PJC & Associates, Inc.
Consulting Engineers & Geologists

February 18, 2021 Job No. 4433.02

Community Housing of Sonoma County
Attention: Paula Cook
131-A Stony Circle, Suite 500
Santa Rosa, CA 95401

Subject: Design Level Geotechnical Investigation
Proposed Hearns Veterans Village
2149 West Hearn Avenue
Santa Rosa, California
APN: 134-011-012

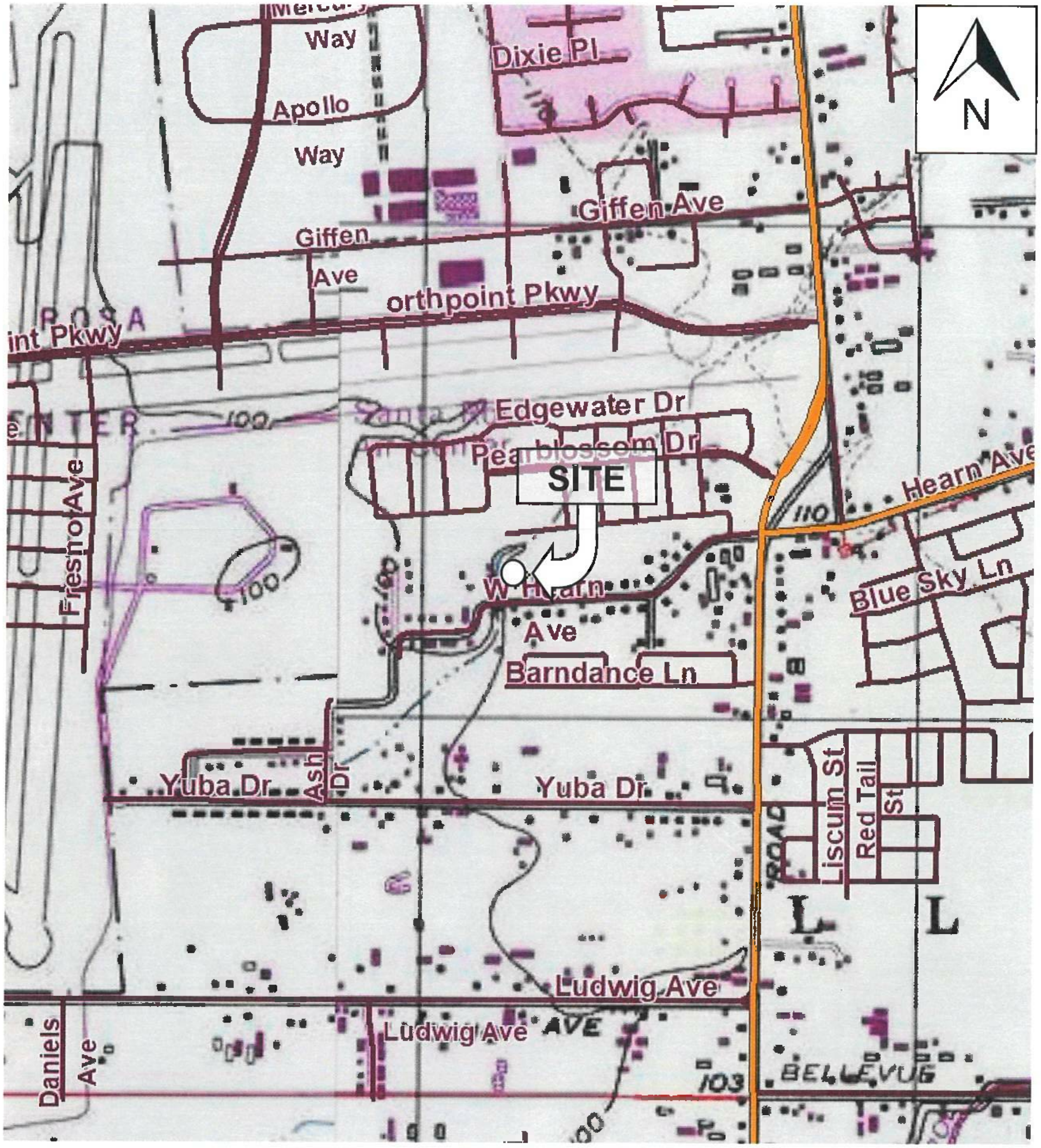
Dear Paula:

PJC & Associates, Inc. (PJC) is pleased to submit the results of our design level geotechnical investigation for the proposed Hearns Veterans Village located at 2149 West Hearn Avenue in Santa Rosa, California. The approximate location of the site is shown on the Site Location Map, Plate 1. The site corresponds to latitudinal and longitudinal coordinates of 38.411° north and 122.746° west, according to field GPS measurements. Our services were completed in accordance with our proposal for geotechnical engineering services dated October 22, 2020 and your authorization to proceed with the work, dated November 10, 2020. This report presents our engineering opinions and recommendations regarding the geotechnical aspects of the design and construction of the proposed project. Based on the results of this study, it is our opinion that the project site can be developed from a geotechnical engineering standpoint provided the recommendations presented herein are incorporated in the design and carried out through construction.

1. PROJECT DESCRIPTION

Based on a preliminary project plans prepared by Fitz Architecture, dated May 20, 2020 and August 29, 2020, it is our understanding that the project will consist of improving the site and constructing a total of eight residential structures. It is our understanding that the structures will consist of four residences and four accessory dwelling units (ADU). The residences are expected to consist of two-story, wood-frame structures with concrete slab-on-grade floors. The ADU's will likely consist of single-story, wood-frame structures with concrete slab-on-grade floors. The project will also include carports and exterior flatwork. We assume that the project will be serviced by underground private and municipal utilities.

Structural loading information was not available at the time of this report. For our analysis, we anticipate that the structural loading will be light with dead plus live



SCALE: 1:24,000

REFERENCE: USGS SANTA ROSA, CALIFORNIA 7.5 MINUTE QUADRANGLE, DATED 1998.

PJC & Associates, Inc. Consulting Engineers & Geologists	SITE LOCATION MAP PROPOSED HEARNS VETERANS VILLAGE 2149 WEST HEARN SANTA ROSA, CALIFORNIA	PLATE 1
	Proj. No: 4433.02 Date: 1/2021 App'd by: PJC	

continuous wall loads less than two kips per lineal foot (plf) and dead plus live isolated column loads less than 50 kips. If these assumed loads vary significantly from the actual loads, we should be consulted to review the actual loading conditions and, if necessary, revise the recommendations of this report.

Grading and drainage plans or finish floor elevations were unavailable at time of this report. Based on the site topography, we anticipate that grading will consist of cuts and fills of three feet and less in order to achieve the finish pad and exterior grades and provide adequate gradients for site drainage. We do not anticipate that engineered retaining walls will be required for the project.

