PJC & Associates, Inc.
Consulting Engineers & Geologists

March 18, 2021 Job No. 9852.01

Falcon Point Associates, LLC
c/o: DRG Builders
Attention: Doyle Heaton
doyle@drgbuilders.com
c/o: Steven J. Lafranchi & Associates, Inc.
Attention: Bobbi Wolff
bobbi@sjla.com

Subject: Geotechnical Report Update
Proposed Residential Development
270 & 280 Casa Grande Road
Petaluma, California
APN: 017-040-008 & 015

References: Report titled, "Geotechnical Investigation, Proposed Residential Subdivision, 270 & 280 Casa Grande Road, Petaluma, California," prepared by PJC & Associates, Inc., dated September 21, 2020.

Civil Engineering Plans titled, "Creekwood," Sheets C-1 through C-8, prepared by Steven J. Lafranchi & Associates, Inc., dated February 9, 2021.

Dear Doyle:

PJC & Associates, Inc. (PJC) is pleased to submit this letter which updates our geotechnical investigation report for the above referenced project. Based on our review of the referenced plans it is our understanding that the project has been revised and will now include the construction of 42 residential condo units, which is a medium dense residential development. Based on our review our report and the referenced plans, we judge that the revised project scope does not change the recommendations and design criteria provided in our report.

We trust that this is the information you require at this time. If you have any questions concerning the content of this report, please feel free to call.

Sincerely,

PJC & ASSOCIATES, INC.

Patrick J. Conway
Geotechnical Engineer
GE 2303, California

REGISTERED PROFESSIONAL ENGINEER
PATRICK J. CONWAY
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PJC:ss

CREEKWOOD

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
270 & 280 CASA GRANDE ROAD
PETALUMA, CALIFORNIA
APN: 017-040-008 & 015

PREPARED FOR:

FALCON POINT ASSOCIATES, LLC
C/O: DRG BUILDERS
ATTN: DOYLE HEATON
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PREPARED BY:

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JOB NO. 9852.01

PJC & Associates, Inc.
Consulting Engineers & Geologists

September 21, 2020 Job No. 9852.01

Falcon Point Associates, LLC
c/o: DRG Builders
Attn: Doyle Heaton
doyle@drgbuilders.com

Subject: Geotechnical Investigation
Proposed Residential Development
270 & 280 Casa Grande Road
Petaluma, California
APN: 017-040-008 & 015

Dear Doyle:

PJC & Associates, Inc. (PJC) is pleased to submit this report presenting the results of our geotechnical investigation for the proposed residential subdivision located at 270 & 280 Casa Grande Road in Petaluma, California. The location of the site is shown on the Site Location Map, Plate 1. The site corresponds to the geographic coordinates of 38.2414° N and 122.5965° W, according to Google Earth Imagery. Our services were completed in accordance with our proposal for geotechnical engineering services, dated March 24, 2020, and your authorization to proceed with the work, dated May 27, 2020. This report presents opinions and recommendations regarding the geotechnical engineering aspects of the design and construction of the proposed project. Based on the results of this study, we judge that the project is feasible from a geotechnical engineering standpoint provided the recommendations and criteria presented in this report are incorporated in the design and carried out through construction.

We appreciate the opportunity to be of service. If you have any questions concerning the content of this report, please contact us.

Sincerely,

PJC & ASSOCIATES, INC.

Patrick J. Conway
Geotechnical Engineer
GE 2303, California

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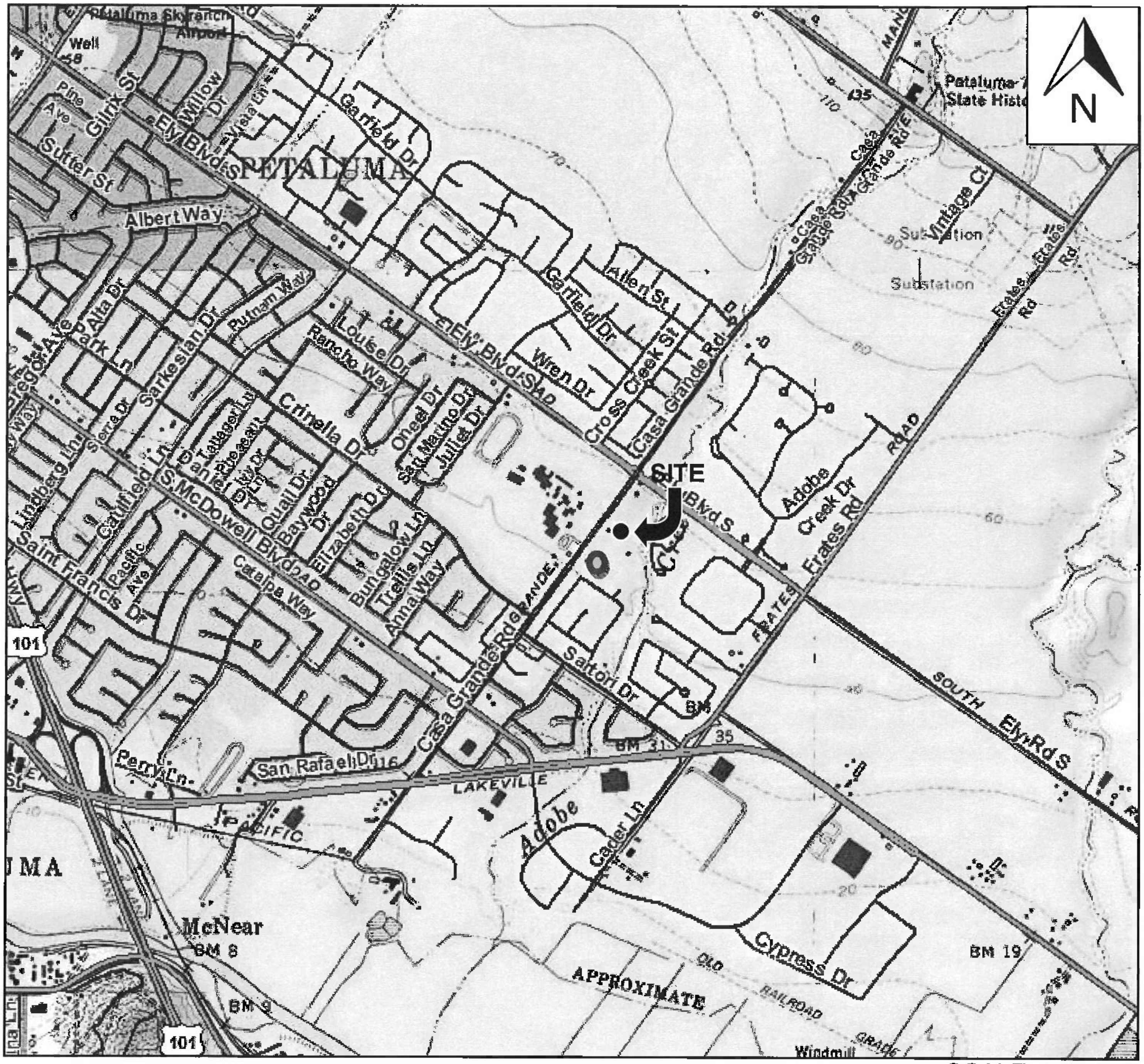

SCALE: 1:24,000

REFERENCE: USGS PETALUMA RIVER, CALIFORNIA 7.5 MINUTE QUADRANGLE, DATED 1978.

