMATION
Tbr	BROINES SANDSTONE
Tmc	MONTEREY FORMATION

N

0 FEET 2000

0 METERS 1000

AXIS OF FOLD

ANTICLINE SYNCLINE

STRIKE AND DIP OF STRATA

INCLINED VERTICAL

BASE MAP SOURCE: DIBBLEE, 2005

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REGIONAL GEOLOGIC MAP
3666, 3672, 3682 MOUNT DIABLO BOULEVARD
LAFAYETTE, CALIFORNIA

PROJECT NO.: 10022.100.000
SCALE: AS SHOWN
DRAWN BY: SRP
CHECKED BY: BF
FIGURE NO. 3

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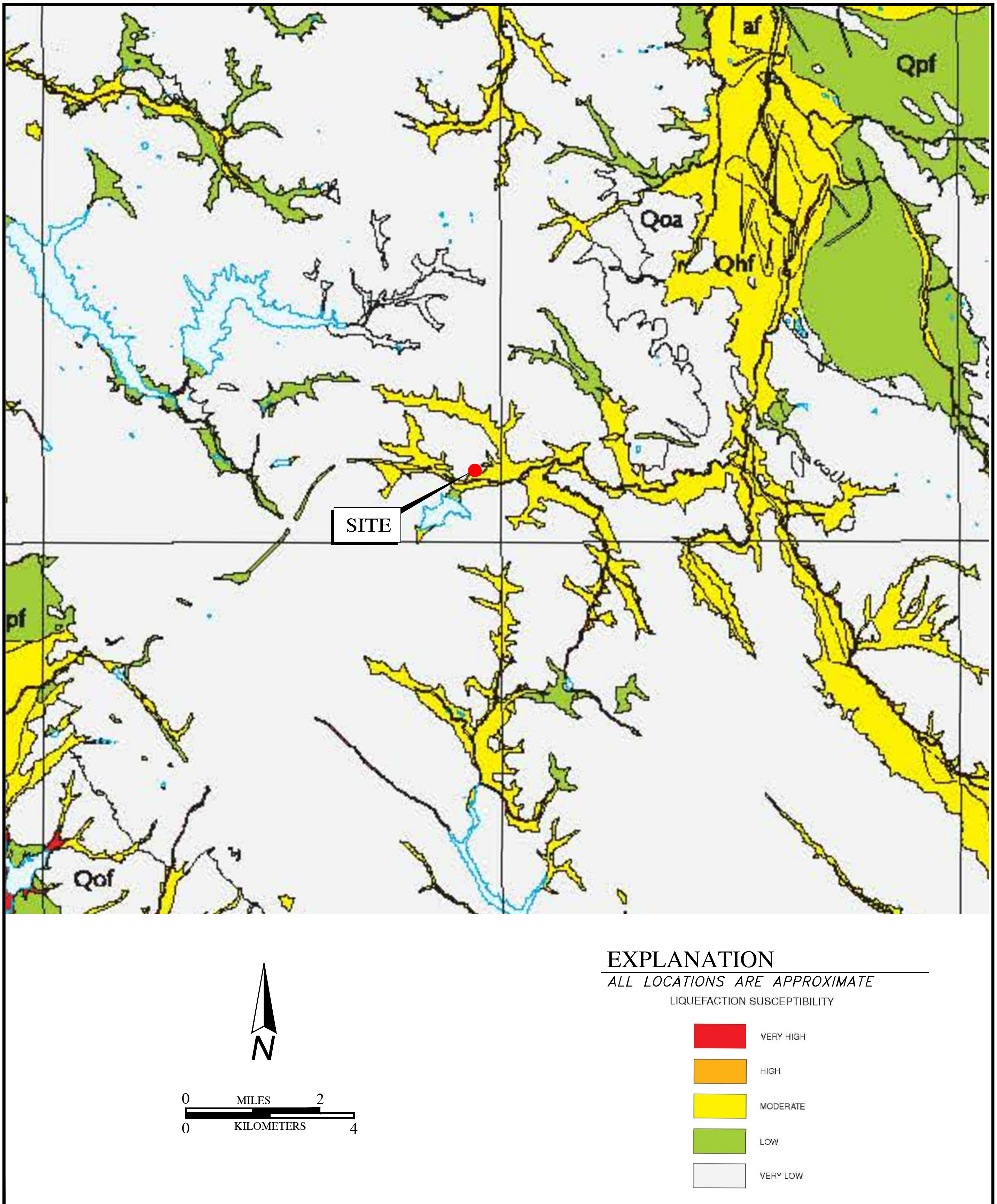
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