2. SCOPE OF SERVICES

The purpose of this study is to provide geotechnical criteria for the design and construction of the proposed project, as described above. Specifically, the scope of our services included the following:

a. Drilling five exploratory boreholes to depths between ten and fifteen feet below the existing ground surface to observe the soil and groundwater conditions underlying the site. Our field geologist was on site during the drilling to log the materials encountered in the boreholes and to obtain representative samples for visual classification and laboratory testing.

b. Laboratory observation and testing of representative samples obtained during the course of our field investigation in order to evaluate the engineering properties of the subsurface soils underlying the site.

c. Review of seismological and geologic literature and discussion of site geology, seismicity and faulting, and evaluation of potential geologic hazards and earthquake effects (i.e., liquefaction, ground rupture, settlement, expansive soils, lurching and lateral spreading, densification, etc.).

d. Performing engineering analyses, to develop geotechnical recommendations for site preparation and grading, foundation type(s) and design criteria, settlement, lateral earth pressures, support of concrete slabs-on-grade, site drainage and construction considerations.

e. Preparation of this report summarizing our work on this project.

3. SITE CONDITIONS

a. General. The site is located on a rectangular parcel on the northern side of West Hearn Avenue in southwest Santa Rosa. The parcel is located approximately a quarter mile west of Stony Point Road in a residential and agricultural portion of Santa Rosa, California. During the time of our subsurface exploration on December 1, 2020, the site was occupied by two existing residences on the southeast portion of the 2.36 acre property. At the time of our investigation of December 1, 2020, the remaining portion of the site was vacant and covered with perennial grasses.

b. Topography and Drainage. The site is located on nearly level ground in the Santa Rosa Plain structural basin. According to the United States Geological Survey (USGS) Santa Rosa, California, 7.5 Minute Quadrangle Map (topographic), the site is situated near an elevation of 100 feet above mean sea level (MSL). Site drainage generally consists of sheet flow to a roadside ditch adjacent to West Hearn Avenue and surface infiltration. Regional drainage is provided by city maintained storm drainage systems and seasonal creeks which extend southwest to the Laguna de Santa Rosa.

4. GEOLOGIC SETTING

The site is located in the Coast Ranges Geomorphic Province of California. This province is characterized by northwest trending topographic and geologic features, and includes many separate ranges, coalescing mountain masses and several major structural valleys. The province is bounded on the east by the Great Valley and on the west by the Pacific Ocean. It extends north into Oregon and south to the Transverse Ranges in Ventura County.

The structure of the northern Coast Ranges region is extremely complex due to continuous tectonic deformation imposed over a long period of time. The initial tectonic episode in the northern Coast Ranges was a result of plate convergence, which is believed to have begun during the late Jurassic period. This process involved eastward thrusting of oceanic crust beneath the continental crust (Klamath Mountains and Sierra Nevada) and the scraping off of materials that are now accreted to the continent (northern Coast Ranges). East-dipping thrust and reverse faults were believed to be the dominant structures formed.

Right lateral, strike slip deformation was superimposed on the earlier structures beginning mid-Cenozoic time, and has progressed northward to the vicinity of Cape Mendocino in Southern Humboldt County (Hart, Bryant and Smith, 1983). Thus, the principal structures south of Cape Mendocino are northwest trending, nearly vertical faults of the San Andreas system.

Based on our review of published geologic literature, the site is underlain by Quaternary alluvial fan and fluvial terrace deposits (Qhf). Alluvial soils generally consist of sand, gravel, silt and clay. The geologic mapping was confirmed by our subsurface exploration. However, the alluvial soils at the site are generally masked by undocumented artificial fill.

5. FAULTING

Geologic structures in the region are primarily controlled by northwest-trending faults. The site is not located within the current Alquist-Priolo Earthquake Fault Zone boundaries (Bryant and Hart, 2007). According the USGS National Seismic

Hazard Map (2008), the closest known active faults to the site are the Rodgers Creek, the Maacama, and the San Andreas faults. The Rodgers Creek is located 3.52 miles to the east, the Maacama is located 11.68 miles to the northeast, and the San Andreas is located 16.56 miles to the west. Table 1 outlines the nearest known active faults and their associated maximum magnitudes.

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance from Site (Miles)	Maximum Earthquakes (Moment Magnitude)
Rodgers Creek	3.52	7.07
Maacama	11.68	7.40
San Andreas	16.56	8.05

Reference – USGS 2008 National Seismic Hazard Maps

6. SEISMICITY

The site is located within a zone of high seismic activity related to the active faults that transverse through the surrounding region. Future damaging earthquakes could occur on any of these fault systems during the lifetime of the proposed project. In general, the intensity of ground shaking at the site will depend upon the distance to the causative earthquake epicenter, the magnitude of the shock, the response characteristics of the underlying earth materials, and the quality of construction. Seismic considerations and hazards are discussed in the following subsections of this report.

7. SUBSURFACE CONDITIONS

a. Soils. The subsurface conditions at the project site were investigated by drilling five exploratory boreholes (BH-1 – BH-5) to depths between ten and fifteen feet below the existing ground surface. The approximate borehole locations are shown on the Borehole Location Plan, Plate 2. The boreholes were performed to observe the subsurface conditions and to collect soil samples of the underlying strata for visual examination and laboratory testing. All soils encountered during the exploration were logged by a staff geologist of PJC. Detailed logs of the subsurface conditions encountered during our exploration are presented on Plates 3 through 7. The drilling and sampling procedures and laboratory procedures are included in Appendices A and B, respectively.

The exploratory boreholes generally encountered artificial fill underlain by heterogenous alluvial soils consisting chiefly of sandy silts, sandy and gravelly clays, clayey sands and sandy gravels which extended to the maximum depths explored. The surface of the site is blanketed by one to three and one-quarter feet of suspected artificial fill, consisting predominantly of low to highly plastic sandy clays and sandy silts. These materials appeared slightly moist and loosely compacted. Underlying the fill layers, the boreholes generally encountered discontinuous and heterogenous alluvial strata of sandy clays and gravels, sandy silts, clayey gravels and clayey sands that extended to the maximum depth explored of 15 feet. The sandy clays appeared moist to very moist, very stiff to hard and generally medium to highly plastic. The sandy gravels appeared wet, dense to hard and high in plasticity. Descriptions of the strata and approximate contacts are presented on the logs of the boreholes, Plates 3 through 9.

b. Groundwater. Groundwater was encountered in BH-4 and BH-5 at 12.5 to 13.5 feet below the existing ground surface during our subsurface exploration on December 1, 2020. No groundwater or groundwater seepage was encountered in the other boreholes. The phreatic groundwater in the Santa Rosa Plain fluctuates by several feet throughout the year due to seasonal rainfall and other factors.