PJC & Associates, Inc. Consulting Engineers & Geologists	SITE LOCATION MAP PROPOSED SUBDIVISION 270 & 280 CASA GRANDE ROAD PETALUMA, CALIFORNIA	PLATE 1
	Proj. No: 9852.01 Date: 8/20 App'd by: PJC	

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GEOTECHNCIAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION
270 & 280 CASA GRANDE ROAD
PETALUMA, CALIFORNIA

1. PROJECT DESCRIPTION

Based on a preliminary site development plan dated October 30, 2019 and information provided by you, it is our understanding that the subdivision will consist of the development of 36 residential lots. The development will consist of the construction of single-family residences expected to be one or two-story, wood-frame structures with post-tension concrete slab-on-grade floors. The project will also include the construction of a new asphaltic concrete paved roadways and driveways and serviced by underground municipal utilities.

Structural loading information was not available at the time of this report. For our analysis, we assume that structural loads for the buildings will be relatively light, with dead plus live continuous wall loads less than two kips per lineal foot, 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.

The project site is situated on nearly level terrain. Based on the site topography, we anticipate that site grading and earthwork will consist of cuts and fills of approximately three feet or less to upgrade the existing site soils, achieve the desired finish pad and roadway grades and to provide adequate gradients for site drainage. However, grading for the stormwater retention basin is shown to consist of cuts and fills up to seven feet and less. We do not anticipate that retaining walls will be required for the project.

2. PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to evaluate the subsurface conditions at the site and develop geotechnical criteria for design and construction of the proposed project as described above. Specifically, the scope of our services consisted of the following:

a. Drilling eight exploratory boreholes to depths up to 50.5 feet below the existing ground surface with a truck mounted drill rig to characterize the soil and groundwater conditions underlying the site. Our project engineer was on site to observe the drilling, log the materials encountered in the boreholes, and obtain representative samples for visual classification and laboratory testing.

b. Laboratory observation and testing were performed on representative soil samples obtained during the course of the field investigation to assist in the evaluation of the engineering properties of the soils underlying the site.

c. Review seismological and geologic literature on the site area, discuss site geology and seismicity, and evaluate potential geologic hazards and earthquake effects (i.e., liquefaction, fault ground rupture, settlement, lurching and lateral spreading, densification, expansive soils, etc.).

d. Perform engineering analyses to develop geotechnical recommendations for site preparation and grading, foundation type(s) and design criteria, support of concrete slabs-on-grade, preliminary pavement design criteria, site surface and subsurface drainage and construction considerations.

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

3. SITE CONDITIONS

a. General. The subject property is located southeast of downtown Petaluma in a fully developed residential area comprised of single-family residences, Casa Grande High School, a Petaluma Ecumenical property and isolated open fields. The project site is located on the southeast side of Casa Grande Road on a partially developed lot. At the time of our investigation on June 29, 2020, the site was occupied by two existing single-family residences and open grassland. It is planned to demolish the residence at 280 Casa Grande Road while the residence at 270 Casa Grande Road will remain. The site is bounded by Casa Grande Road to the west, a planned development to the south, Adobe Creek to the east, and a Petaluma Ecumenical property to the north.

b. Topography. The site is located near the southern end of Petaluma Valley. The project site is located on nearly level terrain. According to the USGS Petaluma River California 7.5 Minute Quadrangle, the site is located near an elevation of 48 feet above mean sea level (MSL).

c. Drainage. Site drainage consists of sheet flow and surface infiltration which migrates in an easterly direction towards Adobe Creek which borders the eastern margin of the property. An evaluation of the flood potential of Adobe Creek is beyond the scope of this report and is being performed by others.

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 late Jurassic time. This process involved eastward thrusting of oceanic crust beneath the continental crust (Klamath Mountains and Sierra Nevada) and the scraping off of materials that were 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 in mid-Cenozoic time, and has progressed northward to the vicinity of Cape Mendocino in Southern Humboldt County. Thus, the principal structures south of Cape Mendocino are northwest-trending, nearly vertical faults of the San Andreas system.

According to the Geologic Map of the Petaluma River Quadrangle prepared by the California Geologic Survey (CGS), the site is underlain by Holocene aged alluvial fan deposits (Qhf). These deposits consist of sand, gravel, silt and clay deposited by streams within canyons emanating onto alluvial valley floors. Our subsurface investigation confirmed the project site is underlain by alluvial fan deposits.

5. FAULTING

Geologic structures in the region are primarily controlled by northwest trending faults. No known active fault passes through the site. Based on published Geologic Maps reviewed, the site is not located in the Alquist-Priolo Earthquake Fault Studies Zone. Based on our review of the Geologic Map of the Petaluma River California 7.5 Minute quadrangle by the California Geologic Survey, a trace of the Tolay Fault exists approximately nine-tenths of one mile northeast of the site. However, the State of California has not classified this particular fault trace as an active fault source during Holocene time (the past 11,000 years).

According to the USGS National Seismic Hazard Map (2008), the three closest known active faults to the site are the Rodgers Creek, the West Napa and the San Andreas faults. The Rodgers Creek fault is located 2.27 miles to the northeast, the West Napa fault is located 15.55 miles to the east and the San Andreas fault is located 16.73 miles southwest of the site. 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	2.27	7.33
West Napa	15.55	6.70
San Andreas	16.73	8.05

Reference: USGS National Seismic Hazard Map (2008).

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 geologic hazards are discussed in Section 8 of this report.

7. SUBSURFACE CONDITIONS

a. Soils. The subsurface conditions at the project site were investigated by drilling eight exploratory boreholes (BH-1 through BH-8) to depths up to 50.5 feet below the existing ground surface. The approximate borehole locations are shown on the Borehole Location Plan, Plate 2. The boreholes were drilled to observe the soil and groundwater conditions underlying the site and collect samples for visual classification and laboratory testing. Complete lithologic descriptions of the subsurface conditions encountered and approximate contacts are presented on the log of the boreholes, Plates 3 through 10. The soils were classified in accordance with the Unified Soil Classification System, as explained on Plate 11. The drilling and sampling procedures and descriptive borehole logs are included in Appendix A of this report. The laboratory procedures are included in Appendix B.

