MILLER PACIFIC
ENGINEERING GROUP

UPDATED GEOTECHNICAL INVESTIGATION
VALLE VERDE AND HERITAGE HOUSE PROJECTS
NAPA, CALIFORNIA

January 29, 2019

Project 1687.01

Prepared for:
Burbank Housing Development Corporation
790 Sonoma Avenue
Santa Rosa, California 95404

Attention: Marianne Lim, Director of Housing

CERTIFICATION

This document is an instrument of service, prepared by or under the direction of the undersigned professionals, in accordance with the current ordinary standard of care. The service specifically excludes the investigation of radon, asbestos, toxic mold and other biological pollutants, and other hazardous materials. The document is for the sole use of the client and consultants on this project. Use by third parties or others is expressly prohibited without written permission. If the project changes, or more than two years have passed since issuance of this report, the findings and recommendations must be reviewed by the undersigned.

MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

REVIEWED BY

REGISTERED PROFESSIONAL ENGINEER
DANIEL S. CALDWELL
No.002006
Exp. 9/30/19
GEOTECHNICAL
STATE OF CALIFORNIA

Monica Thornton
Staff Engineer

Daniel Caldwell
Geotechnical Engineer No. 2006
(Expires 9/30/19)

UPDATED GEOTECHNICAL INVESTIGATION
VALLE VERDE AND HERITAGE HOUSE PROJECTS
NAPA, CALIFORNIA

TABLE OF CONTENTS

1.0 INTRODUCTION

This report summarizes Miller Pacific Engineering Group's (MPEG) Phase 1 updated Geotechnical Investigation for the planned improvements for the Valle Verde and Heritage House Projects at 3700 and 3710 Valle Verde Drive in Napa, California, located as shown on Figure 1.

The purpose of our Phase 1 Investigation is to explore the subsurface soil and groundwater conditions, evaluate geotechnical hazards that may affect the planned development, and provide geotechnical design criteria for the project. We previously prepared a geotechnical investigation report for the project site dated January 13, 2011 (attached in Appendix B). The scope of our Phase 1 Investigation is described in our proposal letter dated June 29, 2018, and includes the following:

- Review of readily available published geologic and geotechnical reference data;
- Exploration of subsurface conditions with eight exploratory soil borings;
- Laboratory testing of select samples to determine the pertinent engineering properties of the soil layers;
- Evaluation of geologic hazards and development of conceptual mitigation measures;
- Development of geotechnical recommendations and design criteria (i.e., site grading, seismic, foundation, etc.) for the project; and,
- Preparation of this report summarizing our findings.

Issuance of this report completes our Phase 1 scope of services. Future phases of work are anticipated to include geotechnical consultation/plan review and geotechnical observation/testing during construction.

2.0 PROJECT DESCRIPTION

The project includes the renovation of existing buildings at 3700 Valle Verde Drive (Heritage House Project) to include 66 bedrooms in three-story buildings. The project also includes the construction of 24 new units in new three-story structures (Valle Verde Project). The development will also include new asphalt paved driveway and parking areas, underground utilities, and landscaping improvements.

3.0 SITE CONDITIONS

3.1 Regional Geology

Napa County lies within the Coast Ranges geomorphic province of California, a region characterized by active seismicity, steep, young topography, and abundant landsliding and erosion owing partly to its relatively high annual rainfall. The regional basement rock consists of sedimentary, igneous, and metamorphic rock of the Jurassic-Cretaceous age (65-190 million years ago) Franciscan Complex and marine sedimentary strata of the Great Valley Sequence, which is of similar age. Within central and northern California, the Franciscan and Great Valley rocks are locally overlain by a variety of late Cretaceous and Tertiary-age sedimentary and volcanic rocks which have been deformed by episodes of folding and faulting. The youngest geologic units in the region are Quaternary-age (last 1.8 million years) sedimentary deposits. These unconsolidated deposits partially fill many of the valleys of the region.

Regional geologic mapping (Clahan, Wagner, Saucedo, Randolph-Loar, Sowers, 2004) indicates that the majority of the project site is underlain by alluvial deposits of Late Pleistocene age, including alluvial fan, stream terrace, basin, and channel deposits, composed of poorly to moderately sorted sand, silt, clay and gravel. A regional geologic map is shown on Figure 3.

3.2 Surface Conditions

We performed a site reconnaissance on October 9, 2018, to observe the existing surface conditions. The site is bounded on the north and east by Salvador Creek, on the south by Shelter Creek Drive, and on the west by Valle Verde Drive. The site is nearly level to slightly sloping, and is currently developed with the Heritage House development, as shown on the Site Plan, Figure 2. Areas around the existing buildings are mostly asphalt paved parking and driveway areas, with associated landscaping areas. Salvador Creek flows in a channel which is approximately 12 to 15 feet deep relative to the adjacent terrace grade. The channel slopes are typically inclined at between one and two horizontal to one vertical, or steeper in areas.

A former house, garage, and pool were located in the central northern portion of the site, northwest of the existing Heritage House structure. The house, garage, and pool were recently demolished, and footing and pool excavations backfilled with undocumented fill.

3.3 Field Exploration and Laboratory Testing

We explored the subsurface conditions in the general vicinity of the planned improvements on October 9th, 10th and 19th, 2018 with eight soil borings, drilled with truck-mounted drilling equipment to a maximum depth of 51.5-feet below the ground surface. The approximate boring locations are shown on the Site Plan, Figure 2. Our staff geologist logged the borings in the field and collected soil samples at select intervals for laboratory testing. Our subsurface exploration program is discussed in more detail in Appendix A. A Soil Classification Chart and the Boring Logs are presented on Figures A-1 through A-14.

In addition, we previously explored the subsurface conditions on the project site on December 9, 2010, with seven exploratory soil borings drilled with truck-mounted drilling equipment to a maximum explored depth of 31.5-feet below the ground surface. The approximate locations of our previous borings are shown on Figure 2 and the boring logs are presented in Appendix B.

Laboratory testing of select soil samples included determination of moisture content, dry density, unconfined compressive strength, percent passing the #200 sieve, and plasticity index. The results of the moisture content, dry density, unconfined compressive strength, and percent passing #200 sieve tests are presented on the boring logs. The results of the plasticity tests are presented on Figure A-15. The laboratory testing program is discussed in further detail in Appendix A.

3.4 Subsurface Conditions

Our subsurface exploration generally confirms the regionally-mapped geologic conditions at the site. The project site is underlain by alluvial deposits variously composed of medium stiff to very stiff clay with silt, sand, and gravel interbedded with occasional lenses of clayey and sandy gravel.

Groundwater was encountered in all our deeper, 2018 borings, at depths between approximately 12.0- and 22.0-feet below the ground surface at the time of drilling. In our 2010 borings, groundwater was encountered at depths between 6.0- and 15.0-feet below the ground surface. Typically, groundwater levels fluctuate seasonally with higher levels expected during the wet winter months.

We researched both the California State Water Resource Control Board's GeoTracker website (http://geotracker.waterboards.ca.gov/) and the Department of Water Resources Water Library website (http://www.water.ca.gov/waterdatalibrary) to determine if existing groundwater elevation data was available in the immediate vicinity of the project site. The databases that were searched did not indicate any groundwater data was available within a half mile radius of the project site. Therefore, we utilized an assumed highest historic groundwater level of 5-feet below the ground surface for our analyses.

3.5 Seismicity

Active Faults in the Region – The project site is located within a seismically active region that includes the Central and Northern Coast Mountain Ranges. Several active faults are present in the area both east and west of the site, including the West Napa, Green Valley, Great Valley, and Cordelia Faults. An "active" fault is defined as one that shows displacement within the last 11,000 years and, therefore, is considered more likely to generate a future earthquake than a fault that shows no sign of recent rupture. The California Department of Conservation, Division of Mines and Geology (1998) has mapped various active and inactive faults in the region. These faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The West Napa Fault is the nearest known active fault, located approximately 3 kilometers west of the site.

Historic Fault Activity – Numerous earthquakes have occurred in the region within historic times. Earthquakes (magnitude 2.0 and greater) that have occurred in the San Francisco Bay Area since 1985 have been plotted on a map shown on Figure 5.

Probability of Future Earthquakes – The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" (USGS, 2003 and 2008; Field et al, 2015) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3 (aka UCERF, UCERF2, and UCERF3, respectively). In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the 2015 UCERF3 indicate that the mean probability of an M>6.7 earthquake occurring on the West Napa Fault by 2045 is about 2.3%, up from less than 1% as reported by the 2008 UCERF2. The 2015 UCERF3 additionally indicates the likelihood of an earthquake of any magnitude occurring on the West Napa Fault by 2045 is 7.6%. It should be noted that these studies consider only the possibility that earthquakes of a given magnitude will occur, and do not consider surface rupture potential or the potential for other effects of such earthquakes.

Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

4.0 GEOLOGIC HAZARDS EVALUATION

4.1 General

The principal geologic hazards which could potentially affect the project site are strong seismic shaking from future earthquakes in the San Francisco Bay Region, liquefaction, slope instability along the Salvador Creek channel, and expansive soil. Other hazards, such as fault rupture and tsunami inundation, are not considered significant at the site. More detailed discussion of each geologic hazard considered, their anticipated impacts, and recommended mitigation measures are discussed below.

4.2 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Geological Survey (CDMG)/California Geologic Survey (CGS) (1972, 2000) produced 1:24,000 scale maps showing all known active faults and defining zones within which special fault studies are required. Based on currently available published geologic information, the project site is not located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2000) nor is within the City's General Plan Fault Rupture Hazard Zone. The potential for fault surface rupture on the project site is therefore considered to be low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.3 Seismic Shaking

The site will likely experience seismic ground shaking from future earthquakes in the San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 4, could cause strong ground shaking at the site.

Deterministic Seismic Hazard Analysis – Deterministic Seismic Hazard Analysis (DSHA) predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. A summary of the principal active faults affecting the site, their closest distance, moment magnitude of characteristic earthquake and probable peak ground accelerations (PGA), which an earthquake on the fault could generate at the site are shown in Table A.

TABLE A
DETERMINISTIC SEISMIC HAZARD ANALYSIS
Valle Verde and Heritage House Projects
Napa, California

Fault	Moment Magnitude[1]	Fault Distance [2]	Fault Mechanism	Median PGA[1,2,3]
West Napa	6.6	3.5 km	Strike Slip	0.40 g
Green Valley	6.8	8.7 km	Strike Slip	0.30 g
Great Valley	6.7	11.9 km	Strike Slip	0.28 g
Cordelia	6.5	13.2 km	Strike Slip	0.21 g
Rodgers Creek	7.3	21.7 km	Strike Slip	0.19 g

Notes:

1.) PGA – Peak Ground Acceleration
2.) Values determined using Caltrans ARS Online (web-based seismic acceleration spectra calculator tool), http://dap3.dot.ca.gov/ARS_Online/, accessed January 2019.
3.) Values determined using Vs^{30} = 270 m/s for Site Class "D" ("Stiff Soil" Conditions) in accordance with the 2016 CBC and 2010 ASCE-7.
4.) Campbell & Borognia (2008) and Choi & Youngs (2008)

Probabilistic Seismic Hazard Analysis – Probabilistic Seismic Hazard Analysis (PSHA) analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the PGA for two separate probabilistic conditions, the 2% chance of exceedance in 50 years (2,475-year statistical return period) and the 10% chance of exceedance in 50 years (475-year statistical return period), utilizing the USGS Unified Hazard Tool (USGS, 2008b). Deterministic methods, as discussed above, or the PGA arising from a probabilistic analysis for a 10% chance of exceedance in 50 years are commonly utilized for residential, commercial, and industrial developments, while the PGA arising from a probabilistic analysis for a 2% chance of exceedance in 50 years is typically used for "critical" facilities such as schools and hospitals. The results of the probabilistic analyses are presented below in Table B.

TABLE B
PROBABILISTIC SEISMIC HAZARD ANALYSES
Valle Verde and Heritage House Projects
Napa, California

	Statistical Return Period	Mean Moment Magnitude[1]	Peak Ground Acceleration (g)[1]
2% in 50 years	2,475 years	6.6	0.71 g
10% in 50 years	475 years	6.7	0.46 g

Notes:

1.) USGS Unified Hazard Tool, Dynamic Conterminous US (2008) model version 3.3.1, https://earthquake.usgs.gov/hazards/interactive, accessed January 2019.

The potential for strong seismic shaking at the project site is high. Due to its close proximity, the West Napa Fault presents the highest potential for severe ground shaking. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Minimum mitigation measures should include designing the structures and foundations in accordance with the most recent version (2016) of the California Building Code.*

4.4 Liquefaction Potential and Related Impacts

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with a high, 35 to 50%, fines content (soil particles that pass the #200 sieve), provided the fines exhibit a plasticity less than 7. Saturated granular layers were observed during our subsurface exploration. Additionally, regional mapping indicates the site lies on the border of a zone of “high liquefaction susceptibility”, as shown on Figure 6.

4.4.1 Liquefaction Evaluation

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation, known as the Cyclic Resistance Ratio (CRR). The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum considered earthquake peak ground acceleration (PGA) and depth. Soil resistance to liquefaction is based on its relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with the Standard Penetration Test (SPT) blow count data measured in the field and corrected for hammer efficiency, overburden and percent fines to determine the $(N_1)_{60,CS}$ value. Gravels encountered in-situ will artificially inflate the blow counts recorded in the field. Therefore, we "capped" the blow counts recorded in the field to 20 blows per foot (bpf) for SPT samples and 30 bpf for Modified California samples.

We analyzed the potential for liquefaction utilizing the data from our borings and the procedures outlined by Idriss and Boulanger (2008 & 2010), considering a magnitude 6.6 earthquake producing a PGA of 0.68-g, which corresponds to the PGA_M value as defined in ASCE 7-10 Section 11.8.3. The results of our liquefaction analyses, including post-liquefaction settlement predictions, are presented on Figures 7 and 8 for our 2018 borings and on Figures 9 and 10 for our 2010 borings and indicate several localized soil layers may liquefy under a strong seismic event. These layers were found at depths between about 10.0 and 15.0-feet, with a maximum layer thickness of roughly seven feet.

4.4.2 Post Liquefaction Settlement

We predicted the amount of post liquefaction settlement utilizing the procedures outlined by Idriss and Boulanger (2008 & 2010), which indicate post liquefaction settlement can occur in soils that exhibit a factor of safety against liquefaction of 2.0 or less. Based on our analyses, we predict up to about 2.5-inches of total settlement and about 1.0-inch of differential settlement over a horizontal distance of about 30-feet may occur during the design seismic event.

Additionally, we utilized the procedures outlined by Ozocak and Sert (2010) to calculate the Liquefaction Potential Index (LPI), which is a gauge to determine if liquefiable layers will impact the ground surface. LPI is a function of the thickness, depth, and factor of safety against liquefaction in the individual layers within a soil column. The resulting LPI value corresponds to a relative potential for surface deformation impacting the ground surface. Typically, an LPI value of zero indicates the liquefiable layer will not impact the ground surface; while a value less than 5 has a low probability, value between 5 and 15 have a moderate probability and an LPI value greater than 15 have a high probability of surface impact. The results of our liquefaction analyses indicate LPI values up to 7.4, suggesting a moderate probability of liquefaction effects (such as sand boils) at the ground surface.

Based on our calculations, as described above, it is our opinion that liquefaction and related settlement present a relatively low risk of significant damage to the planned improvements. We anticipate up to about 2.5 inches of post liquefaction settlement and one inch of differential settlement which may cause damage to brittle surfaces, door and window operations, and other issues, but is unlikely to result in building collapse. The project Structural Engineer should verify our opinion about structural collapse or other "significant" damage.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Foundation systems should be able to accommodate up to 2.5-inches of total settlement and about half of that in differential settlements over a horizontal distance of about 30 feet during the maximum anticipated ground shaking. Foundation design criteria to mitigate the effects of liquefaction induced differential settlement are provided in Section 5.4.*

4.5 Seismically Induced Ground Settlement

Seismic ground shaking can induce settlement of unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. We did not observe loose granular soil layers above the historic high groundwater table. Therefore, we judge seismically induced ground settlement is not considered a significant geologic hazard at the project site.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.6 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. These conditions do not exist at the site, therefore the risk of lurching and ground cracking at the project site is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.7 Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. These conditions, with the exception of the Salvador Creek channel, do not exist at the site. However, there is always some potential for localized erosion due to concentrated surface water flows.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Mitigation measures include designing a site drainage system to collect surface water and discharging it into an established storm drainage system. The project Civil Engineer is responsible for designing the site drainage system and, an erosion control plan could be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook (2003).*

4.8 Seiche and Tsunami

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon fault geometry, offset, and resultant ground motions associated with the source earthquake. The project site is not mapped in a tsunami inundation zone (ABAG, 2019). Therefore, the risk of inundation by seiche or tsunami at the site is judged to be low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are anticipated.*

4.9 Flooding

The site is located approximately 1/2-mile west of the Napa River and Salvador Creek flows along the northeastern edge of the property. The site is mapped on the border of a FEMA 100-year flood zone (ABAG, 2019) and shown on Figure 9. Therefore, the risk of damage due to large-scale flooding is moderate to high.

Evaluation: *Less than significant.*
Mitigation: *The project Civil Engineer or Architect should confirm the final site and finished floor elevations are above potential flood elevations. Careful consideration should be given to design of finished grades at the site to reduce the potential for small-scale flooding and localized ponding of water.*

4.10 Dam Failure Inundation

The site is not located proximal to any significant dams and is not mapped within a dam failure inundation zone per the Safety Element of the Napa County General Plan (County of Napa, 2007). Therefore, the risk of dam failure inundation at the site is judged low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.11 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs, and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, and uneven floors and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures. Based on the results of our plasticity index testing, the surficial soils at the project site are generally of low to medium plasticity. However, there appear to be some areas at the site that are blanketed with soil having moderate plasticity and moderate expansion potential. Therefore, the risk of expansive soil affecting the proposed improvements is low to moderate.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Although widespread expansive soils are not expected at or near the ground surface, if encountered, the soils should be prepared as described in the Site Grading section of this report. In addition, foundation design criteria, as outlined in this report, anticipates that some differential movement will occur due to expansive soil.*

4.12 Settlement/Subsidence

Significant settlement can occur when new loads are placed at sites due to consolidation of soft compressible clays (i.e., Bay Mud) or compression of loose granular soils. Significant deposits of soft compressible materials were not observed during our subsurface exploration. Therefore, the risk of long-term static settlement to the proposed structures at the project site is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.13 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. Instability may be manifested in many forms, ranging from “slope creep”, in which near-surface materials migrate slowly downslope over many years or decades, to slides and flows which transport debris downslope very rapidly.

The project site consists of relatively level terrain, with the exception of the Salvador Creek channel slope bordering the northeastern edge of the site. Based on our recent observations and evaluation of the creek channel slopes, it is apparent that active erosion is occurring along portions of the channel slope adjacent to the project site. Areas of active erosion include two sections of the creek channel adjacent to the existing asphalt paved driveway and parking area at 3700 Valle Verde Drive (Valle Verde project). One area of creek channel erosion extends for approximately 85 lineal feet and is located adjacent to the most northwesterly portion of the existing asphalt paved driveway. A second area of erosion extends for approximately 100 lineal feet adjacent to the most southeasterly portion of the existing paved driveway.

Erosion of the creek channel slope adjacent to portions of the site has resulted in over-steepened slope inclinations. In these areas, lateral creep or yielding of the channel slope has resulted in cracking, settlement, and lateral spreading of the asphalt paved driveway areas located near the top of the creek channel. Cracking and distress of the existing pavement surface extends back approximately 25 to 30 feet from the top of the slope.

In our opinion, unless remedial measures are taken, it is anticipated that additional yielding and lateral creep of the creek channel slope will occur in the future, which will result in additional settlement and cracking of the adjacent asphalt paved driveway surface over time.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Mitigation of the impact of creek channel slope instability and slope creep on adjacent improvements can be provided by design/construction of a stich pier retaining structure located at the existing curb, outside the creek channel.*

4.14 Radon-222 Gas

Radon-222 is a product of the radioactive decay of uranium-238 and raduim-226, which occur naturally in a variety of rock types, mainly phosphatic shales, but also in other igneous, metamorphic, and sedimentary rocks. While low levels of radon gas are common, very high levels, which are typically caused by a combination of poor ventilation and high concentrations of uranium and radium in the underlying geologic materials, can be hazardous to human health.

