APPENDIX D:
GEOTECHNICAL INVESTIGATION

GEOTECHNICAL INVESTIGATION
PROPOSED HEMBREE LANE SUBDIVISION
7842 HEMBREE LANE
WINDSOR, CALIFORNIA

PREPARED FOR:

FALCON POINT ASSOCIATES LLC
DOYLE HEATON
3496 BUSKIRK AVENUE #104
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PREPARED BY:

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JOB NO. 10801.01

PJC & Associates, Inc.
Consulting Engineers & Geologists

May 23, 2022 Job No. 10801.01

Falcon Point Associates LLC
Attention: Doyle Heaton
3496 Buskirk Avenue, Suite 104
Pleasant Hill, CA 94523
doyle@drgbuilders.com

Subject: Geotechnical Investigation
Proposed Hembree Lane Subdivision
7842 Hembree Lane
Windsor, California
APN: 163-080-047

Dear Doyle:

PJC and Associates, Inc. (PJC) is pleased to submit the results of geotechnical investigation for the proposed Hembree Lane residential subdivision located at 7842 Hembree Lane in Windsor, California. The approximate location of the site is shown on the Site Location Map, Plate 1. The site corresponds to latitude and longitudinal coordinates of 38.5360°N and -122.7963°W, according to GPS measurements performed at the site. Our services were completed in accordance with our proposal for geotechnical engineering services, dated March 17, 2022 and your authorization to proceed dated March 18, 2022. 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 is feasible from a geotechnical engineering standpoint provided the recommendations presented herein 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

REGISTERED PROFESSIONAL ENGINEER
PATRICK J. CONWAY
No. 2303
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GEOTECHNICAL
STATE OF CALIFORNIA

PJC:dgn:sms

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GEOTECHNICAL INVESTIGATION
PROPOSED HEMBREE LANE SUBDIVISION
7842 HEMBREE LANE
WINDSOR, CALIFORNIA

1. PROJECT DESCRIPTION

Based on our review of preliminary civil engineering plans by Civil Design Consultants, latest revision dated February 2022 and preliminary architectural plans prepared by Edward C. Novak Architect, dated February 10, 2022, it is our understanding that the project will consist of improving the site and constructing 24 new single-family residences with attached garages on the property. We anticipate the proposed residences will consist of two-story, wood-frame structures with concrete slab-on-grade floors. The project will include asphaltic paved public streets, concrete driveways, and will be serviced by underground municipal utilities.