LIQUEFACTION SUSCEPTIBILITY

🟥 VERY HIGH

🟧 HIGH

🟨 MODERATE

🟩 LOW

⬜ VERY LOW

BASE MAP SOURCE: WITTER, KNUDSEN, 2006

ENGEO
Expect Excellence

REGIONAL LIQUEFACTION SUSCEPTIBILITY MAP

3666, 3672, 3682 MOUNT DIABLO BOULEVARD

LAFAYETTE, CALIFORNIA

PROJECT NO.: 10022.100.000

SCALE: AS SHOWN

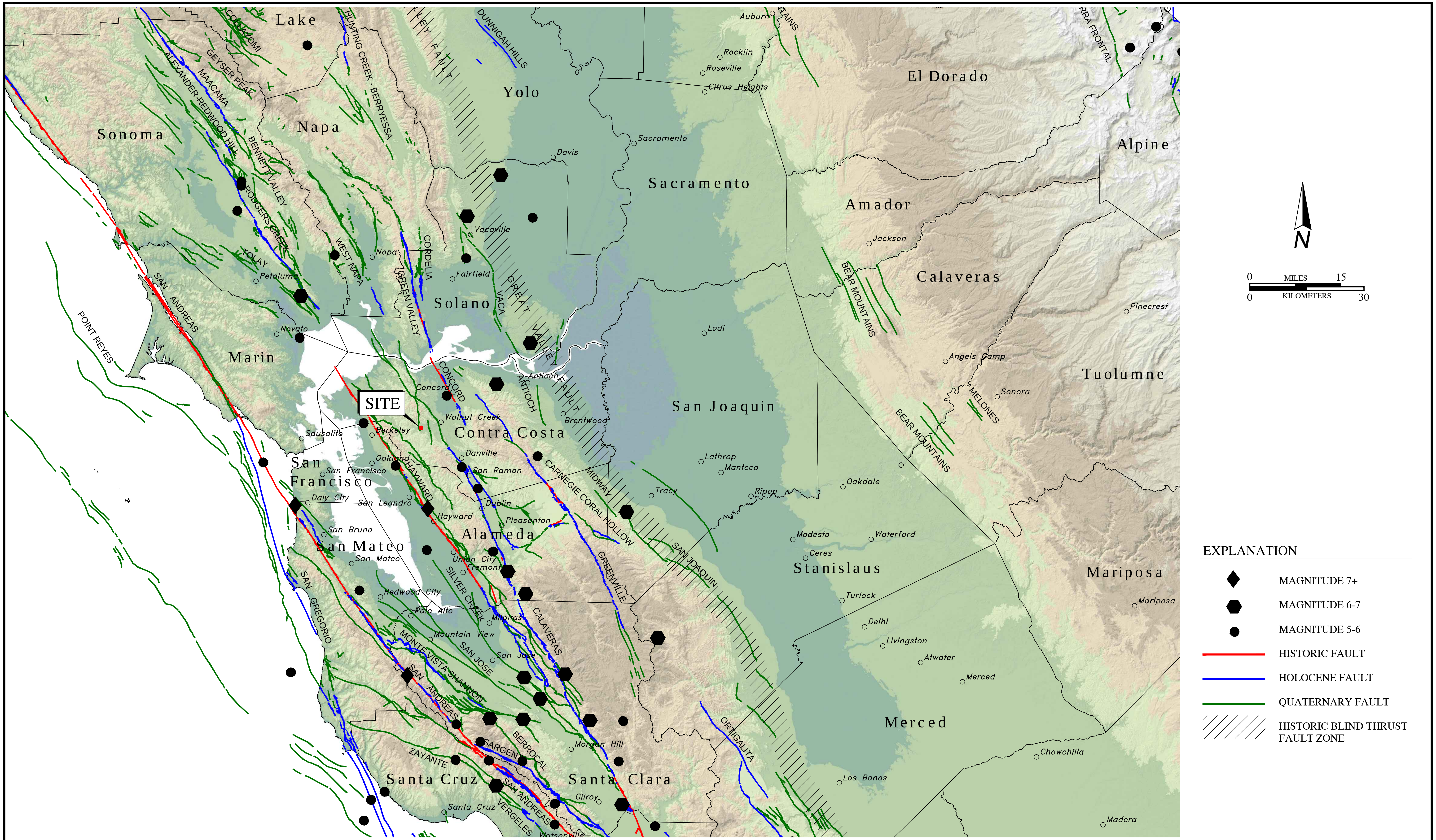
DRAWN BY: SRP

CHECKED BY: BF

FIGURE NO.
4

C:\Users\spatters\appdata\local\temp\AcPublish_4684\10022100000-GEX-4-LiquefactionMap-0414.dwg Plot Date: 4-10-14 spatters

ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

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REGIONAL FAULTING AND SEISMICITY
3666, 3672, AND 3682 MOUNT DIABLO BOULEVARD
LAFAYETTE, CALIFORNIA

PROJECT NO.: 10022.100.000
SCALE: AS SHOWN
DRAWN BY: SRP
CHECKED BY: JF
FIGURE NO. 5

ORIGINAL FIGURE PRINTED IN COLOR

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APPENDIX A

Boring And Test Pit Logs

APPENDIX A

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LOG OF BORING B-1

Geotechnical Exploration
3666 - 3682 Mount Diablo Blvd.
Lafayette, California
10022.100.000

DATE DRILLED: 3/27/2014
HOLE DEPTH: Approx. 10 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 346 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			Asphalt (3 inches)										
			Aggregate Base (1 inch)										
		■	Interbedded SANDSTONE and SILTSTONE, olive brown, extremely weak, closely fractured, thinly bedded, moderately weathered.			27							
	1												
5		■				53/6"							
	2		Some fine-grained sand.										
		■	Same as above.			55/6"							
10	3												
			Bottom of boring at 10 feet, no groundwater encountered during drilling.										