The project site is located in Napa County, California, which is mapped in radon gas Zone 3 by the United States Environmental Protection Agency (USEPA, 2017). Zone 3 is classified by the EPA as exhibiting a “low” potential for Radon-222 gas with average predicted indoor screening levels less than 2 pCi/L. Therefore, we judge the potential for hazardous levels of radon at the project site is low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.15 Volcanic Eruption

Several active volcanoes with the potential for future eruptions exist within northern California, including Mount Shasta, Lassen Peak, and Medicine Lake in extreme northern California, the Mono Lake-Long Valley Caldera complex in east-central California, and the Clear Lake Volcanic Field, located in Lake County approximately 71 miles north of the project site. The most recent volcanic eruption in northern California was at Lassen Peak in 1917, while the most recent eruption at the nearest volcanic center to the project site, the Clear Lake Volcanic Field, was about 10,000 years ago. All of northern California's volcanic centers are currently listed under "normal" volcanic alert levels by the USGS California Volcano Observatory (USGS, 2019). While the aforementioned volcanic centers are considered "active" by the USGS, the likelihood of damage to the proposed improvements due to volcanic eruption is generally low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.16 Naturally Occurring Asbestos (NOA)

Naturally occurring asbestos is commonly found in association with serpentinite and associated ultramafic rock types. These rocks are a major constituent of the Franciscan Complex, which underlies vast portions of the greater San Francisco Bay Area. The site is underlain by relatively thick native alluvial soils. Therefore, the likelihood that significant deposits of naturally-occurring asbestos will be encountered at the site is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.17 Hazardous Materials

The evaluation of hazardous materials is beyond the scope of our investigation. However, we observed nothing at the site or in our exploratory borings that indicated the presence of hazardous materials.

Evaluation: *Less than significant.*
Mitigation: *Evaluation and assessment of hazardous materials is beyond the scope of our current services. If contamination were discovered at the site, removal and replacement of affected soils could be considered, as could "capping" of any contamination with "clean" fill.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on our experience with similar projects in the Napa area, we conclude that, from a geotechnical standpoint, the site is suitable for the planned improvements. The primary geotechnical issues to address in design of the project include strong seismic ground shaking, moderately expansive surface soil, modest differential settlement due to potential liquefaction, and slope instability along portions of the Salvador Creek channel.

5.2 Seismic Design

The project site is located in a seismically active area. Therefore, new structures should be designed in conformance with the seismic provisions of the California Building Code (CBC) to mitigate the potential effects of strong seismic ground shaking to the proposed structures. Based on the weighted average of the blow counts obtained during our subsurface exploration, we judge Site Class "D" is appropriate for the purpose of project seismic design. At a minimum, we recommend the project Structural Engineer utilize the 2016 CBC coefficients shown in Table C below to determine the base shear values.

TABLE C
2016 CBC FACTORS
Valle Verde and Heritage House Projects
Napa, California

Factor Name	Coefficient	2016 CBC Site Specific Value
Site Class[1]	$S_{A,B,C,D,E,\text{ or }F}$	S_D
Site Coefficient	F_a	1.00
Site Coefficient	F_v	1.50
Spectral Acc. (short)	S_S	1.89 g
Spectral Acc. (1-sec)	S_1	0.68 g
Spectral Response (short)	SM_S	1.89 g
Spectral Response (1-sec)	SM_1	1.02 g
Design Spectral Response (short)	SD_S	1.26 g
Design Spectral Response (1-sec)	SD_1	0.68 g
MCE_G[2] PGA adjusted for Site Class	PGA_M	0.68 g
Seismic Design Category	A,B,C,D, or E	D

Notes:
1. Site Class D Description: Stiff soil profile with shear wave velocities between 600 and 1,200 ft/sec, standard blow counts between 15 and 50 blows per foot, and undrained shear strength between 1,000 and 2,000 psf.
2. Maximum Considered Earthquake Geometric Mean

5.3 Site Preparation and Grading

The general grading recommendations presented below are appropriate for construction in the late spring through fall months. From winter through the early spring months, on-site soils may be saturated due to rainfall and may be difficult to compact without drying by aeration or the addition of lime and/or cement (or a similar product) to dry the soils. Site preparation and grading should conform to the recommendations and criteria outlined below. General recommendations for wintertime construction are provided later in this report.

5.3.1 Surface Preparation

Clear all trees, brush, roots, over-sized debris, and organic material from areas to be graded. Trees that will be removed (in structural areas) must also include removal of stumps and roots larger than two inches in diameter. Excavated areas (i.e., excavations for stump removal) should be restored with properly moisture conditioned and compacted fill as described in the following sections. Any loose soil or rock at subgrade will need to be excavated to expose firm natural soils. Debris or rocks larger than four inches and vegetation are not suitable for structural fill and should be removed from the site. Alternatively, vegetation strippings may be used in landscape areas.

A former home, garage, and pool were located northwest of the existing Heritage House structure. The structures and pool were recently demolished, and the old footing and pool excavations were backfilled with undocumented fill. We recommend that all existing fill in the former house/pool area should be overexcavated to expose stiff native soil. The overexcavated areas can then be filled to planned grades with compacted, engineered fill placed as recommended below.

Where fills or other structural improvements are planned on level ground, the subgrade surface should be scarified to a depth of about eight inches, moisture conditioned to above the optimum moisture content, and compacted to a minimum of 90% relative compaction (ASTM D-1557). Relative compaction should be increased to a minimum of 95% where new asphalt pavements are planned. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop." If moderately plastic and/or expansive soils are encountered during construction, they should be moisture conditioned at least three percent over the optimum moisture content and compacted between 90 and 92 percent relative compaction.

5.3.2 Compacted Fill

On-site fill, backfill, and scarified subgrades should be conditioned to above the optimum moisture content. Properly moisture conditioned and cured on-site materials should subsequently be placed in loose horizontal lifts of 8 inches thick or less, and uniformly compacted to a minimum of 90% relative compaction.

5.3.3 Materials

Based on our laboratory testing, on site soil may be suitable for use as fill provided the soil is prepared as described above. If imported fill is required, the material should consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 15, and (3) have a maximum particle size of 4 inches. Any imported fill material needs to be tested to determine its suitability for use as fill material.

5.4 Foundation Design

As described above, minor differential ground movement due to liquefaction induced settlement and expansive soil seasonal shrink/swell are potential hazards at the project site. Therefore, project foundation design should provide for additional stiffness and the ability of foundations to span or cantilever over areas of non-uniform support. Practical alternative foundations include stiff shallow mat slabs or a stiff grid of interconnected shallow footings. Shallow foundation design criteria are shown in Table D.

TABLE D
SHALLOW RIGID FOUNDATION DESIGN CRITERIA
Valle Verde and Heritage House Projects
Napa, California

Concrete Mat Slab Foundation

Minimum thickness:	10 inches
Minimum embedment of perimeter footing:	24 inches
Modulus of subgrade reaction, k_s:	100 pci
Maximum unsupported interior span[1]:	16 feet
Maximum unsupported edge/corner span[1]:	8 feet
Base friction:	0.30

Shallow Grid Foundation

Minimum embedment below existing grade:	24-inches
Minimum width[2]:	
One-story:	12 inches
Two-story:	15-inches
Three-story:	18-inches
Allowable bearing pressure:	
Dead plus live loads:	1,800 psf
Total including wind and seismic	2,400 psf
Base friction coefficient:	0.30
Lateral passive resistance[3,4]:	300 pcf
Maximum unsupported interior span[1]:	16 feet
Maximum unsupported edge/corner span[1]:	8 feet

Notes:
1.) Assumes rigid slab behavior with idealized fixed conditions.
2.) Design shallow foundations to similar bearing pressures, i.e., size footing widths to maintain uniform bearing loads. Maintain above optimum moisture content until concrete slabs are completed.
3.) May increase design values by 1/3 for total design loads including wind and seismic.
4.) Neglect upper 12-inches unless confined by concrete. Equivalent Fluid Pressure, not to exceed 3,000 psf.

If the predicted post-liquefaction settlement is unacceptable, the structures may be supported on deep foundation systems. Suitable deep foundation options include driven piles, torque down piles, etc. We can provide additional deep foundation design criteria for the chosen system upon request.

5.5 Site and Foundation Drainage

The site is relatively flat and there is a possibility that new grading could result in adverse drainage patterns and water ponding around buildings. Careful consideration should therefore be given to design of finished grades at the site. We recommend that landscaped areas adjoining new structures be sloped downward at least 0.25 feet for 5 feet (5%) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2%). Roof gutter downspouts may discharge onto the pavements but should not discharge onto any landscaped areas. Provide area drains for landscape planters adjacent to buildings and parking areas and collect downspout discharges into a non-perforated pipe collection system.

5.6 Concrete Slabs-On-Grade

Interior concrete slab-on-grade floors should be underlain by a minimum 4-inch thick layer of clean, free draining, 3/4-inch angular gravel or crushed rock. The clean angular gravel is placed beneath the interior concrete slabs to form a capillary moisture break. This rock must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15 mils or thicker, should be placed over the compacted base rock. The vapor barrier shall meet the ASTM E 1745 Class A requirements and be installed per ASTM 1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth, or other adverse conditions.

Where exterior concrete slabs are needed, we recommend they be at least 5-inches thick and reinforced with steel bars (not wire mesh). Additionally, contraction joints should be incorporated in the concrete slab in both directions, no greater than 10 feet on center and the reinforcing bars should extend through these control joints. Exterior concrete slabs should be underlain with at least 4-inches of Caltrans Class 2 Aggregate Base compacted to at least 92% relative compaction. For improved performance and in areas with expansive soil at the subgrade level, exterior slabs may be underlain by 12 inches or more of compacted Class 2 Aggregate Base.

5.7 Asphalt Pavements

Typically, asphalt pavement sections are designed utilizing two variables, the R-Value (a measure of the subgrade resistance) and the Traffic Index (a measure of the amount of daily traffic). Based on the subsurface conditions we judge an R-Value of 10 is appropriate for the site. The R-value of the pavement subgrade should be confirmed during construction when the subgrade soil is exposed. We have calculated pavement sections for the project site in accordance with Caltrans procedures for flexible pavement design utilizing the values described above and various Traffic Index (T.I.) values. The resulting supplemental pavement sections are presented in Table E below.

TABLE E
ASPHALT PAVEMENT SECTION
Valle Verde and Heritage House Projects
Napa, California

T.I.	Asphalt Concrete	Aggregate Base Rock
4.0	2.5 inches	7.0 inches
5.0	3.0 inches	9.0 inches
6.0	3.5 inches	11.5 inches

The upper 8 inches of subgrade in pavement areas shall be scarified, moisture conditioned to near the optimum moisture content, and then compacted to at least 95% relative compaction. The compacted subgrade must also be firm and unyielding when proof-rolled with heavy construction equipment.

The aggregate baserock should conform to Caltrans Class 2 Aggregate Baserock (Class 2 AB) outlined in Section 26 of the latest edition of the Caltrans Standard Specifications (2015). The Class 2 AB shall be placed in layers on a properly prepared, firm and unyielding subgrade as described in the previously discussed grading recommendations. The Class 2 AB should be compacted to at least 95% relative compaction. Additionally, the Class 2 AB section should be firm and unyielding under heavy construction equipment.

5.8 Utility Trench Excavations and Backfills

Excavations for utilities will most likely extend into stiff clayey soils. Trench excavations having a depth of five feet or more that will be entered by workers must be sloped, braced, or shored in accordance with current Cal/OSHA regulations. On-site soils appear to be Type C. All excavations where collapse of excavation sidewall, slope or bottom could result in injury or death of workers, should be evaluated by the contractor's safety officer and designated competent person prior to entering in accordance with current Cal/OSHA regulations.

Bedding materials for utility pipes should be well graded sand with 90 to 100% of particles passing the No. 4 sieve and no more than 5% finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically 3 to 6 inches. Trench backfill may consist of on-site soils, moisture conditioned to within 2% of the optimum moisture content, placed in thin lifts and compacted to a minimum of 90% relative compaction. Backfill for trenches within pavement areas should consist of non-expansive granular fill. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits. Where utility lines cross under or through perimeter footings, they should be sealed to reduce moisture intrusion into the areas under the slabs and/or footings.

5.9 Wintertime Construction

Wintertime/wet weather site work is feasible during the construction phase of this project provided weather conditions do not adversely impact the planned grading, and proper erosion control measures are implemented to prevent excessive silt and mud from entering the storm drain system. High soil moisture contents and muddy site conditions may impact placing fills, compacting subgrades, and excavating foundation trenches. Several alternatives may be considered to improve the site conditions to allow site work to proceed in rainy conditions:

- Prior to the onset of winter rains, maintain a drier site by covering the work area and any stockpiled materials with plastic visqueen sheeting or other impermeable membrane. Where asphalt pavements, other hardscape or drainage improvements currently exist in work areas, consider leaving these improvements in place until the last possible moment to maintain a drier subgrade condition.
- Lime/cement treat the subgrade soils when site work commences to “weatherproof” the site. The disadvantage to this alternative is that future landscaping will likely require excavation and replacement of the treated soils for acceptable plant growth.
- Finally, imported, drier fill materials could be used to stabilize the site. Soft or wet on-site materials could be excavated to firm materials and drier (preferably granular) soils with good drainage characteristics would be imported to restore site grades. This alternative might also require future excavation and replacement of landscaping soils.

If construction occurs relatively early in the winter, we judge the first option (covering the site prior to winter rains) could be an effective method of maintaining a workable site. When the construction schedule and weather conditions are known, we can meet with the project team to further discuss alternatives to continuing wintertime construction.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the plans and specifications for the project when they are nearing completion to confirm that the intent of our geotechnical recommendations has been incorporated and provide supplemental recommendations, if needed. During construction, we must observe and test site grading, foundation excavations for the structures and associated improvements to confirm that the soil conditions encountered during construction are consistent with the design criteria.

7.0 LIMITATIONS

This report has been prepared, in our opinion, in conformance with the generally accepted geotechnical engineering practices in the San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of Burbank Housing Development Corporation and/or its assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soils in this geographic area.

Our approved scope of work did not include an environmental assessment of the site. Consequently, this report does not contain information regarding the presence or absence of toxic or hazardous wastes.

The evaluations and recommendations contained in this report do not reflect variations in subsurface conditions that may exist between boring locations or in unexplored portions of the site. Should such variations become apparent during construction, the general recommendations contained within this report will not be considered valid unless MPEG is given the opportunity to review such variations and revise or modify our recommendations accordingly. No changes may be made to the general recommendations contained herein without the written consent of MPEG.

We recommend that this report, in its entirety, be made available to project team members, contractors, and subcontractors for informational purposes and discussion. We intend that the information presented within this report be interpreted only within the context of the report as a whole. No portion of this report should be separated from the rest of the information presented herein. No single portion of this report shall be considered valid unless it is presented with and as an integral part of the entire report.

8.0 LIST OF REFERENCES

American Society of Civil Engineers, “Minimum Design Loads for Buildings and Other Structures,” (ASCE 7), 2010.

American Society for Testing and Materials, “2013 Annual book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics,” ASTM, Philadelphia, 2013.

Association of Bay Area Governments (ABAG), Geographic Information System, http://quake.abag.ca.gov/mitigation/, 2019.

California Building Code, 2016 Edition, California Building Standards Commission, Sacramento, California.

California Department of Conservation, California Geologic Survey, “Official Tsunami Inundation Maps” www.conservation.ca.gov/cgs/geologic_hazards/Tsunami/Inundation_Maps#Interactive, accessed 2019.

California Department of Conservation, California Geologic Survey, “Digital Images of the Alquist Priolo Maps” http://www.conservation.ca.gov/cgs/rghm/ap/Pages/Index.aspx, 2019.

California Department of Transportation (Caltrans), “Acceleration Response Spectra (ARS) Online version 2.3.06” http://dap3.dot.ca.gov/ARS_Online/index.php, 2019.

California Department of Transportation (Caltrans), “Corrosion Guidelines Version 1.0,” September 2010.

California Department of Transportation (Caltrans), “Standard Specifications,” 2010.

California Division of Mines and Geology, Special Publication 42, “Alquist-Priolo Special Studies Zone Act,” 1972 (Revised 1988).

Campbell, K., Bozorgnia, Y., “NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s,” EERI Earthquakes Spectra, Volume 24, Number 1, February 2008.

Chiou, B. and Youngs, R., “An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra,” EERI Earthquakes Spectra, Volume 24, Number 1, February 2008.

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

Idriss, I.M. & Boulanger, R.W. “Soil Liquefaction during Earthquakes”, Earthquake Engineering Research Institute Monograph 12, 2008.

Southern California Earthquake Center (SCEC), “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California,” University of Southern California, March 1999.

Shahi, K.S. & Baker, J.W., “NGA Models for Ground Motion Directionality” May 2013.

United States Geologic Survey (USGS), “Documentation for the 2008 Update of the United States National Seismic Hazard Maps,” Open-File-Report 2008-1128, 2008.

United States Geological Survey & SC/EC, “The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2), 2007 Working Group on California Earthquake Probabilities, USGS Open File Report 2007-1437, CGS Special Report 203, SCEC Contribution #1138, 2008.

United States Geological Survey (USGS), “Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2031 – A Summary of Finding,” The Working Group on California Earthquake Probabilities, Open File Report 99-517, 2003.

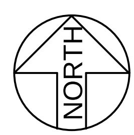
United States Geological Survey (USGS) “Earthquake Hazards Program, Unified Hazard Tool,” https://earthquake.usgs.gov/hazards/interactive/, 2019.

United States Geological Survey (USGS), “U.S. Seismic Design Maps”, web application http://geohazards.usgs.gov/designmaps/us/application.php, 2019.

SITE: LATITUDE, 38.3283°
LONGITUDE, -122.2935°

SITE LOCATION

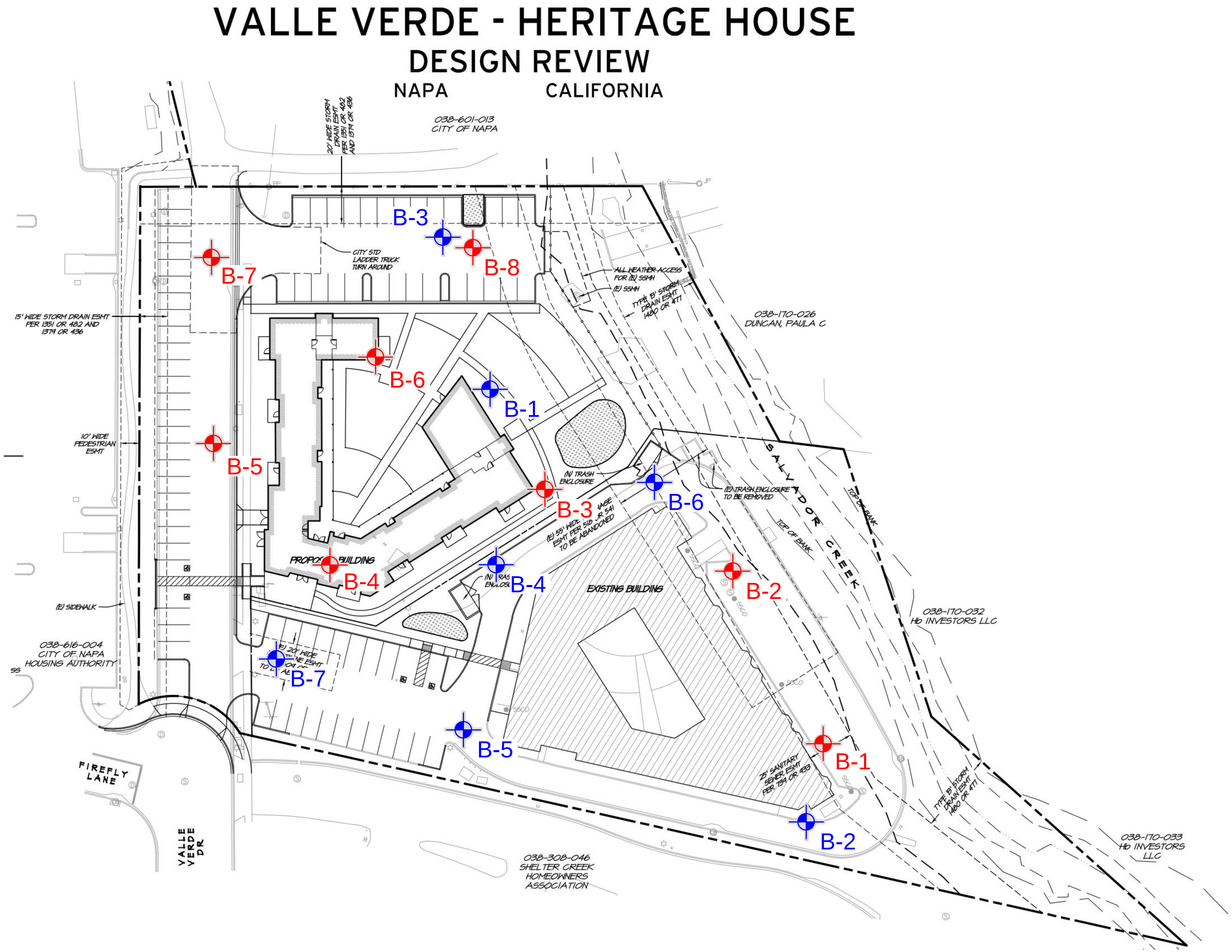
N.T.S.