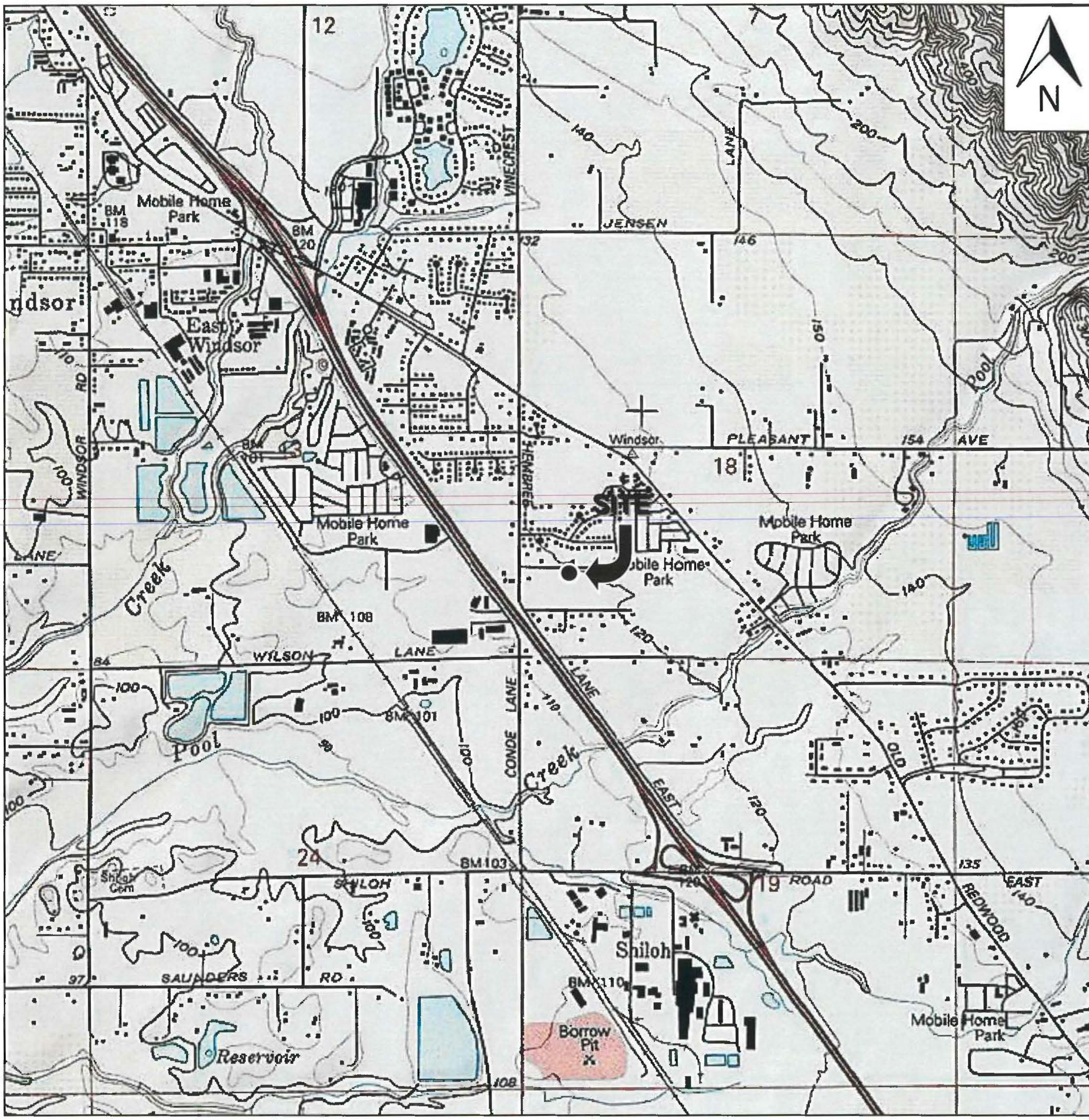
Structural loading information was not available at the time of this investigation. For our analysis, we anticipate that structural foundation loads will be light with dead plus live continuous wall loads less than two kips per lineal foot (plf) and dead plus live isolated column loads less than 50 kips. 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.

Based on the preliminary grading and drainage plans, we anticipate that site grading will consist of cuts and fills of three feet or less to upgrade the site soils, achieve finish pad and street grades and provide adequate gradients for site drainage. We do not anticipate that 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. Specifically, the scope of our services included the following:

a. Drilling ten exploratory boreholes to depths between 5.0 and 30.0 feet below the existing ground surface to observe the soil and groundwater conditions underlying the site. Our certified engineering geologist was on site to log the materials encountered in the boreholes and to obtain representative samples for visual classification and laboratory testing.



REFERENCE: USGS HEALDSBURG, CALIFORNIA
7.5 MINUTE QUADRANGLES, UPDATED 1993.

SCALE: 1:24,000

PJC & Associates, Inc.
Consulting Engineers & Geologists

SITE LOCATION MAP
PROPOSED HEMBREE LANE SUBDIVISION
7842 HEMBREE LANE
WINDSOR, CALIFORNIA

PLATE 1

Proj. No: 10801.01 | Date: 5/2022 | App'd by: PJC

b. Laboratory observation and testing of representative samples obtained during the course of our field investigation to evaluate the engineering properties of the subsurface soils at 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, ground rupture, settlement, lurching and lateral spreading, expansive soils, etc.).

d. Perform engineering analyses to develop geotechnical recommendations for site preparation and earthwork, foundation type(s) and design criteria, lateral earth pressures, settlement, concrete slab-on-grade recommendations, retaining wall design criteria, surface and surface and subsurface drainage control and construction considerations.

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

3. SITE CONDITIONS

a. General. The site is located east and adjacent to Hembree Lane between Cornell Street and Billington Lane, in a fully developed residential area of detached single family homes. The irregular shaped parcel comprises 51 acres of land and is undeveloped. The property is bounded by Cornell Street to the north, single family homes to the south, Hembree Lane to the west and single family homes and a green belt to the east. At the time of our investigation, the site was undeveloped and covered with perennial grasses and trees.

b. Topography and Drainage. The site is located on nearly level terrain in southeastern Windsor. According to the USGS Healdsburg, California, 7.5 Minute Quadrangle, the project site is situated near an elevation of 130 feet above mean sea level (MSL). Terrain at the property is overall level with subtle rolling high and low areas.