8. GEOLOGIC HAZARDS & SEISMIC CONSIDERATIONS

The site is located within a region subject to a high level of seismic activity. Therefore, the site could experience strong seismic ground shaking during the lifetime of the proposed project. The following discussion reflects the possible earthquake effects and geologic hazards, which could result in damage to the improvements at the site.

a. Fault Rupture. Rupture of the ground surface is expected to occur along known active fault traces. No known active faults exist at or near the project site. Therefore, the likelihood of ground rupture at the site due to faulting is considered to be low.

b. Ground Shaking. In the recent past, the site has been subjected to ground shaking by earthquakes on the active fault systems that traverse the region. It is believed that earthquakes with significant ground shaking will occur in the region within the next several decades. Therefore, it must be assumed that the site will be subjected to strong ground shaking during the design life of the structures.

c. Liquefaction. Based on our review of the Association of Bay Area Governments (ABAG) liquefaction susceptibility map, the site is not located in an area prone to liquefaction. Liquefaction is a seismic hazard that occurs in saturated, low density, predominantly granular soils found below the phreatic groundwater. In general, these loose materials experience a rapid, temporary loss in shear strength due to an increase in pore water pressure in response to strong earthquake ground shaking.

Our field exploration revealed no loose, saturated, granular soil strata at the site within 15 feet of the existing ground surface. Therefore, the risk of liquefaction occurring within 15 feet of the existing ground surface is low. Evaluation of the liquefaction potential below a depth of 15 feet is beyond the scope of this report.

d. Lateral Spreading and Lurching. Lateral spreading is normally induced by vibration of near-horizontal alluvial soil layers adjacent to an exposed face. Lurching is an action, which produces cracks or fissures parallel to streams or banks when the earthquake motion is at right angles to them. The site is not located near an exposed face, creek or bank. Therefore, the project is not likely to be impacted by lateral spreading or lurching.

e. Expansive Soils. Based on Atterberg Limits testing (PI=25) the near surface artificial sandy clay site soils should be considered high plasticity and should be considered highly expansive. Expansive soils should be taken into account in design and construction of the project.

9. CONCLUSIONS

Based on the results of our investigation, it is our professional opinion that the project is feasible from a geotechnical engineering standpoint provided the recommendations contained in this report are incorporated into the design and carried out through construction. The primary geotechnical considerations in design and construction of the project are:

a. The presence of weak and compressible suspected undocumented artificial fill that could experience differential settlement under loads of new construction.

b. The presence of highly expansive surface soils.

Weak and compressible undocumented artificial fill was encountered at depths one to three feet below the existing ground surface across the site. The fill is of an unknown source and of variable density. The artificial fill may appear hard and strong when dry. However, these soils could potentially collapse under the loads of foundations, concrete slabs-on-grade or pavements when their moisture content increases and approaches saturation. The moisture content of these soils can increase as the result of rainfall, or when the natural upward migration of water vapor through the soils is impeded by fills, slabs, foundations or pavements. These soils can undergo considerable strength loss and increased compressibility, thus causing irregular and differential settlement under loads. This settlement could be detrimental to concrete elements of the structures and cause intolerable distress to the architectural features of the structures.

Furthermore, the site is underlain by moderately to highly expansive artificial fill and alluvial soils. Expansive soils experience volumetric changes with changes in moisture content. Shrinking and/or swelling of expansive soils due to loss and increase in moisture content can also cause distress and damage to concrete elements and architectural features of structures.

To reduce the detrimental effects of the weak and expansive soils to within tolerable limits, we recommend that the proposed residences and ADU's be supported on post-tension slab foundations designed to resist differential movement of the expansive soils.

As an alternative to post tension slabs, the new structures may also be supported by conventional interior slabs-on-grade, provided all artificial fill is subexcavated and recompacted and the expansive soils are removed from the construction area and replaced with a non-expansive material at least 30 inches thick. The low to non-expansive engineered fill should extend at least five feet beyond the perimeter foundations. The low to non-expansive engineered fill, if constructed, would need to be placed over properly moisture conditioned and compacted native site soils

The plans show that the project will include the construction of carports and asphaltic concrete or exterior concrete slabs-on-grade exterior flatwork. If constructed on the highly expansive soils, these pavements and slabs will be prone to differential movement and cracking, which will reduce the life span of these structures and increase maintenance costs. To reduce the risk of differential movement to within tolerable limits, we recommend that the top 18 inches of soil consist of a low to non-expansive engineered fill. The low to non-expansive engineered fill should extend at least three feet beyond the slabs and pavements.

The following sections present geotechnical recommendations and criteria for design and construction of the project.

10. GRADING AND EARTHWORK

We anticipate the project will include cuts and fills of up to approximately three feet and less to subexcavate and recompact the existing artificial fill, construct the building pads and provide adequate gradients for site drainage. We do not anticipate that retaining walls will be required for the project.

a. Demolition and Stripping. Existing structures to be removed should be completely demolished and removed off site. Structural areas should be stripped of surface vegetation, roots, tree stumps and organic soils. These materials should be moved off site. Organic soils, if suitable, could be stockpiled for later use in landscape areas. Where existing underground utilities pass through the site, we recommend that these utilities be removed in their entirety or rerouted where they exist outside an imaginary plane sloped two horizontal to one vertical (2H:1V) from the outside bottom edge of the nearest foundation element. Any existing wells, septic systems and leach fields should be abandoned and plugged according to regulations set forth by the Sonoma County Health Department. Voids left from the removal of utilities or other obstructions should be replaced with compacted engineered fill under the observation of the project geotechnical engineer. It is important that voids be properly backfilled with compacted materials. Loosely backfilled voids will settle and potentially damage structures built above them.

b. Excavation and Compaction. Following site stripping, excavation should proceed to achieve finish grades or prepare areas to receive fill. Within the building envelopes and areas to receive fill should be prepared by removing artificial fill and weak or porous native soils and exposing firm native soils, as determined by the geotechnical engineer on site during construction. We anticipate subexcavations of one to three feet. Subexcavations scheduled to receive fill should be scarified to a minimum depth of eight inches, moisture conditioned to two to four percent over the optimum moisture content, and recompacted to at least 90 percent of the materials relative maximum dry density as determined by ASTM D 1557 test procedures.

All artificial fill must be removed and recompacted in structural areas, regardless of foundation type. The lateral extent of subexcavation and recompaction of artificial fill and weak native soils should be at least five feet beyond perimeter foundations and three feet beyond the edges of exterior concrete slabs-on-grade and pavements. If conventional interior concrete slabs-on-grade are used, the top 30 inches of the building pad should consist of low to non-expansive engineered fill extending at least five feet beyond the perimeter foundations. All artificial fill present in exterior concrete slab-on-grade and pavement areas should be subexcavated and recompacted. If the risks of differential movement and cracking of the exterior slabs-on-grade and pavements would be unacceptable, the top 18 inches of the slab and pavement subgrade should consist of low to non-expansive engineered fill. Low to non-expansive engineered fill constructed for the support of foundations and concrete slabs-on-grade should be placed and compacted over properly moisture conditioned and compacted site soils.