REFERENCE: Google Earth, 2018

MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE LOCATION MAP Valle Verde Development Napa, California Project No. 1687.01 Date: 10/16/2018	Drawn MMT Checked	1 FIGURE

FILENAME: 1687.010 Figures.dwg



Approximate location of boring completed by MPEG, 2018

Approximate location of boring completed by MPEG, 2010

SCALE

0 30 60 120 FEET

NORTH

REFERENCE: RSA+ Consulting Civil Engineers, Valle Verde - Heritage House Cover Sheet, Sheet DR1.0, April 3, 2018.

MPEG MILLER PACIFIC ENGINEERING GROUP

FILE: 1687.010 Figures.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

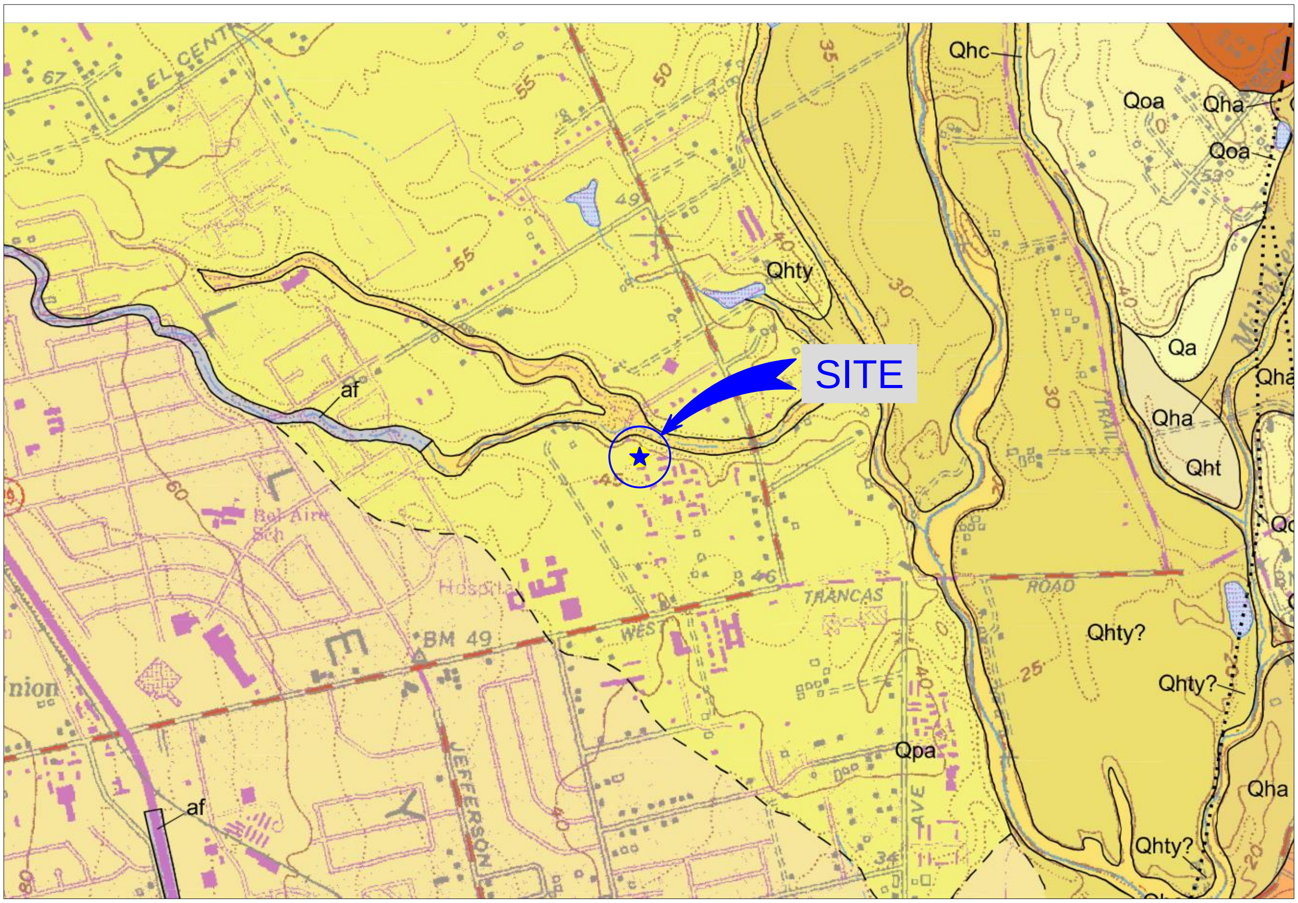
SITE PLAN

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Designed DSC
Drawn MMT
Checked DSC

2
FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

LEGEND

Qpa	Alluvium, undivided (latest Pleistocene): Alluvial fan, stream, terrace, basin and channel deposits, composed of poorly to moderately sorted sand, silt clay and gravel.
af	Artificial Fill (Holocene, historic): May be engineered and/or non-engineered.
Qhty	Stream Terrace Deposits (Latest Holocene): Stream terraces deposited as point bar and overbank deposits along the Napa River.
Qhf2	Alluvial Fan Deposits (Holocene): Alluvial fan sediment deposited by streams emanating from mountain drainages onto alluvial valleys.

REFERENCE: Kevin B. Clahan, David L. Wagner, George J. Saucedo, Carolyn E. Randolph-Loar, and Janet M. Sowers, "Geologic Map of the Napa 7.5' Quadrangle, Napa County, California - A Digital Database, Version 1.0", CGS, 2004, Map Scale 1:24,000.

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.010 Figures.dwg

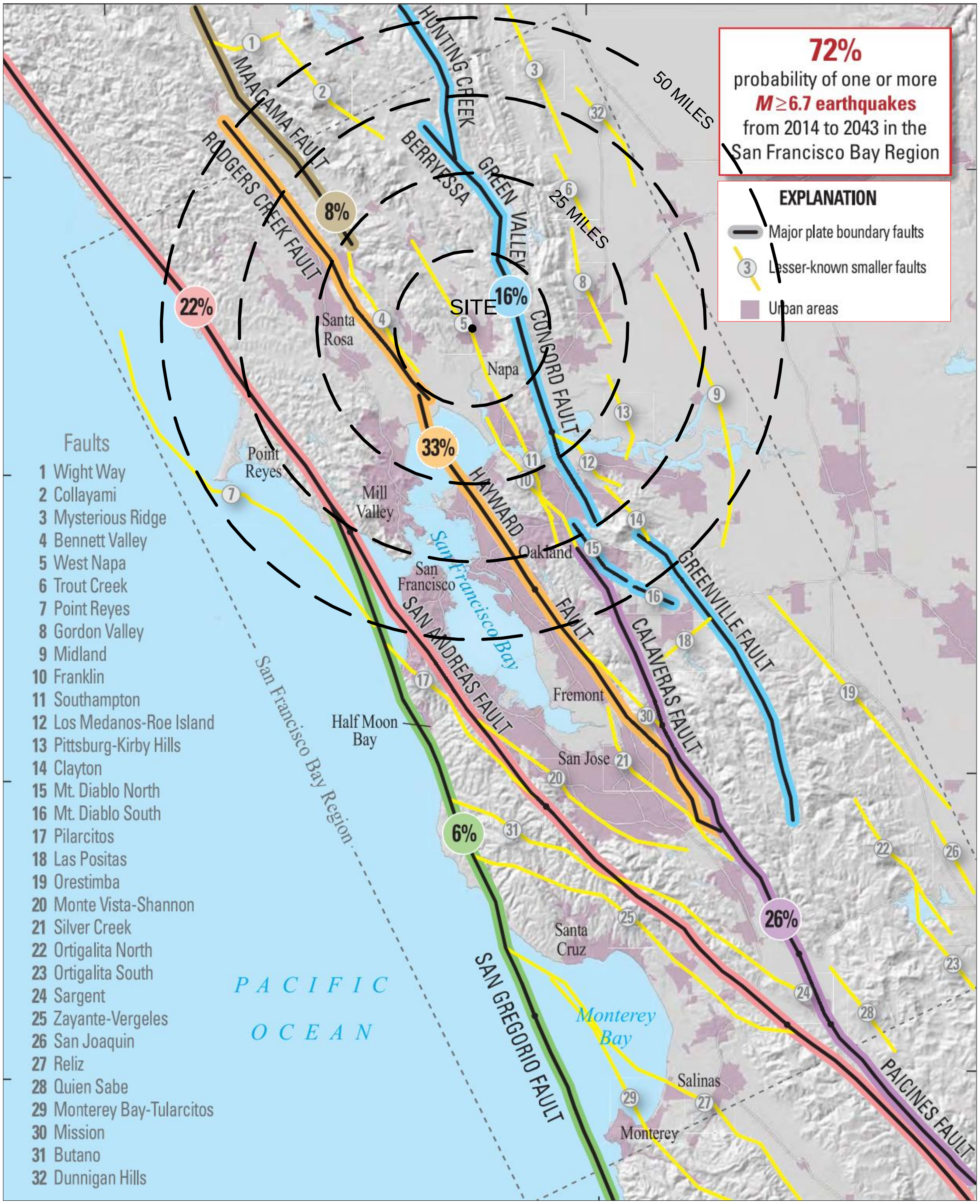
REGIONAL GEOLOGIC MAP

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn MMT
Checked

FIGURE 3



SITE COORDINATES
LAT. 38.3283°
LON. -122.2935°

SCALE
0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

MPEG MILLER PACIFIC ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.010 Figures.dwg

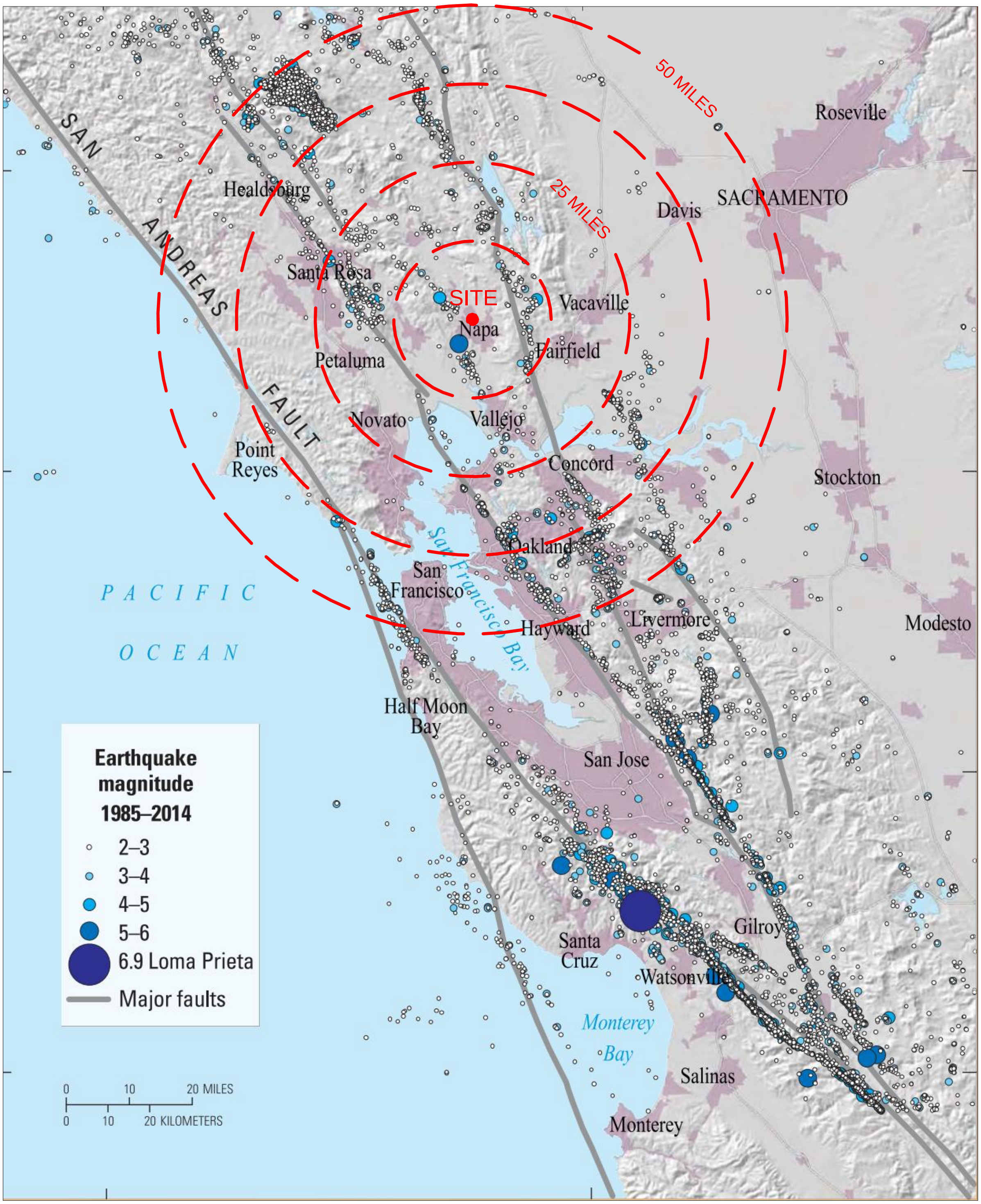
ACTIVE FAULT MAP

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn MMT
Checked

4
FIGURE



SITE COORDINATES
LAT. 38.3283°
LON. -122.2935°

SCALE
0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Earthquakes Greater Than Magnitude 2.0 in the San Francisco Bay Region from 1985-2014, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.010 Figures.dwg

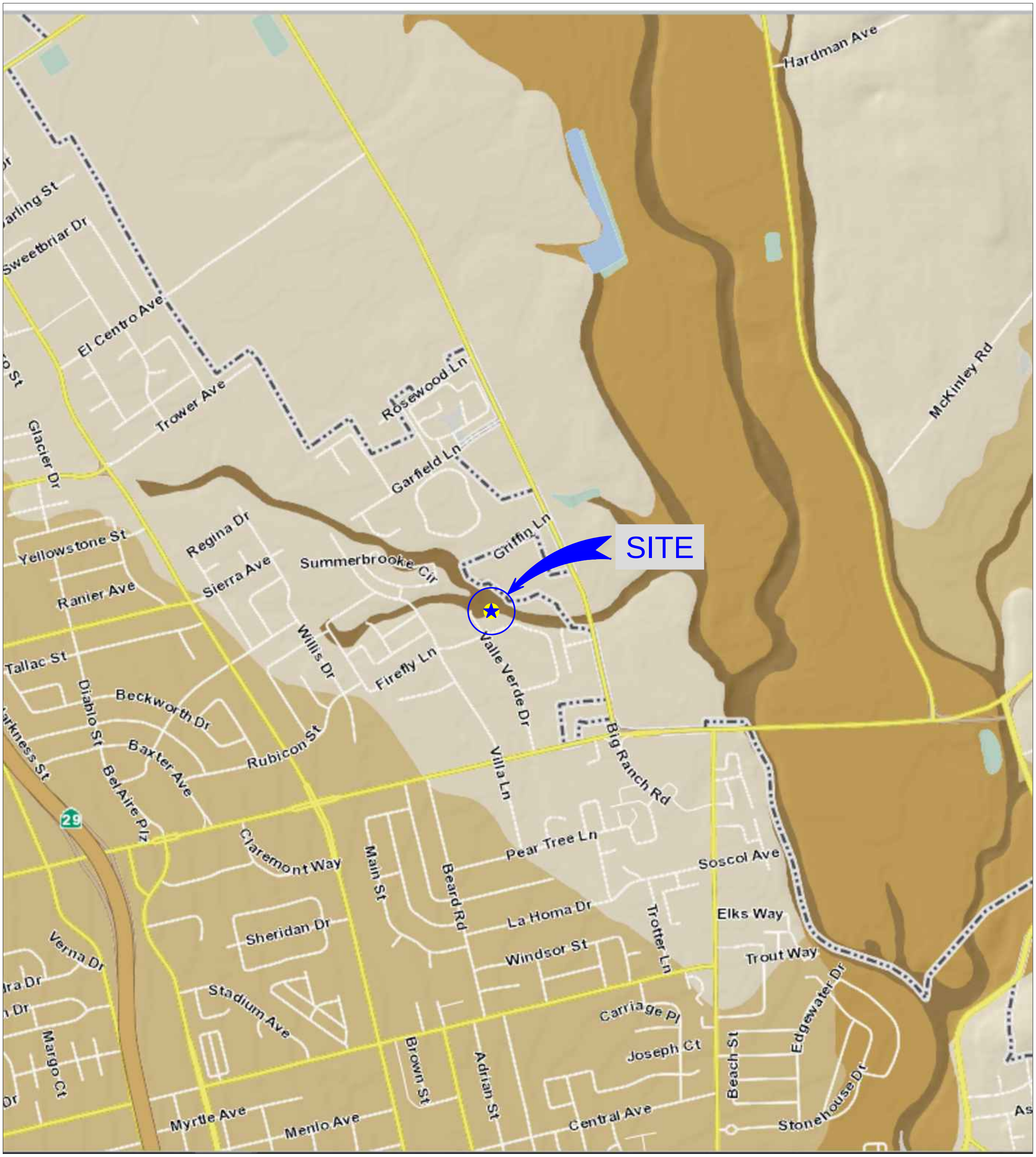
HISTORIC EARTHQUAKE ACTIVITY

Valle Verde Development
Napa, California

Drawn MMT
Checked

5
FIGURE

Project No. 1687.01 Date: 10/16/2018



Susceptibility Level:

No Scale

- Very High
- High
- Moderate
- Low
- Very Low
- Local Road
- Major Road

NORTH

Map Reference: ABAG Geographic Information System.