The exploratory boreholes generally encountered fine grained alluvial deposits which extended to the maximum depths explored. The heterogeneous alluvial deposits consisted of sandy clays, gravelly clays and clayey sands. The cohesive alluvium appeared slightly moist to saturated, stiff to hard, exhibited medium to high plasticity characteristics and included intermittent gravel lenses. The clayey sand appeared saturated, medium dense and fine to coarse grained. Complete lithologic descriptions of the strata encountered are presented on the Logs of the Boreholes, Plates 3 through 10.

b. Groundwater. At the time of our subsurface exploration on June 29, 2020, phreatic groundwater was encountered at a depth of 16.0 feet below the ground surface in BH-1, 13.5 feet below the ground surface in BH-5 and 12.0 feet below the ground surface in BH-7. Groundwater was not encountered in the other boreholes. The phreatic groundwater table rises and falls by several feet throughout the year due to seasonal rainfall and other factors. The phreatic groundwater should not detrimentally impact the project. Perched groundwater zones near the surface are common in the area due to seasonal rainfall, but usually dissipates following the rainy season.

c. Hydrologic Soil Group. Based on our subsurface findings, we judge that the surface and near surface site soils have very low infiltration rates when thoroughly saturated. According to the Natural Resources Conservation Service (NRCS) guidelines, we judge the site soils should be designated as the NRCS Hydrologic Soil Group D.

8. GEOLOGIC HAZARDS AND SEISMIC CONDITIONS

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 designed lifetime of the project. The following discussion reflects the possible geologic hazards and earthquake effects which could result in damage to the proposed structures and improvements at the site.

a. Fault Rupture. Rupture of the ground surface could occur along known active fault traces. No evidence of existing faults or previous ground displacement on the site due to fault movement is indicated in the geologic literature or field exploration. Therefore, the likelihood of ground rupture at the site due to faulting is considered to be low.

b. Ground Shaking. The site has been subjected in the past 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 project. This should be taken into account in the design and construction of the project.

c. Liquefaction. The project site is not located in the State Designated seismic hazard liquefaction zone (Green Zone). Based on our review of the Association of Bay Area Governments (ABAG), interactive liquefaction susceptibility map, the site is considered to have moderate susceptibility to liquefaction during or immediately following a significant seismic event.

Liquefaction is a seismic hazard that occurs in saturated, low density, predominantly granular soils encountered 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. Upon dissipation of pore water pressures following shaking, there is reduction in the void ratio of the impacted soil particles that can cause differential ground settlement and lateral spreading. Low density, fine-grained sandy soils below the phreatic ground water are most susceptible to liquefaction. However, recent case studies have shown that soft silts, low plasticity clays and loose gravels with limited drainage paths are also susceptible to liquefaction. Bedrock materials and plastic clayey soils with a liquid limit (LL) greater than 37 and a plasticity index greater than 18 are generally not known to be prone to liquefaction. In addition, soil deposits older than Holocene time (11,000 years) are generally not prone to liquefaction.

The occurrence of this phenomenon is dependent on many complex factors including the intensity and duration of ground shaking, groundwater elevation at time of shaking, particle size distribution, consistency/relative density of the soil, overburden stress, age of deposit, and many other factors.

In order to evaluate liquefaction potential at the site, our borehole designated BH-1 was drilled to a depth of 50 feet below existing grade. We analyzed the potential for liquefaction of the strata using the simplified method by Seed and Idriss (1971). Based on the results of our analyses, we judge that the strata at the site is not prone to liquefaction due to high relative densities of the granular soils and high plasticities of the clay soils.

d. Densification. The soils encountered in our exploratory boreholes appear to have relatively low densification potential. Therefore, based on the results of our investigation we judge that the risk of soil densification at the site is low.

e. 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. There are no overly-steep exposed faces or banks in close proximity to the project site. Therefore, we judge that the risk of the proposed project being impacted by lateral spreading or lurching is low.

f. Expansive Soils. Based on Atterberg Limits testing (PI=40), Expansion Index testing (EI=146), and our visual observations, the surface soils at the site exhibit high plasticity characteristics and a very high expansion potential. Therefore, the site surface and near surface soils have a high expansion potential. The presence of expansive soils must be considered in design and construction of the project.

g. Stability and Erosion. The project site is not located in the State Designated earthquake induced landslide zone (Blue Zone). According to the Special Report 120 Regional Stability Map, the project site is located in a relatively stable area due to low slope inclinations (Area A). Terrain at the project is nearly level and is not considered to be prone to landsliding. No areas experiencing significant erosion or sediment transport were observed at the project site.

h. Corrosion. Based on our corrosion laboratory testing, it appears that the site soils are mildly alkaline with elevated chlorides, poor resistivity, very mildly elevated sulfates and redox is very mildly reduced. A detailed discussion and recommendations for extending the longevity of building materials and conduits buried in the site soils are presented on Plate 4a.

9. CONCLUSIONS

Based on the results of our geotechnical 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 concerns in design and construction of the project are the presence of weak and compressible surface soils and highly expansive surface soils.

The top two to three feet of surface soils are weak and compressible. Weak and compressible soils appear hard and strong when dry but can lose their strength rapidly and collapse from the loads of fills, foundations or slabs-on-grade as their moisture increases and approaches saturation. The moisture content of these soils can increase as a result of rainfall or when the natural upward migration of water vapor through the pores of the soils is impeded by fills, pavements, slabs-on-grade or foundations. Foundations, concrete slabs and pavements could experience intolerable differential settlement, distress and cracking if constructed on this material in its existing state. Furthermore, the differential settlement could cause architectural distress to the structures. This condition could be mitigated by engineering techniques consisting of subexcavation and replacement with a uniform layer of compacted engineered fill. As an alternative, foundations could be designed to resist deflections from ground movement.

Based on field observations, laboratory testing and our experience with similar projects in the area, the surface soils are highly expansive. Shrinking and/or swelling of expansive soils due to loss and increase in moisture content can cause distress and damage to concrete elements and architectural features of structures as well as asphaltic concretes and exterior flatwork.

To reduce the detrimental effects of these soils to within tolerable limits, we recommend the following geotechnical criteria for foundation support of the structures and support of exterior flatwork and pavements:

a. The proposed residential structures should be supported on a post-tension slab foundation designed to resist differential movement from weak and expansive soils. The upper 12 inches of soils beneath the structures should be scarified, moisture conditioned to three to five percent over optimum moisture content and compacted in accordance with the earthwork and grading section of this report.

b. The top 18 inches of soil beneath exterior flatwork, such as driveways and sidewalks, should consist of an imported low to non-expansive compacted engineered fill. If desired, asphaltic concrete pavements or lime treated soils could be supported by at least 18 inches of imported low to non-expansive compacted engineered fill or lime treated soils. By importing low to non-expansive engineered fill or the exterior flatwork may consist of non-structural slabs-on-grade. If the implementation of this method is not performed, then heave and cracking, which could be severe, should be expected.

c. If importing low to non-expansive fill material is undesirable for asphaltic concrete pavements and exterior flatwork, the upper 18 inches of soils beneath asphaltic concrete and exterior flatwork could be lime treated. Additionally, a moisture cut off wall could be constructed for the sidewalk curbs to prevent the infiltration of water into the subgrade material and reduce cyclic moisture variation as described in section 15 of this report. If the implementation of this method is not performed, then heave and cracking of these asphaltic concrete pavements should be expected, which could be severe.