All fill material should be placed and compacted in accordance to the recommendations presented in Table 2. It is recommended that any import fill to be used on site be of a low to non-expansive nature and meet the following criteria:

Plasticity Index	less than 12
Liquid Limit	less than 35
Percent Soil Passing #200 Sieve	between 15% and 40%
Maximum Aggregate Size	4 inches

All fills should be placed in lifts no greater than eight inches in loose thickness and compacted to the general recommendations provided for engineered fill.

TABLE 2
SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations*
General Engineered Fill (Native)	In lifts, a maximum of eight inches loose thickness, compact to a minimum of 90 percent relative compaction at two to four percent over the optimum moisture content.
General Engineered Fill (Import)	In lifts, a maximum of eight inches loose thickness, compact to a minimum of 90 percent relative compaction at or within two percent of the optimum moisture content.
Trenches**	Compact to at least 90 percent relative compaction at two to four percent over the optimum moisture content. Compact at or within two percent of the optimum moisture content if low to non-expansive fill is used.
Pavement Areas	Compact the top eight inches of the subgrade and the entire aggregate base section to at least 95 percent relative compaction at or within two percent of the optimum moisture content. Moisture conditioned the subgrade to two percent over the optimum moisture content if site soils are used.

*All compaction requirements stated in this report refer to dry density and moisture content relationships obtained through the laboratory standard described by ASTM D-1557-12.

Cut and fill slopes should be no steeper than two horizontal to one vertical (2H:1V). Steeper slopes should be retained.

A representative of PJC should observe all site preparation and fill placement. It is important that during the stripping, grading and scarification processes, a representative of our firm should be present to observe whether any undesirable material is encountered in the construction area.

Generally, grading is most economically performed during the summer months when on site soils are usually dry of optimum moisture content. Delays should be anticipated in site grading performed during the rainy season or early spring due to excessive moisture in on-site soils. Special and relatively expensive construction procedures should be anticipated if grading must be completed during the winter and early spring.

11. FOUNDATIONS: POST-TENSION SLABS

The proposed structures could be supported on post-tensioned slab foundations. The slabs should be designed in accordance with the following recommendations.

a. Vertical Loads. The post-tensioned mat slabs should be designed to be rigid and capable of resisting both positive and negative moments in areas of non-uniform support due to differential movement from the shrink and swell cycles of expansive clay soils. For design purposes, we recommend that the slabs be designed to span areas of non-uniform support for full structural loading in both directions.

The post tension slabs may be designed according to the following criteria, based on the method developed by the Post-Tensioning Institute (PTI) 2012 Edition and subsequent addendums.

i.	Edge Moisture Variation Distance (center lift) =	9.0 feet
ii.	Edge Moisture Variation Distance (edge lift) =	4.0 feet
iii.	Estimated Differential Shrink (center lift) =	1.34 inches
iv.	Estimated Differential Swell (edge lift) =	2.01 inches
v.	Allowable Bearing Capacity (dead plus live loads)	1,500 psf
vi.	Soil modulus of subgrade reaction (K_s) =	50 pci
vii.	Modulus of elasticity of the soil =	3,000 psi

We recommend a minimum slab thickness of 12 inches. The slab perimeter should be provided with a 12-inch wide and 12-inch deep thicken edge to reduce edge drying and storm water intrusion under the slab. The post tension slabs should be underlain by a four-inch layer of three-quarter inch gravel to act as a capillary break. To minimize moisture propagation through the slabs, the gravel should be covered by a 15-mil thick vapor retarder. The vapor retarder should be taped at all utility connections through the slabs to reduce the risk of moisture migration.

Concentrated loads within the slabs should be supported by thickened beams. The soils within the building pads should be maintained at two percent over optimum at all times. The subgrade material should not be allowed to dry out prior to post-tensioned slab construction.

b. Settlement. The majority of elastic settlement is expected to be small and occur during construction and placement of dead loads. Total elastic settlement is expected to be one inch or less. A maximum differential elastic settlement of one-half inch or less. The majority of elastic settlement should be completed within one year of applicable loads.

c. Lateral Loads. Resistance to lateral forces may be computed by using base friction and passive resistance. A friction factor of 0.30 is considered appropriate between the bottom of the concrete structures and the supporting soil. A passive pressure of 250 psf/ft may be used for structural elements embedded below lowest adjacent finish grade. The top six inches should be neglected for passive resistance due to desiccation and soil disturbance.

Special precautions must be taken during the placement and curing of concrete slabs-on-grade. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures and ad mixtures used during either hot or cold weather conditions will lead to excessive shrinkage, cracking or curling of the slabs. High water-cement ratios and/or improper curing also greatly increases water vapor transmission through the concrete which can be detrimental and damaging to floor coverings. Concrete placement and curing operations should be performed in accordance with the American Concrete Institute (ACI) manual.

12. CARPORT FOUNDATIONS — DRILLED PIERS

a. Vertical Loads. The carport structures may be supported on drilled, cast-in-place concrete piers a minimum of 12 inches in diameter and spaced at least three pier diameters center to center. The piers will derive their support through peripheral friction. Perimeter and interior piers should extend at least eight feet below the existing ground surface, regardless of structural loads. The piers should be reinforced and designed by the project structural engineer. All perimeter piers and piers supporting continuous loads should be tied together with grade beams. The grade beams should be designed to span between the piers in accordance with structural requirements.

   The portion of the piers extending at least three feet beneath the finished ground surface may be designed using an allowable dead plus live skin friction of 600 pounds per square foot (psf). This value may be increased by one-third for short duration wind and seismic loads. End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes and the uncertainty of mobilizing end bearing and skin friction simultaneously.

b. Settlement. The maximum settlement for the piers is estimated to be small and within tolerable limits.

c. Lateral Loads. Lateral loads resulting from wind or earthquakes can be resisted by the piers through a combination of cantilever action and passive resistance of the soils surrounding the pier. A passive pressure of 300 pounds per square foot per foot of depth acting on two pier diameters should be used. The upper foot should be neglected for passive resistance.

d. Pier Drilling. Free groundwater and/or caving-prone soils may be encountered within the planned pier depths. If groundwater is encountered or collects in pier holes, it will be necessary to de-water the holes and/or place the concrete using the tremie method. If caving soils are encountered, it may be necessary to case the holes. Furthermore, it may be practical to perform a drill and pour operation where the reinforcing steel and concrete for the caissons are placed immediately after drilling. The drilling subcontractor should review the geotechnical investigation report so they may choose suitable drill rigs to accomplish drilling, and determine the need for casing and de-watering.