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 1687.010 Figures.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

LIQUEFACTION SUSCEPTIBILITY MAP

Valle Verde Development
Napa, California

Drawn MMT
Checked

Project No. 1687.01 Date: 10/16/2018

6
FIGURE

Liquefaction Analysis Utilizing
Standard Penetration Test Data

Project:	Valle Verde and Heritage House Projects	Analyzed by:	MMT
Job No:	1687.010	Date:	1/15/2019

Soil & Seismic Properties

Magnitude, M_w: 6.6
PGA, g: 0.68

Layer	UCS Type	γ_d, pcf	ω%	Depth, ft Upper	Depth, ft Lower	γ_t, pcf	Thickness, ft
1	CL	120	8%	0	4	130	4
2	GC	115	15%	4	20	132	16
3	CL	110	15%	20	30	127	10
4	GC	105	15%	30	40	121	10
5	CH	100	15%	40	60	115	20

GW Depth: 5.0 feet

Cyclic Resistance Ratio (CRR)

MC to SPT: 0.6
Borehole Dia: 6.0
Hammer Eff: 0.8

Sample	Depth, ft	N_{FIELD}, bpf	Fines Cont., %	Sampler Type	Liner Correction	UCS Type	σ, psf	σ', psf	Sampler Corr.	N_{COR}, bpf
B1	10.0	10	18	MC	NO	GC	1312	1000	0.6	6.0
B1	15.5	10	9	SPT	NO	GC	2039	1384	1.0	10.0
B1	19.0	19	47	SPT	NO	GC	2502	1629	1.0	19.0
B4	5.0	21	43	MC	NO	GC	651	651	0.6	12.6
B4	10.0	16	21	MC	NO	GC	1312	1000	0.6	9.6
B5	5.0	27	30	MC	NO	GC	651	651	0.6	16.2
B6	6.0	19	12	MC	NO	GC	783	721	0.6	11.4

Sample	Depth (ft)	C_E	C_B	C_R	C_S	C_N	N_{60}, bpf	$(N_1)_{60}$, bpf	$\Delta(N_1)_{60}$, bpf	$(N_1)_{60CS}$, bpf	$CRR_{M=7.5,\sigma'=1}$
B1	10.0	1.33	1.05	0.80	1.00	1.42	8.0	9.5	4.1	14	0.14
B1	15.5	1.33	1.05	0.85	1.00	1.20	13.3	14.2	0.7	15	0.16
B1	19.0	1.33	1.05	0.85	1.00	1.08	25.3	24.4	5.6	30	0.48
B4	5.0	1.33	1.05	0.75	1.00	1.55	16.8	20.5	5.6	26	0.32
B4	10.0	1.33	1.05	0.80	1.00	1.36	12.8	14.7	4.6	19	0.20
B5	5.0	1.33	1.05	0.75	1.00	1.49	21.6	25.4	5.4	31	0.54
B6	6.0	1.33	1.05	0.75	1.00	1.56	15.2	18.6	2.3	21	0.22

Cyclic Stress Ratio Induced by Earthquake (CSR)

Sample	Depth, ft	σ, psf	σ', psf	r_d	C_σ	K_σ	MSF	$CSR_{M,\sigma'}$	$CSR_{M=7.5,\sigma'=1}$	F.S.
B1	10	1312	1000	0.97	0.11	1.07	1.27	0.56	0.41	0.35
B1	15.5	2039	1384	0.94	0.11	1.04	1.27	0.61	0.46	0.34
B1	19	2502	1629	0.92	0.20	1.04	1.27	0.63	0.47	1.02
B4	5	651	651	0.99	0.17	1.10	1.27	0.44	0.31	1.02
B4	10	1312	1000	0.97	0.13	1.09	1.27	0.56	0.41	0.49
B5	5	651	651	0.99	0.21	1.10	1.27	0.44	0.31	1.72
B6	6	783	721	0.99	0.14	1.10	1.27	0.47	0.34	0.64

Liquefaction Potential Index (LPI)

Boring	Depth (ft)	Depth (m)	F.S.	Fi	Thickness (ft)	Thickness (m)	w(i)	LPI(i)
B1	10	3.05	0.35	0.65	2	0.61	8.48	3.4
B1	15.5	4.72	0.34	0.66	3	0.91	7.64	4.6
B1	19	5.79	1.02	0.00	7	2.13	7.10	0.0
B4	5	1.52	1.02	0.00	6	1.83	9.24	0.0
B4	10	3.05	0.49	0.51	6	1.83	8.48	8.0
B5	5	1.52	1.72	0.00	2	0.61	9.24	0.0
B6	6	1.83	0.64	0.36	4	1.22	9.09	4.0

Boring	LPI
B1	8.0
B4	8.0
B5	0.0
B6	4.0

Liquefaction Severity asa function of Liquefaction Potential Index (LPI)

Severity	Little to none	Minor	Moderate	Major
LPI	LPI = 0	0 < LPI < 5	5 < LPI < 15	15 < LPI

Liquefaction damage appears typically when LPI > 5

Reference: Dixit, Dewaikar, & Jangid 2012 and Ozocak & Sert 2010

http://www.nat-hazards-earth-syst-sci.net/12/2759/2012/nhess-12-2759-2012.pdf

http://www.geoengineer.org/onlinelibrary/CPT10/3-30ozoeol.pdf

Post Liquefaction Settlement Analysis

Sample	Depth, ft	$(N_1)_{60CS}$, bpf	F.S.	γ_{LIM}	F_α	γ_{MAX}	ε_v	Thickness, ft	Settlement, in
B1	10	14	0.35	0.32	0.81	0.32	3.1%	2.0	0.7
B1	15.5	15	0.34	0.28	0.76	0.28	2.9%	3.0	1.0
B1	19	30	1.02	0.05	-0.09	0.03	0.7%	7.0	0.6
B4	5	26	1.02	0.08	0.16	0.03	0.8%	6.0	0.6
B4	10	19	0.49	0.17	0.55	0.17	2.4%	6.0	1.7
B5	5	31	1.72	0.04	-0.14	0.01	0.1%	2.0	0.0
B6	6	21	0.64	0.14	0.47	0.14	2.2%	4.0	1.1

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 1687.010 Figures.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

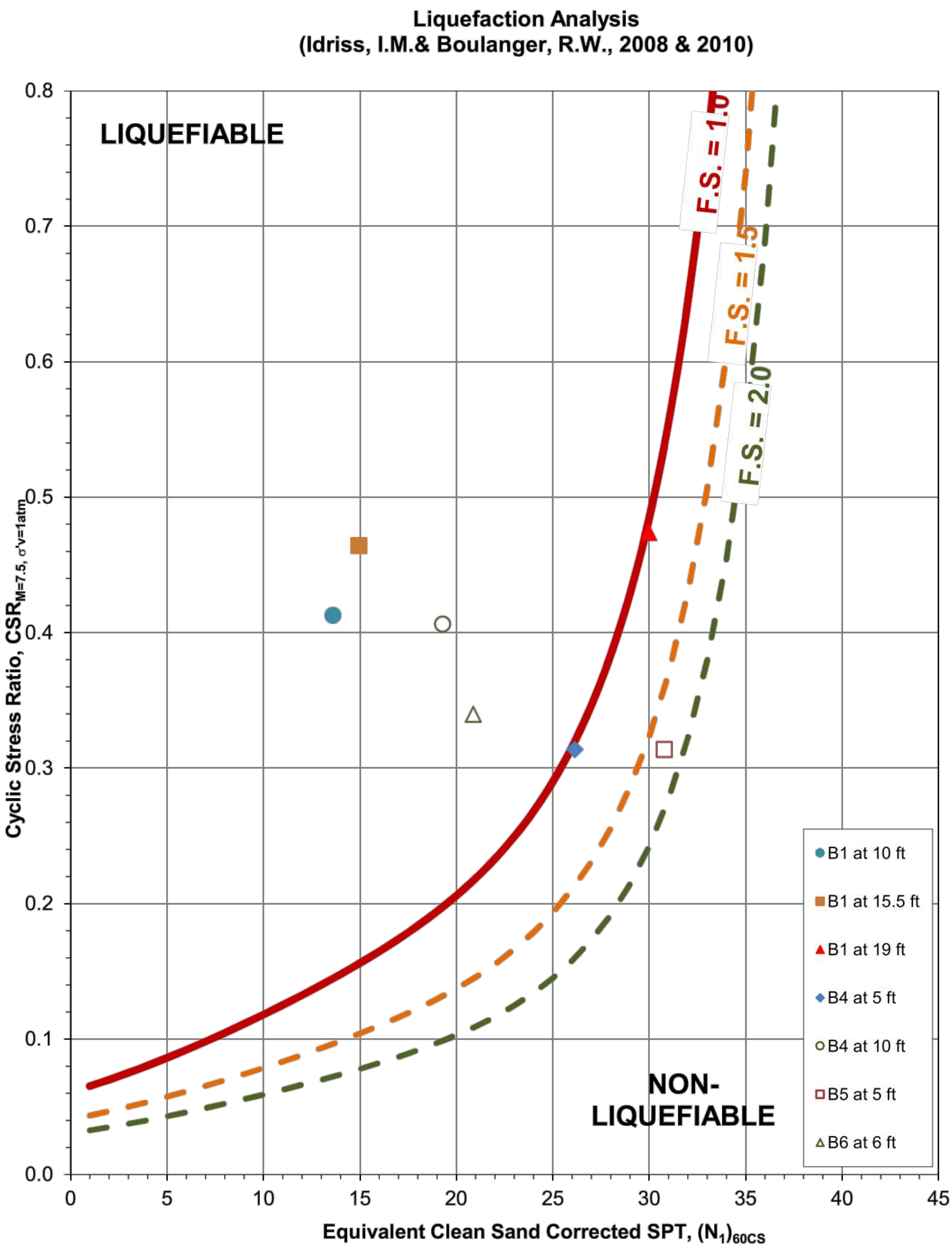
2018 BORINGS LIQUEFACTION ANALYSIS

Valle Verde Development
Napa, California

Drawn MMT
Checked

7
FIGURE

Project No. 1687.01 Date: 10/16/2018



MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	2018 BORINGS LIQUEFACTION ANALYSIS RESULTS Valle Verde Development Napa, California Project No. 1687.01 Date: 10/16/2018	Drawn MMT Checked 8 FIGURE

FILENAME: 1687.010 Figures.dwg

Liquefaction Analysis Utilizing
Standard Penetration Test Data

Project:	Valle Verde and Heritage House Projects	Analyzed by:	MMT
Job No:	1687.010	Date:	1/15/2019

Soil & Seismic Properties

Magnitude, M_w: 6.6
PGA, g: 0.68

Layer	UCS Type	γ_d, pcf	ω%	Depth, ft Upper	Depth, ft Lower	γ_t, pcf	Thickness, ft
1	CL	120	8%	0	4	130	4
2	SC	115	15%	4	20	132	16
3	CL	110	15%	20	30	127	10
4	SC	105	15%	30	40	121	10
5	CH	100	15%	40	60	115	20

GW Depth: 5.0 feet

Cyclic Resistance Ratio (CRR)

MC to SPT: 0.6
Borehole Dia: 6.0
Hammer Eff: 0.8

Sample	Depth, ft	N_{FIELD}, bpf	Fines Cont., %	Sampler Type	Liner Correction	UCS Type	σ, psf	σ', psf	Sampler Corr.	N_{COR}, bpf
B2	8.0	20	15	MC	NO	SC	1047	860	0.6	12.0
B3	15.0	12	22	SPT	NO	SC	1973	1349	1.0	12.0
B4	15.0	23	15	SPT	NO	SC	1973	1349	1.0	23.0
B5	7.0	23	15	MC	NO	SC	915	790	0.6	13.8
B6	10.0	13	21	MC	NO	SC	1312	1000	0.6	7.8
B7	5.0	30	15	MC	NO	SC	651	651	0.6	18.0
B7	8.5	55	15	MC	NO	SC	1114	895	0.6	33.0

Sample	Depth (ft)	C_E	C_B	C_R	C_S	C_N	N_{60}, bpf	$(N_1)_{60}$, bpf	$\Delta(N_1)_{60}$, bpf	$(N_1)_{60CS}$, bpf	$CRR_{M=7.5,\sigma'=1}$
B2	8.0	1.33	1.05	0.75	1.00	1.44	16.0	18.1	3.3	21	0.22
B3	15.0	1.33	1.05	0.85	1.00	1.18	16.0	16.9	4.8	22	0.23
B4	15.0	1.33	1.05	0.85	1.00	1.14	30.7	31.2	3.3	34	0.99
B5	7.0	1.33	1.05	0.75	1.00	1.46	18.4	21.1	3.3	24	0.28
B6	10.0	1.33	1.05	0.80	1.00	1.39	10.4	12.1	4.6	17	0.17
B7	5.0	1.33	1.05	0.75	1.00	1.49	24.0	28.1	3.3	31	0.59
B7	8.5	1.33	1.05	0.75	1.00	1.24	44.0	42.8	3.3	46	53.54

Cyclic Stress Ratio Induced by Earthquake (CSR)

Sample	Depth, ft	σ, psf	σ', psf	r_d	C_σ	K_σ	MSF	$CSR_{M,\sigma'}$	$CSR_{M=7.5,\sigma'=1}$	F.S.
B2	8	1047	860	0.98	0.14	1.10	1.27	0.53	0.38	0.59
B3	15	1973	1349	0.94	0.14	1.06	1.27	0.61	0.46	0.50
B4	15	1973	1349	0.94	0.25	1.10	1.27	0.61	0.44	2.28
B5	7	915	790	0.98	0.16	1.10	1.27	0.50	0.36	0.77
B6	10	1312	1000	0.97	0.12	1.08	1.27	0.56	0.41	0.42
B7	5	651	651	0.99	0.22	1.10	1.27	0.44	0.31	1.87
B7	8.5	1114	895	0.97	0.30	1.10	1.27	0.54	0.38	3.00+

Liquefaction Potential Index (LPI)

Boring	Depth (ft)	Depth (m)	F.S.	Fi	Thickness (ft)	Thickness (m)	w(i)	LPI(i)
B2	8	2.44	0.59	0.41	6	1.83	8.78	6.5
B3	15	4.57	0.50	0.50	3	0.91	7.71	3.5
B4	15	4.57	2.28	0.00	3	0.91	7.71	0.0
B5	7	2.13	0.77	0.23	3	0.91	8.93	1.9
B6	10	3.05	0.42	0.58	5	1.52	8.48	7.5
B7	5	1.52	1.87	0.00	3	0.91	9.24	0.0
B7	8.5	2.59	3.00+	0.00	3	0.91	8.70	0.0

Boring	LPI
B2	6.5
B3	3.5
B4	0.0
B5	1.9
B6	7.5
B7	0.0

Liquefaction Severity asa function of Liquefaction Potential Index (LPI)

Severity	Little to none	Minor	Moderate	Major
LPI	LPI = 0	0 < LPI < 5	5 < LPI < 15	15 < LPI

Liquefaction damage appears typically when LPI > 5

Reference: Dixit, Dewaikar, & Jangid 2012 and Ozocak & Sert 2010

http://www.nat-hazards-earth-syst-sci.net/12/2759/2012/nhess-12-2759-2012.pdf

http://www.geoengineer.org/onlinelibrary/CPT10/3-30ozoeol.pdf

Post Liquefaction Settlement Analysis

Sample	Depth, ft	$(N_1)_{60CS}$, bpf	F.S.	γ_{LIM}	F_α	γ_{MAX}	ε_v	Thickness, ft	Settlement, in
B2	8	21	0.59	0.14	0.44	0.14	2.2%	6.0	1.6
B3	15	22	0.50	0.13	0.43	0.13	2.2%	3.0	0.8
B4	15	34	2.28	0.02	-0.40	0.00	0.0%	3.0	0.0
B5	7	24	0.77	0.10	0.27	0.06	1.5%	3.0	0.6
B6	10	17	0.42	0.23	0.68	0.23	2.7%	5.0	1.6
B7	5	31	1.87	0.04	-0.18	0.00	0.1%	3.0	0.0
B7	8.5	46	3.00+	0.00	-1.27	0.00	0.0%	3.0	0.0

MPEG
MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.010 Figures.dwg

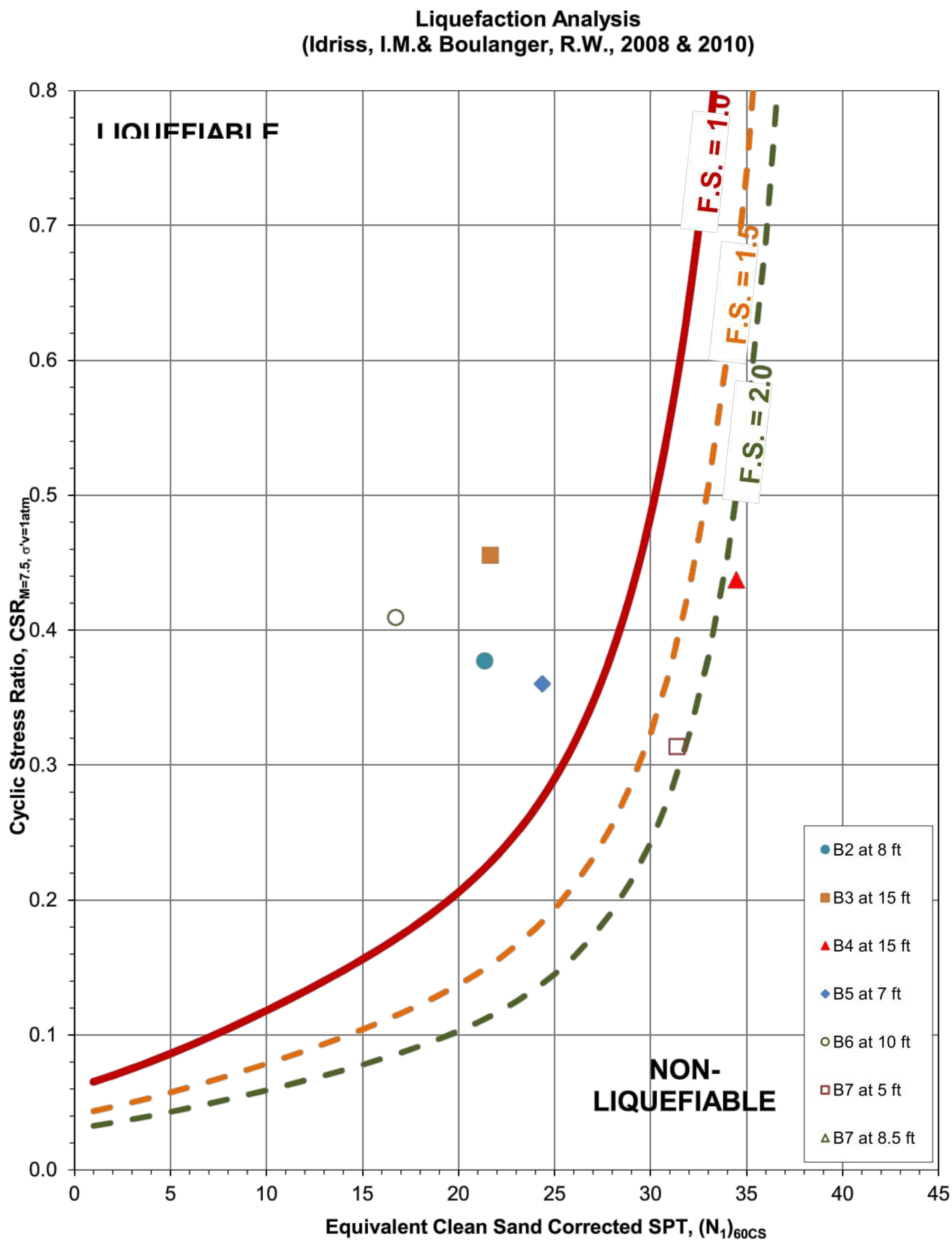
2010 BORINGS LIQUEFACTION ANALYSIS

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn MMT
Checked

9
FIGURE



MILLER PACIFIC ENGINEERING GROUP

FILENAME: 1687.010 Figures.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

2010 BORINGS LIQUEFACTION ANALYSIS RESULTS

Valle Verde Development
Napa, California

Project No. 1687.01

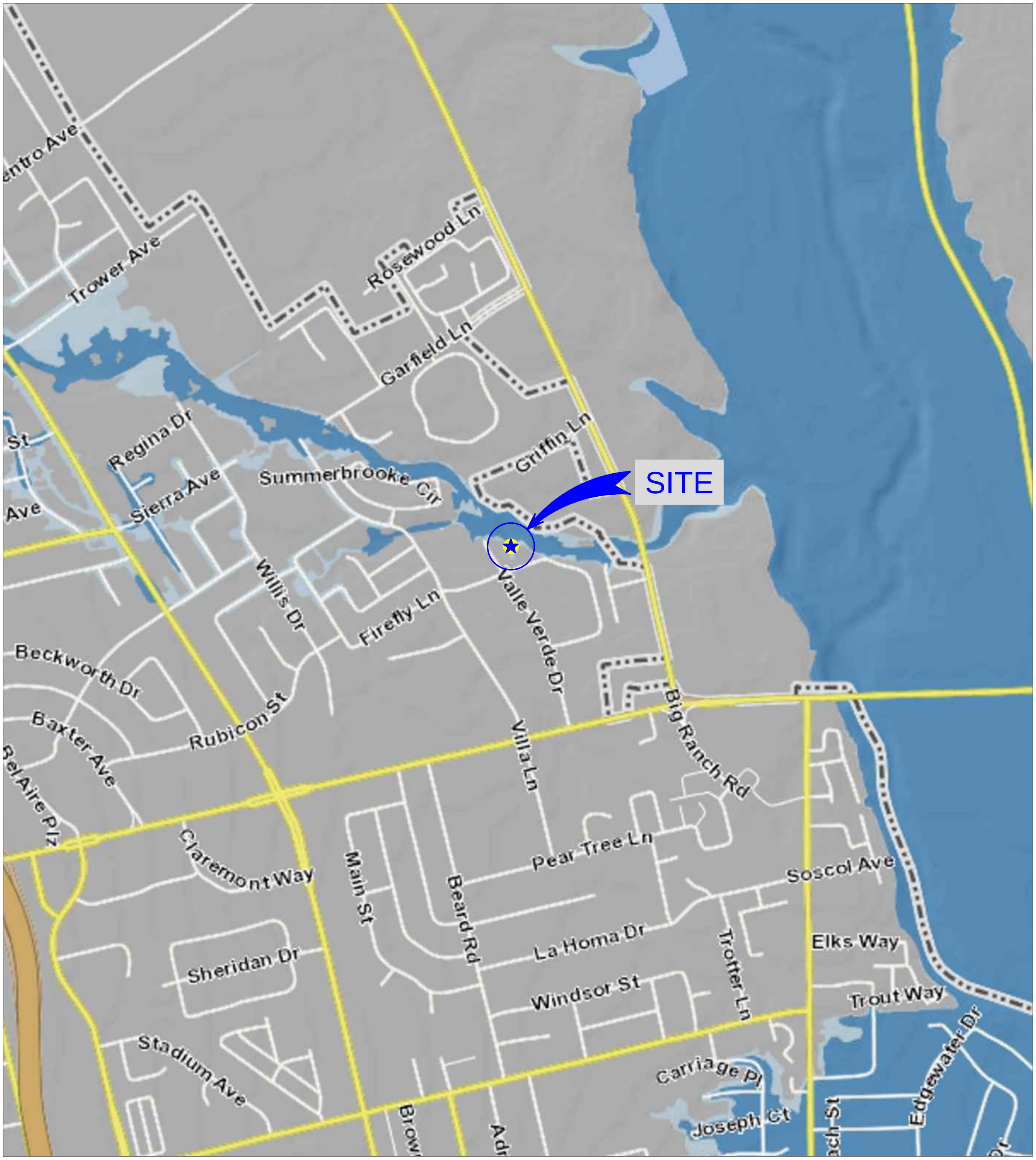
Date: 10/16/2018

Drawn MMT

Checked

10

FIGURE



Flood Hazard Area:

Zone V- 100yr.	Zone X - 500yr.
Zone A - 100yr.	Urbanized Area

No Scale

Zone V: This code identifies an area inundated by 1% annual chance flooding with velocity hazard (wave action).
Zone A: This code identifies an area inundated by 1% annual chance flooding.

