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GEOTECHNICAL INVESTIGATION
VARTNAW ESTATES
PETALUMA, CALIFORNIA

August 31, 2020

Job No. 2211.003

Prepared For:
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CERTIFICATION

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TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
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1.0 INTRODUCTION

This report presents the results of our Geotechnical Investigation for the Vartnaw Estates subdivision planned in southern Petaluma. As shown on the Site Location Map, Figure 1, the project area is located just east of McNear Avenue and south of Petaluma Boulevard.

Our work was performed in accordance with our Agreement for Professional Services authorized on July 14, 2020. We previously prepared a Geotechnical Feasibility Evaluation (Miller Pacific, 2018) which addressed potential geotechnical and geologic issues and provided preliminary geotechnical recommendations for using in planning and permitting. The purpose of our investigation was to explore subsurface conditions within the proposed project area and to develop geotechnical recommendations and criteria for use in design and construction of the project. The scope of our services includes:

- Reviewing published geologic and geotechnical background information.
- Exploring subsurface conditions with eight borings located within the general vicinity of the planned structures, roadways and related improvements.
- Laboratory testing to estimate pertinent engineering properties of the soils encountered during our subsurface exploration.
- General evaluation and discussion of relevant geologic hazards including seismic shaking, expansive soils, liquefaction, and other hazards.
- Engineering analyses to develop geotechnical recommendations and design criteria related to seismic design, foundations, site grading, retaining walls, new pavements and concrete flatwork and other geotechnical-related items.
- Preparation of this Geotechnical Investigation report which summarizes the subsurface exploration and laboratory testing programs, evaluation of relevant geologic hazards, and geotechnical recommendations and design criteria.

Issuance of this report completes our current phase of services. Subsequent phases of work should include geotechnical plan review and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

Based on our review of preliminary plans (WHA, 2020), the proposed development includes constructing 15 single-family residences and 52 multi-family townhome and residential flats. Detached accessory dwelling units (ADUs) are also planned within the backyard of several of the single-family homes along the east side of the site. While grading plans and detailed structural information are not yet available, the new structures are expected to be two to three stories in height and will impose relatively light to moderate foundation loads. Site improvements will also include constructing new roadways, parking areas and utilities. Site grading is expected include cuts and fills of a few feet to create level building pads for the new structures, to construct new roadways and parking areas, and to develop appropriate surface drainage patterns. Ancillary improvements will likely include new concrete flatwork, site drainage, landscaping, a common park/open space area, bioretention areas and other improvements. The proposed improvements are shown on the Site Plan, Figure 2.

3.0 SITE CONDITIONS

3.1 Regional Geology

The project site lies within the Coast Ranges geomorphic province of California. Regional topography within the Coast Ranges province is characterized by northwest-southeast trending mountain ridges and intervening valleys that parallel the major geologic structures, including the San Andreas Fault System. The province is also generally characterized by landsliding and erosion owing in part to its typically high levels of precipitation and seismic activity.

The oldest rocks in Sonoma County are the sedimentary, igneous, and metamorphic rocks of the Mesozoic-age (225- to 65-million years old) Franciscan Assemblage. Within Sonoma County, Franciscan rocks are in fault contact with marine sedimentary rocks of the Great Valley Sequence, which are of similar age. Locally, a variety of sedimentary and volcanic rocks of Tertiary (1.8- to 65-million years old) and Quaternary (less than 1.8-million years old) age overlie the basement rocks of the Franciscan Assemblage and Great Valley Sequence. The late Miocene to Pliocene-age (approximately 2.6- to 11.6-million years old) Sonoma Volcanics comprise the majority of these rocks.

The project site is located within relatively level to gently sloping terrain, approximately 800 feet south of the Petaluma River. Regional geologic mapping by the California Geological Survey (CGS, 2002) indicates the majority of the project site is underlain by Holocene-age, alluvial fan deposits which generally consist of sand, gravel, silt and clay deposited by streams. The mapping further indicates that the northernmost portion of the site is underlain by schist, phyllite and semischist of the Jurassic- to Cretaceous-age Franciscan Complex. A Regional Geologic Map and descriptions of the mapped geologic units are shown on Figure 3.

3.2 Seismicity

The project site is located within the seismically active San Francisco Bay 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 they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust but are typically comprised of localized shear zones which link together to form larger fault zones. 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 becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

3.2.1 Regional Active Faults

The California Geological Survey (previously known as the California Division of Mines and Geology), defines a "Holocene-active fault" as one that has had surface displacement within Holocene time (the last 11,700 years). CGS has mapped various faults in the region as part of their Fault Activity Map of California (CGS, 2010). Many of these faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known Holocene-active faults are the Rodgers Creek and San Andreas Faults. The Rodgers Creek Fault is located roughly 8.2 kilometers (5.1 miles) northeast of the site, while the San Andreas Fault is located approximately 24.4 kilometers (15.2 miles) to the southwest[1].

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. The results of our USGS earthquake search catalogue indicates that at least ten earthquakes with a Richter Magnitude of 5.0 or larger have occurred within 100 kilometers (62 miles) of the site between 1900 and 2020. The approximate locations of many of these and other earthquakes are shown on the Historic Earthquake Map, Figure 5.

3.2.3 Probability of Future Earthquakes

The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. 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, 2008, 2013) 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. 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.

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[1] Distances to faults estimated using Caltrans ARS Online (v2.3.09), accessed January 18, 2018.

Conclusions from the most recent UCERF3 and USGS indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system. The Rodgers Creek Fault is located approximately 8.2 kilometers (5.1 miles) northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 24.4 kilometers (15.2 miles) southwest of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. 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.

3.3 Surface Conditions

The project site encompasses an approximately 4.5-acre, roughly rectangular-shaped property which comprises seven separate parcels (APN's 019-210-021, 022, 025, 029, 032, 033 and 034). The property is bordered to the north by Petaluma Boulevard, to the west by McNear Avenue, and to the south and east by private residences. Two residences exist within the property along the McNear Avenue frontage and a separate residence exists near the north end of the property adjacent to Petaluma Boulevard. Several sheds and other outbuildings are located behind the existing residences. The ground surface is level to gently sloping to the northeast with surface elevations ranging from about 20 to 40 feet[2]. The ground surface is largely covered with mature trees, blackberry bushes, low-lying grasses and other shrubbery. Several small mounded areas also exist toward the central part of the site behind the residences

3.4 Field Exploration and Laboratory Testing

We explored subsurface conditions near the proposed improvements on July 29 and 30, 2020 with eight borings at the approximate locations shown on Figure 2. The borings were excavated using track-mounted drilling equipment to approximate depths ranging from 10 to 20 feet below ground surface. The borings were logged by our Field Engineer and samples were obtained for classification and laboratory testing. We prepared boring logs based on soil and rock descriptions in the field, as well as visual examination and testing of the soil and rock samples in our laboratory. The boring logs are presented in Appendix A.

Laboratory testing of soil samples from the exploratory borings included determination of moisture content, dry density, unconfined compressive strength, Atterberg limits and Expansion Index. The results of our laboratory tests are presented on the boring logs with the exception of the Atterberg limits and Expansion Index test results which are presented on Figures A-11 and A-12, respectively. Our laboratory testing program is discussed in greater detail in Appendix A.

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[2] Surface elevations based on contours obtained from Sonoma Veg Map accessed on January 18, 2019. Elevations referenced herein are based on NAVD 88.

3.5 Subsurface Conditions & Groundwater

Based on our field exploration, subsurface conditions are generally consistent with the regional geologic mapping and consist of alluvial soils over Franciscan bedrock. The alluvial soils are about three- to eight-feet-thick and primarily consist of stiff to very stiff sandy clay. Laboratory testing and visual classification of samples collected during our subsurface exploration indicate the clayey soils exhibit low to high plasticity. Expansion index testing of the higher plasticity soils indicates the soils also exhibit a "very high" expansive potential. While not definitively identified in our borings, it is possible that some of the near-surface soils consist of fill which was derived from native soils that were reworked during previous site grading. The underlying Franciscan bedrock generally consists of schist and meta-schist that are highly to completely weathered and exhibit low to moderate hardness and strength.

With the exception of Borings 1 and 7, groundwater was encountered in the borings at approximate depths ranging from 7 to 15 feet. The borings were excavated in late July several months after any significant rainfall events. Because the borings were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. Groundwater elevations fluctuate seasonally and higher groundwater levels may be present during or following periods of intense rainfall. Perched water tables may also exist within the soil and bedrock materials.

4.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards and discusses their potential impacts on the planned improvements. The primary geologic hazards which could affect the proposed development include strong seismic ground shaking and potentially expansive near-surface soils. Other geologic hazards are judged less than significant with regard to the proposed project. Each significant geologic hazard considered is discussed in further detail in the following paragraph.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest Alquist-Priolo Earthquake Fault Zone is associated with the Rodgers Creek Fault located approximately 8.2 kilometers (5.1 miles) to the northeast. Based on currently available published geologic information, the site is not located within an Alquist-Priolo Earthquake Fault Zone. We therefore judge the potential for fault surface rupture in the development area to be low.

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

4.2 Seismic Shaking

The site 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 are often used for residential developments.

Deterministic methods use empirical attenuation relations that 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 1. The calculated accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Table 1 – Estimated Peak Ground Accelerations for Principal Active Faults

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (km)	Median PGA (g)	Median PGA + 1 Std Dev (g)
Rodgers Creek	7.3	8.2	0.35	0.63
San Andreas	8.0	24.4	0.23	0.41
Hayward	7.3	29.4	0.14	0.26
Maacama	7.4	34.0	0.13	0.24
San Gregorio	7.4	36.6	0.12	0.23

Reference: Abrahamson & Silva, Boore & Atkinson, Campbell & Bozorgnia, and Chiou & Youngs, 2014 using V_{s30} = 560 m/s.

The calculated bedrock accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations. Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements (such as light fixtures, shelves, cornices, etc.) to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the most recent version of the California Building Code (2019 CBC) should result in structures that do not collapse in an earthquake. Damage may still occur and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to their proximity and historic rates of activity, the Rodgers Creek and San Andreas 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.

Evaluation: *Less than significant with mitigation.*
Recommendation: *Design new structures in accordance with the provisions of the 2019 California Building Code or subsequent codes in effect when final design*

occurs. Recommended seismic design coefficients and spectral accelerations are presented in Section 5.1 of this report.

4.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. The strength loss occurs as a result of the build-up of excess pore water pressures and subsequent reduction of effective stress. While liquefaction most commonly occurs in saturated, loose, granular deposits, recent studies indicate that it can also occur in materials with relatively high fines content provided the fines exhibit lower plasticity. The effects of liquefaction can vary from cyclic softening resulting in limited strain potential to flow failure which cause large settlements and lateral ground movements.