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

10. GRADING AND EARTHWORK

a. <u>Demolition and Stripping</u>. The existing structures, except for the residence that is to remain at the site, should be demolished and removed off site. Following demolition and removal of the existing undesired structures, structural areas should be stripped of surface vegetation, old fills, debris, tree stumps, underground utilities, etc. These materials should be removed from the site. Some of the stripped soils, if suitable, could be stockpiled for later use in landscape areas. If underground utilities pass through the site, they should 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 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.

b. Excavation and Compaction. Following site stripping, excavation should be performed to achieve finish grades and/or to prepare areas to receive fill. Where imported fill is proposed for exterior flatwork and/or pavements, we recommend the upper 18 inches of expansive site soils be removed and replaced with low to non-expansive engineered fill. For the residential structures, we recommend that the weak surface soils within the proposed building envelopes be scarified to a depth of 12 inches moisture conditioned to three to five percent over optimum moisture content and recompacted to 85 percent relative compaction. All desiccation cracks should be closed. The lateral extent of the subexcavation/scarification should extend at least five feet beyond perimeter foundations of the structures and three feet beyond exterior flatwork and pavements.

The bottoms of subexcavations scheduled to receive fill should be scarified to a depth of eight inches, moisture conditioned to a moisture content of three to five percent over optimum moisture content, and recompacted to a minimum of 85 percent of the materials relative maximum dry density as determined by ASTM D-1557 test procedures. All desiccation cracks must be closed. All fill material should be placed and compacted in accordance to the recommendations presented in Table 2. Imported fill to be used on site and should be of a low to non-expansive nature and should meet the following criteria:

Plasticity Index	12 or less
Liquid Limit	38 or less
Percent Soil Passing #200 Sieve	between 15% and 35%
Maximum Aggregate Size	4 inches

The excavated material onsite, free of organics, and rock fragments greater than four inches would be suitable for use as engineered fill in landscape areas. In exterior flatwork and pavement areas, the top 18 inches should consist of low to non-expansive material approved by the geotechnical engineer prior to importing to the site or consist of lime treated soil.

All fills should be placed and compacted to the general recommendations provided below.

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 at two to four percent over the optimum moisture content.
Import Fill (Low to Non-Expansive)	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 (Import)	Compact to at least 90 percent relative compaction at or within two percent of the optimum moisture content.
Driveways and Parking Areas (Low to Non-Expansive)	Compact the top eight inches of subgrade and the entire base rock section to at least 95 percent relative compaction at or within two percent of optimum moisture content.

*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.

c. <u>Temporary Slopes.</u> We do not anticipate that a mass excavation will be required for the project. However, temporary slopes may be required for underground utility construction. Based on our findings, we recommend that temporary slopes should not exceed three fourths horizontal to one vertical (3/4H:1V). If steeper slopes are required, shoring should be used. The geotechnical engineer should observe the excavation to determine if steeper cut slopes are feasible or shoring is necessary during construction. Temporary cut slopes should not be left exposed longer than absolutely necessary. The highly plastic clay soils will dry out and be prone to planar shear failures.

Permanent cut and fill slopes for the stormwater retention basin should be no steeper than two horizontal to one vertical (2H:1V). Steeper slopes should be retained. The cut slopes will expose highly plastic clays. Over time, due to desiccation, the slopes likely will experience shallow failures unless treated. Therefore, maintenance of these slopes will be necessary. If optimum performance is required, 30 inches of compacted low to non-expansive fill should be placed over the slopes or retaining structures should be implemented.

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. LIME TREATMENT OPTION

If the importation of low to non-expansive engineered fill material is undesirable for support of asphaltic concrete pavements and/or exterior flatwork, the expansive site soils should be lime treated. If post tension slab-on-grade foundations are used, it would not be necessary to lime treat soils within building pads.

a. Subexcavation. The highly expansive site soils beneath pavements and exterior slabs should be subexcavated to a depth of 18 inches below the subgrade elevation. The lateral extent of lime treatment should be a minimum of three feet beyond the edges of exterior concrete slabs and pavements. The bottoms of all of the subexcavations should be scarified to a depth of eight inches, moisture conditioned to a moisture content of three to five percent over optimum moisture content, and recompacted to a minimum of 88 percent of the materials relative maximum dry density.

b. Staging. A staging area for mixing of the highly expansive site soils and powdered lime should be established at the site. The highly expansive soils should be transported to the staging area and be moisture conditioned and amended with powdered lime. The lime treated site soils should then be transported back to the subexcavation and be spread in loose, eight inch thick lifts. We recommend that the lime-treated soils be moisture conditioned to two percent over optimum moisture content, and compacted to at least 90 percent relative compaction. With proper mixing equipment, it is possible that the on-site expansive clays could be treated in place. However, this should be evaluated in the field during grading and earthwork by the geotechnical engineer.

c. Lime Application. The expansive soils should be treated with high calcium or dolomitic quick lime. For preliminary budgeting purposes, we recommend a blend of at least five percent powdered lime (by dry weight) be evenly mixed with the site soils. Laboratory testing should be performed on trial samples to establish the percentage, by dry weight, of lime to be used. Ten days should be allowed to perform the testing prior to bidding and construction. The performance of lime stabilized soil is critically dependent on uniform mixing of the lime into the highly expansive soils and providing a proper curing period following amendment with the lime.

d. Quality Control. An experienced lime stabilization contractor, along with a comprehensive quality control program, is required to achieve the best possible stabilized soils. PJC should also perform laboratory testing during and following lime application. The powdered lime purchase order receipts should be provided to PJC to be kept on record.

12. FOUNDATION: POST-TENSION SLAB-ON-GRADE

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

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

The post tension slab 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) =	5.0 feet
iii.	Estimated Differential Shrink (center lift) =	1.50 inches
iv.	Estimated Differential Swell (edge lift) =	2.00 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 slab 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 slab, the gravel should be covered by a 15-mil thick vapor retarder. The membrane should be taped at all utility connections through the slabs to reduce the risk of moisture migration.

Concentrated loads within the slab should be supported by thickened beams. The soils within the building pad should be maintained at two to four percent over optimum at all times. The subgrade material should not be allowed to dry out prior to post-tensioned slab construction. If the slab subgrade is allowed to dry, all desiccation cracks should be moisture conditioned and closed.

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. Concrete placement and curing operations should be performed in accordance with the American Concrete Institute (ACI) manual.

b. Post Construction 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 less than one inch. A maximum differential elastic settlement of one-half inch is anticipated.

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 supporting soil. A passive pressure of 250 psf/ft may be used for structural elements embedded in the clay soils. The top 12 inches should be neglected for passive resistance due to desiccation and soil disturbance.

13. NON-STRUCTURAL CONCRETE SLABS-ON-GRADE

Non-structural concrete slabs-on-grade may be used for exterior flatwork provided the slabs are underlain by at least 18 inches of a low to non-expansive compacted fill or lime treated soils. The low to non-expansive fill should extend at least three feet beyond exterior slab edges and pavements.

All slab subgrades should be moisture conditioned and rolled to produce a firm and uniform 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.