Zone X 500yr: This code identifies an area inundated by .02% annual chance flooding and area inundated by 1% annual chance of flooding with average depth of less than 1 foot of with drainage areas less than 1 square mile or an area protected by levees form 1% annual chance flooding.

NORTH

Map Reference: ABAG Geographic Information System.

MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.010 Figures.dwg

FEMA DIGITAL FLOOD INSURANCE RATE MAP (DFIRM)

Valle Verde Development
Napa, California

Project No. 1687.01

Date: 10/16/2018

Drawn MMT
Checked

FIGURE 11

APPENDIX A
SUBSURFACE EXPLORATION (BORINGS) AND LABORATORY TESTING

1.0 Subsurface Exploration

We explored subsurface conditions at the site by drilling eight test borings utilizing truck mounted drilling equipment with either 6-inch diameter hollow stem augers or 4-inch diameter solid stem augers on October 9th, 10th, and 19th, 2018. The approximate boring locations are shown on Figure 2. The borings were drilled to a maximum depth of 51.5-feet below the ground surface.

The soils encountered were logged and identified in the field in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Figure A-1, Soil Classification Chart. The boring logs are presented on Figures A-2 through A-14.

We obtained “undisturbed” samples using a 3-inch diameter, split-barrel modified California sampler with 2.5 by 6-inch brass tube liners or with a 2-inch diameter, split-barrel Standard Penetration Test (SPT) sampler. The sampler was driven with a 140-pound hammer falling 30 inches. The number of blows required to drive the samplers 18 inches was recorded and is reported on the boring logs as blows per foot for the last 12 inches of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

2.0 Laboratory Testing

We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate engineering properties. The following laboratory tests were conducted in accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166; and
- Liquid and Plastic Limits of Soil, ASTM D 4318.

The moisture content, dry density, and unconfined compressive strength results are shown on the exploratory Boring Logs and the results of our Plasticity Index tests are presented on Figure A-15. The exploratory boring logs, description of soils encountered, and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage.

MAJOR DIVISIONS		SYMBOL		DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW		Well-graded gravels or gravel-sand mixtures, little or no fines
		GP		Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVEL with fines	GM		Silty gravels, gravel-sand-silt mixtures
		GC		Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW		Well-graded sands or gravelly sands, little or no fines
		SP		Poorly-graded sands or gravelly sands, little or no fines
	SAND with fines	SM		Silty sands, sand-silt mixtures
		SC		Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <50%	ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL		Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL		Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH		Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH		Inorganic clays of high plasticity, fat clays
		OH		Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT		Peat, muck, and other highly organic soils
ROCK				Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

PI	PLASTICITY INDEX
LL	LIQUID LIMIT
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

- MODIFIED CALIFORNIA
- STANDARD PENETRATION TEST
- THIN-WALLED / FIXED PISTON
- HAND SAMPLER
- ROCK CORE
- X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

STRENGTH TESTS

TV	FIELD TORVANE (UNDRAINED SHEAR)
UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress

SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25	sampler driven 12 inches with 25 blows after initial 6-inch drive
85/7"	sampler driven 7 inches with 85 blows after initial 6-inch drive
50/3"	sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

SOIL CLASSIFICATION CHART

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-1
FIGURE

BORING 1

EQUIPMENT: Truck Mounted Mobile B-53 with 6.0-Inch Hollow Stem Auger
DATE: 10/10/18
ELEVATION: 37 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters / feet	SAMPLE	SYMBOL (4)	Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0 – 0			2" Asphalt Concrete						
			8" Aggregate Base			13.8			
			Sandy CLAY (CL) Gray mottled orange, moist, stiff, medium plasticity clay, ~10-20% very fine grained sand. [Alluvium]	23	113	15.8	UC 2700	LL:37 PI:18	
1			Sandy CLAY (CH) Brown, moist, stiff, high plasticity clay, ~10-20% very fine grained sand. [Alluvium]						
5				18	108	18.7	UC 1950		
2			Grades medium stiff.	12	103	20.9	UC 950		
3 10			Clayey GRAVEL (GC) Brown, moist, loose, subangular gravel, ~15-20% low plasticity clay. [Alluvium]	10	112	8.8		P200 17.7%	
∇				7		19.4			
4			CLAY (CL) Gray mottled orange, moist, medium stiff, medium plasticity clay, ~10-20% very fine grained sand. [Alluvium]						
15			Clayey GRAVEL with Sand (GC) Brown to medium brown, loose to medium dense, subangular gravel, ~5-10% medium plasticity clay, ~5-10% fine to medium grained sand. [Alluvium]	20	112	19.1	UC 350		
5				10		12.7		P200 8.6%	
			Clayey GRAVEL with Sand (GC) Brown to medium brown, wet, medium dense, subangular gravel, ~45-50% medium plasticity clay, ~5-10% fine to medium grained sand. [Alluvium]						
6 20				19		19.9		P200 46.5%	

∇ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-2
FIGURE

BORING 1 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	20			Clayey GRAVEL with Sand (GC) Brown to medium brown, wet, medium dense, subangular gravel, ~45-50% medium plasticity clay, ~5-10% fine to medium grained sand. [Alluvium]						
7										
				Silty CLAY (CH) Gray mottled orange, wet, stiff to very stiff, high plasticity clay and silt. [Alluvium]						
	25				25	91	32.0			
8										
9				Grades stiff.						
	30				19	98	27.2			
10										
	35									
11										
12										
	40				14	96	28.5			

∇ Water level encountered during drilling

▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-3
FIGURE

BORING 1 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	40			Silty CLAY (CH) Gray mottled orange, wet, stiff, high plasticity clay and silt. [Alluvium]						
13										
	45									
14										
15	50	■			23		27.9			
				Boring terminated at 50.0 feet. Groundwater encountered at 12.5 feet during exploration.						
16										
	55									
17										
18	60									

▽ Water level encountered during drilling

▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-4
FIGURE

BORING 2

EQUIPMENT: Truck Mounted Mobile B-53 with 6.0-Inch Hollow Stem Auger
DATE: 10/10/18
ELEVATION: 39 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			2.5" Asphalt Concrete						
				6" Aggregate Base						
				Silty CLAY (CL) Medium brown, moist, very stiff, medium plasticity clay and silt. [Alluvium]			14.0			
					51	120	12.3	UC 8900		
1					57	117	15.0	UC 7650		
	5				27		11.7			
2										
3	10				18		11.1			
4										
	15 ∇				17		15.6			
5										
				CLAY (CH) Dark brown mottled orange, moist, stiff to very stiff, high plasticity clay. [Alluvium]						
6	20				15		26.9			

∇ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 1687.01BL.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-5
FIGURE

BORING 2 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	20			CLAY (CH) Dark brown mottled orange, moist, stiff to very stiff, high plasticity clay. [Alluvium]	15		26.9			
				Boring terminated at 21.5 feet. Groundwater encountered at 15.0 feet during exploration.						
7										
	25									
8										
9	30									
10										
	35									
11										
12	40									

Water level encountered during drilling

Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-6
FIGURE

BORING 3

EQUIPMENT: Truck Mounted Mobile B-53 with 6.0-Inch Hollow Stem Auger
DATE: 10/10/18
ELEVATION: 38 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			Sandy CLAY with Gravel (CL) Medium brown, moist, stiff to very stiff, medium plasticity clay, ~20% subangular gravel, ~15% fine grained sand. [Alluvium]			7.0			
1					25	110	11.6	UC 1100	LL:36 PI:15	
	5			Silty CLAY (CH) Medium brown, moist, medium stiff, high plasticity clay and silt. [Alluvium]	14	108	16.1	UC 950		
2					12	89	31.5	UC 1100		
3	10				9	124	26.9	UC 1200		
4										
	∇			Sandy CLAY (CH) Medium brown, moist, stiff, high plasticity clay, ~30% fine grained sand. [Alluvium]	14	93	30.8			
5	15				13		23.3			
6	20			Boring terminated at 17.0 feet. Groundwater encountered at 14.0 feet during exploration.						

∇ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 1687.01BL.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-7
FIGURE

BORING 4

EQUIPMENT: Truck Mounted Mobile B-53 with 6.0-Inch Hollow Stem Auger
DATE: 10/10/18
ELEVATION: 44 - feet*
*REFERENCE: Google Earth, 2018

DEPTH (meters / feet)	SAMPLE	SYMBOL (4)	Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0 – 0			CLAY with Gravel (CL) Brown, moist, stiff, medium plasticity clay, ~25% subrounded gravel. [Fill]	20	105	13.2			
1			Clayey GRAVEL with Sand (GC) Brown, moist, medium dense, subangular gravel, ~40-45% medium plasticity clay, ~10% very fine grained sand. [Alluvium]	16	95	12.6			
5 / 2				21	96	12.9		P200 43.1%	
3 / 10			Grades ~20-25% medium plasticity clay, medium to coarse grained sand.	16	114	15.2	UC 700	P200 20.5%	
4 / 15 / 5			CLAY (CH) Brown mottled orange, moist, medium stiff, high plasticity clay. [Alluvium]	10	91	32.3			
			Boring terminated at 16.5 feet. No groundwater encountered during exploration.						
6 / 20									

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-8
FIGURE

BORING 5

EQUIPMENT: Truck Mounted Mobile B-53 with 6.0-Inch Hollow Stem Auger
DATE: 10/10/18
ELEVATION: 44 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			2.5" Asphalt Concrete						
				9.5" Aggregate Base						
		■		Clayey GRAVEL (GC) Light to medium brown, moist, dense, subrounded to subangular gravel, ~30% medium plasticity clay. [Fill]	57	118	10.7			
1		■		Grades medium dense.	27	114	9.6	UC 1300		
	5	■			27	117	14.7			
2				Boring terminated at 6.5 feet. No groundwater encountered during exploration.						
3	10									
4										
	15									
5										
6	20									

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-9
FIGURE

BORING 6

EQUIPMENT: Truck Mounted Mobile B-53 with 6.0-Inch Hollow Stem Auger
DATE: 10/10/18
ELEVATION: 41 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0	Gravelly SILT with Sand (ML) Brown, dry, very stiff, low to medium plasticity silt, ~30% subangular gravel, ~15% very fine grained sand. [Fill]			8.8			
1			56	114	7.5			
	5	Clayey GRAVEL (GC) Medium brown, moist, medium dense, subrounded to subangular gravel, ~10-20% medium plasticity clay. [Alluvium]	39	112	9.5	UC 900	P200 17.6%	
2			19	103	10.0		P200 12.4%	
3	10	CLAY (CL) Gray mottled orange and brown, moist, medium stiff, medium to high plasticity clay. [Alluvium]	9	91	32.3	UC 900		
4								
	15	Grades stiff.	14	99	27.3			
5								
6	20		19	98	27.4			

SAMPLE | SYMBOL (4)

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-10
FIGURE

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	BORING 6 (CONTINUED)	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	20			CLAY (CL) Gray mottled orange and brown, moist, stiff, medium to high plasticity clay. [Alluvium]						
7										
	25				17	95	28.1			
8										
				CLAY (CH) Gray mottled orange and brown, moist, stiff, medium to high plasticity clay. [Alluvium]						
9	30				20	87	34.6			
10										
	35									
11										
12										
	40				18	91	31.9			

▽ Water level encountered during drilling

▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-11
FIGURE

BORING 6 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	40			CLAY (CH) Gray mottled orange and brown, moist, stiff, medium to high plasticity clay. [Alluvium]						
13										
	45									
14										
15				CLAY with Gravel (CH) Gray mottled orange and brown, moist, very stiff, medium to high plasticity clay, ~20-30% subangular to subrounded gravel. [Alluvium]	43	104	23.6			
	50			Boring terminated at 50.0 feet. Groundwater encountered at 22.5 feet during exploration.						
16										
	55									
17										
18	60									

Water level encountered during drilling

Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-12
FIGURE

BORING 7

EQUIPMENT: Truck Mounted Mobile B-53 with 6.0-Inch Hollow Stem Auger
DATE: 10/10/18
ELEVATION: 44 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			2.5" Asphalt Concrete						
				4" Aggregate Base						
				Clayey SILT (ML) Light brown mottled orange, moist, very stiff, medium plasticity silt and clay. [Fill]	43	109	9.7 9.6	UC 2200		
1				Gravelly SILT (ML) Light brown mottled orange, moist, very stiff, medium plasticity silt, ~30-40% subangular gravel. [Alluvium]	38	106	17.3	UC 8150		
	5									
2					56	117	11.3			
				Boring terminated at 7.5 feet. No groundwater encountered during exploration.						
3	10									
4										
	15									
5										
6	20									

∇ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-13
FIGURE

BORING 8

EQUIPMENT: Truck Mounted Mobile B-53 with 6.0-Inch Hollow Stem Auger
DATE: 10/10/18
ELEVATION: 44 - feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			Silty CLAY (CL) Medium brown, moist, very stiff, medium to high plasticity clay and silt. [Fill]						
					47		8.9			
1					41	117	13.6	UC 6850		
	5			Grades medium stiff.	11	111	14.9			
2				Boring terminated at 6.5 feet. No groundwater encountered during exploration.						
3	10									
4										
	15									
5										
6	20									

Water level encountered during drilling
Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

BORING LOG

Valle Verde Development
Napa, California

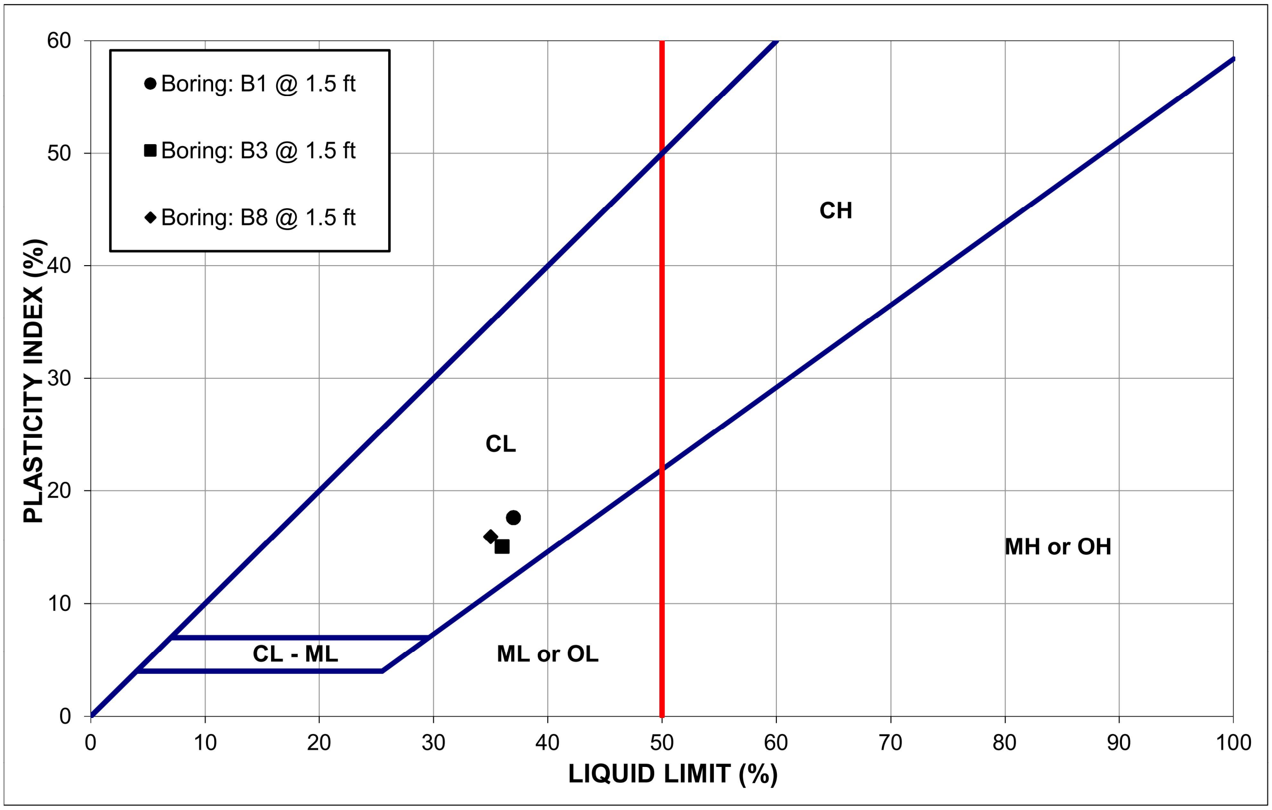
Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-14
FIGURE

MILLER PACIFIC ENGINEERING GROUP

ATTERBERG LIMITS TEST (ASTM D 4318)



Sample	Classification	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Boring: B1 @ 1.5 ft	Sandy CLAY (CL) gray mottled orange	37	19	18
Boring: B3 @ 1.5 ft	Sandy CLAY with Gravel (CL) medium brown	36	21	15
Boring: B8 @ 1.5 ft	Sandy CLAY with Gravel (CL) medium brown	35	19	16

PI = 0-3: Non-Plastic
PI = 3-15: Slightly Plastic
PI = 15-30: Medium Plasticity
PI = >30: High Plasticity

MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 1687.01BL.dwg

PLASTICITY INDEX TEST RESULTS

Valle Verde Development
Napa, California

Project No. 1687.01 Date: 10/16/2018

Drawn NAR
Checked

A-15
FIGURE

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, California 94947
T 415 / 382-3444
F 415 / 382-3450

GEOTECHNICAL INVESTIGATION
MULTI UNIT RESIDENTIAL DEVELOPMENT
3700 VALLE VERDE
NAPA, CALIFORNIA

January 13, 2011

Job No. 1687.01

Prepared For:
BRIDGE Housing
345 Spear Street
San Francisco, California

CERTIFICATION

This document is an instrument of service, prepared by or under the direction of the undersigned professionals, in accordance with the current ordinary standard of care. The service specifically excludes the investigation of polychlorinated byphenols, radon, asbestos or any other hazardous materials. The document is for the sole use of the client and consultants on this project. No other use is authorized. If the project changes, or more than two years have passed since issuance of this report, the findings and recommendations must be updated.

MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

REVIEWED BY:

Timothy J. Reynolds
Geotechnical Engineer No. 2686
(Expires 12/31/12)

Daniel S. Caldwell
Geotechnical Engineer No. 2006
(Expires 9/30/11)

Napa Valley Business Park, 135 Camino Dorado, Suite 3, Napa, California 94558
Phone – 707.265.7936 / Fax – 707.265.7982

Miller Pacific ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
MULTI UNIT RESIDENTIAL DEVELOPMENT
3700 VALLE VERDE
NAPA, CALIFORNIA

TABLE OF CONTENTS

FIGURES

APPENDIX A, SUBSURFACE EXPLORATION AND LABORATORY TESTING

GEOTECHNICAL INVESTIGATION
MULTI UNIT RESIDENTIAL DEVELOPMENT
3700 VALLE VERDE
NAPA, CALIFORNIA

I. INTRODUCTION

This report presents the results of our geotechnical investigation for a proposed multi-unit residential development at 3700 Valle Verde in Napa, California. A site location map is included as Figure 1. This report was prepared for the exclusive use of BRIDGE Housing and their development team for this project and site. No other use is authorized without written consent of Miller Pacific Engineering Group.

The purpose of our investigation was to explore subsurface conditions and develop geotechnical criteria for design of new buildings, asphalt-paved access driveways and parking areas, and other associated new improvements. Our current scope of services does not include geotechnical evaluation of exiting structures. The scope of our geotechnical investigation as described in our proposal letter dated November 9, 2010 included subsurface exploration with seven test borings, laboratory testing, engineering evaluation, and development of recommendations appropriate for the project and site. This report completes our Phase 1 Geotechnical Investigation services and includes the following:

(1) Brief summary of geologic and geotechnical setting, surface, and expected subsurface conditions;

(2) Discussion of geologic hazards;

(3) Recommendations for site preparation including placement, quality, and compaction of new fill (if needed);

(4) Seismic Design Criteria, including 2010 CBC Coefficients and near-source factors;

(5) Recommendations for foundation type and design criteria;

(6) Soil engineering drainage recommendations;

(7) Driveway and parking structural pavement sections; and,

(8) Recommendations for backfill of utility trenches.