Regional liquefaction hazard maps indicate the site is mapped within a zone of “moderate” susceptibility to liquefaction (Association of Bay Area Governments, 2018), as shown on Figure 6. The alluvial fan deposits encountered predominantly stiff to very stiff clayey soils over relatively shallow bedrock which are not susceptible to liquefaction. Therefore, we judge there is generally a low risk of liquefaction during future seismic events.

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

4.4 Seismic Densification

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. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Loose, granular soils were not encountered in our borings. Therefore, we judge the likelihood of damage to the new structures due to seismically induced settlement is low.

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

4.5 Expansive Soils

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, uneven floors, and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures. Laboratory testing and visual classification of samples collected during our subsurface exploration indicate near-surface soils are clayey and exhibit low to high plasticity. Expansion index testing of the higher plasticity soils indicates the soils also exhibit a “very high” expansive potential.

Evaluation: Less than significant with mitigation.

Recommendation: *As a minimum, soils should be moisture conditioned to slightly above the optimum moisture content during site grading and maintained at this moisture content until imported aggregate base and/or surface flatwork is completed to "seal" in the higher moisture content and therefore reduce the expansive potential. Removing and replacing expansive soils with non-expansive fill or soil improvement using lime or cement may also be considered to mitigate expansive soils and to reduce the thickness of pavement sections. Additionally, building foundations and slab floors should be designed to account for some expansive soil movement as discussed under Sections 5.3 and 5.5.*

4.6 Settlement

Significant settlement can occur when new loads are placed over soft, compressible clays (e.g. Bay Mud) or loose soils. The site is underlain by stiff to very stiff clayey soils and relatively shallow bedrock and settlement is not considered a significant hazard. Site grading will likely include cuts and fills of a few feet. Some differential settlement could occur where new structures cross cut/fill transitions due to variations in material properties between the native soil/bedrock and new fill. Provided the thickness of new fills is less than about five feet, we estimate the magnitude of differential settlement across the building pad for new structures that cross cut/fill transitions would be less than one inch.

Evaluation: *Less than significant with mitigation.*

Recommendation: *The thickness of new fills should be minimized to the extent possible as to reduce the potential for differential settlements for structures sited over cut/fill transitions. The foundations for new structures should be designed in accordance with the recommendations in Section 5.3*.

4.7 Erosion

Sandy soils on most slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity. The work area is relatively level and it is anticipated that much of the site will be covered with new buildings, pavements, or concrete flatwork. Therefore, erosion is not considered to be a significant long-term geologic hazard. However, care should be taken during construction to prevent excess erosion when the soils are exposed.

Evaluation: *Less than significant with mitigation.*

Recommendation: *The site drainage system should be designed to collect surface water and discharge it into an established storm drainage system. The project Civil Engineer is responsible for designing the site drainage system. An erosion control plan should be developed prior to construction per the City of Petaluma's current guidelines*.

4.8 Flooding

The project site is located at elevations ranging from about 20 to 40 feet above sea level and is not mapped within a FEMA-designated Special Flood Hazard Area (Federal Emergency Management Agency, 2015). Therefore, large scale flooding is not considered a significant hazard at the project site. The project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation measures.

Evaluation: Less than significant. No mitigation measures are required

4.9 Tsunami/Seiche

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 would be dependent upon ground motions and fault offset from nearby active faults. While the project site is roughly 800 feet south of the Petaluma River, it is at an elevation of about 20 to 40 feet and is not mapped within a designated Tsunami Inundation Area (California Geological Survey, 2009). Therefore, the risk of tsunami inundation following a future seismic event is low.

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

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration, we judge that construction of the proposed development is feasible from a geotechnical standpoint. Primary geotechnical considerations for the project will include providing uniform foundation support for the new structure and designing new structures to accommodate potentially expansive soil conditions and to resist strong seismic ground shaking. Additional discussion and recommendations addressing these and other considerations are presented in the following sections.

5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2019) 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 of several nearby faults, we recommend the CBC coefficients and site values shown in Table 2 be used to calculate the design base shear of the new construction.

Table 2 – 2019 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	C
Site Latitude	38.227°N
Site Longitude	-122.622°W
Spectral Response (short), S_S	1.50 g
Spectral Response (1-sec), S_1	0.60 g
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.4
Spectral Response (Short), S_{MS}	1.80 g
Spectral Response (1 sec), S_{M1}	0.84 g
Design Spectral Response (short), S_{DS}	1.20 g
Design Spectral Response (1 sec), S_{D1}	0.56 g
MCE_G PGA Adjusted, PGA_M	0.75 g

Reference: ATC Hazard by Location, accessed on August 14, 2020.

5.2 Site Grading

Site grading is expected to include cuts and fills of a few feet to create level building pads for the new structures, to construct new driveways and parking areas, and to develop appropriate surface drainage patterns. Site grading should be performed in accordance with the recommendations and criteria outlined in the following sections.

5.2.1 Site Preparation

Clear pavements, old foundations, over-sized debris, and organic material from areas to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site. Trees that are located within the building areas should be removed and the root systems excavated. Existing foundations and utilities which are to be abandoned as part of the work should be removed from structural areas. In non-structural areas, utilities could be abandoned in place in many cases provided cement grout completely fills any void in the utility.

Where fills or other structural improvements are planned on level ground, the subgrade surface should be scarified to a depth of eight inches, moisture conditioned to at least two percent above the optimum moisture content and compacted to at least 90 percent relative compaction. Subgrade preparation should extend a minimum of five feet beyond the planned building envelopes in all directions. The subgrade should be firm and unyielding when proof-rolled with heavy, rubber-tired construction equipment. If soft, wet or otherwise unsuitable materials are encountered at subgrade elevation during construction, we will provide supplemental recommendations to address the specific condition.

5.2.2 Excavations

Site excavations for new foundations, grading, utilities, and other improvements will generally encounter predominantly stiff to very stiff clayey soils and shallow schist bedrock. In unsupported excavations, the clayey soils and bedrock are expected to exhibit firm behavior. Temporary (steeper) cut slopes may be required during construction. For planning purposes, cut slopes into these materials should be designed for an OSHA Type B soil. Permanent cut slopes should be inclined no steeper than 2:1 (horizontal:vertical).

Based on our subsurface exploration, we judge the majority of site excavation can be performed with conventional equipment, such as medium-size dozers and excavators. However, Franciscan bedrock often contains inclusions and zones of harder, more resistant rock which may require specialized techniques or equipment to excavate (e.g., jackhammers or hydraulic breakers). Therefore, we recommend inclusion of a line item and clear definition for “hard rock excavation” in the project bid documents. If hard rock is encountered during construction which prohibits excavation to the required depths, we should be consulted to observe conditions and revise our recommendations and/or design criteria as appropriate. Reducing planned excavation depths will also reduce the potential for hard rock excavation and resulting costs.

5.2.3 Fill Placement and Compaction

Fill materials should generally consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 45 (ASTM D 4318), a Plasticity Index of less than 20 (ASTM D 4318), and a minimum R-value of 20 (California Test 301). The fill material should contain no more than 50 percent of particles passing a No. 200 sieve and should have a maximum particle size of four inches. Onsite soils may be used for fill provided that they meet the criteria described above. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to at least two percent above the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of eight-inches-thick or less and uniformly compacted to at least 90 percent relative compaction. Where fill thicknesses are greater than five feet, fill materials should be compacted to at least 92 percent relative compaction. In pavement areas, the upper 12 inches of fill should be compacted to at least 95 percent relative compaction. The maximum dry density and optimum moisture content of fill materials should be determined in accordance with ASTM D1557.

5.2.4 Expansive Soil Mitigation

As previously discussed, near-surface soils are predominantly clayey and are potentially expansive. While visual classification and the results of our laboratory testing suggest the expansive potential of the near-surface soils is variable, the new structures and other improvements should be designed to mitigate potentially expansive conditions. As a minimum, building pads should be thoroughly moisture conditioned to at least two percent above the optimum moisture content and maintained at this moisture content until the residence, hardscape and other improvements are constructed. Building foundations must also be designed as described in Section 5.3.1.

One alternative to further mitigate potentially expansive conditions could include grading the new building pads so that the upper three feet consists of non-expansive soils. If onsite soils are used to grade the building pads, we anticipate significant laboratory testing would be required to evaluate the expansion potential of soils encountered during grading. Additional handling and sorting of expansive and non-expansive materials would likely be required which may reduce the efficiency of grading operations. While this approach is feasible, it may be challenging considering the expansive potential of the clayey soils appears to be highly variable.

Lime treatment could also be considered to improve the clayey, near-surface soils. Treating the soils with lime will reduce the expansion potential and increase the strength, thereby significantly reducing the required thickness of new asphalt pavement sections. Lime-treated soils also perform relatively well during wet weather conditions which allows for site grading and construction to proceed more easily throughout the rainy season. A disadvantage of lime treatment is that it will increase the soil pH which may adversely affect future landscaping at the site. If lime treatment is used, treated soils may need to be removed from planned landscaping areas to allow for future planting.

If used, lime treatment should be performed in accordance with Section 24 of the most recent version of Caltrans Standard Specifications. Soil-lime proportioning testing should be performed in general accordance with ASTM D6276 to estimate the optimum lime content for treatment. Based on experience with similar projects in the Petaluma area, a minimum of four to five percent of high-calcium lime by weight is typically mixed with the on-site soils to the treatment depth. The actual lime content required for treatment should be determined by laboratory testing. R-value testing should also be performed on a sample of lime-treated soil as a basis for design of new pavement sections. The recommended minimum treatment depth is 36 inches under buildings and 18 inches in pavement and concrete flatwork areas. The lime-treatment should extend at least five feet beyond building pads, and three feet beyond flatwork and pavement areas.

5.3 <u>Foundations</u>

Provided site grading results in new fill thickness of less than five feet, new structures can be supported on a shallow foundation system consisting of spread footings or a thickened mat or post-tensioned slab. If spread footings are used, the footings should bear on firm soils or bedrock and should be relatively rigid, continuous and interconnected. Isolated spread footings should not be used. Geotechnical design criteria for spread footings and thickened mat or post-tensioned slabs are presented in Tables 3 and 4, respectively.

Table 3 – Spread Footing Design Criteria

Parameter	Design Value
Minimum Embedment	See Sections 5.3.1 to 5.3.3
Minimum Width	18 inches
Allowable Bearing Pressure[1,2]	2,500 psf
Base Friction Coefficient	0.35
Lateral Passive Resistance[3,4]	300 pcf

(1) Design shallow foundations to similar bearing pressures (i.e., size footing widths to maintain relatively uniform bearing loads).

(2) Increase design values by 1/3 for total design loads including seismic.

(3) Equivalent fluid pressure, not to exceed 3,000 psf. Neglect uppermost six inches of embedment unless footing is confined by concrete.

(4) Where slopes exist below the building pad, provide a minimum of seven feet of horizontal confinement between bottom edge of footing and adjacent slope.