For slabs-on-grade with moisture sensitive surfacing, we recommend that a vapor retarder at least 15 mils thick be placed over the drain rock to prevent migration of moisture vapor through the concrete slabs. Control joints should be provided to induce and control cracking. The slabs should be cast and maintained separate of adjacent foundations.

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. 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 = 1.754 g S_1 = 0.666 g
c.	Spectral Response Acceleration Parameters:	S_{MS} = 2.105 g S_{M1} = null
d.	Design Spectral Acceleration Parameters:	S_{DS} = 1.403 g S_{D1} = null

15. ASPHALTIC CONCRETE PAVEMENTS

An R-value of 5 was assigned to the site soils for the project. We recommend that the pavement base rock section should be underlain by at least 18 inches of low to non-expansive compacted engineered fill or lime treated soils to reduce the risk of severe cracking. Pavement sections should be constructed according to Table 3 if native soils are used to support the pavement. Table 4 may be used if the subbase consists of import or lime treated soils. If treatment if treatment is not implemented, cracking and high level of maintenance should be expected.

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 Indexes (TI) 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 the optimum moisture content, 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 course should comply with the minimum requirements specified in Caltrans Standard Specifications, Section 26, for Class 2 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 geotechnical 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 and subgrade 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" LOW TO NON-EXPANSIVE ENGINEERED FILL OR LIME TREATED SOIL
(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	7.0

16. UTILITY TRENCHES

Shallow excavations for utility trenches can be readily made with either a backhoe or trencher; larger earth moving equipment should be used for deeper excavations. We expect the walls of trenches less than five feet deep, excavated into engineered fill or native soils, to remain in a near-vertical configuration during construction provided no equipment or excavated spoil surcharges are located near the top of the excavation. If the trench extends deeper than five feet, then the trench walls may become unstable and may require shoring. All trenches should conform to the current CAL-OSHA requirements for worker safety.

The trenches may be backfilled with import soils and compacted to at least 90 percent of maximum dry density. The backfill soils should be moisture conditioned according to Table 2 of this report before compacting. Jetting should not be used.

Special care should be taken in the control of utility trench backfilling in structural areas. Substandard compaction may result in excessive settlements resulting in damage to structures constructed above them.

17. 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 adjacent to the building foundations, slabs or pavements. Surface runoff should be directed away from 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 soils near the structures. Downspouts should be connected to closed conduits and discharged away from structures.

18. LIMITATIONS

The data, information, interpretations and recommendations contained in this report are presented solely as bases and guides to the preliminary geotechnical design of the proposed residential subdivision located at 270 & 280 Casa Grande Road in Petaluma, 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 building loads and 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 is carried out during construction; these services should include observation of grading and earthwork, approving slab subgrade, approving pavement sections, and observing the installation of drainage provisions. 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 the content of this report or if we may be of further assistance.

Sincerely,

PJC & ASSOCIATES, INC.

APPENDIX A
FIELD INVESTIGATION

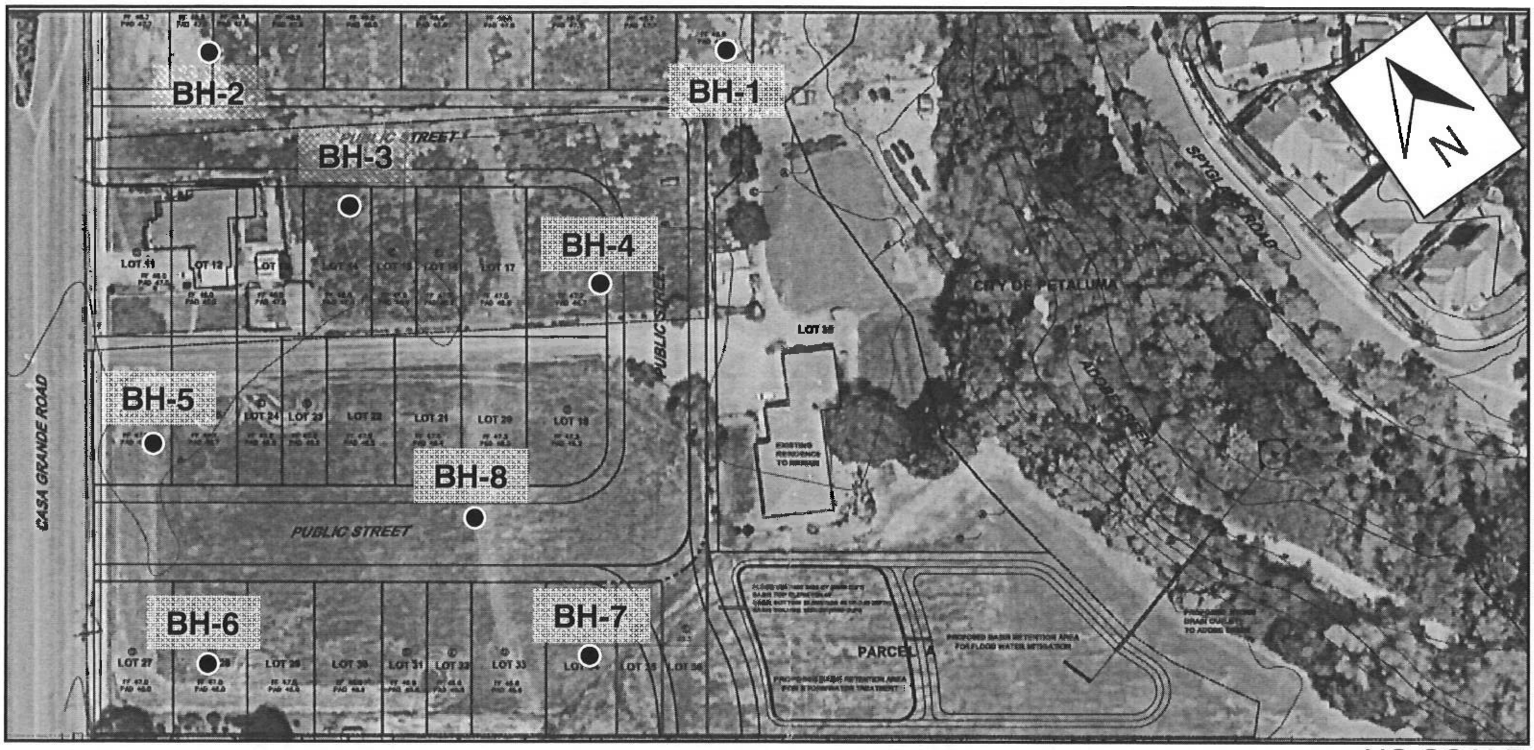
1. INTRODUCTION

The field program performed for this study consisted of drilling eight exploratory boreholes (BH-1 through BH-8) within the project area. The exploration was completed on June 29, 2020. The approximate 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 10.

2. BOREHOLES

The boreholes were advanced using truck mounted drill rigs with hollow and solid stem flight augers. The drilling subcontractor on the project was Pearson Drilling of Forestville, California. The drilling was performed under the observation of a project engineer of PJC who maintained a continuous log of the soil conditions and obtained samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System, as explained on Plate 11.