ENGEO INCORPORATED

LOG OF BORING B-2

Geotechnical Exploration
3666 - 3682 Mount Diablo Blvd.
Lafayette, California
10022.100.000

DATE DRILLED: 3/27/2014
HOLE DEPTH: Approx. 10 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 350 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			Asphalt (3 inches)	■									
			Aggregate Base (3 inches)										
		■	SANDSTONE CONGLOMERATE, olive brown to reddish brown, extremely weak, closely fractured, moderately weathered, poorly cemented, fine to coarse subrounded gravel/pebbles.			52/6"							
	1		Interbedded SILTSTONE and CLAYSTONE, red and dark olive brown, extremely weak, closely fractured, moderately weathered.	▨									
5		■				59					20.68	102.2	1.78
	2												
10	3	■				56/6"					29.06	87.4	2.01
			Bottom of boring at 10 feet, no groundwater encountered during drilling.										

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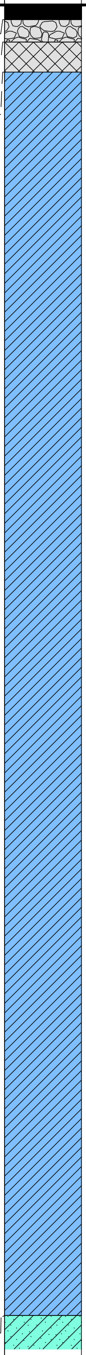
LOG OF BORING B-3

Geotechnical Exploration
3666 - 3682 Mount Diablo Blvd.
Lafayette, California
10022.100.000

DATE DRILLED: 3/27/2014
HOLE DEPTH: Approx. 45 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 346 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead



Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			Asphalt (4 inches)										
			Aggregate Base (6 inches)										
		■	LEAN CLAY (CL), dark brown, stiff, moist, with gravel and rock fragments. (Fill)			33	46	17	29				2.5*
	1		LEAN CLAY (CL), very dark brown, very stiff, moist, few fine gravels. (Qal)										
5		■	With fine gravel.			55					19.8		3.25*
	2												
10	3	■	Becomes dark brown, fine to coarse gravel, <5% fine-grained sand.			77					19.8	108.2	3.17
	4												
15		■	Becomes reddish brown, 5-10% fine-grained sand.			63					20.9		3.25*
	5												
					▽								
20	6	■	Stiff, moist to wet.			45							1.75*
	7												
25		■	Medium stiff, wet.			15					28.1	96.6	0.61
	8												
30	9	■	Same as above.			34							

ENGEO
INCORPORATED

LOG OF BORING B-3

Geotechnical Exploration
3666 - 3682 Mount Diablo Blvd.
Lafayette, California
10022.100.000

DATE DRILLED: 3/27/2014
HOLE DEPTH: Approx. 45 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 346 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
		■	CLAYEY SAND (SC), brown, very dense, wet, with fine gravel.	▨							15.1		2.0*
			LEAN CLAY (CL), brown, stiff, moist, few coarse gravels.	▨									
	10												
35		■	Stiff, with sand.			50					24.1		1.75*
	11												
	12												
40		◣	Very stiff, few fine gravels.			20							3.0*
	13												
		◣	Same as above.			45							
45			SANDSTONE CONGLOMERATE, brown, extremely weak, moderately weathered, poorly cemented.	▨									
			Bottom of boring at 45.5 feet, groundwater encountered at 19 feet during drilling.										

ENGEO
INCORPORATED

LOG OF BORING B-4

Geotechnical Exploration
3666 - 3682 Mount Diablo Blvd.
Lafayette, California
10022.100.000

DATE DRILLED: 3/27/2014
HOLE DEPTH: Approx. 40 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 351 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			Asphalt (3 inches)	■									
			Aggregate Base (3 inches)										
		■	LEAN CLAY (CL), black, very stiff, moist, few fine gravels. (Qal)	▨		28							2.75*
	1			▨									
5		■	Becomes very dark brown.	▨		36					20.4	107	2.29
	2			▨									
10	3	■	Becomes dark brown, with fine gravel.	▨		62	48	16	32				3.5*
	4			▨									
15		■	Reddish brown, 5-10% sand.	▨		54					19.7		3.5*
	5			▨									
20	6	■	Same as above.	▨		25					23.5		2.0*
	7			▨	∇								
25		■	Medium stiff, wet.	▨		17					20.4	108.5	.57
	8			▨									
30	9	■		▨		16							

ENGEO
INCORPORATED

LOG OF BORING B-4

Geotechnical Exploration
3666 - 3682 Mount Diablo Blvd.
Lafayette, California
10022.100.000

DATE DRILLED: 3/27/2014
HOLE DEPTH: Approx. 40 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (msl): Approx. 351 ft.

LOGGED / REVIEWED BY: J. White / JBR
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
		■	LEAN CLAY (CL), black, very stiff, moist, few fine gravels. (Qal) Same as above.	▨									1.25*
	10			▨									
35	11	■	Same as above.	▨		30					22.7		2.75*
	12	■	Interbedded SANDSTONE and SILTSTONE, brown, extremely weak, moderately weathered.	▥		50/6"							
40			Bottom of boring at 40 feet, groundwater encountered at 23 feet during drilling.										