Table 4 – Thickened Mat or Post-Tension Slab Design Criteria

Parameter	Design Value
Modulus of Subgrade Reaction	150 psi per inch
Minimum Edge Thickness	12 inches
Maximum Unsupported Interior Span	10 feet
Maximum Unsupported Edge Cantilever	7 feet
Edge Moisture Variation, Center Lift	10 feet
Edge Moisture Variation, Edge Lift	7 feet
Differential Soil Movement, Edge & Center Lift	1.5 inches

(1) Assumes rigid slab behavior with idealized fixed end conditions.

The design of new foundations must also account for potentially expansive soils and for potential nonuniform support conditions that exist where new structures traverse cut/fill transitions. Additional recommendations and design criteria addressing these conditions are summarized in the following sections.

5.3.1 Untreated Building Pads

Where building pads are constructed without lime treatment or grading which provides at least 36 inches of non-expansive soils in the upper portion of the pad, spread footings should be embedded at least 36 inches below the lowest adjacent grade so that they are bottomed beneath the zone in which significant seasonal moisture fluctuations (and resultant shrink/swell behavior) typically occur. The footing excavations or mat/post-tensioned slab subgrade should also be thoroughly wetted to at least two percent above the optimum moisture content and maintained at that moisture content until concrete is placed.

If mat or post-tensioned slabs or used on untreated building pads, we estimate that movements of up to about two inches could occur as the underlying soils shrink/swell with seasonal changes in moisture.

5.3.2 Treated Building Pads

Where building pads are constructed using lime treatment or grading which results in at least 36 inches of non-expansive soils in the upper portion of the pad, spread footing excavations should be embedded at least 18 inches below the lowest adjacent grade. If mat or post-tensioned slabs are constructed over treated building pads, we estimate that movement due to seasonal changes in moisture would be less than one inch.

5.3.3 Cut/Fill Building Pads

New structures that extend across cut/fill transitions will be susceptible to differential settlement due to non-uniform support provided by the compacted fill and native soil or bedrock. Where new fills are less than five feet, we estimate differential settlements across the new building pads would be less than one inch and shallow spread footings or mat/post-tensioned slabs could be used. The minimum footing embedment would be as described previously for treated or untreated building pads.

We generally recommend drilled pier foundations be used in areas where the new structure crosses cut/fill transitions and new fill thicknesses exceed five feet. The drilled piers should be designed using an allowable skin friction of 500 pounds per square foot for soil and 1,500 pounds per square foot for bedrock, a minimum diameter of 18 inches, and a minimum embedment of ten feet into firm rock. Lateral resistance for drilled piers should be calculated using a passive resistance of 300 pounds per cubic foot (equivalent fluid pressure) applied over two pier diameters. The upper three feet of embedment should be ignored in calculating the lateral and vertical capacity of drilled piers.

Where drilled piers are used, individual piers should be interconnected using grade beams. The grade beams should be designed for an uplift pressure of 1,500 pounds per square foot to account for seasonal volume changes in the potentially expansive near-surface soils. Uplift pressures can be eliminated if a void box is placed below the grade beams. Additionally, the top there feet of drilled piers should be formed with sonotubes to prevent “mushrooming” at the top of the piers and resultant uplift forces due to potentially expansive soils.

5.4 Retaining Walls

Retaining walls may also be required to support new cuts and/or fills. If required, the walls should be designed using the foundation design criteria presented in Section 5.3 and the lateral earth pressures shown in Table 5. Retaining walls that can slightly deflect at the top can be designed using the unrestrained criteria shown below. Walls that are structurally connected and not allowed to deflect (e.g. tied-back walls) are restrained and are commonly designed using a uniform active earth pressure distribution rather than an equivalent fluid pressure.

Table 5 – Lateral Earth Pressures for Retaining Wall Design

Backfill Inclination[1]	Unrestrained[2]	Restrained[3]
Level	45 pcf	30 x H psf
3:1	50 pcf	35 x H psf
2:1	60 pcf	40 x H psf

(1) Interpolate earth pressures for intermediate slopes.
(2) Equivalent fluid pressure.
(3) Rectangular distribution, H is wall height in feet.

In addition to the pressures noted above, we also recommend the walls be designed to resist a uniform seismic surcharge equal to ten times the retained height (in psf). The factor of safety used in the retaining wall design should be reduced under seismic conditions as permitted by the governing code that is used for design. A minimum uniform surcharge of 100 psf should also be applied to the upper five feet to account for surcharge loads due to light vehicles, compaction, equipment or other surcharges. The wall designer should adjust the surcharge load at their discretion commensurate with the specific loading conditions that are anticipated.

Soil nail and shotcrete retaining walls may be a relatively efficient retention system in areas where new cuts are planned. Mechanically stabilize earth (MSE) retaining walls may also be an efficient system where new fills are planned. If used, soil nail and MSE retaining walls should be designed using the criteria presented in Table 6.

Table 6 – MSE and Soil Nail Wall Design Criteria

Parameter	Wall Backfill	Clayey Soils	Bedrock
Friction Angle	30 degrees	N/A	38 degrees
Cohesion	200 psf	1,000 psf	1,000 psf
Unit Weight	125 pcf	120 pcf	130 pcf
Soil Nail Bond Strength	N/A	500 psf	1,500 psf

Wall drainage is required for all retaining walls taller than three feet. Either Caltrans Class 1B permeable material within filter fabric or Caltrans Class 2 permeable material can be used for wall drainage. The drainage should be collected in a four-inch perforated PVC drain line at the base of the wall. The permeable material should extend at least 12 inches from the back of the wall and be continuous from the bottom of the wall to within 12 inches of the ground surface. Alternatively, drainage panels, such as Mirafi 100N, may be utilized. A typical wall backdrain detail is presented on Figure 7. The Architect or waterproofing consultant should also specify a waterproofing membrane on retaining walls where interior seepage or moisture vapor would be problematic.

5.5 Interior Concrete Slabs

Reinforced concrete slab floors are judged to be appropriate for the new structures provided the building pads are prepared in accordance with our recommendations. Differential settlement should be anticipated where the concrete slab floors traverse cut/fill transitions. Where these conditions exist, we recommend that the interior floors be designed as structural slabs which span between adjacent foundations. The concrete slab floors may be poured monolithically or separated with a cold joint at the Structural Engineer's discretion. We recommend that interior concrete slabs have a minimum thickness of five inches and be reinforced with steel reinforcing bars (not mesh). Slabs subgrades should be moisture conditioned and compacted as discussed in Section 5.2 to reduce potential for future expansive behavior. Where interior slabs are constructed over untreated building pads, the slabs should also be designed for an uplift pressure of 1,500 pounds per square foot and up to two inches of movement due to seasonal changes in moisture. The project Structural Engineer should specifically design the concrete slabs, including locations of crack control joints.

To reduce the potential for moisture to move upward through the slab, a four-inch-thick layer of clean, free draining, ¾-inch angular gravel should be placed beneath interior concrete slabs to form a capillary moisture break. The gravel 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 free draining gravel. The vapor barrier should meet the ASTM E1745 Class A requirements and be installed per ASTM E1643. 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.

We note that over time, placing sand between the vapor barrier and concrete is becoming less common because of elevated interior moisture contents. If sand is used, it should be dry, and if it is not used, the slab should be carefully designed with a lower water-cement ratio since eliminating the sand can cause cracking or "curling" of the new concrete. For slabs that are not sensitive to moisture vapor, we recommend at least four inches of Class 2 Aggregate Base (Caltrans, 2015) compacted to at least 95 percent relative compaction.

5.6 Exterior Concrete Slabs

Exterior concrete walkway slabs and other concrete slabs that are not subjected to vehicle loads should be a minimum of four-inches-thick and underlain with four inches or more of Class 2 Aggregate Base. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction. The upper eight inches of subgrade on which aggregate base is placed should be prepared as previously discussed under Section 5.2.

Where improved performance is desired (i.e. reduced risks of cracking or offsets due to seasonal movements), exterior slabs can be thickened to five inches, underlain with eight inches of aggregate base compacted to 95 percent relative compaction, and reinforced with steel reinforcing bars (not welded wire mesh). Driveways and slabs subject to vehicle loads should be a minimum of five-inches-thick and designed to resist traffic loading. We recommend crack control joints no farther than six feet apart in both directions and that the reinforcing bars extend through the control joints. Some movement or offset at sidewalk joints should be expected as the underlying soils expand and shrink from seasonal moisture changes.

5.7 Site and Foundation Drainage

New grading could result in adverse drainage patterns causing water to pond around the structures. 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 five feet (five 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 five feet (two percent).

Roof gutter downspouts may discharge onto the pavements but should not discharge onto landscaped areas immediately adjacent to the structures. Provide area drains for landscape planters adjacent to buildings and parking areas and collect downspout discharges into a tight pipe collection system that discharges well away from the building foundations. Site drainage should be discharged away from the building area and outlets should be designed to reduce erosion. Site drainage improvements should be connected into an established storm drainage system.

5.8 Underground Utilities

Excavations for utilities will be in stiff to very stiff clayey soils and variably weathered schist bedrock and may encounter groundwater if wintertime or early spring work is performed. Trench excavations having a depth of five feet or more must be excavated and shored in accordance with OSHA regulations. Bedding materials for utility pipes should be poorly graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than five percent finer than the No. 200 sieve. Crushed rock or pea gravel may also be considered for pipe bedding. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically three to six inches. Trench backfill may consist of on-site soils, moisture conditioned and 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.

5.9 Pavements

We have calculated pavement sections in accordance with Caltrans procedures for flexible pavement design (Caltrans Highway Design Manual, 2015). Our calculations assume an R-value of ten for untreated soils and 40 for lime treated soils. If lime treatment is used, additional laboratory testing should be performed prior to construction to confirm the assumed R-value for the treated soils is appropriate. We have provided a range of Traffic Indices from 4.0 to 7.0 depending on the expected traffic loads for a twenty-year design life. In general, areas expected to experience loading from heavy vehicles should be designed using the higher Traffic Index, while parking areas and other lightly-loaded areas can utilize a thinner pavement section based on the lower Traffic Index. The recommended pavement sections are presented in Table 6.

Table 6 – Preliminary Asphalt-Concrete Pavement Sections

Traffic Index	Untreated Subgrade (R-value = 10)		Lime-Treated Subgrade[1] (R-value = 40)	
	Asphalt Concrete (inches)	Aggregate Base (inches)	Asphalt Concrete (inches)	Aggregate Base (inches)
4.0	3.0	7.0	2.5	4.0
5.0	3.5	8.0	3.0	4.0
6.0	5.0	8.5	3.5	6.0
7.0	5.0	13.0	4.0	7.0

(1) Calculated using a minimum lime treatment depth of 18 inches.