13. NON-STRUCTURAL CONCRETE SLABS-ON-GRADE

Non-structural slabs-on-grade may be used for the structures provided they are supported on a 30 inch thick layer of low to non-expansive engineered fill. Non-structural slabs-on-grade may be used for exterior flatwork provided they are underlain by at least 18 inches of low to non-expansive engineered fill. The low to non-expansive fill material should extend at least five feet beyond perimeter foundations and three feet beyond the edges of exterior slabs.

All slab subgrades should be moisture conditioned according to the geotechnical engineer, and rolled to produce a firm, uniform and unyielding subgrade. The slab subgrade should not be allowed to dry. Non-structural slabs should be at least five inches thick and underlain with a capillary moisture break consisting of at least four inches of clean, free-draining crushed rock or gravel. The rock should be graded so that 100 percent passes the one-inch sieve and no more than five percent passes the No. 4 sieve.

Control joints should be provided to induce and control cracking. Exterior slabs should not be tied to foundations. We recommend that slabs be reinforced to reduce cracking due to thermal and curling stresses. However, some cosmetic cracking will likely occur. Special care should be taken to ensure that reinforcement is placed and maintained at least two inches below the top of the slab.

Special precautions must be taken during the placement and curing of concrete slabs-on-grade. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures and ad mixtures used during either hot or cold weather conditions will lead to excessive shrinkage, cracking or curling of the slabs. High water-cement ratios and/or improper curing also greatly increases water vapor transmission through the concrete which could be detrimental and damaging to floor coverings. Concrete placement and curing operations should be performed in accordance with the American Concrete Institute (ACI) manual.

14. SEISMIC DESIGN

Based on criteria presented in the 2019 edition of the California Building Code (CBC) and ASCE (American Society of Civil Engineers) STANDARD ASCE/SEI 7-16, the following minimum criteria should be used in seismic design:

a.	Site Class:	D		
b.	Mapped Acceleration Parameters:	S_S	=	1.751 g
		S_1	=	0.652 g
c.	Site Adjusted Spectral Response Acceleration Parameters:	S_{MS}	=	1.719 g
		S_{M1}	=	null
d.	Design Spectral Acceleration Parameters:	S_{DS}	=	1.146 g
		S_{D1}	=	null

According to section 11.4.8, Site-Specific Ground Motion Procedure, ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for structures on Site Class D and E sites with S1 greater than or equal to 0.2. The S1 for the subject site falls into this category. An exception to this criteria is provided in this section. We assumed that the exemption will be implemented for the project.

15. DRAINAGE

We recommend that the structures be provided with roof gutters and downspouts. Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly above slopes or adjacent to the building foundations or slabs. Surface runoff should be directed away from slopes and foundations. If the drainage facilities discharge onto the natural ground, adequate means should be provided to control erosion and to create sheet flow. Care must be taken so that discharges from the roof gutter and downspout systems are not allowed to infiltrate the subsurface near the structure or in the vicinity of slopes. Downspouts should be connected to closed conduits and discharged away from structures. Storm water must not be discharged on or near slopes; or it will cause erosion and stability problems.

16. PRELIMINARY ASPHALTIC CONCRETE PAVEMENT DESIGN

As previously discussed, the site is underlain by highly expansive soils. Pavements constructed on the highly expansive soils will be prone to differential settlement, heave, and cracking. We assumed an R-value of 5 in our pavement design. As a result, we judge that the existing soils will have a low supporting capacity (after properly compacted) when used as a pavement subgrade. Pavement designs sections are presented in Table 3. If low to non-expansive engineered fill is used for the top 18 inches of the pavement subgrade, pavement sections could be constructed according to Tables 4.

Pavement thicknesses were computed from Chapter 633 of the Caltrans Highway Design Manual and are based on a pavement life of 20 years. The Traffic Indices (TI's) used are judged representative of the anticipated traffic but are not based on actual vehicle counts. The actual traffic indexes should be determined and provided by the project civil engineer.

Prior to placement of the aggregate base material, the top eight inches of the pavement subgrade should be scarified to at least eight inches deep, moisture conditioned to within two percent of optimum (if engineered fill is used) or two to four percent over optimum (if native subgrade is used), and compacted to a minimum of 95 percent relative compaction. Aggregate base material should be spread in thin layers and compacted to at least 95 percent relative compaction to form a firm and unyielding base. The subgrade and aggregate base section should visually pass a firm unyielding proof-roll inspection.

The material and methods used should conform to the requirements of the Caltrans Standard Specifications, except that compaction requirements for the soil subgrade and aggregate baserock should be based on ASTM D-1557-12. Aggregate used for the base coarse should comply with the minimum requirements specified in Caltrans Standard Specifications, Section 26, for Class II aggregate base.

In general, the pavements should be constructed during the dry season to avoid the saturation of the subgrade and base materials, which often occurs during the wet winter months. If pavements are constructed during the winter and early spring, a cost increase relative to drier weather construction should be anticipated. The soils engineer should be consulted for recommendations at the time of construction.

Where pavements will abut landscaped areas, water can seep below the concrete curb and into the base rock within the pavement section. Continued saturation of the base rock leads to permanent wetness towards the lower elevation of the pavement where water ponds. Soft subgrade conditions and pavement damage can occur as a result.

Several precautionary measures can be taken to minimize the intrusion of water into the base rock; however, the cost to install the protective measures should be balanced against the cost of repairing damaged pavement sections. An alternative, which can be taken to extend the life of the pavement, would be to construct a cutoff wall along the perimeter edge of the pavement. The wall should consist of a lean concrete mix. The trench should be four inches wide and extend at least 36 inches deep.

Where trees are located adjacent to pavement areas, we recommend that a suitable impervious root barrier be included to minimize water mitigation into the pavement layer.

TABLE 3
PAVEMENT DESIGN FOR PAVEMENT AREAS
(Subgrade R-Value = 5)

Traffic Index	Asphaltic Concrete (in)	Class II Aggregate Base (in)
4.0	2.0	8.5
5.0	2.5	11.0
6.0	3.0	13.5
7.0	3.5	16.5

TABLE 4
PAVEMENT DESIGN FOR 18 INCHES OF LOW TO NON-EXPANSIVE ENGINEERED FILL
(Subgrade R-Value = 50)

Traffic Index	Asphaltic Concrete (in)	Class II Aggregate Base (in)
4.0	2.0	6.0
5.0	2.5	6.0
6.0	3.0	6.0
7.0	3.5	6.0

17. CONCRETE PAVEMENTS

We anticipate that concrete pavements may be used for the project. As with the case for asphaltic pavements, concrete pavements constructed on the weak and highly expansive soils may be prone to differential movement, heave and cracking. For optimum performance the recommendations provided in the earthwork section of this report should be followed.