Relatively undisturbed and disturbed samples were obtained from the exploratory boreholes. A 2.43 in I.D. California Modified Sampler was driven into the underlying soil using a 140 pound hammer falling 30 inches to obtain an indication in the field of the density of the soil and to allow visual examination of at least a portion of the soil column. A standard penetration sampler was used in the granular soils. 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 the borehole logs. All samples collected were labeled and transported to PJC's office for examination and laboratory testing.



NO SCALE

EXPLANATION

● BOREHOLE LOCATION AND DESIGNATION

REFERENCE: PRELIMINARY SITE DEVELOPMENT PLAN, PREPARED BY STEPHEN J. LAFRANCHI & ASSOCIATES, INC., DATED OCTOBER 30, 2019.

PJC & Associates, Inc. Consulting Engineers & Geologists	BOREHOLE LOCATION PLAN PROPOSED SUBDIVISION 270 & 280 CASA GRANDE ROAD PETALUMA, CALIFORNIA	PLATE 2
	Proj. No: 9852.01 Date: 8/20 App'd by: PJC	

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-1

CLIENT Falcon Point Associates, LLC		PROJECT NAME Proposed Residential Subdivision	
JOB NUMBER 9852.01	LOCATION 270 & 280 Casa Grande Road		
DATE STARTED 6/29/20	COMPLETED 6/29/20	GROUND ELEVATION 48 ft	HOLE SIZE 6
DRILLING CONTRACTOR Pearson Drilling		GROUND WATER LEVELS:	
DRILLING METHOD B-53 Hollow Stem Auger with 140lb hammer		∇ AT TIME OF DRILLING 24.00 ft / Elev 24.00 ft	
LOGGED BY B.C.	CHECKED BY PJC	AT END OF DRILLING ---	
NOTES		▼ AFTER DRILLING 16.00 ft / Elev 32.00 ft	

DEPTH (ft)	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	(CH) 0.0'-5.0'; SANDY CLAY; brownish black, slightly moist to moist, hard, high plasticity, many gravels. (Qhf) Expansion Index = 146								
		MC	15	4.5					
		MC	17			16			
5	(CH) 5.0'-9.5'; SANDY CLAY; brownish gray, moist, hard, high plasticity. (Qhf)								
		MC	25	4.5	96	22			
10	(CL) 9.5'-13.0'; SANDY CLAY, yellowish gray with white and orange mottling, moist, hard, low plasticity, with gravels and cobbles. (Qhf)	MC	31	4.5	95	15			
	(CL) 13.0'18.5'; SANDY CLAY, brownish gray, moist to saturated, hard, low plasticity, with gravels. (Qhf)								
15		SPT	44			14			
	(CH) 18.5'-29.5'; SANDY CLAY; light gray, saturated, stiff to very stiff, high plasticity. (Qhf)								
20		SPT	10			32			
25									

(Continued Next Page)

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CLIENT Falcon Point Associates, LLC **PROJECT NAME** Proposed Residential Subdivision

JOB NUMBER 9852.01 **LOCATION** 270 & 280 Casa Grande Road

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	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	FINES CONTENT (%)
25		(CH) 18.5'-29.5'; SANDY CLAY; light gray, saturated, stiff to very stiff, high plasticity. (Qhf) *(continued)*	MC		24	2.5	87	32				
30		(SC) 29.5'-34.0; CLAYEY SAND; dark brownish gray, saturated, medium dense, fine to coarse grained, with gravels (Qhf)	MC		41		105	20				33
35		(CL) 34.0'-50.5'; SANDY CLAY, moderate gray, saturated, stiff to very stiff, medium plasticity, trace gravels. (Qhf)	SPT		15			21				
40			MC		31	2.5	94	26				
45			MC		22	2.5	93	28				
50			MC		14	1.5	95	27				

Bottom of borehole at 50.5 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-2

CLIENT Falcon Point Associates, LLC
PROJECT NAME Proposed Residential Subdivision
JOB NUMBER 9852.01
LOCATION 270 & 280 Casa Grande Road
DATE STARTED 6/29/20
COMPLETED 6/29/20
GROUND ELEVATION 48 ft
HOLE SIZE 6
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer
AT TIME OF DRILLING --- Not Encountered
LOGGED BY B.C.
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		(CH) 0.0'-5.5'; SANDY CLAY; brownish black, slightly moist to moist, hard, high plasticity, few gravels. (Qhf)								
			MC	17			19			
5			MC	30	4.5	101	21			
		(CL) 5.5'-10.0'; SANDY CLAY; brownish gray, moist, hard, medium plasticity, trace gravels. (Qhf)								
			MC	37	4.0	97	25			
10		(CH) 10.0'-13.5'; SANDY CLAY; orangish brown, moist, hard, high plasticity. (Qhf)								
			MC	32	4.0	105	20			

Bottom of borehole at 13.5 feet.

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL MIN RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
08414-1	CG1/P	Native Soil BH-1 – 8 @ 0'-5'	7.26	590	[1695]	210	345.0
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
08414-1	CG1/P	Native Soil BH-1 – 8 @ 0'-5'				+335.7	
Method	Detection	Limits --->	---	0.01	0.01	1	0.1

COMMENTS

Resistivity is under 1,000 ohm-cm, i.e., poor, but soil reaction (i.e., pH) is mildly alkaline; sulfate is very mildly elevated (i.e., @ >200 ppm), and chloride is also elevated (i.e., @ >100 ppm); this soil is only very mildly reduced (@ 300-400 mV); [see table below on the right for assigned point values and ranges]. CalTrans (CT) times to perforation of galvanized steel and full depth pitting times for unprotected steel (following Uhlig) for this soil are determined based on pertinent parameters [see table at left below]. Sulfate could have some very mild adverse impact on concrete, cement, mortar or grout; likewise, chloride could have some adverse impact on rebar or buried steel over the very long term. In principle, lime or cement treatment could be of benefit in that raising soil pH to the 7.5-8.5 range would increase perf and pitting times (as indicated in the table at left below). This improvement tends to be minimal for unprotected steel. Otherwise, to increase metals longevity any more in this soil would require steel upgrading or other actions. At times, structural strength considerations may require heavier gauge steel than is used in the presented examples such that perf and pitting to depth times can be beyond the specified life span. Where this is not the case, cathodic protection along with the coating or wrapping of steel assets can be one potential solution. Other options include increased and/or specialized engineering fill, use of a polymer coating, or use of plastic, fiberglass or concrete assets. Based on these results, some upgrading of concrete (e.g. to ASTM Type II) and steel (e.g. heavier gauge) would probably be desirable primarily due to the low resistivity which is indicative of a somewhat elevated total mineral salts content.