The aggregate base should conform to the most recent version of Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction. Additionally, the aggregate base should be firm and unyielding under heavy, rubber-tired construction equipment. If heavier truck traffic or "superior" performance is desired, the thickness of the aggregate base and asphalt thickness may be increased.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

As project plans are nearing completion, we should review them to confirm that the intent of our geotechnical recommendations has been incorporated. We can also consult with project team to supplement or clarify geotechnical recommendations, if needed. During construction, we should be present intermittently to observe foundation excavations, retaining wall drainage and backfill, subgrade preparation and compaction, proper moisture conditioning of soils, lime treatment (if used), fill placement and compaction and other geotechnical-related work items. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to confirm that the Contractor's work is performed in accordance with the project plans and specifications.

7.0 LIMITATIONS

We believe this report has been prepared in accordance with 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 Vesta Pacific Development and/or their 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 a detailed environmental assessment of the site. We recommend that an environmental consultant be retained to evaluate environmental-related issues.

The evaluations and recommendations 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 Miller Pacific 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 Miller Pacific.

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 Concrete Institute, “318-14: Building Code Requirements for Structural Concrete and Commentary”.

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

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Association of Bay Area Governments (ABAG), Geographic Information System, http://quake.abag.ca.gov/mitigation/, 2020.

California Building Code, 2019 Edition, California Building Standards Commission/International Conference of Building Officials, Whittier, California.

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California Geological Survey, "Geologic Map of the Petaluma River 7.5' Quadrangles, Sonoma and Marin Counties: A Digital Database", 2002.

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Federal Emergency Management Agency (FEMA), “Flood Insurance Rate Map, Sonoma County, California and Incorporated Areas, Panel 1001 of 1150 (Map Number 06097C1001G)”, dated October 2, 2015

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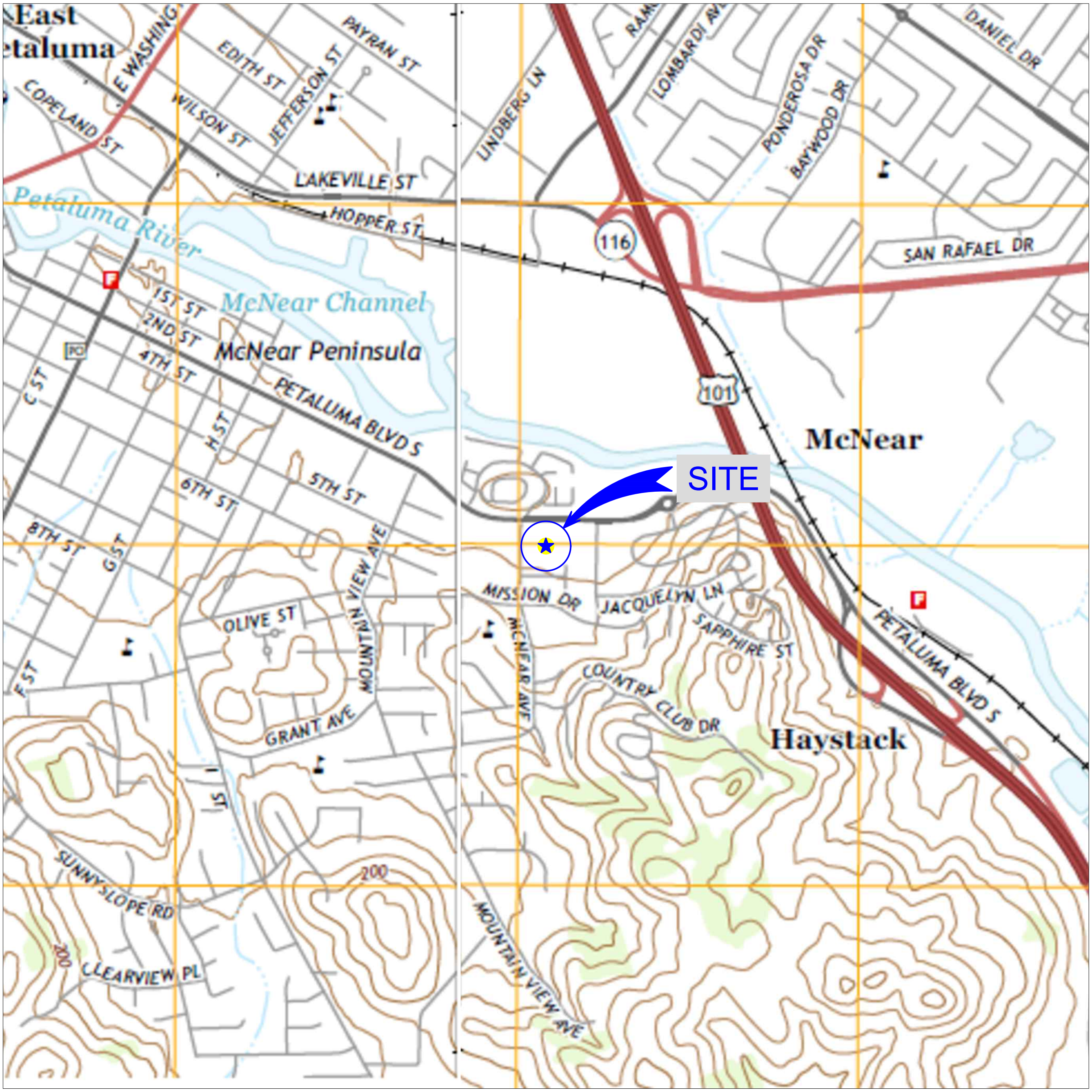
United States Geological Survey, Earthquake Hazards Program, Earthquake Circular Area Search http://neic.usgs.gov/neis/epic/epic_circ.html, accessed August 8, 2019.

WHA, “Conceptual Site Plan, Vartnaw Estates, Petaluma, California”, June 9, 2020.



SITE COORDINATES
LAT. 38.227°
LON. -122.622°

SITE LOCATION

(NO SCALE)

NORTH

REFERENCE: United States Geological Survey, Topographic Maps for Petaluma and Petaluma River 7.5' Quadrangles, 2015.

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SITE LOCATION MAP

Vartnaw Estates
McNear Ave & Petaluma Blvd
Petaluma, California

Project No. 2211.003

Date: 8/25/2020

Drawn RCA
Checked

1
FIGURE

Project Summary

Gross Site Area: + 4.09 Acres (178,126 SF)
Internal Street Area: + 0.52 Acres (22,787SF)
Net Site Area: + 3.57 Acres (155,339 SF)
Existing Zoning: R4 (8.1-18.0 du/ac)
Density Bonus: 15% BMR - 10% Density Bonus
Allowed Total Units: 80 Units = 73 Units per R4 + 7 Density Bonus Units@
Proposed Total Units: 67 Dwelling Units (Including 10 BMR Units@15%)

- (16) Flat Units
- (36) Townhome Units
- (15) Single Family Detached Units + 5 Detached

Density: 16.4 Dwelling Units per Gross Acre
18.8 Dwelling Units per Net Acre

Unit Mix:

TH / Flats - 52 Units

(8) Plan 1 (Flat @ L2) | 2 Bdrm/1 Bath | 1-Car Garage | 741
(8) Plan 2 (Flat @ L3) | 2 Bdrm/2 Bath | 2-Car Garage | 836
(32) Plan 3 (TH) | 2 to 3Bdrm/2.5 Bath | 2-Car Garage | 1,46
(4) Plan 4 (TH) | 3 Bdrm/3.5 Bath | 2-Car Garage | 1,600 E

Single Family Detached - 15 Units

(3) Plan 1 | 3-4 Bd/3.5 Ba | 2-Car Tandem Garage |1,990
(1) Plan 1X | 3-4 Bd+ADU/4.5 Ba | 2-Car Tandem Garage |
(7) Plan 2 | 3-4 Bd/3.5 Ba | 2-Car Garage |2,089 Est. SF
(4) Plan 2X | 3-4 Bd+ADU/4.5 Ba | 2-Car Garage |2,369 Es

Parking:

Required: 201 Spaces

- 67 Dwelling Units x 3 Spaces / Unit = 201 Spaces

Proposed: 161 Spaces (2.4 Spaces per Dwelling Unit)

- Garage: 126 Spaces
- Head In (Standard Size): 11 Spaces (9' x 18')
- Head In (Compact Size): 8 Spaces (8' x 16')
- Parallel: 16 (8' x 22')

Open Space:

Required: 20,100 S.F. Total (300 S.F. per home)

- Combination of common and private

Provided: TBD S.F. Total (TBD S.F. per home)

- Common: + 33,630 S.F. (10' Min. Dimension)
- Private (SFD - Rear Yard): + 7,700 S.F. (10' Min.
- Private (TH / Flats - Decks): TBD S.F.

Lot Coverage: + 58,093 S.F. (32.6% of Gross Site; 60% Permitted)

Notes:

1. Site plan is for conceptual purposes only.
2. Site plan must be reviewed by planning, buildi departments for code compliance.
3. Base information per civil engineer.

LEGEND:

APPROXIMATE LOCATION OF BORING BY MILLER PACIFIC, 2020.

NORTH

SCALE
0 30 60 120 FEET

REFERENCE: WHA, "Conceptual Site Plan", dated June 9, 2020.

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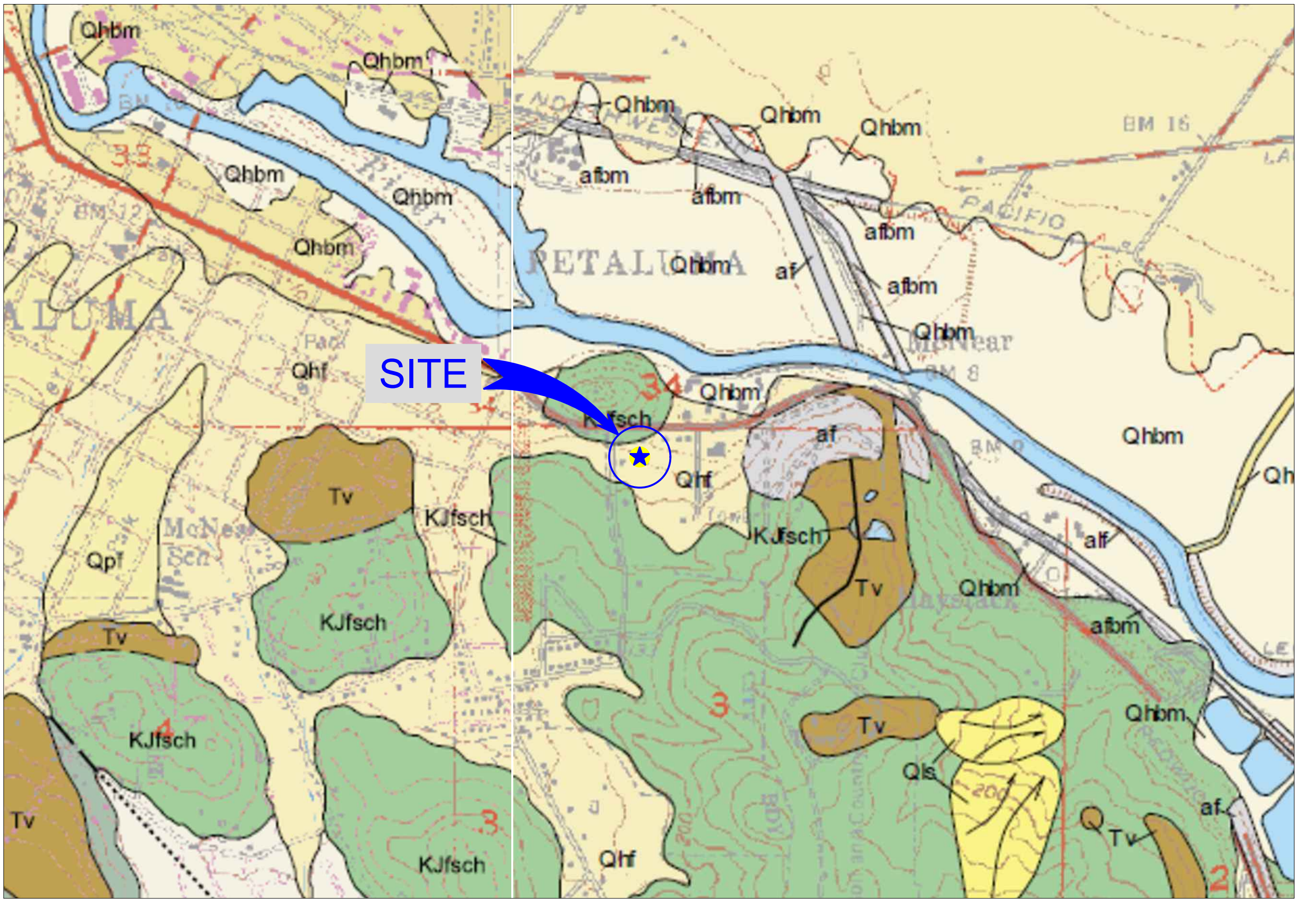
SITE PLAN