II. PROJECT DESCRIPTION

The project will include construction of several two- or three-story, multi-unit residential structures. We anticipate the new structures will have concrete slab-on-grade floors with relatively light, wood- or metal-framed construction. A couple of general development plans are being considered. One would re-use an existing three-story residential building at the southeastern end of the site with new units constructed at the central and north of the site. Alternatively, the existing structure could be demolished with new buildings and other infrastructure built within its footprint. Underground service utilities, asphalt-paved driveways, and landscaping will be located between and around the buildings. A site plan showing the development plan that would re-use the existing three-story residential structure is shown on Figure 2. We anticipate that site preparations will consist of minor grading to create the building pads and new access driveway and to create positive surface drainage patterns. Cuts and fills will likely be limited to a few feet or less in thickness.

III. SITE CONDITIONS

A. Geologic Setting

The project site is located within the Coast Range Geomorphic Province of California. The regional bedrock geology consists of complexly folded, faulted, sheared and altered sedimentary, igneous and metamorphic bedrock of the Jurassic-Cretaceous age (65-190 million years ago) Franciscan Complex.

The regional topography is characterized by northwest-southeast trending mountain ridges and intervening valleys that were formed from tectonic activity between the North American Plate and the (offshore) Pacific Plate. Extensive faulting during the Pliocene Age (1.8-7 million years ago) formed the uneven depression that is now the San Francisco Bay. The more recent tectonic activity within the Coast Range Geomorphic Province is concentrated along the San Andreas Fault zone, a complex group of generally parallel faults.

More locally, published geologic mapping by the California Geological Survey (Clahan, et al., 2004) indicate that the site is underlain by older (latest Pleistocene, approx. 30,000 years old) alluvial deposits comprised of various mixtures of sand, gravel, silt, and clay (see Figure 3). Stream Channel Deposits (younger, less consolidated sands and silt) are mapped near the northern boundary, adjacent to Salvador Creek. The older geologic deposits tend to be more highly consolidated than more recent (Holocene) deposits common through much of the Napa Valley. Mapping by the United States Geological Survey (USGS, Sowers, et al., 1998) shows the site is located in an area of Very Low liquefaction potential.

B. Seismicity

The proposed site is located within a seismically active area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated or it may be accumulated and stored for long periods of time. Individual releases may be so small that only sensitive instruments detect them, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust but typically are braids of breaks that comprise shatter zones which link to form networks of major and minor faults. Studies are continually in-progress to more precisely locate currently-known faults and to identify previously un-known faults. Within the Bay Area, faults are concentrated along the San Andreas fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy movement becomes a long, high-amplitude motion when moving through soft ground materials, such as alluvial soil.

An "active" fault is one that shows evidence of displacement within the last 11,000 years, and therefore, is considered more likely to generate a future earthquake than a fault that shows no sign of recent rupture. The locations of the currently known active faults relative to the project site are shown on Figure 4.

The Richter or Moment Magnitude Scale provides a method to deduce the magnitude of an earthquake from seismologic instruments. The measurement of magnitude provides a rating that is independent of the place of observation and thus allows a comparison of seismic events. Magnitude is measured on a logarithmic scale; every one-unit increase indicates an increment of roughly 30 times the energy. For example, an 8.0 magnitude earthquake would have an energy level 30 times that of a 7.0 magnitude and 900 times that of a 6.0 magnitude earthquake.

Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that 30 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 100 kilometers of the site area between 1735 and 2010. Within a radius of 200 kilometers of the site, 105 earthquakes (Richter Magnitude 5.0 or larger) have occurred in the same time period. Using these earthquakes and empirical attenuation relationships, the maximum historic ground acceleration (median peak) at the project site is approximately 0.19g. The five most significant historic earthquakes to affect the project site are summarized in Table A.

TABLE A
SIGNIFICANT EARTHQUAKE ACTIVITY
SAN FRANCISCO BAY AREA REGION

Fault	Historic Richter Magnitude	Year	Distance	Peak Ground Acceleration
Rodgers Creek	6.2	1898	13 km	0.19 g
San Andreas	8.3	1906	68 km	0.12 g
Unnamed/Mt. Veeder	5.2[1]	2000	13 km	0.09 g
Hayward	6.8	1836	55 km	0.07 g
Hayward	6.8	1868	68 km	0.06 g

1) Moment Magnitude

References: Sources: USGS (2001), Abrahamson & Silva (1997)

The calculated accelerations in Table A should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations. Significant deviation from the values presented is possible due to geotechnical and geologic variations from the typical conditions used in the empirical correlations.

Probability of Future Earthquakes

The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probability in this region, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" to estimate the probabilities of earthquakes on active faults. Potential sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, and micro-seismicity to arrive at estimates of probabilities of earthquakes with a Moment Magnitude greater than 6.7 by 2032.

The probability studies focus on seven "fault systems" within the Bay Area. Fault systems are composed of different, interacting fault segments capable of producing earthquakes within the individual segment or in combination with other segments of the same fault system. The probabilities for the individual fault segments in the San Francisco Bay Area are presented on Figure 4.

In addition to the seven fault systems, the studies included probabilities of "background earthquakes." These earthquakes are not associated with the identified fault systems and may occur on lesser faults (i.e. West Napa) or previously unknown faults (i.e. the 1989 Loma Prieta and 2000 Mt. Veeder Earthquake, Napa). When the probabilities on all seven fault systems and the background earthquakes are combined mathematically, there is a 60 percent chance for a magnitude 6.7 or larger earthquake to occur in the Bay Area by the year 2032. Smaller earthquakes (between magnitudes 6.0 and 6.7), capable of considerable damage depending on proximity to urban areas, have about an 80 percent chance of occurring in the Bay Area by 2032 (USGS, 2002).

Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are on going. These current evaluations include data from additional active faults and updated geological data.

C. Surface Conditions

The property consists of two parts. The southern half was previously used as an assisted care facility (Sunrise Facility), with asphalt paved driveway and parking, underground service utilities, and the three-story residential building noted previously. The northern half of the site includes an abandoned single-family residence with garage, swimming pool, and paved access driveway at the northeast corner. The project area is relatively flat, but with a general, gentle gradient down towards a creek along the northeastern property boundary.

D. Field Exploration and Subsurface Conditions

Prior to our subsurface exploration, we contacted Underground Service Alert (USA) to mark the locations of underground utilities within the area of our proposed exploration. Seven test borings were drilled at the site on December 9 and 20, 2010 using truck-mounted drilling equipment. The boring locations are shown on the Site Plan, Figure 2. The soils encountered were logged and samples were obtained for laboratory testing. The subsurface exploration program is discussed in more detail in Appendix A. A Soil Classification Chart and boring logs are presented on Figures A-1 through A-14 of Appendix A.

Our subsurface exploration generally confirms the mapped geology described and referenced earlier. Our borings drilled within the Sunrise Property encountered a consistent pavement section of 3-inches of asphalt concrete over 9-inches of aggregate baserock. Below the pavement at Borings 2 and 6 (northeast and northwest corners of existing Sunrise building, respectively), we encountered six to nine feet of a sandy clay artificial fill, likely placed during construction of the existing Sunrise facility. Below the fill at Borings 2 and 6 and directly below the pavement section at Borings 5 and 7 (drilled within the parking area southwest of the Sunrise building) we encountered approximately five feet of clayey sand and gravel mixtures (classified as either alluvium of stream deposits) over clayey alluvial deposits with varying amounts of sand and gravel and intermittent sand/gravel lenses to the total depths explored (31.5 feet).

At Borings 1 and 3, drilled within the northwesterly (abandoned residence) portion of the site, we encountered an older pavement section of 2-inches of asphalt concrete over 4-inches of aggregate baserock. Below the pavement at these borings and starting at the surface in Boring 4, we encountered variable mixtures of sandy clay and clayey sand with minor amounts of gravel and intermittent sand/gravel lenses to the depths explored (31.5 feet). With the exception of the upper one to two feet (which could have been minor old fills or disturbed native soil), the soils encountered within the northwestern part of the site were classified as natural alluvium.

E. Laboratory Testing

Laboratory testing of both relatively undisturbed and bulk samples from the exploratory borings included moisture content, dry density, -(200) wash, Plasticity Index (PI), and unconfined compressive strength. The results of the laboratory testing are presented on the Boring Logs and on Figure A-15. The laboratory-testing program is described in detail in Appendix A.

F. Groundwater

Free water was observed as shallow as 6 feet below ground surface in Boring 7 and was generally encountered between 8 and 11 feet within the remaining borings. Borings were backfilled after completion, so a stabilized water level may not have been observed. Groundwater levels will likely fluctuate throughout the year and will likely perch on top of shallow clayey layers during the winter or after periods of intense rainfall.

IV. GEOLOGIC HAZARDS EVALUATION

A. Summary

Potential geologic hazards which could affect the planned improvements at the project site include seismic shaking, liquefaction, moderately expansive soils, and differential settlement if new buildings are founded across differing foundation soil conditions (i.e., older alluvium and artificial fill/stream deposits). Other hazards, such as fault surface rupture, are not judged to be Significant at the subject site. Geologic hazards, impacts, and mitigation measures are described below.

B. Fault Surface Rupture

The proposed development area is not located within an Alquist-Priolo Earthquake Fault Zone[1]. We therefore judge the potential for fault surface rupture in the development area to be low.

No mitigation measures are required for structures in the development area.

C. Seismic Shaking

The project area will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical relations developed for stiff soil sites (Abrahamson & Silva, 1997) to provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the center of the planning area, and probable peak ground accelerations are summarized in Table B.

[1] The Alquist Priolo Earthquake Fault Zoning Act prohibits placing most structures for human occupancy across traces of active faults. These fault zones are shown on maps issued by the Department of Conservation's Division of Mines and Geology.

TABLE B
ESTIMATED PEAK GROUND ACCELERATION
FOR PRINCIPAL ACTIVE FAULTS
3700 VALLE VERDE
NAPA, CALIFORNIA

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (kilometers)	Median Peak Ground Acceleration (g)[1]
West Napa	6.5	1	0.50
Concord – Green Valley	6.8	13	0.24
Rodgers Creek	7.0	18	0.21
Hunting Creek – Berryessa	6.9	21	0.17
Hayward	7.1	26	0.16

(1) Determined from attenuation relationship by Abrahamson & Silva (1997) for stiff soil sites.

Reference: USGS (1996)

The calculated accelerations in Table B should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

The potential for strong seismic shaking at the project site is high. Due to their close proximity, the West Napa, Concord–Green Valley, and Rodgers Creek faults present the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Seismic Shaking Mitigation Measures – Mitigation measures should include, as a minimum, designing the structure in accordance with the most recent (2010) version of the California Building Code. Recommended CBC seismic coefficients are provided in Section V of this report.

D. Liquefaction Potential and Related Impacts

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. This phenomenon can occur where there are saturated, loose, granular (sandy) deposits subjected to seismic shaking. Liquefaction-related phenomena include seismically-induced settlement, flow failure, and lateral spreading.

Our test borings encountered one, three to five foot thick, layer of granular stream deposits capped by eight to ten feet of clayey artificial fill under the northeast end of the existing Sunrise building and isolated lenses of loose to medium-dense sands and gravels beneath other areas of the site, generally below 15 feet. The stream deposit layer below the existing Sunrise facility was relatively dense and only partially located below likely high ground water levels. However, some settlements could result from volumetric strain in a severe seismic event. The deeper lenses are relatively thin, not continuous, and confined by a minimum 15-foot thick stiff clay layer, thus surface effects due to liquefaction are judged to be unlikely and would be limited to isolated minor settlement.

Liquefaction Potential Mitigation Measures – The risk for damage to new structures associated with liquefaction is considered to be low and limited to modest total and differential settlements. Mitigation of the risk should include a combination of site grading and foundation design criteria provided in Sections V-C and V-E, respectively.

E. Seismic Induced Ground Settlement

Ground shaking can also induce settlement of non-saturated loose granular soils. We did not encounter these materials within planned development areas.

Seismic Induced Ground Settlement Mitigation Measures – No mitigation measures are required.

F. Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. Ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep channel banks. Because the site topography is essentially flat, there is very little potential for lurching or ground cracking.

Lurching and Ground Cracking Mitigation Measures – No mitigation measures are required.

G. Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flows. At the project site, the surface soils are relatively clayey and slopes are relatively flat. However, concentrated surface water flows could result in erosion.

Mitigation measures include the project Civil Engineer designing a site drainage system to collect surface water and discharge at an appropriate location.

H. Expansive Soils

Moderate and highly plastic silts and clays, when located near the ground surface, can exhibit expansive characteristics (shrink-swell) that can be detrimental to structures and flatwork during periods of fluctuating soil moisture content. Our past experience in the area indicates near-surface soils are not highly expansive. However, our lab testing indicates that some near-surface silty and clayey soils at the project site are moderately plastic.

Expansive Soil Mitigation Measures – No special mitigation measures are recommended to mitigate expansive soil. However, recommendations for site preparation, grading, drainage, and foundation design provided in the following Sections V-C, V-D, and V-E must be followed to mitigate the slight to moderate shrink/swell potential of near-surface clayey soil.

I. Settlement

Near surface soils (to depths of 15+ feet) encountered during our exploration were generally medium stiff/dense to stiff/dense and will not be prone to significant settlement due to the anticipated static loads from new construction. As noted, the existing Sunrise structure appears to rest in part on an artificial fill wedge. Our current work has not included a settlement study of the existing building. Planned development will require demolition and backfill of existing structures, utilities, and foundations. This could result in differential foundation soil conditions beneath new construction.

Settlement Mitigation Measures – Site Grading and Foundation design recommendations previously discussed for liquefaction and expansive soils will serve as mitigation for any slight potential settlement under static loading. Foundations should be designed in accordance with Section V-E.

J. Slope Stability

The project site is generally flat or gently sloping. Slope instability is not judged to be a significant hazard. Localized sloughing along the banks of Salvador Creek northeast of the site could occur. However, existing and proposed building setbacks should provide a more than adequate buffer zone.

Slope Stability Mitigation Measures – No mitigation measures are required.

V. CONCLUSIONS AND RECOMMENDATIONS

A. Conclusions

Based on our subsurface exploration, laboratory testing, analysis, and experience with similar projects in this area, we judge the site may be developed as described, provided site grading and foundation design is incorporated into the project. The primary geotechnical concerns at the site include strong seismic ground shaking, moderately expansive near-surface soil, and modest settlement under static and seismic loading.

B. Seismic Design

Minimum mitigation of ground shaking includes seismic design of the structure in conformance with the provisions of the most recent version (2010) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity to faults, as shown on Figure 4, we recommend the CBC coefficients and site values shown in Table C below.

TABLE C
2010 CBC SEISMIC DESIGN FACTORS
3700 VALLE VERDE
NAPA, CALIFORNIA

Factor Name	Coefficient	CBC Table/ Figure	Site Specific Value
Site Class[A]	$S_{A,B,C,D,E,\text{ or }F}$	1613.5.2	S_D
Spectral Acc. (short)	S_s	1613.5(3)	1.89 g
Spectral Acc. (1-sec)	S_1	1613.5(4)	0.68 g
Site Coefficient	F_a	1613.5.3(1)	1.0
Site Coefficient	F_v	1613.5.3(2)	1.5

(A) Soil Profile Type S_D Description: Stiff soil profile with shear wave velocity between 600 (180) and 1200 (360) feet per second (m/s), Standard Penetration Test N value between 15 and 50, and Undrained Shear Strength between 1000 and 2000 psf.

The effects of earthquake shaking (i.e. protection of life safety) can be mitigated by close adherence to the seismic provisions of the current edition of the CBC. However, some building damage may still occur during strong ground shaking.

C. Site Grading

1. Site Preparation

Site preparation should include scraping grass, weeds, and their root crowns from the ground surface. Old concrete slabs, foundations, swimming pool, and underground tanks (if encountered) must also be removed from the site. Excavated areas (i.e., excavations for removal of old pipes, existing foundations, old septic tanks, etc.) should be restored with properly moisture conditioned and compacted fill as described in the following sections. Concrete and asphalt debris (unless processed to substantially reduce particle sizes) and vegetation scrapings will not be suitable for structural fill and should be removed from the site. Alternatively, vegetation scrapings may be used in landscape areas. Also, if the existing Sunrise building is to be demolished, old fill and any unsuitable soils should be removed, reprocessed, and replaced as structural fill as outlined within new structural areas.

Where fills are planned, the subgrade surface should be moisture conditioned to near or slightly above the optimum moisture content and compacted to a minimum of 90 percent relative compaction (ASTM D-1557) and a firm and unyielding surface. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop."

2. Fill Materials

The on-site materials appear suitable for use as compacted fill. Imported fill may also be considered if it meets the criteria below.

3. Imported Fill

If imported fill is used to raise building site grades, it shall be free of organic matter, have a Liquid Limit of less than 40, a Plasticity Index of less than 20, have a minimum R-value of 15, and conform to the gradation limits in Table D.

TABLE D
IMPORTED FILL GRADATION LIMITS
3700 VALLE VERDE
NAPA, CALIFORNIA

Particle Size	Percent Finer by Dry Weight
4 inch	100
No. 4 sieve	20 - 100
No. 200 sieve	0 - 50

4. Compaction

All on-site and imported fill and backfill should be conditioned to near or slightly above the optimum moisture content. Properly moisture conditioned and cured materials should subsequently be placed in loose horizontal lifts of 8 inches thick or less, and uniformly compacted to a minimum of 90 percent relative compaction to produce a firm non-yielding surface.

5. Cut and Fill Slope Construction

If minor cut or fill slopes are planned to level the building pads, the slopes should be limited to 2:1 (horizontal: vertical).

D. Site and Foundation Drainage

Careful consideration should be given to design of finished grades at the site. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for 5 feet (5 percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2 percent). Roof gutter downspouts may discharge onto the pavements, but should not discharge onto any landscaped areas. Provide area drains for landscape planters adjacent to buildings and parking areas and collect downspout discharges into a tight pipe collection system.

Site drainage should be discharged away from the building area and outlets should be designed to reduce erosion of the soils immediately downslope. Where possible, site drainage improvements should be connected into the existing City storm drainage system.

E. Foundation Design Criteria

As described previously, minor differential ground movements due to static and seismic loading and expansive soil shrink/swell are potential hazards at the site. Further, the need to demolish old structures, including a swimming pool, could result in some differential soil conditions below new structures. While our site preparation and grading recommendations are intended to mitigate these hazards, some potential for isolated areas of non-uniform foundation support will be inevitable. Thus, project foundation design should provide for some additional stiffness and the ability of structures to span isolated areas of non-uniformity. Alternatives include deep foundations (such as drilled piers) or stiffened shallow foundations, such as reinforced and thickened concrete slabs (structural slab) or inter-connected grade beams (which can be used to support wood floors or with concrete slab-on-grade floors). Drilled piers will be difficult to construct because of the high groundwater and granular soils. We therefore recommend the use of a structural slab or inter-connected grade beams for the building foundations.

Structural Slabs –Mat slabs can be designed as: (1) slabs with underlying cross beams, (2) waffle-mat slabs with cross-beam stiffeners in a waffle pattern, or (3) uniform-thickness slabs. A uniform slab thickness is typically preferred because construction is simpler and trenching and forming of underlying cross beams can be avoided. The Structural Engineer should determine and design the most appropriate means of stiffening the slab. For this project, “Uniform” slabs can be thickened as needed to provide added stiffness where heavy loads (bearing walls or columns) are planned while still limiting the need for extra trenching and forming. The typical thickness of a uniform mat slab is on the order of 8 to 10 inches with heavy top and bottom reinforcing steel in each direction. The project Structural Engineer should design the mat slab with sufficient thickness and reinforcement to resist unsupported spans or cantilevers shown in Table E.

Continuous Spread Footings – The objective for continuous spread footings will be to provide stiffness in two directions by inter-connecting the grade beams in a rectangular grid pattern. Footings should be designed for both positive and negative bending and to span and cantilever a distance of 5 feet over localized areas of non-uniform support. Where footings support a raised floor with an 18-inch crawl space, we expect that 18-inches of embedment (total footing height of 36 inches (mud sill to footing bottom) will provide adequate stiffness, if reinforced.

TABLE E
FOUNDATION DESIGN CRITERIA
3700 VALLE VERDE
NAPA, CALIFORNIA

Minimum Embedment (Grade Beams):	18 inches
Minimum Width (Grade Beams):	12 inches
Allowable bearing pressure:	
Dead plus live loads	2000 psf
Total design loads	3000 psf
Base friction coefficient:	0.35
Lateral Passive Resistance[1]:	300 pcf
Modulus of Subgrade Reaction, k:	120 psi per inch
Maximum unsupported interior span:	8 feet
Maximum unsupported corner cantilever:	4 feet

(1) Equivalent fluid pressure. Ignore upper 6-inches, unless exterior concrete or asphalt pavement confines the slab.