A creek runs through the eastern portion of the property. Site drainage consists of surface infiltration and sheet flow. Run-off from the property is provided by town maintained gutters and storm drains located on the nearby public streets.

4. GEOLOGIC SETTING

a. Regional Geology. 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.

b. Local Geology. According to a Geologic Map of the Healdsburg 7.5 Minute Quadrangle prepared by the California Geological Survey (CGS) the site has been mapped to be underlain by early to late Pleistocene alluvial type soil deposits (Qoa). Alluvial type soil deposits are generally characterized to consist of poorly sorted and unconsolidated soil strata of sand, gravel, silt and clay. Alluvial soil strata are typically discontinuous and heterogenous. Our subsurface exploration confirmed that the site is underlain by alluvial soil deposits. The alluvial soils likely extend to great depths below the site.

5. FAULTING

Geologic structures in the region are primarily controlled by northwest-trending dextral faults. The site is not located within the Alquist-Priolo Earthquake Fault Zone boundaries. According the USGS National Seismic Hazard Map (2008), the closest known active faults to the site are the

Rodgers Creek, the Maacama, and the Collayomi. The Rodgers Creek is located 1.7 miles to the northeast, the Maacama is located 6.0 miles to the west, and the Collayomi is located 17.8 miles to the northwest. 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	1.7	7.3
Maacama	6.0	7.4
Collayomi	17.8	6.7

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 ten exploratory boreholes (BH-1 through BH-10) to depths of 5.0 to 30.0 feet below the existing ground surface. The approximate borehole locations are shown on the Borehole Location Plan, Plate 2. The boreholes were used to collect soil samples of the underlying strata for visual examination and laboratory testing. The drilling and sampling procedures and descriptive borehole logs are included in Appendix A. The laboratory procedures are included in Appendix B.

The exploratory boreholes encountered a topsoil deposit and alluvial soil strata which extended to the maximum depths explored. The topsoil deposit extended approximately one and one-half to six feet below the existing ground surface. The topsoil consisted of a sandy silt soil stratum which appeared medium stiff to hard, slightly moist to very moist, and exhibited low plasticity characteristics. Underlying the topsoil, the exploratory boreholes encountered alluvial soil strata which extended to the maximum depths explored. The alluvial soils varied from granular soil strata to cohesive soil strata. The granular alluvium appeared slightly moist to saturated, medium dense to dense and fine to coarse grained. The cohesive alluvium appeared moist to saturated, very stiff to hard and exhibited low plasticity characteristics.

b. Groundwater. Groundwater was encountered at a depth of 13.0 feet below the ground surface in BH-1, 5.0 feet in BH-5 and 12.0 feet in BH-9 at the time of our subsurface exploration in March and April of 2022. Groundwater was not encountered in BH-2, BH-3, BH-4, BH-6, BH-7, BH-8 and BH-10. Due to the high groundwater conditions and seasonal standing water conditions at the site drainage mitigation measures should be implemented into the design and construction of the project. However, based on our experience on nearby projects we judge the shallow groundwater table was likely a perched condition which should dissipate following seasonal rainfall.

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

8. SEISMIC AND GEOLOGIC 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 project. The following discussion reflects the possible earthquake and geologic effects which could result in damage to the proposed project.

a. Fault Rupture. Rupture of the ground surface is expected to 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. However, it cannot be completely dismissed because the site is located in an active tectonic area.

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.

c. Liquefaction. Based on our review of the Association of Bay Area Governments (ABAG) liquefaction susceptibility map, the site is underlain by soils which are considered to have low liquefaction potential. In order to evaluate liquefaction potential at the site, our borehole designated BH-1 was drilled to a depth of 30 feet below the existing ground grade. The borehole encountered cohesive soil and relatively dense granular soil to the maximum depth explored. Therefore, the potential for liquefaction during a seismic event impacting the site is considered to be low.