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TEST PIT LOG

3666, 3672 and 3682 Mount Diablo Boulevard
Lafayette, California
10022.100.000

Logged By: J. Brooks Ramsdell
Logged Date: 3/27/2014

Test Pit Number	Depth (Feet)	Description
TP1	0 – 6	INTERBEDDED SANDSTONE, CLAYSTONE AND CONGLOMERATE SANDSTONE, yellowish brown, weak, poorly cemented, well graded, fine to medium grained, highly weathered, very closely fractured to crushed. CLAYSTONE, reddish brown, very weak, thin bedded, highly weathered, closely fractured. Bedding attitude – N65W 50NE and N60W 48NE CONGLOMERATE, gray brown to yellowish brown, weak, poorly cemented, well graded, fine to coarse gravel, fine to coarse grained sand matrix, clasts are subrounded and comprise chert, quartzite, greenstone and greywacke, red clay films on clasts, thick bedded, scattered cobbles Bottom at 6 feet. No groundwater encountered.

APPENDIX B

Laboratory Testing

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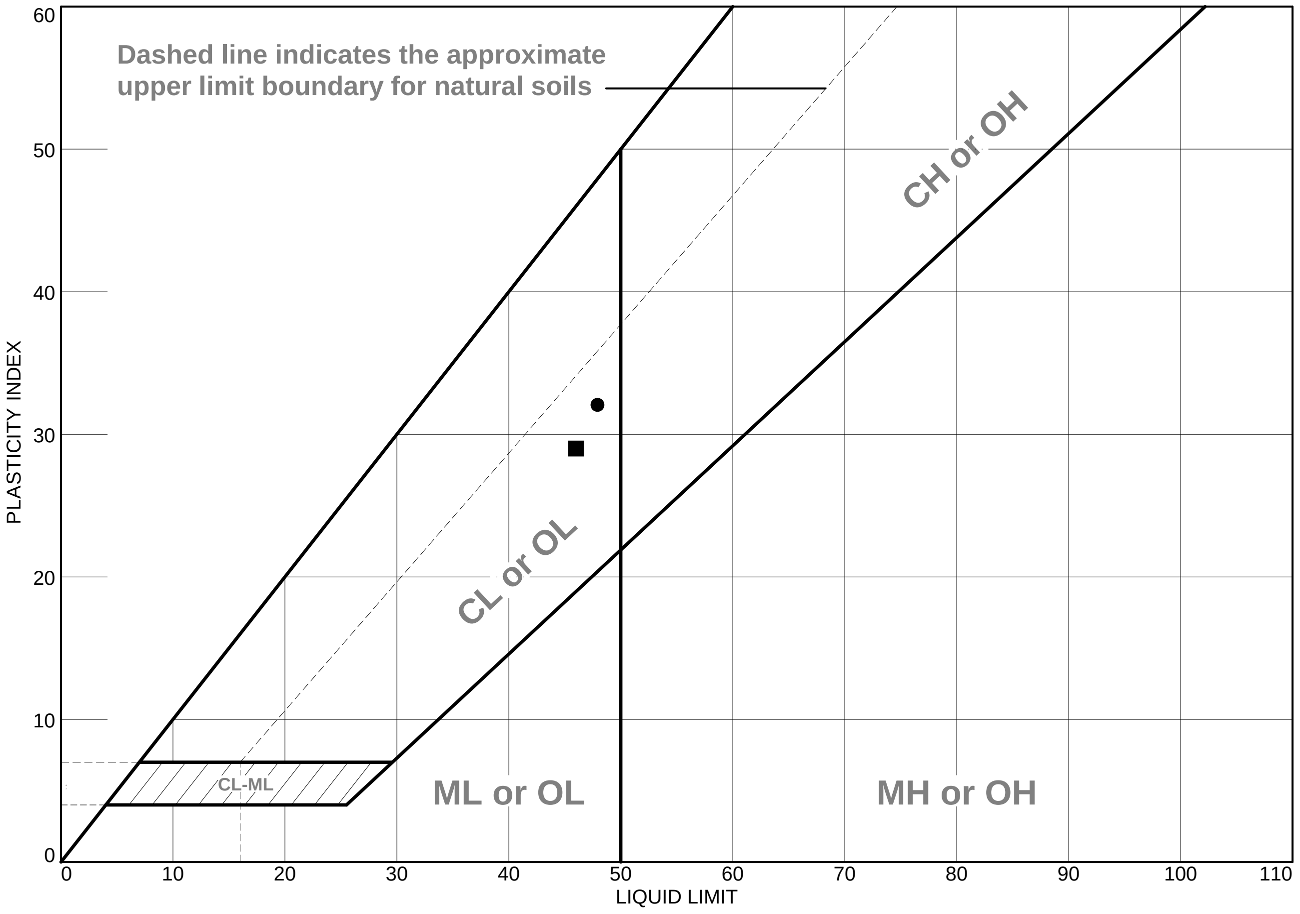
B

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	48	16	32			
■	See exploration logs	46	17	29			

Project No. 10022.100.000 **Client:** Lennar Homes

Project: 3666-3682 Mt. Diablo Blvd GEX Lafayette

●**Depth:** 10.0 feet **Sample Number:** B4@ 10

■**Depth:** 2.0 feet **Sample Number:** B3@ 2

Remarks:

ENGEO INCORPORATED

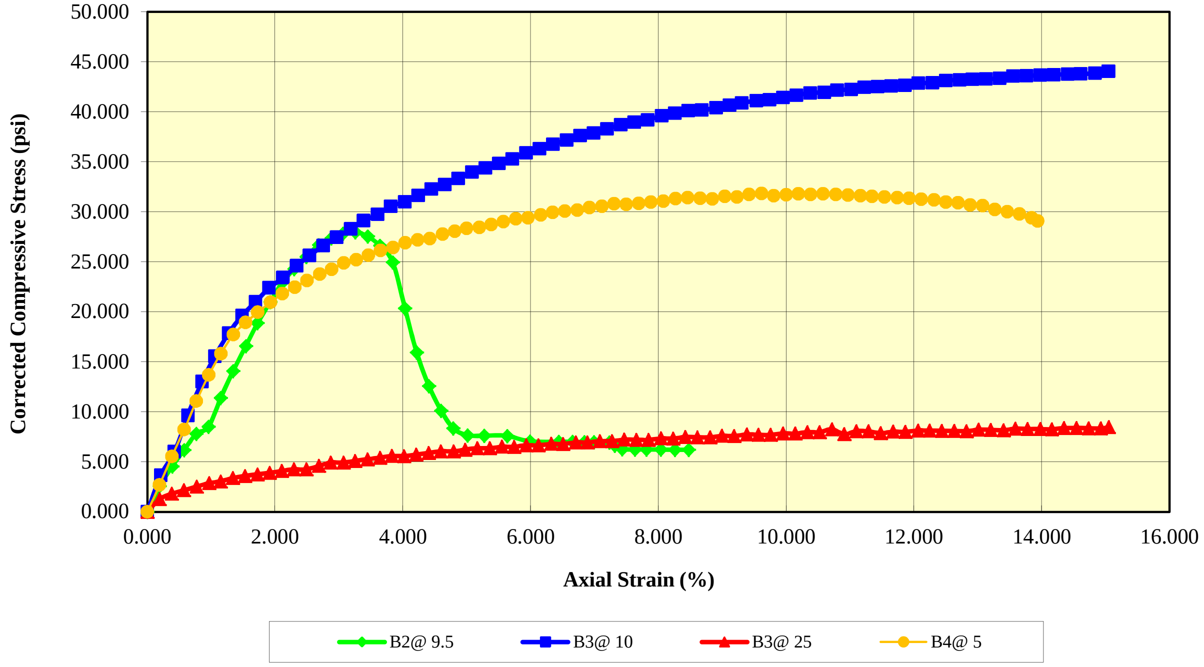
Tested By: JAL **Checked By:** GC

ENGEO Incorporated

Unconfined Compression Test Report (ASTM D2166)

Date 4/1/2014
Checked By G. Criste
Date 4/1/2014
Computed By J.Lawton
Date 4/1/2014
Tested By J.Lawton

Compressive Stress Axial Strain Curve



Before Test	Specimen			
	B2@ 9.5	**B3@ 10**	**B3@ 25**	**B4@ 5**
Water Content (%)	29.06	19.77	28.14	20.38
Dry Density (pcf)	87.400	108.200	96.600	107.026
Saturation (%)	86.24	99.04	100.00	98.96
Void Ratio	0.89	0.53	0.71	0.55
Diameter (in)	2.387	2.389	2.396	2.408
Height (in)	5.263	4.763	5.271	5.245
Height-to-Diameter Ratio	2.205	1.994	2.200	2.178
Test Data	**B2@ 9.5**	**B3@ 10**	**B3@ 25**	**B4@ 5**
Unconfined Compressive Strength (psf)	4025.697	6343.902	1221.318	4584.741
Undrained Shear Strength (psf)	2012.848	3171.951	610.659	2292.371
Strain Rate (in./min.)	0.05000	0.05000	0.05000	0.05000
Strain at Failure (%)	3.258	15.043	15.049	9.614

Project Information		Sample ID	Description
Project Number	10022.100.000	**B2@ 9.5**	See exploration logs
Project Name	3666-3682 Mt Diablo Blvd GEX Lafayette	**B3@ 10**	See exploration logs
		B3@ 25	See exploration logs
Test Date	4/1/2014	**B4@ 5**	See exploration logs
Project Location	Lafayette, CA		
Client	Lennar Homes		
Remarks			

Index Properties	B2@ 9.5	B3@ 10	B3@ 25	B4@ 5
Specific Gravity:	2.65	2.65	2.65	2.65
Liquid Limit:				
Plastic Limit:				

ENGEO Incorporated 2010 Crow Canyon Place Suite 250 San Ramon, CA 94583
Laboratory address: 2057 San Ramon Valley Blvd., San Ramon, CA 94583 (925) 837-2973

ENGEO Incorporated

Unconfined Compression Test Report (ASTM D2166)