SAMPLE ID	CT 18 ga	CT 12 ga	2 mm (Uhlig)	PARAMETER/ID	B1/SR
CGR1/P	~ 21.5yrs	~47 yrs	~10 yrs	pH	Ø
treated	~30 yrs	~70 yrs	~12 yrs	Rs	8-10
				SO4	Ø-1
				Cl	Ø-3
				Redox	Ø-3.5
				TOTALS	8-17.5

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Consulting Engineers & Geologists

CORROSION TEST RESULTS
PROPOSED SUBDIVISION
270 & 280 CASA GRANDE ROAD
PETALUMA, CALIFORNIA

PLATE 4a

Proj. No: 9852.01 Date: 08/20 App'd by: PJC

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BORING NUMBER BH-3

CLIENT Falcon Point Associates, LLC

PROJECT NAME Proposed Residential Subdivision

JOB NUMBER 9852.01

LOCATION 270 & 280 Casa Grande Road

DATE STARTED 6/29/20 COMPLETED 6/29/20 GROUND ELEVATION 48 ft HOLE SIZE 6

DRILLING CONTRACTOR Pearson Drilling

GROUND WATER LEVELS:

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

AT TIME OF DRILLING --- Not Encountered

LOGGED BY B.C. CHECKED BY PJC

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	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	FINES CONTENT (%)
0.0		(CH) 0.0'-5.0'; SANDY CLAY; brownish black, slightly moist to moist, hard, high plasticity, few gravels. (Qhf) Bulk sample										
2.5			AU									
5.0												

Bottom of borehole at 5.0 feet.

ORIGINAL GEOTECH BH COLUMNS - GINT STD US.GDT - 9/21/20 13:54 - C:\USERS\PUBLIC\DOCUMENTS\BENTLEY\GINT\PROJECTS\9852.01 270 & 280 CASA GRANDE ROAD.GPJ

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-4

CLIENT Falcon Point Associates, LLC
PROJECT NAME Proposed Residential Subdivision
JOB NUMBER 9852.01
LOCATION 270 & 280 Casa Grande Road
DATE STARTED 6/29/20
COMPLETED 6/29/20
GROUND ELEVATION 48 ft
HOLE SIZE 6
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer
AT TIME OF DRILLING --- Not Encountered
LOGGED BY B.C.
CHECKED BY PJC
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	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	FINES CONTENT (%)
0		(CH) 0.0'-5.0; SANDY CLAY; brownish black, slightly moist to moist, hard, high plasticity, trace gravels. (Qhf)										
			MC		10			14				
			MC		18	4.5	97	19				
5		(CH) 5.0'-15.0'; SANDY CLAY; dark gray to yellowish gray, moist, very stiff to hard, high plastiicty, trace gravels. (Qhf)										
			MC		22	2.5	92	27				
10			MC		37	4.5	96	25				
15			MC		67	4.5	99	23				

Bottom of borehole at 15.0 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-5

CLIENT Falcon Point Associates, LLC

PROJECT NAME Proposed Residential Subdivision

JOB NUMBER 9852.01

LOCATION 270 & 280 Casa Grande Road

DATE STARTED 6/30/20

COMPLETED 6/30/20

GROUND ELEVATION 48 ft

HOLE SIZE 6

DRILLING CONTRACTOR Pearson Drilling

GROUND WATER LEVELS:

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

∇ AT TIME OF DRILLING 13.50 ft / Elev 34.50 ft

LOGGED BY L.C.

CHECKED BY PJC

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	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	FINES CONTENT (%)
0		(CH) 0.0'-6.5'; SANDY CLAY; brownish black, slightly moist to moist, very stiff, high plasticity, trace gravels. (Qhf)										
			MC		16	4.5	104	12	57	17	40	
			MC		17	4.5	96	21				
5												
			MC		26	4.0	98	23				
		(CH) 6.5'-14.5'; SANDY CLAY; brownish gray, moist to saturated, very stiff to hard, high plasticity, trace gravels. (Qhf)										
10												
			MC		51	4.5	94	28				
	∇											
15		(CH) 14.5'-15.5'; SANDY CLAY; yellowish gray, saturated, hard, high plasticity, few gravels. (Qhf)	MC		70	4.4U	107	18				

Bottom of borehole at 15.5 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-6

CLIENT Falcon Point Associates, LLC
PROJECT NAME Proposed Residential Subdivision
JOB NUMBER 9852.01
LOCATION 270 & 280 Casa Grande Road
DATE STARTED 6/30/20
COMPLETED 6/30/20
GROUND ELEVATION 48 ft
HOLE SIZE 6
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer
AT TIME OF DRILLING --- Not Encountered
LOGGED BY L.C.
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		(CH) 0.0'-7.0'; SANDY CLAY; brownish black, slightly moist to moist, hard, high plasticity, trace gravels and organics. (Qhf)								
			MC	23	4.5	97	21			
5			MC	31	4.5	98	23			
		(CH) 7.0'-10.0'; SANDY CLAY, gray, moist, hard, high plasticity. (Qhf)								
			MC	38	4.0	95	25			
10		(CH) 10.0'-13.5'; SANDY CLAY; grayish brown, moist, hard, high plasticity. (Qhf)								
			MC	89	4.5	111	16			

Bottom of borehole at 13.5 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-7

CLIENT Falcon Point Associates, LLC
PROJECT NAME Proposed Residential Subdivision
JOB NUMBER 9852.01
LOCATION 270 & 280 Casa Grande Road
DATE STARTED 6/30/20
COMPLETED 6/30/20
GROUND ELEVATION 48 ft
HOLE SIZE 6
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD B-53 Solid Stem Auger with 140lb Hammer
AT TIME OF DRILLING 11.50 ft / Elev 36.50 ft
LOGGED BY L.C.
CHECKED BY PJC
AT END OF DRILLING 12.00 ft / Elev 36.00 ft
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		(CH) 0.0'-5.0'; SANDY CLAY; brownish black, slightly moist to moist, hard, high plasticity, trace gravels and organics. (Qhf)	MC	13			15			
			MC	19	4.0	94	24			
5		(CH) 5.0-8.5'; SANDY CLAY; dark gray, moist, very stiff, high plasticity, trace gravels and organics. (Qhf)	MC	21	3.0	92	27			
10		(CH) 8.5'-14.5'; SANDY CLAY; yellow brown, moist to saturated, very stiff, high plasticity, trace gravels. (Qhf)	MC	40	4.5	94	29			
			MC	34	2.2U	99	24			

Bottom of borehole at 14.5 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-8

CLIENT Falcon Point Associates, LLC

PROJECT NAME Proposed Residential Subdivision

JOB NUMBER 9852.01

LOCATION 270 & 280 Casa Grande Road

DATE STARTED 6/30/20

COMPLETED 6/30/20

GROUND ELEVATION 48 ft

HOLE SIZE 6

DRILLING CONTRACTOR Pearson Drilling

GROUND WATER LEVELS:

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

AT TIME OF DRILLING --- Not Encountered

LOGGED BY L.C.

CHECKED BY PJC

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	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	FINES CONTENT (%)
0		(CH) 0.0'-5.0'; SANDY CLAY; brownish black, slightly moist to moist, hard, high plasticity, few gravels. (Qhf) Bulk sample	AU									
5												

Bottom of borehole at 5.0 feet.