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. A five foot tall creek runs through the project site that could be prone to lateral spreading or lurching. Structures should be placed an adequate distance from the creek bank.

e. Expansive Soils. Based on our findings and laboratory testing (PI= 2, 11, 11), the sandy silt topsoil and near surface alluvial soils exhibit low plasticity characteristics. Therefore, the site soils are considered to exhibit low expansion potential. The presence of expansive soils at the site is not a consideration.

f. Flooding. According to the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map panel 06097CO566E, effective December 2, 2008, the project site is located in Zone X. According to FEMA, a designation in Zone X indicates that the site is located within the 0.2% annual chance flood hazard with an average depth less than one foot or with drainage areas of less than one square mile. According to FEMA, this flood zone designation is considered a low risk area.

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 followed. The primary geotechnical considerations in design and construction of the project are the following:

a. Weak and compressible surface soils.

b. Potential seasonal high groundwater and standing water conditions.

The boreholes encountered weak and compressible surface soils extending to depths of up to three to four feet below the existing ground surface. Weak and compressible soils may appear hard and strong when dry. However, they could potentially collapse under the load of foundations, engineered fill, concrete slabs or pavements when their moisture content increases and approaches saturation. These soils can undergo considerable strength loss and increased compressibility thus causing irregular and erratic ground settlement under loads. This ground movement manifests in the form of cracked foundations and slabs and distress to architectural features of the structures.

To reduce the detrimental effects of these soils to within tolerable limits, we recommend the following options for pad preparation and foundation support:

a. The soils within the building pads could be upgraded by subexcavation of the top three feet and recompaction. The top 36 inches should consist of a low to non-expansive soil. We judge that the site soils meet the low to non-expansive criteria. By upgrading the site soils as previously described, the structures may be supported on shallow spread footing foundations and non-structural slabs-on-grade may be used.

b. As an alternative that would reduce the amount of earthwork and eliminate the need to subexcavate and recompact the surface weak soils, the structures could be supported on a post-tension slab foundations designed to resist differential movement.

The pavements and exterior slabs may be supported on 12 inches of low to non-expansive engineered fill. The lateral extent of the low to non-expansive engineered fill should be a minimum of three feet beyond the edges of exterior concrete slabs and pavements.

During our subsurface exploration on March 25, 2022 and April 7, 2022, groundwater was encountered in BH-1 at a depth of 25 feet below the existing ground surface and in BH-5 at a depth of five feet and BH-9 at a depth of 12 feet below the existing ground surface. The groundwater conditions will fluctuate by several feet throughout the year depending on seasons and environmental conditions. Based on our experience on nearby projects, we judge the shallow groundwater table was likely a perched condition which should dissipate following seasonal rainfall. The condition should not impact the project provided that grading is performed during the dry season.

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

10. SITE GRADING AND EARTHWORK

We anticipate that the project will include cuts and fills of three feet or less to upgrade the site soils, achieve finish pad grades and provide adequate gradients for site drainage.

a. Stripping. We recommend that structural areas be stripped of all disturbed soil, debris, surface vegetation, roots, and the upper few inches of soil containing organic matter. These materials should be moved off site; some of them, if suitable, could be stockpiled for later use in landscape areas. If underground utilities or any other obstructions pass through the site, we recommend that these utilities or obstructions 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 or septic systems not included in the project should be abandoned in accordance with the requirements of the Town of Windsor or County of Sonoma 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. Loosely backfilled voids will settle excessively and cause damage to structures constructed above them.

b. Subexcavation and Compaction. Following site stripping and demolition, excavation should proceed to achieve finish grades or prepare areas to receive fill. If shallow spread footings and non-structural slabs-on grade are desired for the project, we recommend that the weak and compressible soils be completely removed to full depth and width, and be replaced as compacted engineered fill. For budgetary purposes the depth of excavation is anticipated to be 36 inches below the existing ground surface.