Date 4/1/2014

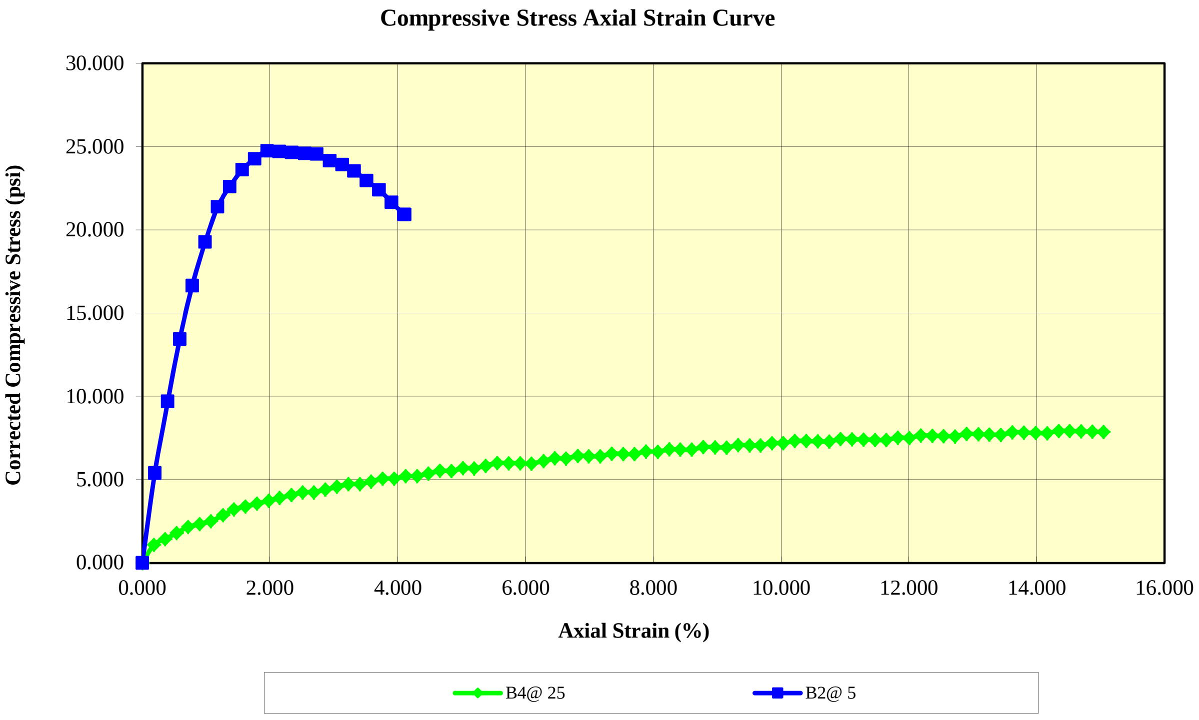
Checked By G. Criste

Date 4/1/2014

Computed By J.Lawton

Date 4/1/2014

Tested By J.Lawton



Before Test	Specimen			
	B4@ 25	B2@ 5		
Water Content (%)	20.35	20.68		
Dry Density (pcf)	108.500	102.200		
Saturation (%)	100.00	88.53		
Void Ratio	0.52	0.62		
Diameter (in)	2.395	2.404		
Height (in)	5.629	5.168		
Height-to-Diameter Ratio	2.350	2.150		
Test Data	**B4@ 25**	**B2@ 5**		
Unconfined Compressive Strength (psf)	1142.885	3564.285		
Undrained Shear Strength (psf)	571.442	1782.143		
Strain Rate (in./min.)	0.05000	0.05000		
Strain at Failure (%)	14.341	1.952		

Project Information		Sample ID	Description
Project Number	10022.100.000	B4@ 25	See exploration logs
Project Name	3666-3682 Mt Diablo Blvd GEX Lafayette	B2@ 5	See exploration logs
Test Date	4/1/2014		
Project Location	Lafayette, CA		
Client	Lennar Homes		
Remarks			

Index Properties	B4@ 25	B2@ 5		
Specific Gravity:	2.65	2.65		
Liquid Limit:				
Plastic Limit:				

ENGEO Incorporated 2010 Crow Canyon Place Suite 250 San Ramon, CA 94583
Laboratory address: 2057 San Ramon Valley Blvd., San Ramon, CA 94583 (925) 837-2973

APPENDIX C

Previous Boring Logs - Allwest Geoscience Inc.

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C

PROJECT: 3672-3680 MT. DIABLO
LAFAYETTE, CALIFORNIA

BORING NO. 1

BORING SUPERVISOR:

TYPE OF BORING: 3" Auger

DATE OF BORING: Nov 2008

HAMMER WEIGHT/DROP #70/ 30"
TYPE: free fall weight

SURFACE ELEVATION:

GROUNDWATER DEPTH: GW N.E.