F. Concrete Slabs-On-Grade

Interior concrete slabs should be underlain by at least 4-inches of clean, free-draining (3/4 inch) aggregate to act as a capillary moisture break. Where moisture would be detrimental to the interior floor coverings, a vapor barrier consisting of a minimum 10-mil plastic sheeting should cover the aggregate. The vapor barrier should meet the requirements of ASTM E-1745. To aid concrete curing and protect the vapor barrier, cover the membrane with about 2-inches of dry sand.

Exterior concrete walkway slabs should be at least 4 inches thick and underlain with 4 inches or more of Class 2 Aggregate Base (Caltrans Standard Specifications, 1999). The aggregate base should be moisture conditioned and compacted to at least 95 percent relative compaction. The upper 8-inches of subgrade on which aggregate base is placed should be prepared as described in Section C, Site Grading.

If superior performance is desired, exterior slabs can be thickened to 5 inches and reinforced with steel reinforcing bars (not welded wire mesh). We recommend crack control joints no farther than 6 feet apart in both directions and that the reinforcing bars, if used, extend through the control joints.

G. Utility Trench Excavations and Backfill

Excavations for utilities will be in medium stiff to stiff soils to depth of greater than 10 feet. Trench excavations having a depth of 5 feet or more must be excavated and shored in accordance with OSHA regulations. Pursuant to OSHA classifications, on-site soils to a depth of 10 feet are a Type B. Bedding materials for utility pipes should be well graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 5 percent finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically 3 to 6 inches. Trench backfill may consist of on-site soils that are moisture conditioned, placed in thin lifts, and compacted to at least 90 percent. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

H. Driveway Subgrade Preparation and Pavement Structural Section

Site grading for the new paved access driveways and parking areas should be performed as described in Section C, Site Grading. If soft or unsuitable materials are encountered in the surface soils, these soils must be removed or moisture conditioned and recompacted at the direction of the Geotechnical Engineer.

Using a design Traffic Index (TI) of 4 for parking areas and minimum R-value of 10, we recommend a pavement section of 2.5 inches of asphalt concrete over 7 inches of aggregate base. For a TI of 5 and R-value of 10, we recommend a section of 2.5 inches of asphalt concrete over 10 inches of aggregate base. The R-value of pavement subgrade should be confirmed during construction and when the subgrade soils are exposed. If larger TI's are anticipated in the driveway area, the pavement section should be re-evaluated. As larger TI's can generate significantly thicker structural pavement sections, treatment of pavement subgrade with lime or cement could be considered to increase the design R-value and thus reduce the aggregate baserock thickness.

The upper 6 inches of subgrade in pavement areas must be scarified, moisture conditioned to near the optimum water content, and then compacted to a minimum 95 percent relative compaction. The compacted surface must also be non-yielding when proof-rolled with heavy construction equipment.

The base rock should consist of compacted Class 2 Aggregate Base (Caltrans, 2000) or approved alternate compacted to achieve at least 95 percent relative compaction and a non-yielding surface when proof-rolled with heavy construction equipment.

VI. SUPPLEMENTAL SERVICES

We must review the grading plans and specifications for site development and foundation design when they are nearing completion to confirm that the intent of our recommendations has been understood and incorporated, and to provide supplemental recommendations, if needed.

During construction, we must inspect site preparation and foundation excavations. We must verify subgrade preparation and compaction, proper moisture conditioning of soils, and fill placement and compaction. We should also inspect pavement subgrade preparation and placement and compaction of base rock materials.

LIST OF REFERENCES

American Society for Testing and Materials; 2001 Annual Book of ASTM Standards, Section 4, Construction, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics, ASTM, Philadelphia, 1999.

California Department of Conservation, Division of Mines and Geology, "Fault Activity Map of California and Adjacent Areas," 1998.

California Department of Conservation, Division of Mines and Geology, “Probabilistic Seismic Hazards Assessment” website.

California Department of Transportation, A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (Based on Maximum Credible Earthquakes), by Lalliana Mualchin, July 1996.

California Department of Conservation, California Geologic Survey, “Geologic Map of the Napa 7.5’ Quadrangle”, 2004.

California Department of Conservation, Division of Mines and Geology, “Guidelines for evaluating and Mitigating Seismic Hazards in California”, Special Publication 117, 2010.

Johnson, Michael J., “Ground Water Hydrology of the Lower Miliken-Sarco-Tulucay Creeks Area, Napa County, California,” 1977.

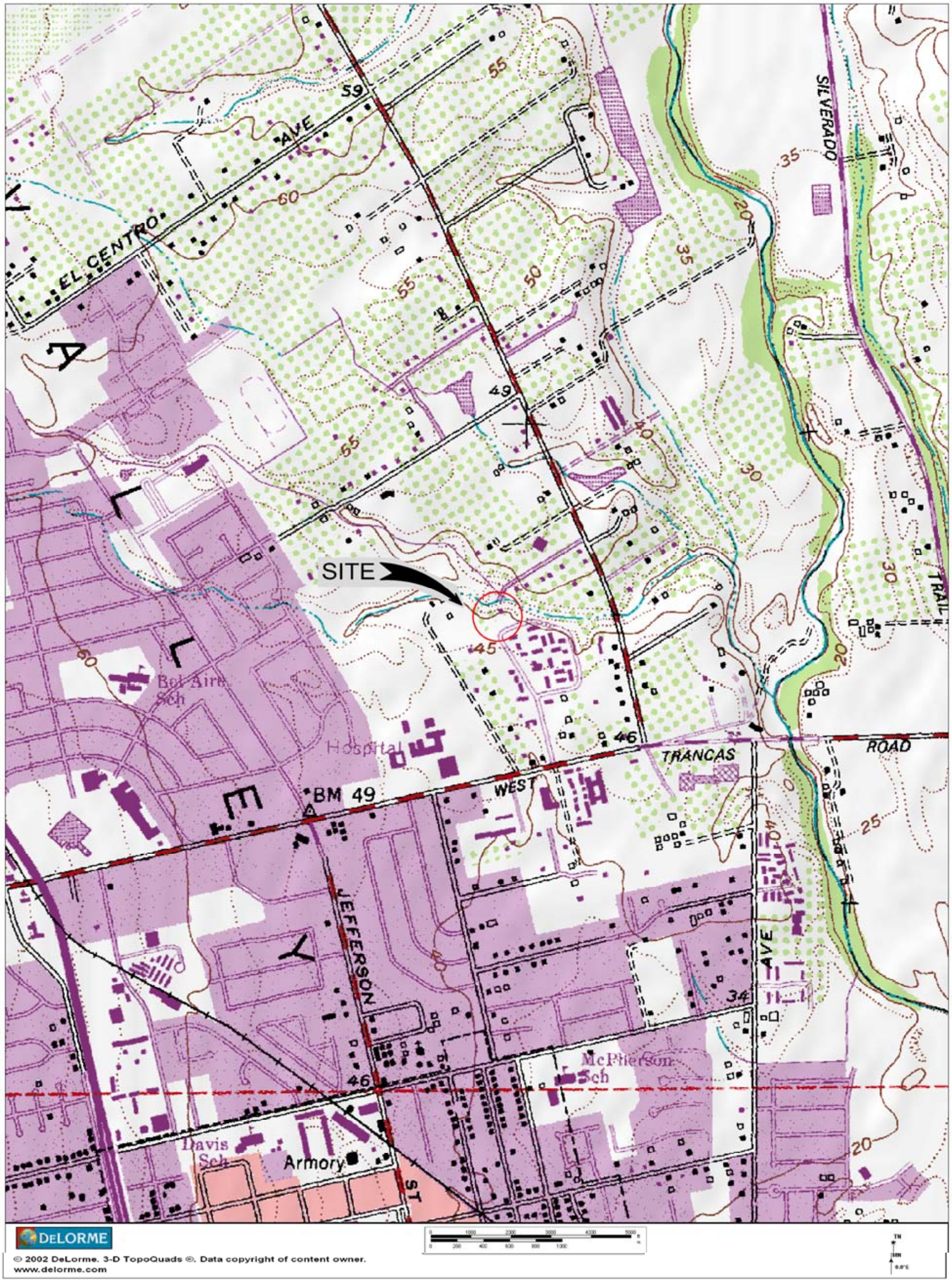
Tokimatsu, K. and Seed, H. B., 1987 “Evaluation of Settlements in Sands Due to Earthquake Shaking,” Journal of the Geotechnical Engineering Division, ASCE, Vol. 113, No. 8, August.)

U.S. Geological Survey, 7.5 Minute Quadrangle, Napa, California, 1980.

U.S. Geological Survey, Earthquake Probabilities in the San Francisco Bay Region, 2000 to 2032 – A Summary of Finding, The Working Group on California Earthquake Probabilities, Open File Report 99-517, 2002.

Wagner, D.L. and Bortungo, E.J., “Geologic Map of the Santa Rosa Quadrangle,” 1982, CDMG Regional Map Series No. 2A.

Youd, T., Hansen, C., and Bartlett, S., “Revised MLR Equations for Predicting Lateral Spread Displacement,” Proceedings, 7th U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Liquefaction, Seattle, Washington, MCEER-99-0019, 1999.



Miller Pacific
ENGINEERING GROUP

1333 N. McDowell Blvd.
Suite C
Petaluma, CA 94954
T 707 / 765-6140
F 707 / 765-6222
www.millerpac.com

FILE: 1687.01 S:LM.dwg

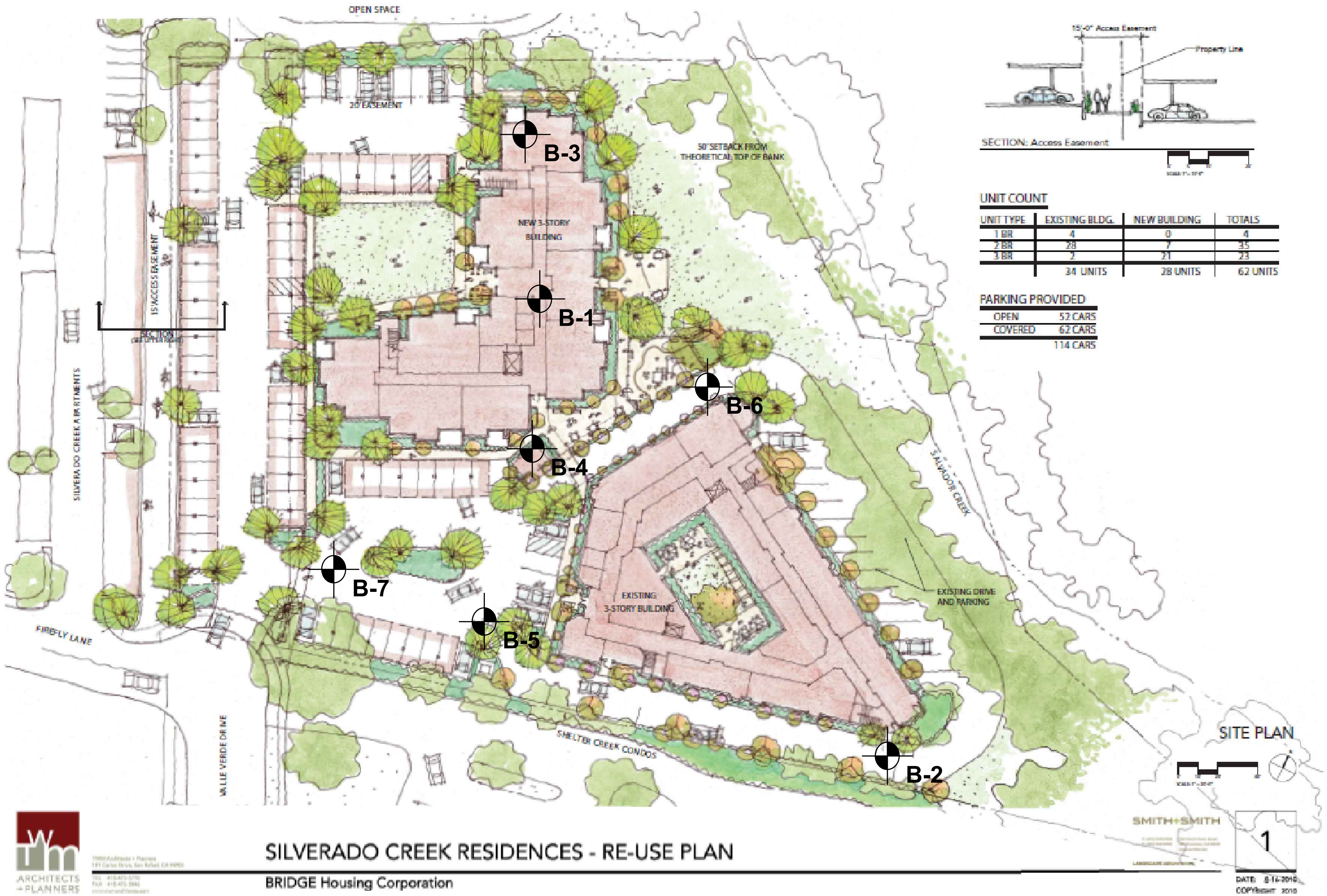
SITE LOCATION MAP

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/21/10

Drawn JBG
Checked JJR

1
FIGURE



Boring by MPEG
December 2010.

Map Reference: TWM Architects + Planners
181 Carlos Drive San Rafael, CA, 94903

SCALE
0 25 50 100 FEET

NORTH

Miller Pacific
ENGINEERING GROUP

FILE: 1687.01 SP.dwg

1333 N. McDowell Blvd.
Suite C
Petaluma, CA 94954
T 707 / 765-6140
F 707 / 765-6222
www.millerpac.com

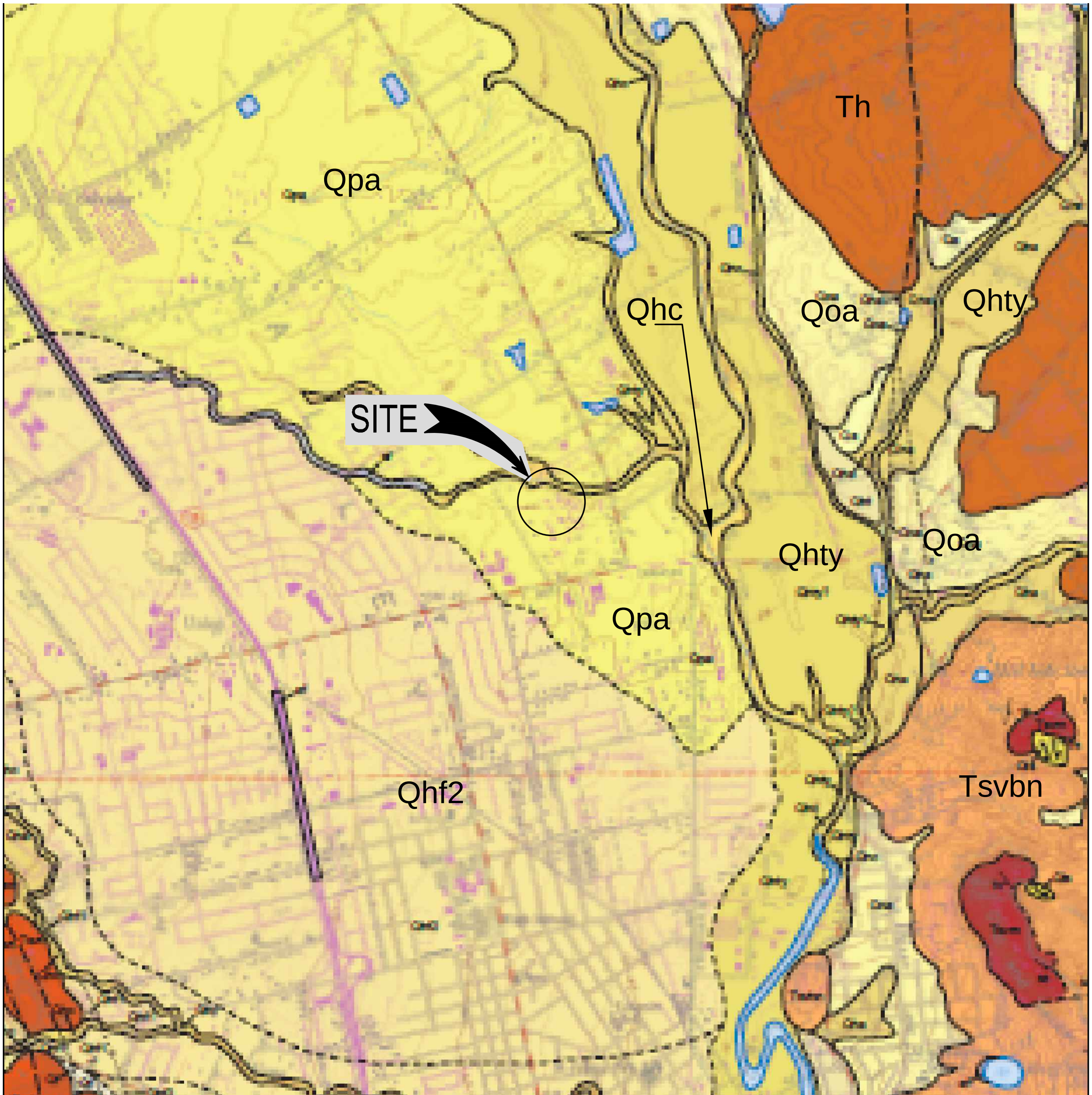
SITE PLAN

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Designed
Drawn JBG
Checked JJR

2
FIGURE



Key:

Qpa	Alluvium, undivided (latest pleistocene)- Alluvium fan, stream terrace, basin, and Channel deposites, composed of poorly to moderately sorted sand, silt, clay and gravel.
Qhty	Stream terrace deposits (Latest Holocene) Stream terrace deposited as point bar and over bank deposits along napa river,composed of moderately sorted clayey sandand sandy clay with gravel.
Qhc	Stream Channel deposits (Latest Holocene < 1,000 years) Deposits in active, natural stream, channels, consist of loose alluvial sands and silts.
Qhf2	Alluvial fan deposits (Holocene) alluvial fan sediments deposited by streams emanating from mountain drainages onto alluvial valleys, composed of moderately to poorly sorted sand, gravel, silt and clay.

Map Reference: Geological Map of the Napa 7.5' Quadrangle Napa County, CA: A Digital Database. Version 1.0, Copyright 2004. by Carlos I. Gutierrez

SCALE

0 1000 2000 4000 FEET

NORTH

Miller Pacific
ENGINEERING GROUP

FILE: 1687.01GM.dwg

1333 N. McDowell Blvd.
Suite C
Petaluma, CA 94954
T 707 / 765-6140
F 707 / 765-6222
www.millerpac.com

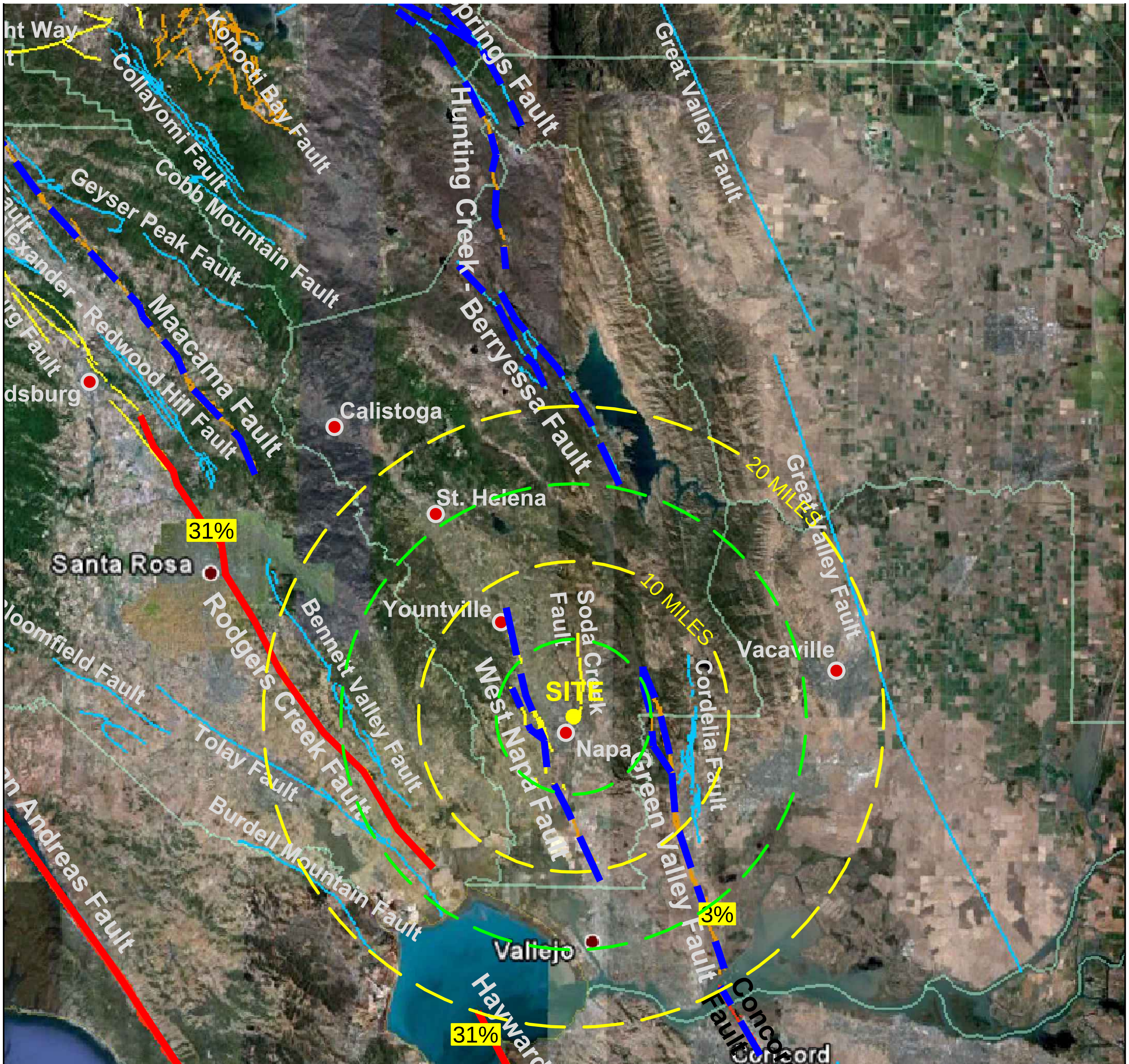
GEOLOGICAL MAP

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/21/10

Drawn JBG
Checked JJR

3
FIGURE



LEGEND

SITE: LATITUDE, 38°19' 41.40"
LONGITUDE, -122°17' 35.56"

NORTH

FAULT	TYPE	CBC DESCRIPTION
(red solid line)	"A"	CAPABLE OF LARGE MAGNITUDE EARTHQUAKES AND HIGH RATE OF SEISMIC ACTIVITY
(blue dashed line)	"B"	CAPABLE OF LARGE MAGNITUDE EARTHQUAKES OR HIGH RATE OF SEISMIC ACTIVITY

21% PROBABILITY OF M ≥6.7 BETWEEN 2008-2038 FOR FAULTS SHOWN. OVERALL PROBABILITY OF 63% IN BAY AREA OF ONE OR MORE M≥6.7 EARTHQUAKES FROM 2008-2038.