However, the geotechnical engineer should determine the actual subexcavation depths in the field during construction. Total engineered fill thicknesses should not vary more than 2 feet across the individual pads.

the weak soils should be subexcavated and firm native soils exposed as determined by the geotechnical engineer on site during construction. The exposed surface should be scarified to a depth of eight inches; moisture conditioned to within two percent of the optimum moisture content, and compacted to a minimum of 90 percent of the maximum dry density of the materials, as determined by the ASTM D 1557-12 laboratory compaction test procedures. The sites soils should be moisture conditioned to two to four percent over the optimum moisture content, and compacted to a minimum of 90 percent of the maximum dry density of the materials, as determined by the ASTM D 1557-12 laboratory compaction test procedures. The excavated material free of organics and rocks four inches or less maybe re-used as engineered fill as approved by the geotechnical engineer in the field during grading. The fill should be spread in eight-inch thick loose lifts, moisture conditioned to within two percent of the optimum moisture content, and compacted to at least 90 percent of the maximum dry density of the materials. The engineered fill should extend at least five feet beyond perimeter foundations and at least three feet beyond exterior flatwork and pavements. Imported fill, if required, should be evaluated and approved by the geotechnical engineer before importation.

It is recommended that any import fill should be of a low to non-expansive nature and should 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 below.

TABLE 2
SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations*
Low to Non Expansive Engineered Fill Building Pad, Exterior Flatwork & Pavement (Native or 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 (Import)	Compact to at least 90 percent relative compaction at or within two percent of the optimum moisture content.
Driveways and Parking Areas	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.
Exterior Flatwork	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.

* 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. Temporary Slopes. We do not anticipate that a mass excavation will be required for the project. However, temporary slopes will be required for underground utility construction. Based on our findings we recommend that temporary slopes should not exceed one horizontal to one vertical (1H:1V). If steeper slopes are required, shoring should be used. The geotechnical engineer should observe the excavations 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.

Permanent 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. FOUNDATION OPTION: POST-TENSION SLAB-ON-GRADE

An option that would eliminate sub-excavation and re-compaction of the weak soils would be to support the structures on a post-tensioned mat slab designed to resist differential settlement. 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 settlement. The slabs should be designed according to the following criteria:

i. Allowable Bearing Capacity = 1,500 psf

ii. A maximum differential settlement of one and one-half inches.

b. Lateral Loads. Resistance to lateral forces may be computed by using base friction or adhesion. A friction factor of 0.30 is considered appropriate between the bottom of the concrete structures and the subgrade soils. A passive pressure of 250 psf/ft may be used for structural elements embedded below grade. The top six inches should be neglected for passive resistance.

We recommend a minimum slab thickness of 10 inches. The slab perimeter should be provided with a 12-inch wide and six-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 membranes 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 pads should be maintained within two percent of optimum moisture content at all times. The subgrade material should not be allowed to dry out prior to post-tensioned slab construction.

c. Settlement. As mentioned, the post tension slabs should be designed to resist a differential settlement of one and one-half inches. They should also be designed for to span at least eight at the corners and the center as determined by the project structural engineer.

12. FOUNDATION OPTION: SPREAD FOOTINGS

a. Vertical Loads. Provided that the weak soils are upgraded by subexcavation and re-compaction, we judge that spread footings may be used for foundation support. Continuous wall footings should be a minimum of 18 inches wide and 18 inches deep. Isolated column footings should be at least 24 inches square and 18 inches deep. Footing excavations should be observed and approved by the geotechnical engineer before reinforcing steel is placed. All footings should be reinforced. The recommended bearing pressures, depth of embedment and minimum widths of footings are presented in Table 3. The bearing values provided have been calculated assuming that all footings uniformly bear on a uniform layer of compacted engineered fill.

TABLE 3
FOUNDATION DESIGN CRITERIA

Footing Type	Bearing Pressure (psf)*	Minimum Embedment (in)**	Minimum Width (in)
Continuous wall	2,000	18	18
Isolated Column	2,500	24	18

* Dead plus live load.
**into engineered fill

The allowable bearing pressures are net values. The weight of the foundation and backfill over the foundation may be neglected when computing dead loads. Allowable bearing pressures may be increased by one-third for transient applications such as wind and seismic loads.

b. Lateral Loads. Resistance to lateral forces may be computed by using friction and passive pressure. A friction factor of 0.30 is considered appropriate between the bottom of the concrete structures and the bearing soils. A passive pressure of 300 pounds per square foot per foot of depth (psf/ft) is recommended. Unless restrained at the surface, the top six inches should be neglected for passive resistance.