DESCRIPTION OF MATERIALS	DEPTH IN FEET	SAMPLE	SAMPLE NUMBER SAMPLE DIAMETER	DRIVING RESISTANCE BLOWS PER FT. (SPT)	DRY DENSITY (PCF)	MOISTURE CONTENT %	UNCONFINED-COMPRESSION STRENGTH (PSF)	OTHER TESTS (Field, Lab)
Gray/ brown Clayey SILT, dry–moist, w/0.25"–0.75" gravels (Fill, ML – CL)	1	■	1–1	25/23/18 (27) SPT				Pene=3.5
	2	■						
Stiff dark brown Silty CLAY, dry–moist, w/light brown mottling, moderately expansive (Native, CH)	3							P.I.=30 L.L.=55
	4							
	5							
	6							
	7							
	8							
	9	■	3–2	13/23/29 (35) SPT				PI=4.25
Light Olive Sandy highly weathered CLAYSTONE, w/roots, dry–moist, w/ white trace (Bedrock)	10	■						
	11							
	12							
	13							
	14	■	3–3	17/27/45 (48) SPT				Pene=4.5
	15	■						
Bottom of Boring = 15 ft	16							
	17							
	18							

Job No. 08–7912GP

ALLWEST GESCIENCES INC.

Figure 4

PROJECT:	3672-3680 MT. DIABLO LAFAYETTE, CALIFORNIA	BORING NO. 2
BORING SUPERVISOR:	TYPE OF BORING: 3" Auger	DATE OF BORING: Nov 2008
HAMMER WEIGHT/DROP #70/ 30" TYPE: free fall weight		
SURFACE ELEVATION:		
GROUNDWATER DEPTH: GW	N.E.	

DESCRIPTION OF MATERIALS	DEPTH IN FEET	SAMPLE	SAMPLE NUMBER SAMPLE DIAMETER	DRIVING RESISTANCE BLOWS PER FT. (SPT)	DRY DENSITY (PCF)	MOISTURE CONTENT %	UNCONFINED–COMPRESSION STRENGTH (PSF)	OTHER TESTS (Field, Lab)
Red orange Silty CLAY to Clayey SILT, dry–moist, non–expansive (Native, ML)	1							
Stiff red orange CLAYSTONE to SILTSTONE, dry–moist	2	■	1 – 2"	18/23/23 (30) SPT				
(bedrock)	3	■						
	4							
	5							
	6	■	2 – 2"	18/21/22 (28) SPT				
	7							
	8							
	9							
	10							
	11	■	3 – 2"	35/$\frac{50}{6''}$ (65) SPT				
Bottom of Boring = 11 ft	12							
	13							
	14							
	15							
	16							
	17							
	18							

Job No. 08–7912GP	ALLWEST GESCIENCES INC.	Figure 5

PROJECT: 3672-3680 MT. DIABLO
LAFAYETTE, CALIFORNIA

BORING NO. 3

BORING SUPERVISOR:

TYPE OF BORING: 3" Auger

DATE OF BORING: Nov 2008

HAMMER WEIGHT/DROP #70/ 30"
TYPE: free fall weight

SURFACE ELEVATION:

GROUNDWATER DEPTH: GW N.E.

DESCRIPTION OF MATERIALS	DEPTH IN FEET	SAMPLE	SAMPLE NUMBER SAMPLE DIAMETER	DRIVING RESISTANCE BLOWS PER FT. (SPT)	DRY DENSITY (PCF)	MOISTURE CONTENT %	UNCONFINED– COMPRESSION STRENGTH (PSF)	OTHER TESTS (Field, Lab)
Asphalt Pavement (4")								
Light brown very Sandy Clayey SILT, dry–moist,w/0.5"–1"gravel, expansive (fill)	1							
Dark brown Silty CLAY, moist, w/0.125"–0.5"rocks, w/ worange mottling (Colluvium)	2							
Gray/brown Silty CLAY, w/0.25"–0.5" rocks, w/ red brown mottling (Colluvium)	3		3–1	Bulk Sample				P.I.=40 L.L.=70
Stiff dark brown Silty CLAY (CH), dry–moist, w/pebbles, expansive Qlc	4							
	5		3–2	Bulk Sample				
	6							
Turns light brown Siltier (Alluvium)								
	7							
	8							
	9		3–3	Bulk Sample				
	10							
	11	■	3–4	10/15/20 (23) SPT				Pene=2.5ksf
	12	■						
Bottom of Boring = 12 ft								
	13							
	14							
	15							
	16							
	17							
	18							

Job No. 08–7912GP

ALLWEST GESCIENCES INC.

Figure 6

PROJECT: 3672-3680 MT. DIABLO LAFAYETTE, CALIFORNIA

BORING NO. 4

BORING SUPERVISOR:

TYPE OF BORING: 3" Auger

DATE OF BORING: Nov 2008

HAMMER WEIGHT/DROP #70/ 30"
TYPE: free fall weight

SURFACE ELEVATION:

GROUNDWATER DEPTH: GW N.E.

DESCRIPTION OF MATERIALS	DEPTH IN FEET	SAMPLE	SAMPLE NUMBER SAMPLE DIAMETER	DRIVING RESISTANCE BLOWS PER FT. (SPT)	DRY DENSITY (PCF)	MOISTURE CONTENT %	UNCONFINED-COMPRESSION STRENGTH (PSF)	OTHER TESTS (Field, Lab)
Asphalt Pavement (4")								
Dark brown Clayey SILT, dry-moist, w/ 0.5"-1" gravel (fill)	1							
Dark brown Silty CLAY, dry-moist, expansive soil, CH	2							
			4-1	Bulk Sample				P.I.=35 L.L.=62
	3							
	4							
Dark brown Silty CLAY, w/0.125"-0.25" rocks, w/ light brown mottling, w/pebbles, expansive	5							
Orange green mottling	6		4-2	Bulk Sample				P.I.=25 L.L.=47
(Alluvium, CL)	7							
Gray/brown Silty CLAY, dry-moist, w/.125"-.25"rocks, expansive, CH	8							
	9							
	10			35/50 6"				
Bottom of Boring = 10 ft								
	11							
	12							
	13							
	14							
	15							
	16							
	17							
	18							