REFERENCES:

1) ACTIVE FAULT MAP MODIFIED FROM SUMMARY OF EARTHQUAKE PROBABILITIES IN THE S.F. BAY REGION, 2008-2038, THE 2007 WORKING GROUP ON CALIFORNIA EARTHQUAKE PROBABILITIES, 2008.

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 FM.dwg

ACTIVE FAULT MAP

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked JJR

4
FIGURE

APPENDIX A

SUBSURFACE EXPLORATION AND LABORATORY TESTING

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. Soil and Rock Classification Systems

We have classified soils for engineering purposes in general conformance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)" and the Unified Soil Classification System. These systems enable geotechnical engineers to correlate soil stratigraphy and compare physical soil properties. The soil classification system and symbols used for the test borings and in discussions throughout this report are briefly explained on Figure A-1, Soil Classification Chart.

B. Test Borings and Sampling

Seven test borings were excavated on December 9 and 20, 2010 at the locations shown on the Site Plan, Figure 2. The purpose of the test borings was to examine the materials encountered and obtain representative samples for laboratory testing. The test borings were excavated with a truck-mounted drill rig using 6-inch diameter solid-stem augers. The exploration was done under the technical supervision of our Field Geologist, who examined and logged the soil materials encountered and obtained samples. The subsurface conditions encountered in the test borings are summarized and presented on the Boring Logs, Figures A-2 through A-14.

The depth to groundwater, if encountered, was noted during the exploration and measured before backfilling the borings. The borings were backfilled or grouted with cement after the exploration was completed.

To obtain representative soil samples, a standard 2-1/2-inch inner-diameter "California sampler" was used. This sampler has a split barrel which contains 6-inch long thin-walled brass liner tubes and a nose with a sharpened cutting edge. The assembled sampler was lowered into the boring and driven 18 inches into the materials at the bottom of the boring with a 140-pound hammer and a 30-inch drop. The blow counts required to drive the sampler were recorded at 6-inch intervals. The total blow count for the last 12 inches is reported on the boring logs and is used as an indication of formation density or consistency. The sampler was then withdrawn from the boring and disassembled. The liners containing the soil "core" were removed, examined, trimmed and sealed with tight plastic caps to prevent moisture loss.

C. Laboratory Testing

We re-examined the samples in the laboratory to confirm field classification and suitability for testing. We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate physical engineering properties. The following laboratory tests were conducted in general accordance with ASTM standard test methods modified as appropriate for local conditions and practice to provide the data needed for our engineering judgment:

- Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937; and,
- Plasticity Index, ASTM D 4318.

The test results are reported on the boring logs, Figures A-2 through A-14 and Plasticity Chart, Figure A-15.

The boring logs, description of soils encountered, and the laboratory test data reflect conditions only at the location of the boring or sampling at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate, and changes in surface and subsurface drainage.

MAJOR DIVISIONS		SYMBOL	DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVEL with fines	GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
	SAND with fines	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <50%	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, muck, and other highly organic soils
ROCK			Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

PI PLASTICITY INDEX
LL LIQUID LIMIT
SA SIEVE ANALYSIS
HYD HYDROMETER ANALYSIS
P200 PERCENT PASSING NO. 200 SIEVE
P4 PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

- MODIFIED CALIFORNIA
- HAND SAMPLER
- STANDARD PENETRATION TEST
- ROCK CORE
- THIN-WALLED / FIXED PISTON
- X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

STRENGTH TESTS

TV FIELD TORVANE (UNDRAINED SHEAR)
UC LABORATORY UNCONFINED COMPRESSION
TXCU CONSOLIDATED UNDRAINED TRIAXIAL
TXUU UNCONSOLIDATED UNDRAINED TRIAXIAL
UC, CU, UU = 1/2 Deviator Stress

SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25 sampler driven 12 inches with 25 blows after initial 6-inch drive

85/7" sampler driven 7 inches with 85 blows after initial 6-inch drive

50/3" sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

Miller Pacific
ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

SOIL CLASSIFICATION

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-1
FIGURE

BORING 1

EQUIPMENT: Truck Mounted B-53 Drill Rig with 6in. Auger.

DATE: 12/9/10

ELEVATION: 39-Feet*

*REFERENCE: Google Earth Used For Elevation

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	DESCRIPTION
						0	0	ASPHALTIC CONCRETE 2in.
								AGGREGATE BASEROCK 4in.
								SANDY CLAY (CL) Dark brown, dry to moist, stiff, low to medium plasticity clay, ~20-30% fine to medium sand. [Alluvium]
		2130 UC	29	14.8	104			
						1		
PI			16					
							5	
						2		SILTY CLAY (CL) Gray to medium brown, moist, stiff, low to medium plasticity clay, trace of fine sand and rounded fine gravel. [Alluvium]
		2810 UC	28	17.8	111			
			14					
						3	10	
						4		
							15	
	P200 50%		10					CLAYEY SAND WITH GRAVEL (SC) Dark to medium brown, moist, loose to medium dense, fine to coarse subrounded sand,low to medium plasticity clay~10-15% fine subrounded gravel. [Alluvium]
						5		
								CLAY (CL) Medium brown to gray, moist, medium stiff, low to medium plasticity clay, trace fine sand. [Alluvium]
						6	20	

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Drawn JBG
Checked

Project No. 1687.01 Date: 12/22/10

A-2
FIGURE

BORING 1
(CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	
							20			CLAY (CL) Gray to medium brown, saturated, medium stiff to stiff, medium plasticity clay, trace fine sand.[Alluvium]
			10			7				
							25			
						8				
										Grades to ~10-15% fine sand.
	P200 87.0%		17							
										Bottom of boring at 28.5 feet. Groundwater encountered at 16 feet.
						9				
							30			
						10				
							35			
						11				
						12				
							40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-3
FIGURE

BORING 2

EQUIPMENT: Truck Mounted B-53 Drill Rig with 6in. Auger.

DATE: 12/20/10

ELEVATION: 40-Feet*

*REFERENCE: Google Earth Used For Elevation

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			ASPHALTIC CONCRETE 3in.
										AGGREGATE BASEROCK 9in.
		1340 UC	32	16.4	113	1	5			SANDY CLAY WITH GRAVEL (CL) Mottled medium brown, moist, stiff, low to medium plasticity clay, ~30-40% fine to coarse sand, ~5-10% fine subrounded gravel. [Fill]
			20			2, 3	10			CLAYEY SAND/CLAYEYGRAVEL (SC/GC) Medium brown, saturated, medium dense to dense, fine to coarse subangular sand, ~20-25% fine to medium subrounded gravel, ~15% low to medium plasticity clay. [Alluvium/Stream Deposit]
	P200 54%		11			4	15			SANDY CLAY (CL) Medium brown, saturated, medium stiff, medium plasticity clay, ~45% fine to medium sand. [Alluvium]
						5				CLAYEY SAND WITH GRAVEL (SC) Dark to medium brown, moist, medium dense, fine to coarse subrounded sand, ~20-30% low to medium plasticity clay,~15% fine subrounded gravel. [Alluvium]
						6	20			SANDY CLAY (CL) Medium brown, saturated, medium stiff, medium plasticity clay, ~35-45% fine to medium sand. [Alluvium]

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-4
FIGURE

BORING 2 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH feet	DEPTH meters	SAMPLE	SYMBOL (3)	DESCRIPTION
						20				SANDY CLAY (CL) Medium brown to gray, saturated, medium stiff, medium plasticity clay, ~25-35% fine to medium sand. [Alluvium]
			10							
							7			
						25				Grades to medium brown to orangish gray, ~10-20% fine sand at 25 feet.
			10				8			
							9			Grades to gray at 29 feet.
						30				
			13							
										Bottom of boring at 31.5 feet. Groundwater encountered at 10 feet.
							10			
						35	11			
							12			
						40				

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

BORING 3

EQUIPMENT: Truck Mounted B-53 Drill Rig with 6in. Auger.

DATE: 12/9/10

ELEVATION: 40-Feet*

*REFERENCE: Google Earth Used For Elevation

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			ASPHALTIC CONCRETE 2in.
										AGGREGATE BASEROCK 4in.
			26	10.4	113					SANDY CLAY (CL) Medium brown, moist, soft to medium stiff, medium plasticity clay, ~20-30% fine sand. [Possible Fill]
			10			1	5			CLAYEY SAND WITH GRAVEL (SC) Medium brown to mottled dark brown, moist, medium dense, fine to coarse angular sand, ~25-35% fine to medium subangular gravel, ~10-20% low to medium plasticity clay. [Fill]
		1310 UC	10	31.7	93	2				SILTY CLAY WITH SAND (CL) Medium brown to gray, moist, medium stiff, medium plasticity clay, ~10-20% fine sand. [Alluvium]
			7			3	10			
						4				
	P200 22%		12			5	15			POORLY GRADED SAND WITH CLAY(SC) Medium brown, saturated, medium dense, fine to coarse subrounded sand, ~10-20% low to medium plasticity clay. [Alluvium]
						6	20			CLAY (CL) Medium brown to gray, moist, medium stiff, medium plasticity clay, trace fine sand. [Alluvium]

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-6
FIGURE

BORING 3 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						20			CLAY (CL)
			16						Medium brown to gray, moist, stiff, medium plasticity clay, trace fine sand. [Alluvium]
						7 (m)			
						25			Grades to gray to mottled brown, ~20-30% fine sand at 25.5 feet.
	P200 73%		8			8 (m)			
						9 (m)			
						30			
			12						
									Bottom of boring at 31.5 feet. Groundwater encountered at 15 feet.
						10 (m)			
						35			
						11 (m)			
						12 (m)			
						40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Drawn JBG
Checked

Project No. 1687.01 Date: 12/22/10

A-7
FIGURE

BORING 4

EQUIPMENT: Truck Mounted B-53 Drill Rig with 6in. Auger.

DATE: 12/9/10

ELEVATION: 43-Feet*

*REFERENCE: Google Earth Used For Elevation

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			CLAYEY SILT WITH SAND (ML) Medium brown, moist, medium dense fine to coarse sand, ~15-25% low to medium plasticity clay, ~5-10% fine gravels, trace fine rootlets. [Fill/Topsoil]
		1230 UC	17	10.4	98					
			17			1				CLAYEY SAND (SC) Mottled brown to gray, moist, medium dense, fine sand, ~30% medium plasticity clay. [Alluvium]
							5			
		5290 UC	68	18.3	107	2				SILTY CLAY (CL) Mottled orange to gray, moist, very stiff, slightly cemented, medium plasticity clay. [Alluvium]
			33							
						3	10			
						4				
							15			CLAYEY SAND WITH GRAVEL (SC) Medium reddish brown, moist to saturated, medium dense to dense, fine to coarse subrounded sand, low to medium plasticity clay, ~15-25% fine subrounded gravels. [Alluvium]
	P200 15%		23			5				
										CLAY WITH SAND (CL) Medium brown to gray, moist, medium stiff, medium plasticity clay, ~10-20% fine sand. [Alluvium]
						6	20			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

FILE: 1687.01 BL.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-8
FIGURE

BORING 4 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
							20			CLAY WITH SAND (CL)
	P200 80%		12							Medium brown to gray, moist, medium stiff, medium plasticity clay, ~10-20% fine sand. [Alluvium]
						7				Bottom of boring at 21.5 feet. Groundwater encountered at 13 feet.
							25			
						8				
						9				
							30			
						10				
							35			
						11				
						12				
							40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Drawn JBG
Checked

Project No. 1687.01 Date: 12/22/10

BORING 5

EQUIPMENT: Truck Mounted B-53 Drill Rig with 6in. Auger.

DATE: 12/20/10
ELEVATION: 41-Feet*
*REFERENCE: Google Earth Used For Elevation

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			ASPHALTIC CONCRETE 3in.
										AGGREGATE BASEROCK 9in.
			22			1				CLAYEY SAND WITH GRAVEL (SC) Medium brown, moist, medium dense to dense, fine to coarse subangular sand, ~20-30% medium plasticity clay, ~10-20% fine to medium subrounded gravel. [Alluvium]
							5			
			23	14.6	117	2				GRAVELLY SAND WITH CLAY (SC) Medium brown, moist, medium dense to dense, fine to coarse subrounded sand, ~ 30-40% fine to medium subrounded gravel, ~10-15% low to medium plasticity clay. [Alluvium]
						3	10			CLAY WITH SAND (CL) Medium brown to gray, moist to saturated, medium stiff, medium plasticity clay, ~10-15% fine sand. [Alluvium]
			12							
						4				
			11				15			Grades to medium brown to orange at 15 feet.
						5				
						6	20			Grades to gray at 20 feet.

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-10
FIGURE

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	BORING 5 (CONTINUED)
			16			20			CLAY WITH SAND (CL) Medium gray, moist to saturated, stiff, medium plasticity clay, ~10-15% fine sand. [Alluvium]
						7			Bottom of boring at 21.5 feet. Groundwater encountered at 11 feet.
						25			
						8			
						9			
						30			
						10			
						35			
						11			
						12			
						40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-11
FIGURE

BORING 6

EQUIPMENT: Truck Mounted B-53 Drill Rig with 6in. Auger.

DATE: 12/20/10

ELEVATION: 40-Feet*

*REFERENCE: Google Earth Used For Elevation

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0 / 0			ASPHALTIC CONCRETE 3in.
									AGGREGATE BASEROCK 9in.
									SANDY CLAY (CL) Medium brown, moist, medium stiff to stiff, medium plasticity clay, ~25-35% fine to medium sand,trace fine gravel. [Fill]
		1190 UC	19	21.5	104	1 / 5			SANDY CLAY (CL) Dark brown, moist, medium stiff to stiff, low plasticity clay, ~30-40% fine to medium sand. [Fill]
	P200 21%		13			2 / 3 / 10			CLAYEY SAND (SC) Medium brown to orange, wet, medium dense, fine sand, ~20-30% low plasticity clay. Disturbed sample? [Alluvium/Stream Deposit]
			12			4 / 15			
						5 / 6 / 20			CLAY WITH SAND (CL) Medium gray to brown, moist to saturated, medium stiff, medium plasticity clay, ~10-20% fine sand. [Alluvium]

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-12
FIGURE

BORING 6 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
							20			CLAY WITH SAND (CL)
			21							Medium gray to brown, moist to saturated, stiff, medium plasticity clay, ~10-20% fine sand. [Alluvium]
						7				Bottom of boring at 21.5 feet. Groundwater encountered at 10 feet.
							25			
						8				
						9				
							30			
						10				
							35			
						11				
						12				
							40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-13
FIGURE

BORING 7

EQUIPMENT: Truck Mounted B-53 Drill Rig with 6in. Auger.

DATE: 12/20/10

ELEVATION: 43-Feet*

*REFERENCE: Google Earth Used For Elevation

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	DESCRIPTION
						0	0	ASPHALTIC CONCRETE 3in.
								AGGREGATE BASEROCK 9in.
			23					CLAYEY SAND WITH GRAVEL (SC) Medium brown to orange, moist, medium dense, fine to coarse sand, ~15-25% medium plasticity clay, ~20-30% fine to medium subangular gravel. [Alluvium]
						1		
			30				5	CLAYEY SAND (SC) Medium brown, moist to saturated, dense, fine to coarse subrounded sand, ~10% fine to coarse subangular gravel, ~10-20% low plasticity clay. [Alluvium]
						2		
			55					
						3	10	Bottom of boring at 10 feet. Groundwater at 6 feet.
						4		
							15	
						5		
						6	20	

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

Miller Pacific
ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILE: 1687.01 BL.dwg

BORING LOG

Bridge Housing Valle Verde Dr.
Napa, California

Project No. 1687.01 Date: 12/22/10

Drawn JBG
Checked

A-14
FIGURE

MPEG **MILLER PACIFIC ENGINEERING GROUP**

April 8, 2019
File: 1687.010cltr.doc

Burbank Housing Development Corporation
790 Sonoma Avenue
Santa Rosa, California 95404

Attention: Marianne Lim, Director of Housing

Re: Addendum to Updated Geotechnical Report Dated January 29, 2019
Clarification of Report Findings
Valle Verde and Heritage House Projects
3700 and 3710 Valle Verde Drive
Napa, California

Introduction

The purpose of this letter is to clarify some of the geotechnical findings as summarized in the Miller Pacific Engineering Group Updated Geotechnical Report (January 29, 2019) for the proposed Valle Verde project at 3710 Valle Verde Drive and the proposed Heritage House project at 3700 Valle Verde Drive in Napa, California.

Section 4.4 of January 29, 2019 Updated Geotechnical Report, Liquefaction Potential and Related Impacts, Page 6.

Based on exploratory borings and laboratory testing conducted in 2010 and 2018, we concluded that seismic induced liquefaction and liquefaction induced ground surface settlement is a moderate risk at the project site. The report provided an estimate for post liquefaction differential vertical settlement of approximately one inch over a horizontal distance of 30 feet.

The report noted that a potential impact of liquefaction is lateral spreading, which can occur if relatively continuous layers of liquefiable soil and sloping terrain are both present.

In our opinion, the risk that liquefaction induced lateral spreading will occur at the project site during a seismic event is very low. Based on exploratory borings drilled in 2010 and 2018, the potentially liquefiable layers are isolated, relatively thin, and are not continuous. The liquefiable layers are interrupted by clayey deposits and confined by clayey deposits. Therefore, the risk of liquefaction induced lateral spreading toward the creek channel slope is considered to be very low.

Section 4.6 of January 29, 2019 Updated Geotechnical Report, Lurching and Ground Cracking, Page 8.

The wording in Section 4.6 of the report is unclear. The wording, as given below, conveys the intended meaning more clearly.

504 Redwood Blvd., Suite 220 ■ Novato, California 94947 ■ T (415) 382-3444 F (415) 382-3450

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes, such as creek channel slopes, where stiff soils are underlain by soft deposits. This combination of conditions, including stiff soil underlain by soft soil in proximity to slopes, does not exist at the project site. Therefore, the risk of lurching and ground cracking at the project site is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

We are pleased to have been of service to you. If you have any questions, please call us at your convenience.

Very truly yours,
MILLER PACIFIC ENGINEERING GROUP

REGISTERED PROFESSIONAL ENGINEER
DANIEL S. CALDWELL
No.002006
Exp. 9/30/19
GEOTECHNICAL
STATE OF CALIFORNIA

Daniel S. Caldwell
Geotechnical Engineer No. 2006
(Expires 9/30/19)