Vartnaw Estates
McNear Ave & Petaluma Blvd
Petaluma, California
Project No. 2211.003 Date: 8/25/2020

Designed
Drawn RCA
Checked MPM

2
FIGURE



REGIONAL GEOLOGIC MAP

(NO SCALE)

NORTH

LEGEND

Qhbm	**BAY MUD (Holocene)** Silt, clay, peat and fine sand deposited at or near sea level in San Pablo Bay.
Qhf	**ALLUVIAL FAN DEPOSITS (Holocene)** Sand, gravel, silt and clay deposited by streams emanating from canyons onto alluvial valley floors. Sediment is poorly to moderately sorted and bedded.
Tv	**VOLCANIC ROCKS (Tertiary)** Mafic volcanic rocks, mostly basaltic andesite, similar to and probably part of Mt. Burdell volcanics.
KJsch	**FRANCISCAN COMPLEX (Jurassic to Cretaceous)** Franciscan Complex schist, phylite and semischist.

REFERENCE: California Geological Survey, "Geologic Map of the Petaluma and Petaluma River 7.5' Quadrangles, Sonoma and Marin Counties: A Digital Database", 2002.

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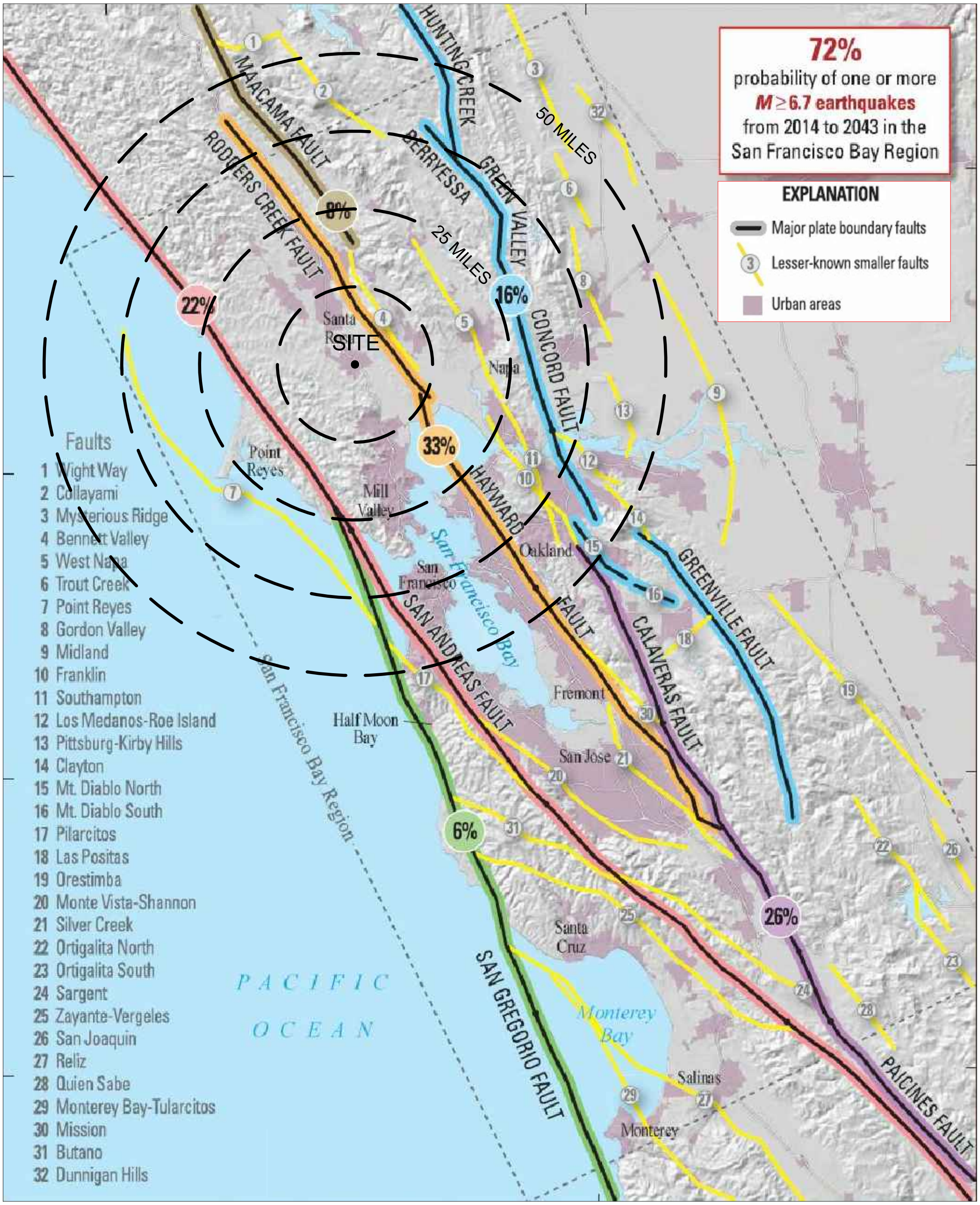
REGIONAL GEOLOGIC MAP

Vartnaw Estates
McNear Ave & Petaluma Blvd
Petaluma, California

Project No. 2211.003 Date: 8/25/2020

Drawn RCA
Checked

3
FIGURE



SITE COORDINATES
LAT. 38.0000°
LON. -122.0000°

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).

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ACTIVE FAULT MAP

Vartnaw Estates
McNear Ave & Petaluma Blvd
Petaluma, California
Project No. 2211.003
Date: 8/25/2020

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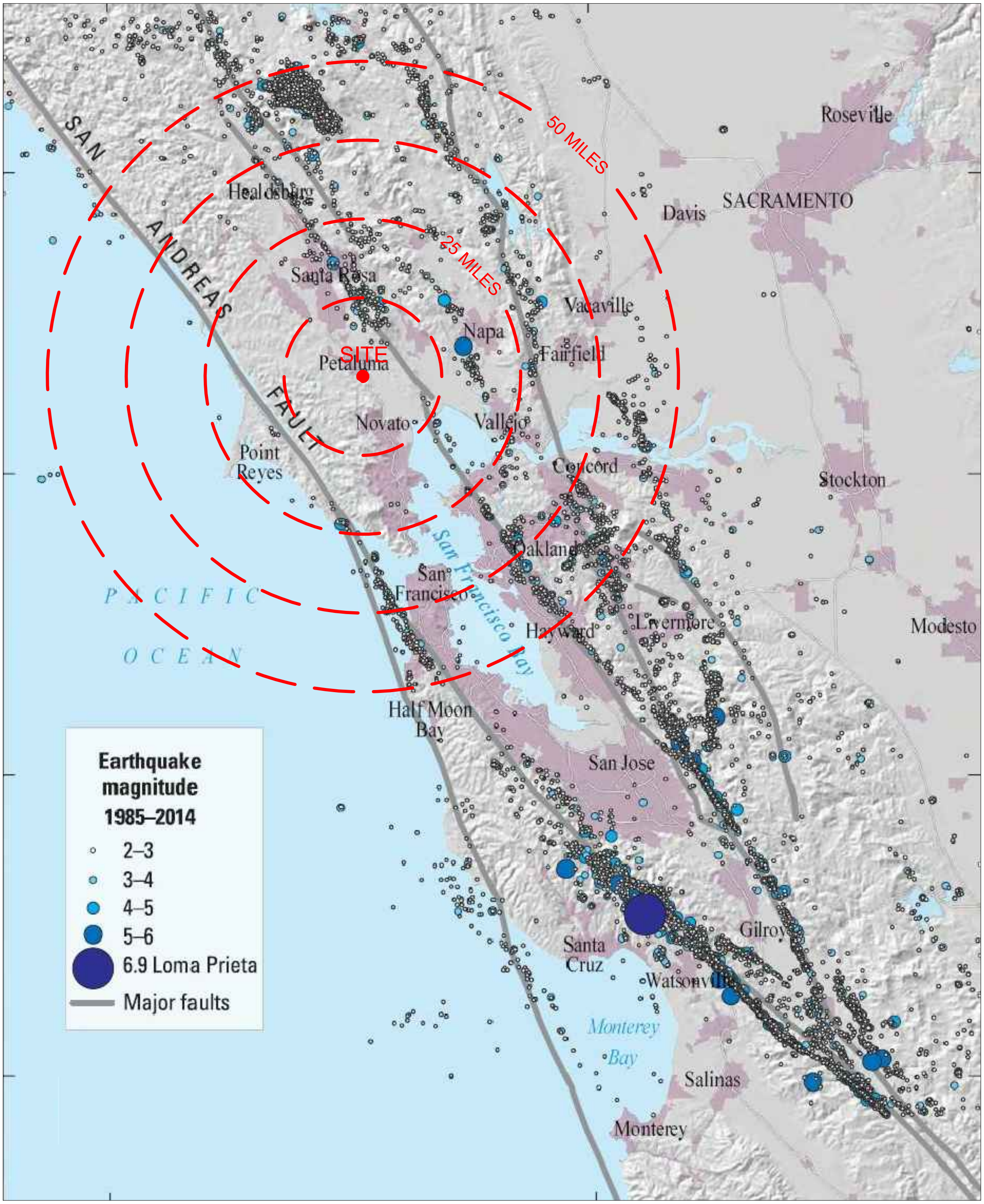
4
FIGURE



SITE COORDINATES
LAT. 38.0000°
LON. -122.0000°

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).