ORIGINAL GEOTECH BH COLUMNS - GINT STD US.GDT - 9/21/20 13:54 - C:\USERS\PUBLIC\DOCUMENTS\BENTLEY\GINT\PROJECTS\9852.01 270 & 280 CASA GRANDE ROAD.GPJ

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

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 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, pocket penetrometer, grain-size distribution and Atterberg Limits testing.

a. <u>Natural Water Content and Dry Density</u>. Natural water content and dry density of the soils were determined, often in conjunction with other tests, on selected undisturbed samples. The samples were extruded and visually classified, trimmed to obtain a smooth flat face, and accurately measured to obtain volume and wet weight. The samples were then dried in accordance with the procedures of ASTM 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. The water content and dry density results are summarized on the borehole logs.

b. <u>Pocket Penetrometer</u>. Pocket Penetrometer tests were performed on all 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 are summarized on the borehole logs.

c. Grain-Size Distribution. The gradation characteristics of a selected sample were determined in accordance with ASTM D422-63. The sample was soaked in water until individual soil particles were separated and then washed on the No. 200 mesh sieve. That portion of the material retained on the No. 200 mesh sieve was oven-dried and then mechanically sieved. The results are presented as Plate 12.

d. Atterberg Limits Determination. Liquid and plastic limits were determined on selected samples in accordance with ASTM D4318-83. The results of the limits are summarized on the borehole logs.

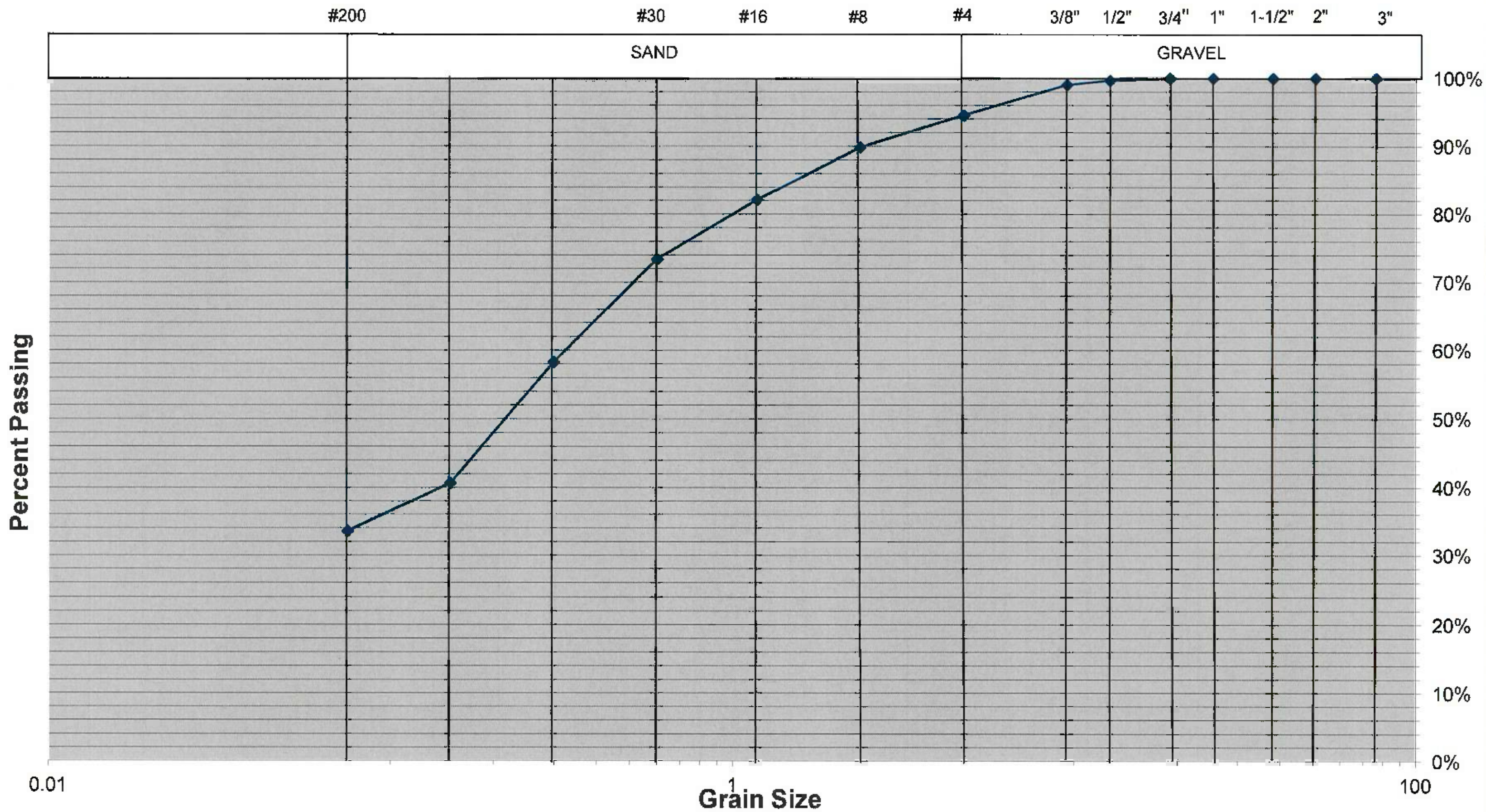
e. Expansion Index. In the Expansion Index test a sample is compacted into a metal ring and moisture conditioned to 40 to 60 percent of saturation and placed into a consolidometer. A vertical confining pressure is applied and the sample is inundated with distilled water. The deformation rate is recorded. The test was performed in accordance with ASTM D 4829.

3. ENGINEERING PROPERTIES TESTING

The engineering properties testing consisted of unconfined compression tests.

a. Unconfined Compression Test. Unconfined compression tests were performed on intact samples obtained in BH-5 and BH-7. In the unconfined compression test, the shear strength is determined by axially loading the sample under a slow constant strain rate until failure is obtained. Failure stress is defined as the maximum stress at ten percent strain. The results of the tests are presented on the borehole logs.

GRADATION CURVE



Sample: BH-1 at 30.0 feet; olive brown clayey sand (SC).

SIEVE ANALYSIS
PROPOSED SUBDIVISION
270 & 280 CASA GRANDE
PETALUMA, CALIFORNIA

PLATE 12

PJC

Proj. No: 9852.01

App'd by: PJC

Date: 8/2020

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. USGS Petaluma River, California Quadrangle 7.5-Minute Topographic Map, photo revised 1978.

4. Association of Bay Area Governments, Interactive Liquefaction Susceptibility Map, dated June 2009.

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

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

7. ASCE STANDARD ASCE/SEI 7-16, prepared by the American Society of Civil Engineers.

8. USGS National Seismic Hazard Maps, 2008.

9. Geologic Map of the Petaluma River 7.5 Minute Quadrangle, prepared by the California Geological Survey, dated 2002.

10. Preliminary Site Development Plan, prepared by Steven J. LaFranchi & Associates, Inc., dated October 30, 2019.