Concrete pavements should be underlain by 18 inches of low to non-expansive fill. In addition, we recommend a minimum section of at least six inches of Class II base rock be placed under concrete pavements. The Class II base rock should be compacted to at least 95 percent at or within two percent of optimum moisture content. We recommend a minimum concrete pavement thickness of six inches for a TI of 4 and seven inches for a TI of 5 and 6.

Asphalt pavements near trash enclosures can be prone to depressions and rutting conditions. Therefore, we recommend that a reinforced concrete slab-on-grade pavement should be constructed within 10 feet of trash enclosures.

18. LIMITATIONS

The data, information, interpretations and recommendations contained in this report are presented solely as bases and guides to the geotechnical design of the proposed Veterans Village located at 2149 West Hearn Avenue in Santa Rosa, California. The conclusions and professional opinions presented herein were developed by PJC in accordance with generally accepted geotechnical engineering principles and practices. No warranty, either expressed or implied, is intended.

This report has not been prepared for use by parties other than the designers of the project. It may not contain sufficient information for the purposes of other parties or other uses. If any changes are made in the project as described in this report, the conclusions and recommendations contained herein should not be considered valid, unless the changes are reviewed by PJC and the conclusions and recommendations are modified or approved in writing. This report and the figures contained herein are intended for design purposes only. They are not intended to act by themselves as construction drawings or specifications.

Soil deposits may vary in type, strength, and many other important properties between points of observation and exploration. Additionally, changes can occur in groundwater and soil moisture conditions due to seasonal variations or for other reasons. Therefore, it must be recognized that we do not and cannot have complete knowledge of the subsurface conditions underlying the subject site. The criteria presented are based on the findings at the points of exploration and on interpretative data, including interpolation and extrapolation of information obtained at points of observation.

19. ADDITIONAL SERVICES

Upon completion of the project plans, they should be reviewed by our firm to determine that the design is consistent with the recommendations of this report. During the course of this investigation, several assumptions were made regarding development concepts. Should our assumptions differ significantly from the final intent of the project designers, our office should be notified of the changes to assess any potential need for revised recommendations. Observation and testing services should also be provided by PJC to verify that the intent of the plans and specifications are carried out during construction; these services should include observation of site grading, field density testing of engineered fill and aggregate base rock and observation of the foundation excavations. Special inspection services could also be needed including reinforcing, concrete, and/or welding.

These services will be performed only if PJC is provided with sufficient notice to perform the work. PJC does not accept responsibility for items we are not notified to observe.

It has been a pleasure working with you on this project. Please call if you have any questions regarding this report or if we can be of further assistance.

Sincerely,

PJC & ASSOCIATES, INC.

Patrick J. Conway
Geotechnical Engineer
GE 2303, California

REGISTERED PROFESSIONAL ENGINEER
PATRICK J. CONWAY
No. 2303
Exp. 3/31/22
GEOTECHNICAL
STATE OF CALIFORNIA

PJC:ab

APPENDIX A
FIELD INVESTIGATION

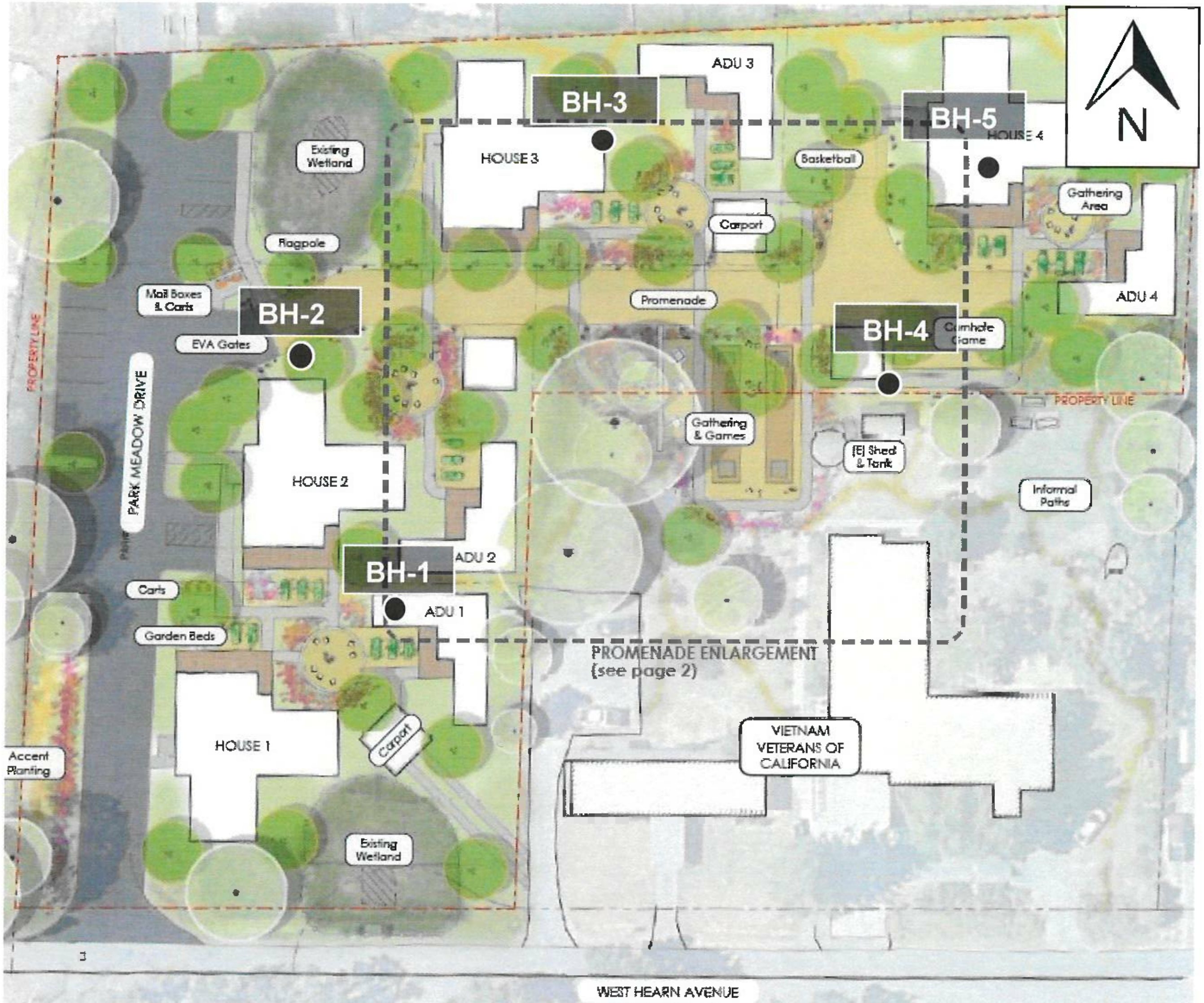
1. INTRODUCTION

The field program performed for this study consisted of advancing five exploratory boreholes within the project area. The exploration was completed on December 1, 2020. The borehole locations are shown on the Borehole Location Plan, Plate 2. Descriptive logs of the boreholes are presented in this appendix as Plates 3 through 7.