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HISTORIC EARTHQUAKE MAP

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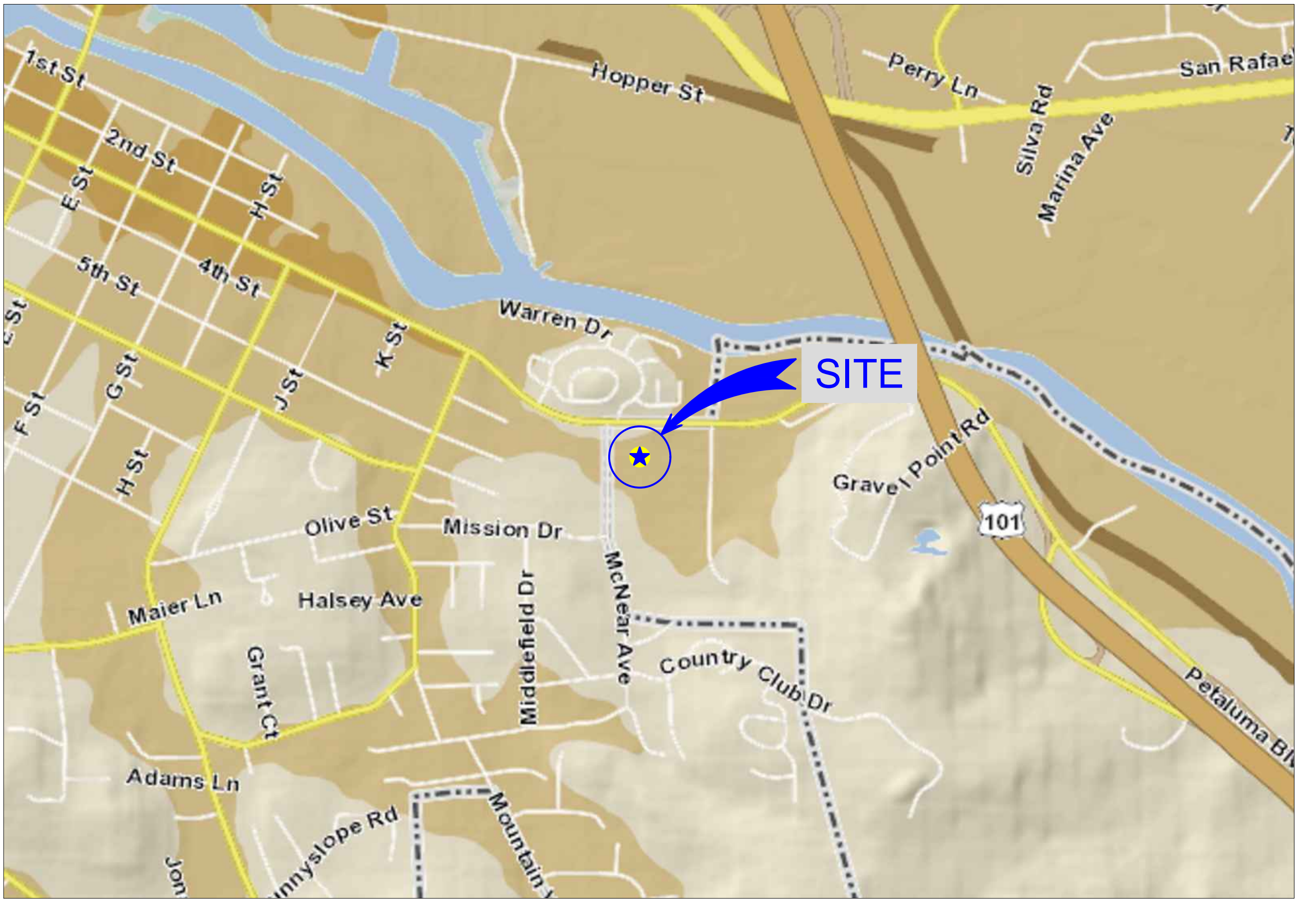
5
FIGURE



LIQUEFACTION SUSCEPTIBILITY

(NO SCALE)

NORTH

Liquefaction Susceptibility

Liquefaction Susceptibility Hazard

- ■ Very High Susceptibility
- ■ High Susceptibility
- ■ Moderate Susceptibility
- ■ Low Susceptibility
- ■ Very Low Susceptibility

REFERENCE: Association of Bay Area Governments (ABAG) website, accessed January 4, 2019.

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LIQUEFACTION SUSCEPTIBILITY MAP

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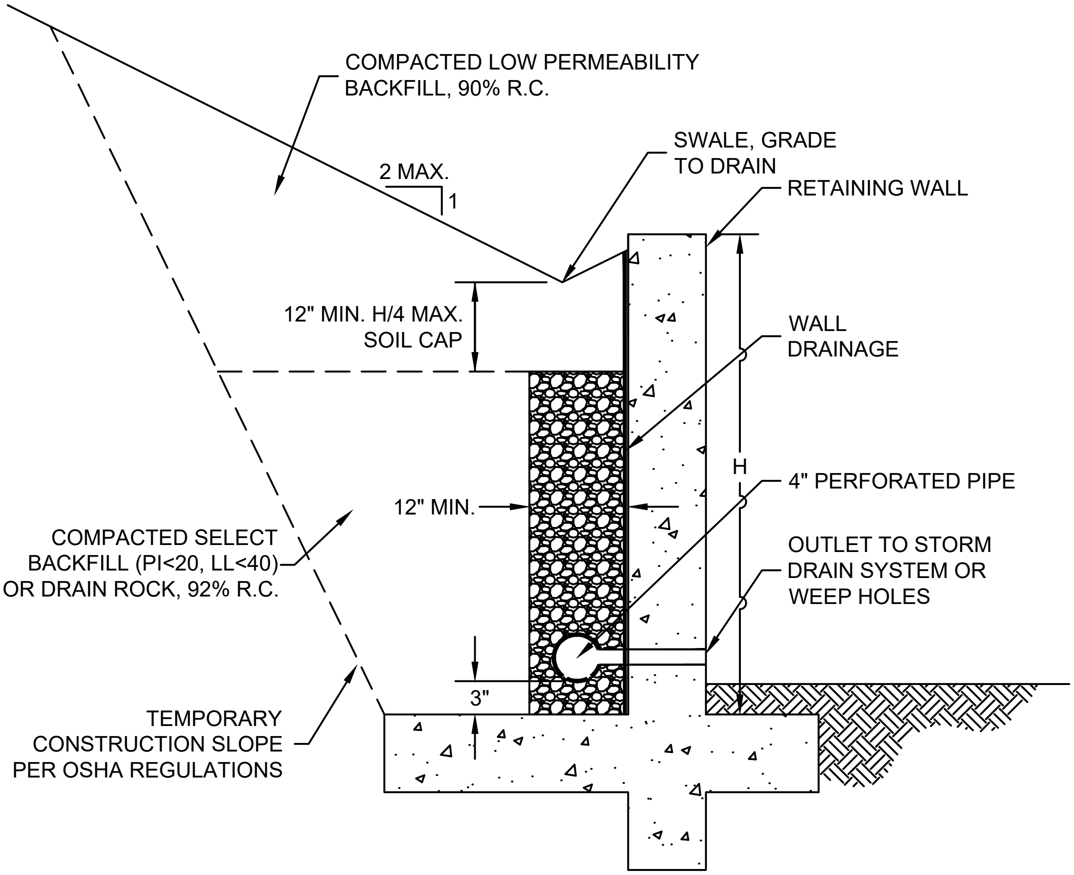
6
FIGURE



NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, pre-fabricated drainage panels (Miradrain G100N or equivalent), installed per the manufacturers recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over-stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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SCHEMATIC WALL DRAINAGE DETAIL

Vartnaw Estates
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Petaluma, California

Project No. 2211.003 Date: 8/25/2020

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7
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. SUBSURFACE EXPLORATION

We explored subsurface conditions with eight exploratory borings drilled with a track-mounted drill rig on July 29 and 30, 2020 at the approximate locations shown on the Site Plan, Figure 2. The exploration was conducted 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-1 through A-10.

Relatively “undisturbed” samples were obtained using a three-inch diameter, split-barrel Modified California Sampler with 2.5 by six-inch tube liners or a Standard Penetration Test (SPT) Sampler. The samplers were driven by a 140-pound hammer at a 30-inch drop. 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.

B. LABORATORY TESTING

We conducted laboratory tests on selected intact samples to classify soils and to estimate engineering properties. The following laboratory tests were conducted in general 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 D2937
- Unconfined Compressive Strength of Cohesive Soil, ASTM D2166
- Liquid Limit, Plastic Limit and Plasticity Index, ASTM D4318
- Expansion Index, ASTM D4829

The results of our laboratory testing are shown on the exploratory boring logs. 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

UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress
DS (2.0)	DRAINED DIRECT SHEAR (NORMAL PRESSURE, ksf)

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

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SOILS CLASSIFICATION CHART

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Project No. 2211.003
Date: 8/25/2020

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A-1
FIGURE

FRACTURING AND BEDDING

Fracture Classification	Spacing	Bedding Classification
Crushed	less than 3/4 inch	Laminated
Intensely fractured	3/4 to 2-1/2 inches	Very thinly bedded
Closely fractured	2-1/2 to 8 inches	Thinly bedded
Moderately fractured	8 to 24 inches	Medium bedded
Widely fractured	2 to 6 feet	Thickly bedded
Very widely fractured	greater than 6 feet	Very thickly bedded

HARDNESS

Low	Carved or gouged with a knife
Moderate	Easily scratched with a knife, friable
Hard	Difficult to scratch, knife scratch leaves dust trace
Very hard	Rock scratches metal

STRENGTH

Friable	Crumbles by rubbing with fingers
Weak	Crumbles under light hammer blows
Moderate	Indentations <1/8 inch with moderate blow with pick end of rock hammer
Strong	Withstands few heavy hammer blows, yields large fragments
Very strong	Withstands many heavy hammer blows, yields dust, small fragments

WEATHERING

Complete	Minerals decomposed to soil, but fabric and structure preserved
High	Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates
Moderate	Fracture surfaces coated with weathering minerals, moderate or localized discoloration
Slight	A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation
Fresh	Rock unaffected by weathering, no change with depth, rings under hammer impact

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.