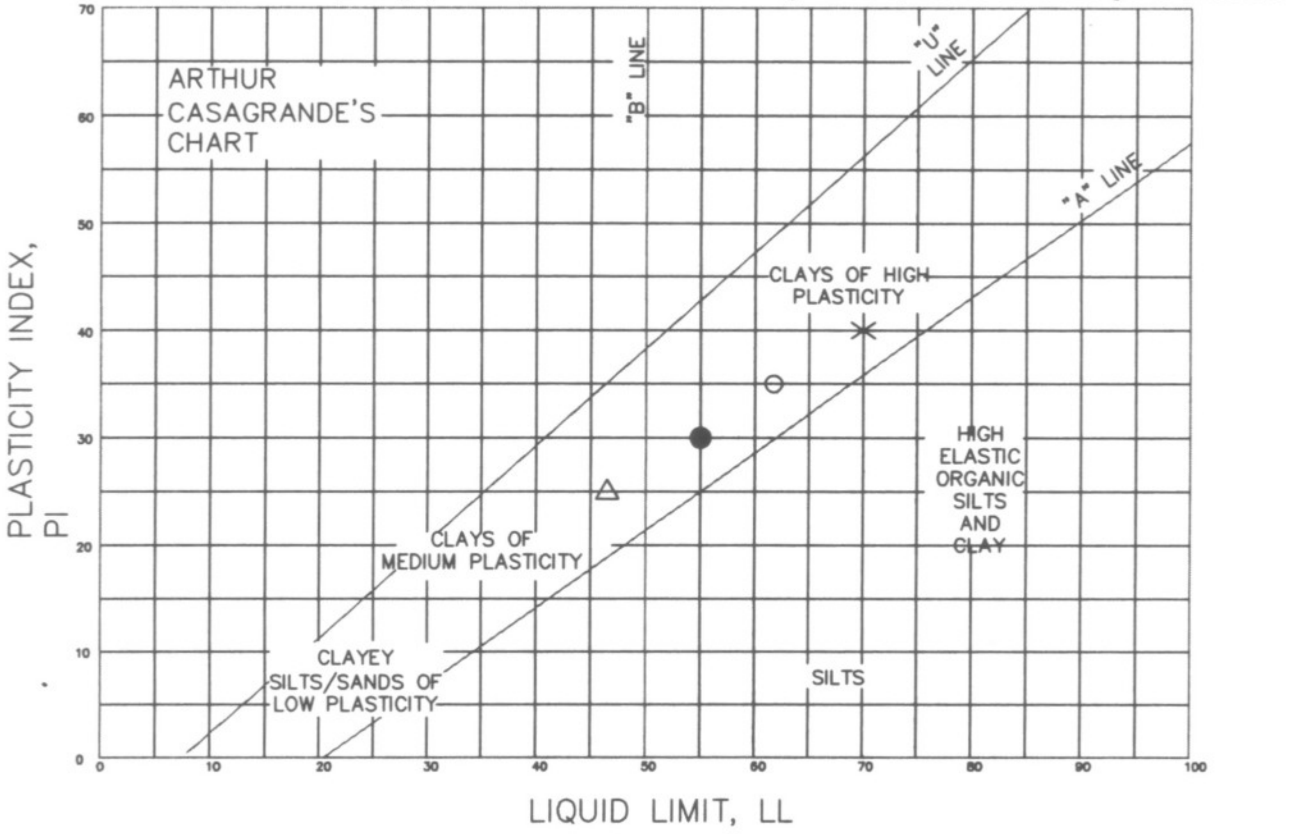
Job No. 08-7912GP

ALLWEST GESCIENCES INC.

Figure 7

PLASTICITY CLASSIFICATION

Remolded; only fraction passing #40 sieve, fine sand, silt, and clay fractions, coarse sand and gravel discarded



Reference CBC2007 1802.3.2 and ASTM D4318

CLASSIFACTION TEST RESULTS

SAMPLE IDENTIFICATION			ATTERBERG LIMITS			GRAIN SIZES % DRY WT.				
SAMPLE NO. (Boring/Pit/Bulk)	LETTER DESIGNATION	DESCRIPTION	LIQUID LIMIT	PLASTICITY INDEX	SHRINKAGE LIMIT	ESTIMATED % PASSING #40 SIEVE	SAND	SILT	CLAY	COLLOIDAL
Boring #1	●	Brown Silty CLAY	55	30	---	---	---	---	---	---
					---	---	---	---	---	---
3–1	×	Gray Brown Silty CLAY	70	40						
4–1	○	Dark Brown Silty CLAY	62	35	---	---	---	---	---	---
4–2	△	Olive Brown Silty CLAY	47	25	---	---	---	---	---	---
					---	---	---	---	---	---

Job No. 07–7912GP

ALLWEST GEOSCIENCE INC.

Figure 8