2. BOREHOLES

The boreholes were advanced were advanced using a Mobile B-53 drill with solid hollow flight augers. The drilling subcontractor on the project was Pearson Drilling of Forestville, California. The drilling was performed under the observation of a field geologist of PJC who maintained a continuous log of soil conditions and obtained soil samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System, as explained in Plate 8.

Relatively undisturbed and disturbed samples were obtained from the exploratory boreholes. A 2.43 in I.D. California Modified Sampler or a 1.5 in I.D. Standard Sampler was driven into the underlying soil using a 140 pound hammer falling 30 inches to obtain an indication in the field of the soil density and to allow visual examination of at least a portion of the soil column. Soil samples obtained with the split-spoon sampler were retained for further observation and testing. The number of blows required to drive the sampler at six-inch increments was recorded on each borehole log. During our subsurface exploration Shelby tubes were pushed at the desired elevations. All samples collected were labeled and transported to PJC's office for examination and laboratory testing.



NO SCALE

EXPLANATION

● BOREHOLE LOCATION AND DESIGNATION

REFERENCE: SITE PLAN PREPARED BY QUADRIGA, DATED OCTOBER 1, 2020.

PJC & Associates, Inc. Consulting Engineers & Geologists	BOREHOLE LOCATION PLAN PROPOSED HEARNS VETERANS VILLAGE 2149 WEST HEARN SANTA ROSA, CALIFORNIA	PLATE 2
	Proj. No: 4433.02 Date: 1/2021 App'd by: PJC	

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-1

CLIENT Community Housing of Sonoma County

PROJECT NAME Veteran's Village

JOB NUMBER 4433.02

LOCATION 2149 West Hearn Avenue

DATE STARTED 12/1/20 COMPLETED 12/1/20 GROUND ELEVATION ______ HOLE SIZE 4"

DRILLING CONTRACTOR Pearson Drilling

GROUND WATER LEVELS:

DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer

AT TIME OF DRILLING --- No free groundwater encountered

LOGGED BY AB CHECKED BY PJC

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0.0		0.0' - 1.5'; SANDY SILT (ML); moderate brown, slightly moist, loosely compacted, low plasticity (FILL).								
		1.5' - 3.5'; SANDY CLAY (CL); mottled orange to light gray, moist, hard, medium plasticity (Qal).	MC	25	4.5	102	8			
2.5										
		3.5' - 5.0'; SANDY CLAY (CH); grayish brown, moist, hard, high plasticity (Qal).	MC	19	4.5	107	17			
5.0		5.0' - 7.5'; CLAYEY GRAVEL (GC); orange-gray, moist, medium dense, fine to medium grained, subrounded gravels (Qal).								
			MC	29		114	11			
7.5		7.5' - 13.5'; SANDY CLAY (CH); olive brown, moist, very stiff, high plasticity (Qal).								
10.0			MC	25	3.5	100	23			
12.5										
		13.5' - 15.0'; SANDY GRAVEL (GW); grayish orange, moist, very dense, fine to coarse grained, subrounded gravel (Qal).	MC	65		118	12			
15.0										

Bottom of borehole at 15.0 feet.

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PJC & Associates, Inc.

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BORING NUMBER BH-2

CLIENT Community Housing of Sonoma County
PROJECT NAME Veteran's Village
JOB NUMBER 4433.02
LOCATION 2149 West Hearn Avenue
DATE STARTED 12/1/20
COMPLETED 12/1/20
GROUND ELEVATION
HOLE SIZE 4"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer
AT TIME OF DRILLING --- No free groundwater encountered
LOGGED BY AB
CHECKED BY PJC
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX
0.0		0.0' - 2.75'; SANDY CLAY (CL); dark brown to gray, slightly moist, loosely compacted, high plasticity (FILL).								
			MC	18	4.5	91	11	41	16	25
2.5										
		2.75' - 4.5'; SANDY CLAY (CL); mottled orange to light gray, slightly moist, hard, medium plasticity (Qal).	GB							
			MC	16	4.5	97	8			
5.0		4.5' - 10.0'; SANDY CLAY (CH), olive gray to orange, very moist, very stiff, high plasticity (Qal).								
7.5										
			MC	15	2.5	92	30			
10.0										

Bottom of borehole at 10.0 feet.

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BORING NUMBER BH-3

CLIENT Community Housing of Sonoma County

PROJECT NAME Veteran's Village

JOB NUMBER 4433.02

LOCATION 2149 West Hearn Avenue

DATE STARTED 12/1/20 COMPLETED 12/1/20 GROUND ELEVATION HOLE SIZE 4"

DRILLING CONTRACTOR Pearson Drilling

GROUND WATER LEVELS:

DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer

AT TIME OF DRILLING --- No free groundwater encountered

LOGGED BY AB CHECKED BY PJC

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX
0.0		0.0' - 3.25'; SANDY CLAY (CL); dark brown, slightly moist, loosely compacted, high plasticity (FILL).	MC	19	4.5	95	12			
2.5		3.25' - 5.0'; SANDY CLAY (CL); mottled orange, slightly moist, hard, medium plasticity (Qal).	MC	23		96	6			
5.0		5.0' - 10.0'; GRAVELY CLAY (CH); mottled olive to gray-brown, moist, hard, high plasticity (Qal).	MC	49	4.5	101	18			
7.5										
10.0		10.0' - 11.0'; CLAYEY SAND WITH GRAVEL (SC); orange-gray, slightly moist, medium dense, fine to coarse sands (Qal).	SPT	27			8			

Bottom of borehole at 11.0 feet.

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PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-4

CLIENT Community Housing of Sonoma County
PROJECT NAME Veteran's Village
JOB NUMBER 4433.02
LOCATION 2149 West Hearn Avenue
DATE STARTED 12/1/20
COMPLETED 12/1/20
GROUND ELEVATION
HOLE SIZE 4"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer
▽ AT TIME OF DRILLING 12.50 ft
LOGGED BY AB
CHECKED BY PJC
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0.0		0.0' - 2.5'; SANDY CLAY (CL); dark brown, slightly moist, loosely compacted, high plasticity (FILL).								
2.5			MC	14	4.5	98	8			
		2.5' - 4.0'; SANDY CLAY (CL); mottled orange to olive brown, moist, hard, medium plasticity (Qal).								
		4.0' - 6.5'; SANDY CLAY (CH); mottled light brown to olive, moist, hard, high plasticity, trace gravels and roots (Qal).	MC	33	4.5	109	16			
5.0										
			MC	56	4.5	116	13			
		6.5' - 9.5'; SANDY CLAY (CH); olive brown to gray, moist, hard, high plasticity (Qal).								
7.5										
10.0		9.5' - 15.0'; SANDY GRAVEL (GW); mottled gray to olive, moist to saturated, dense, fine to coarse grained, subrounded gravel (Qal).	MC	60			14			
12.5	▽									
15.0			SPT	35						