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ROCK CLASSIFICATION CHART

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A-2
FIGURE

BORING 1

EQUIPMENT: Track Mounted Drill Rig with 4-inch Solid Flight Auger
DATE: 7/29/2020
ELEVATION: 44 - feet*
*REFERENCE: Google Earth, 2020

DEPTH meters feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
0 – 0		▨	Sandy CLAY/SILT (CL-ML) brown, moist, stiff, low plasticity						
1	■	▨		19	113	16.0	UC 9400	LL 27 PL 21 PI 6	
5		▩	SCHIST tan with dark gray mineral grains, moderate hardness, moderate strength, highly weathered						
2	■	▩		96/9"	125	7.7	UC 1650		
		▩							1.0
3 10	▨	▩		50/5"		5.2			
			Boring terminated at 10.5 feet No groundwater encountered during drilling						
15									
4									
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

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BORING LOG

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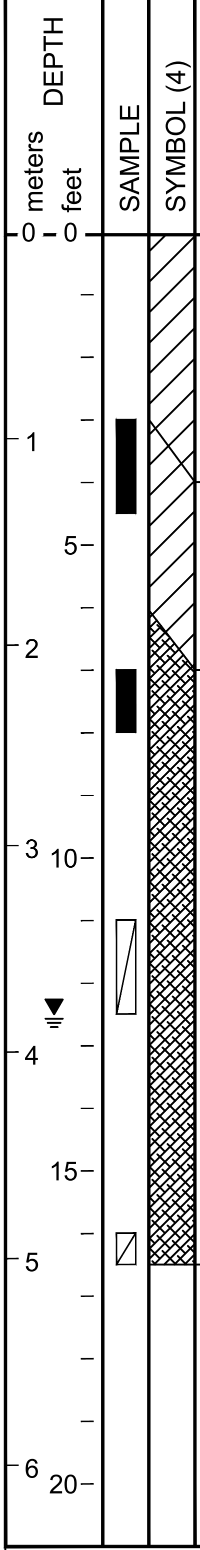
A-3
FIGURE

BORING 2

EQUIPMENT: Track Mounted Drill Rig with 4-inch Solid Flight Auger
DATE: 7/29/2020
ELEVATION: 34 - feet*
*REFERENCE: Google Earth, 2020



	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
Sandy CLAY with Gravel (CH) brown, moist, stiff, medium to high plasticity						
Sandy CLAY (CH) light brown, moist, stiff, high plasticity	18	109	17.2	UC 5200		
SCHIST tan with dark gray mineral grains, moderate hardness, moderate strength, highly weathered	81/11"	129	7.6	UC 500		
	76		6.0			
						1.0
grades strong	50/6"		7.4			

Boring terminated at 16.5 feet
No groundwater encountered during drilling
Groundwater measured at 12.5 feet approximately 24 hours after drilling was complete

▽ 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

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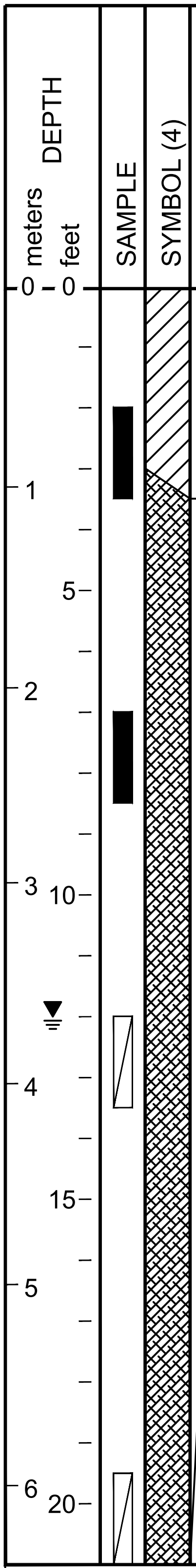
A-4
FIGURE

BORING 3

EQUIPMENT: Track Mounted Drill Rig with 4-inch Solid Flight Auger
DATE: 7/29/2020
ELEVATION: 28 - feet*
*REFERENCE: Google Earth, 2020



	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
Sandy CLAY (CL) brown, slightly moist, stiff, low to medium plasticity	29	114	15.1	UC 8300		
SCHIST tan with green-gray and dark gray mineral grains, moderate hardness, moderate strength, highly weathered, contains white/chalky mineralization	96	126	11.8	UC 3300		
						1.0
	71		11.9			
						1.0
Boring terminated at 20.5 feet. No groundwater encountered during drilling Groundwater measured at 12 feet approximately 24 hours after drilling was complete	62		11.2			

▽ 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

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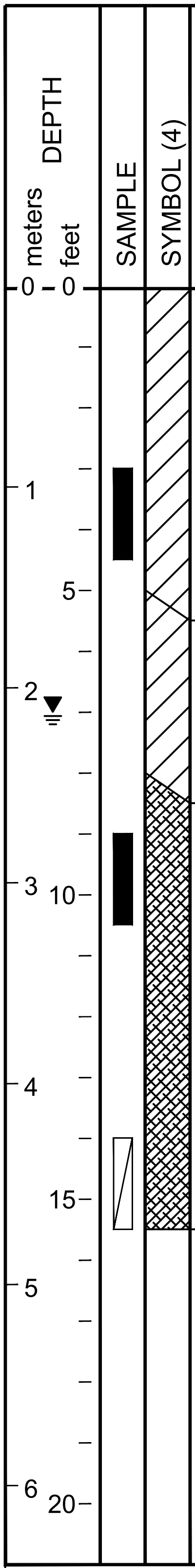
A-5
FIGURE

BORING 4

EQUIPMENT: Track Mounted Drill Rig with 4-inch Solid Flight Auger
DATE: 7/29/2020
ELEVATION: 32 - feet*
*REFERENCE: Google Earth, 2020



DEPTH feet	Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
0	**Sandy CLAY with Gravel (CH)** medium to dark brown, moist, stiff, medium to high plasticity						
4		42	116	13.5	UC 7300		
5.5	**Sandy CLAY (CH)** tan, moist, stiff, high plasticity						
8.5	**SCHIST** tan with dark gray mineral grains, moderate harness, moderate strength, highly to completely weathered						
10		51	122	13.7			
10.5–14							1.0
15		48		10.4			

Boring terminated at 15.5 feet.
No groundwater encountered during drilling.
Groundwater measured at 7 feet approximately
24 hours after drilling was complete

▽ 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

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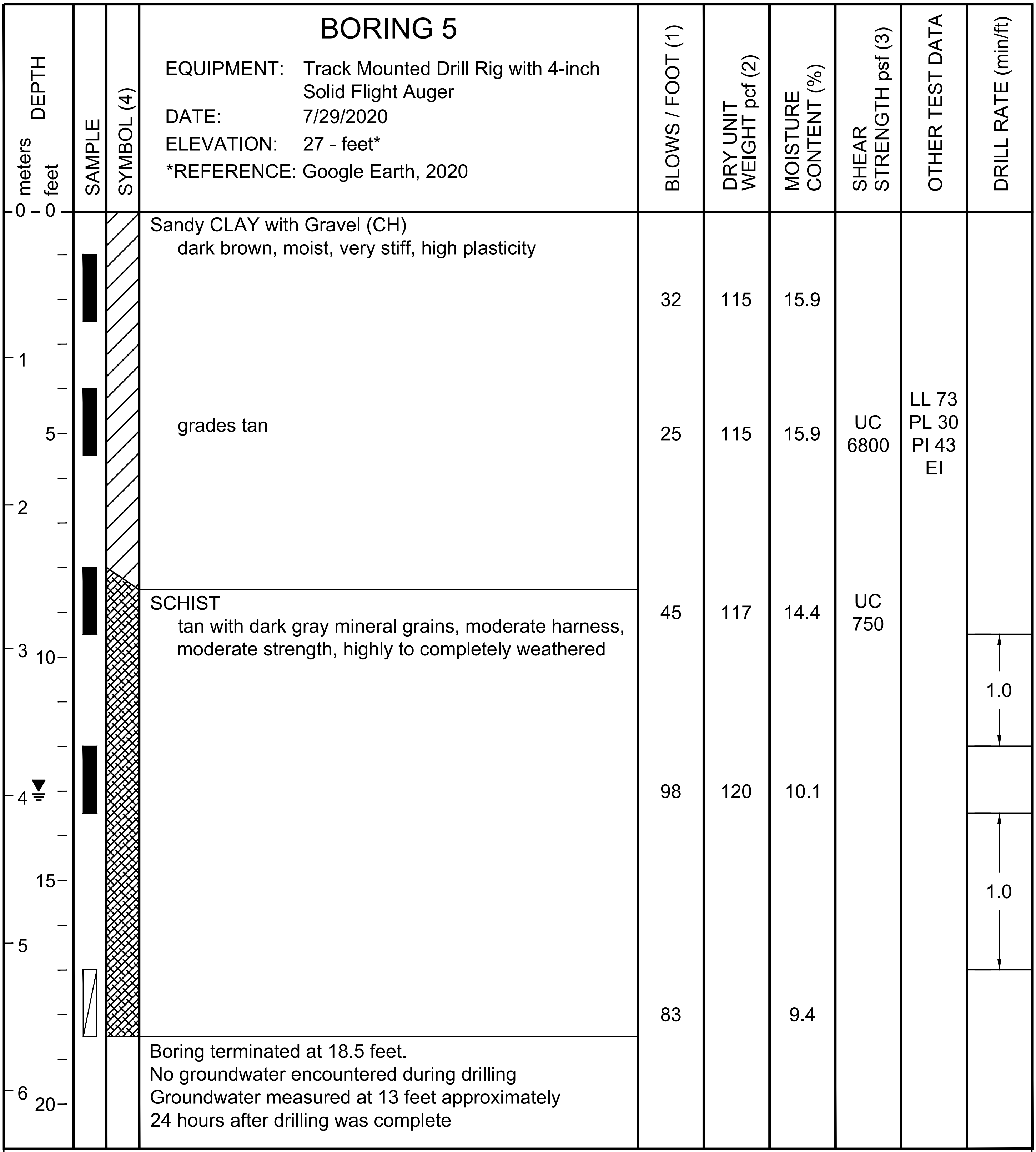
A-6
FIGURE

BORING 5

EQUIPMENT: Track Mounted Drill Rig with 4-inch Solid Flight Auger
DATE: 7/29/2020
ELEVATION: 27 - feet*
*REFERENCE: Google Earth, 2020



Sandy CLAY with Gravel (CH)
dark brown, moist, very stiff, high plasticity

grades tan

SCHIST
tan with dark gray mineral grains, moderate harness, moderate strength, highly to completely weathered

Boring terminated at 18.5 feet.
No groundwater encountered during drilling
Groundwater measured at 13 feet approximately
24 hours after drilling was complete

BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
32	115	15.9			
25	115	15.9	UC 6800	LL 73 PL 30 PI 43 EI	
45	117	14.4	UC 750		
					1.0
98	120	10.1			
					1.0
83		9.4			

▽ 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

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A-7
FIGURE

BORING 6

EQUIPMENT: Track Mounted Drill Rig with 4-inch Solid Flight Auger
DATE: 7/30/2020
ELEVATION: 31 - feet*
*REFERENCE: Google Earth, 2020

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
0	0		▨	Sandy CLAY (CL) brown, slightly moist, stiff, low to medium plasticity						
1										
	5		▩	SCHIST tan with dark gray mineral grains, moderate hardness, moderate strength, highly weathered						
2		■	▩		52/6"	120	6.9	UC 1100		1.0
3	10	▯	▩	grades hard, strong	50/2"		5.3			
	▼		▩							
4			▩							
	▽ 15	▯	▩		50/5"		6.0			
5				Boring terminated at 15.5 feet. Groundwater encountered at 15 feet during drilling Groundwater measured at 12 feet 4 hours after drilling was complete						
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

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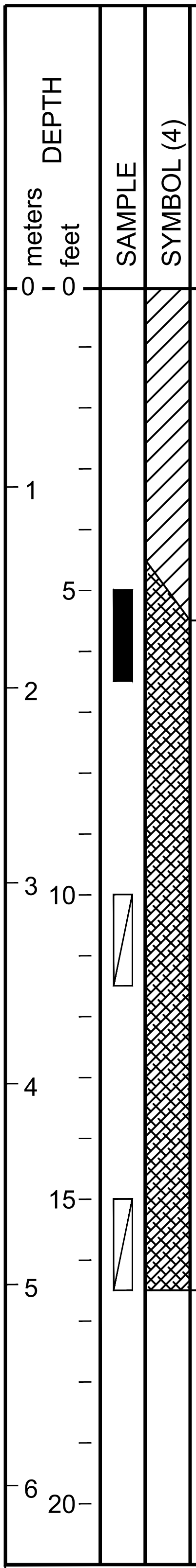
A-8
FIGURE

BORING 7

EQUIPMENT: Track Mounted Drill Rig with 4-inch Solid Flight Auger
DATE: 7/30/2020
ELEVATION: 28 - feet*
*REFERENCE: Google Earth, 2020



BORING 7	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
Sandy CLAY with Gravel (CH) brown, moist, stiff, medium to high plasticity						
SHCIST tan with dark gray mineral grains, moderate hardness, moderate strength, highly weathered	92	125	10.6			
grades hard	89		6.2			1.5
grades moderate hardness	50		6.2			
Boring terminated at 16.5 feet No groundwater encountered during drilling						

▽ 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

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A-9
FIGURE

BORING 8

EQUIPMENT: Track Mounted Drill Rig with 4-inch Solid Flight Auger
DATE: 7/30/2020
ELEVATION: 28 - feet*
*REFERENCE: Google Earth, 2020

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
0	0		▨	Sandy CLAY with Gravel (CL/CH) dark brown, moist, stiff, medium plasticity						
		■	▨		15	122	10.3	UC 2050		
1	5		▨	Sandy CLAY (CH) tan, moist, very stiff, high plasticity						
2		■	▦	SCHIST tan with dark gray mineral grains, moderate hardness, moderate strength, highly to completely weathered	60	120	14.3	UC 1250		
3	10		▦							1.0
4			▦							
	▽ 15	☐	▦		63		6.0			
5				Bottom of boring at 15.5 feet. Groundwater measured at 15 feet upon completion of drilling						
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

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A-10
FIGURE

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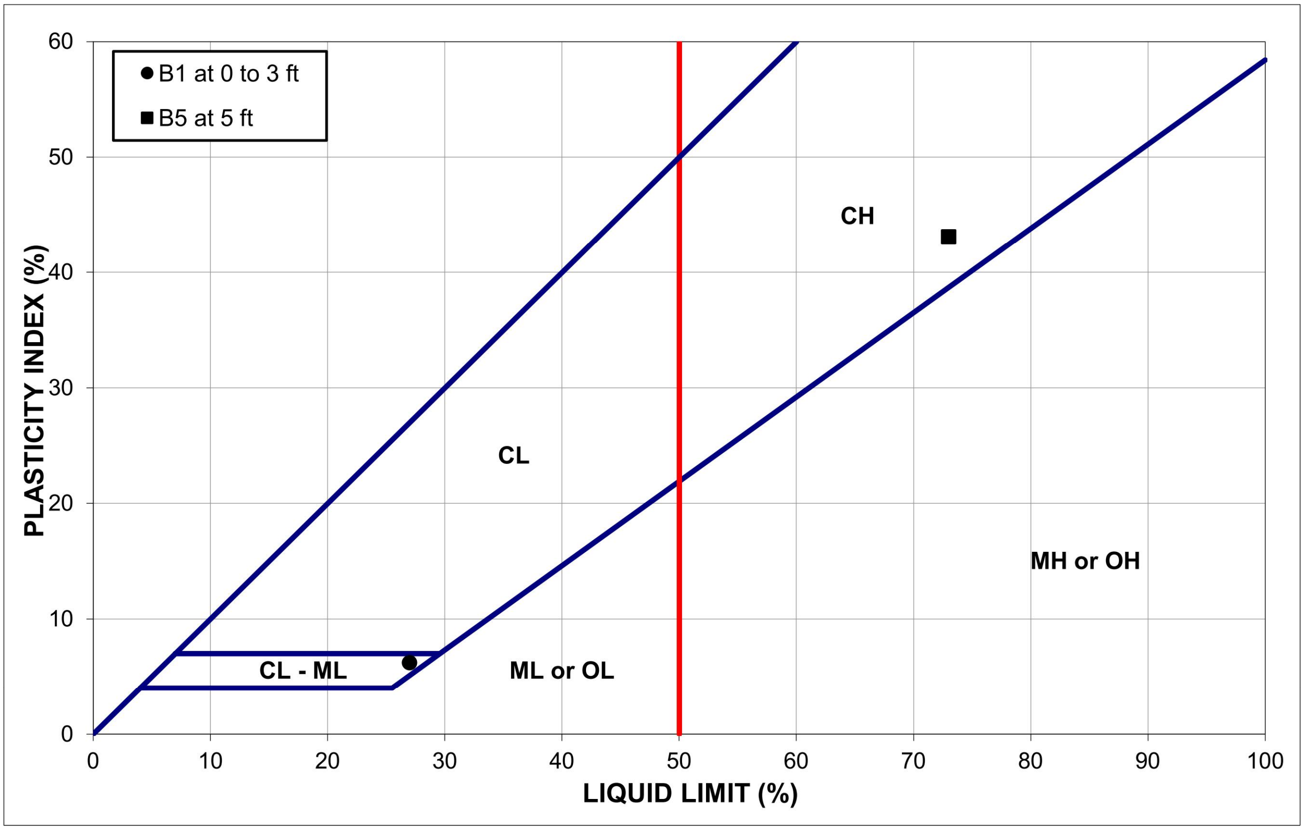
ATTERBERG LIMITS TEST (ASTM D 4318)



Sample	Classification	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
B1 at 0 to 3 ft	Sandy CLAY/SILT (CL-ML) brown	27	21	6
B5 at 5 ft	Sandy CLAY with Gravel (CH) tan	73	30	43

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ATTERBERG LIMITS TEST RESULTS

Vartnaw Estates
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Project No. 2211.003 Date: 8/25/2020

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A-11
FIGURE

EXPANSION INDEX (ASTM D4829)

Project Name: Vartnaw Estates
Tested By: BPC
Date: 8/17/2020

Project Number: 2211.003
Sample Source: Boring B-5 at 5 feet

Sample Description: Sandy CLAY with Gravel (CH), tan

Sample Height before Saturation (in.): 1.000

Sample/Ring Diameter: (in.): 4.000

Sample Volume before Saturation (cu.ft.): 0.007272

Ring Number: EI4

Ring Tare (gm): 367.1

Weight Ring + Moist Soil (gm): 775.1

Approx. Moisture Content (%):
Estimated Specific Gravity (2.60-2.70): 2.68
Approx. EI Dry Density (pcf):

Approximate Saturation: 50.0%

Dial Readings with 1 psi Load:

Start Time:	7:11	Dial Reading T0:	0.0000
Time 1:	7:14	Dial Reading T1:	0.0084
Time 2:	7:50	Dial Reading T2:	0.0527
Time 3:	9:36	Dial Reading T3:	0.1210
Time 4:	12:30	Dial Reading T4:	0.1376
Time 5:	14:00	Dial Reading T5:	0.1382

Final Height of Sample (in.): 1.138

Pan Identification: 9T

Weight of Ring + Wet Soil + Pan (gm): 1255.0

Pan Tare (gm): 425.3

Weight of Ring + Dry Soil + Pan (gm): 1158.9

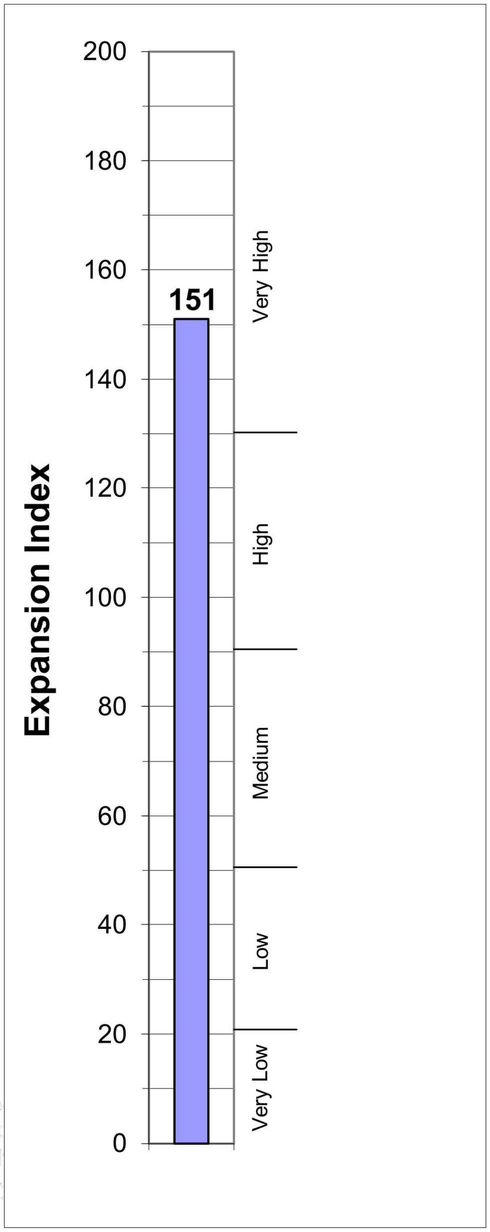
Initial Moisture Content: 11.3%
Initial Dry Density (pcf): 111.1
Prepared Sample Saturation: 60.1%

100
1000
0.0%

Percent Expansion: 13.8%

Final Moisture Content: 26.2%
Final Dry Density (pcf): 97.6
EI $_{50}$: 151
Potential Expansion Very High

20.5 Low
50.5 Med
90.5 Hig
130.5 Ver



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EXPANSION INDEX TEST RESULTS

Vartnaw Estates
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A-12
FIGURE