Bottom of borehole at 15.0 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-5

CLIENT Community Housing of Sonoma County **PROJECT NAME** Veteran's Village

JOB NUMBER 4433.02 **LOCATION** 2149 West Hearn Avenue

DATE STARTED 12/1/20 **COMPLETED** 12/1/20 **GROUND ELEVATION** **HOLE SIZE** 4"

DRILLING CONTRACTOR Pearson Drilling **GROUND WATER LEVELS:**

DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer ∇ **AT TIME OF DRILLING** 11.50 ft

LOGGED BY AB **CHECKED BY** PJC **AT END OF DRILLING** ---

NOTES **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX
0.0		0.0' - 1.0'; SANDY SILT (ML); light brown, slightly moist, loosely compacted, medium plasticity (FILL).								
		1.0' - 6.0'; SANDY CLAY (CL); mottled orange and brown, slightly moist to moist, hard, medium plasticity (Qal).	MC	32	4.5	98	7			
2.5										
			MC	56						
5.0										
			MC	66	4.5	100	19			
		6.0' - 8.5'; SANDY GRAVEL (GW); mottled orange and brown, moist, dense, fine to coarse grained, subrounded gravel (Qal).								
7.5			MC	57		128	10			
		8.5' - 11.5'; SANDY GRAVEL (GW); olive brown, moist, dense, fine to coarse grained, subrounded gravel (Qal).								
10.0			MC	65		113	13			
	∇									

Bottom of borehole at 11.5 feet.

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MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than half is larger than #200 sieve	GRAVELS more than half coarse fraction is larger than no. 4 sieve size	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND MIXTURES
	SANDS more than half coarse fraction is smaller than no. 4 sieve size	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVEL-SAND MIXTURES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than half is smaller than #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, SILTY OR CLAYEY FINE SANDS, VERY FINE SANDS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS OR LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

KEY TO TEST DATA

LL — Liquid Limit (in %)
PL — Plastic Limit (in %)
G — Specific Gravity
SA — Sieve Analysis
Consol — Consolidation

- ■ "Undisturbed" Sample
- ☒ Bulk or Disturbed Sample
- ☐ No Sample Recovery

	Shear Strength, psf	Confining Pressure, psf	
*Tx	320	(2600)	Unconsolidated Undrained Triaxial
Tx CU	320	(2600)	Consolidated Undrained Triaxial
DS	2750	(2000)	Consolidated Drained Direct Shear
FVS	470		Field Vane Shear
*UC	2000		Unconfined Compression
LVS	700		Laboratory Vane Shear

Notes: (1) All strength tests on 2.8" or 2.4" diameter sample unless otherwise indicated
(2) * Indicates 1.4" diameter sample

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Consulting Engineers & Geologists

USCS SOIL CLASSIFICATION KEY
PROPOSED HEARNS VETERANS VILLAGE
2149 WEST HEARN
SANTA ROSA, CALIFORNIA

PLATE 8

Proj. No: 4433.02 Date: 1/2021 App'd by: PJC

APPENDIX B
LABORATORY INVESTIGATION

1. INTRODUCTION

This appendix includes a discussion of the test procedures of the laboratory tests performed by PJC for use in the geotechnical study. The testing was carried out employing, whenever practical, currently accepted test procedures of the American Society for Testing and Materials (ASTM).

Undisturbed and disturbed samples used in the laboratory investigation were obtained from various locations during the course of the field investigation, as discussed in Appendix A of this report. Identification of each sample is by borehole number, sample number and depth. All of the various laboratory tests performed during the course of the investigation are described below.

2. INDEX PROPERTY TESTING

In the field of soil mechanics and geotechnical engineering design, it is advantageous to have a standard method of identifying soils and classifying them into categories or groups that have similar distinct engineering properties. The most commonly used method of identifying and classifying soils according to their engineering properties is the Unified Soil Classification System as described by ASTM D-2487-83. The USCS is based on a recognition of the various types and significant distribution of soil characteristics and plasticity of materials.

The index properties tests discussed in this report include the determination of natural water content and dry density and Atterberg Limits testing.

a. <u>Natural Water Content and Dry Density</u>. The natural water content and dry density of the soils were determined on selected samples. The samples were extruded, visually classified, and accurately measured to obtain the volume and wet weight. The samples were then dried, in accordance with ASTM D-2216-80, for a period of 24 hours in an oven maintained at a temperature of 100 degrees C. After drying, the weight of each sample was determined and the moisture content and dry density calculated. A similar procedure was used to determine the water content only for disturbed samples.

b. <u>Atterberg Limits Determination</u>. The liquid and plastic limits of selected fine-grained soil samples were determined by air drying and breaking down the sample. The results of the limits are shown on the borehole logs.

3. ENGINEERING PROPERTIES TESTING

The engineering properties tests discussed in this report include pocket penetrometer testing.

a. <u>Pocket Penetrometer</u>. Pocket Penetrometer tests were performed on cohesive samples. The test estimates the unconfined compressive strength of a cohesive material by measuring the materials resistance to penetration by a calibrated, spring-loaded cylinder. The maximum capacity of the cylinder is 4.5 tons per square foot (tsf). The results of these test are indicated on the borehole logs.

APPENDIX C
REFERENCES

1. "Foundations and Earth Structures" Department of the Navy Design Manual 7.2 (NAVFAC DM-7.2), dated May 1982.

2. "Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction" Department of the Navy Design Manual 7.3 (NAVFAC DM-7.3), dated April 1983.

3. Geologic Map of the Santa Rosa Quadrangle, Scale: 1:250,000, compiled by D.L. Wagner and E.J. Bortugno, 1982.

4. Geology for Planning in Sonoma County, Special Report 120, California Division of Mines and Geology, 1980.

5. "Soil Mechanics" Department of the Navy Design Manual 7.1 (NAVFAC DM-7.1), dated May 1982.

6. McCarthy, David. Essential of Soil Mechanics and Foundations. 5th Edition, 1998.

7. Bowels, Joseph, Engineering Properties of Soils and Their Measurement. 4th Edition, 1992.

8. California Building Code (CBC), 2019 edition.

9. Report titled, "Design Level Geotechnical Investigation, Proposed Low Income Veteran Housing & Convalescent Home Addition, 2149 West Hearn Avenue, Santa Rosa, California", prepared by PJC & Associates, Inc., dated December 7, 2009.

10. Project Plans, titled "Hearn Veterans Village," Sheets 1 through 4 and A3.0-A though A3.2-B, prepared by Fritz Architecture, dated May 20, 2020 and August 29, 2020.