Footing concrete should be placed neat against engineered fill. Footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavations, the soil should be thoroughly moistened prior to concrete placement.

c. Settlement. Total settlement of individual foundations will vary depending on the width of the foundation and the actual load supported. Foundation settlements have been estimated based on the foundation loads and bearing values provided. Maximum settlements of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to one inch or less. Differential settlement between similarly loaded, adjacent footings is expected to be one-half inch or less. The majority of the settlement is expected to occur during construction and placement of dead loads.

We should be retained to review the spread footing excavations, to review the actual soil conditions exposed, and provide modifications in the field, if necessary.

13. NON-STRUCTURAL CONCRETE SLABS-ON-GRADE

Non-structural concrete slabs-on-grade may be used for the project provided the slabs are underlain by a uniform layer of compacted engineered fill. The low to non-expansive fill should extend at least three feet beyond exterior slab and pavement edges and at least five feet beyond perimeter foundation edges.

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 exterior slabs should be cast and maintained separate of 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. 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 from gutters should be discharged onto an impermeable surface such as pavement or into a closed conduit discharging a minimum of eight feet away from the structures. Storm water must not be discharged on or near slopes. Discharge of storm water will cause erosion and stability problems of slopes.

15. ASPHALTIC CONCRETE PAVEMENTS

Based on our investigation, the existing surface soils will have a low supporting capacity (even after properly compacted) when used as a pavement subgrade. Based on our findings, ions could be revised by actual R-value testing an R-value of 5 was assigned to the site soils. We recommend the 12 inches below the pavement section should be subexcavated and replaced with compacted onsite soils extending three feet beyond the perimeter of the pavement. Pavement designs sections based on R-Value of 5 are presented in Table 3. The pavement sections could be revised by R-value testing of the exposed soils after they have been exposed at pavement subgrade.

Pavement thicknesses were computed from Chapter 630 of the Caltrans Highway Design Manual (2010), 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. Both the subgrade and base rock sections should pass firm and unyielding proof rolls with a fully loaded water truck.

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-1557-09. Aggregate used for the base coarse 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 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.

TABLE 4
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

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 will require shoring. Furthermore, where excavations extend deeper than five feet, groundwater could be encountered depending on when work is performed. Dewatering could be required. All trenches should conform to the current CAL-OSHA requirements for worker safety.

The trenches should 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 on top of them. The settlement and damage may not occur until years after completion of the structures.

17. 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 Site Class and Site Coefficients should be used:

a. Site Class: D

b. Mapped Acceleration Parameters: $S_S = 2.109g$, $S_1 = 0.811g$

c. Acceleration Parameters: $S_{MS} = 2.109g$, $S_{M1} = null$

d. Design Spectral Acceleration Parameters: $S_{DS} = 1.406g$, $S_{D1} = null$

18. RETAINING WALLS

a. Static Lateral Earth Pressures. Retaining walls should be supported on footings as described in section 12 of this report. Walls free to rotate on the top should be designed to resist active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation or supporting compacted engineered fill, they should be designed for "at rest" lateral earth pressures.

Retaining walls should be designed to resist the following earth equivalent fluid pressures (triangular distribution):

Active Pressure (level backfill) (5H:1V or less)............40 psf/ft
At Rest Pressure (level backfill) (5H:1V or less)............55 psf/ft
Active Pressure (2H:1V maximum slope backfill)..........55 psf/ft
At Rest Pressure (2H:1V maximum slope backfill)........70 psf/ft

b. Lateral Earth Pressures from Surcharge Loads. Retaining walls subjected to vehicle loads, if applicable, should be designed to resist additional induced lateral earth pressures due to traffic surcharge loads. We recommend surcharge loads of 120 psf (rectangular distribution).

The use of heavy, multi-ton compaction equipment such as large sheepsfoot rollers should not be allowed within a distance equal to the total wall height from the back face of retaining walls or the walls should be designed for additional induced lateral earth pressures. Excessive pressures will cause the walls to deflect excessively.

c. Pseudostatic Force. For walls taller than six feet, the horizontal pseudostatic force acting upon the retaining wall during a seismic event should be calculated from the following equation:

$P_E = 15.9 H^2$

where, P_E = Pseudostatic Force (lbs)

H = retained height (ft)

The location of the pseudostatic force is assumed to act at a distance of 0.33H above the base of the wall.

Static and pseudostatic force listed above do not include surcharge loads resulting from adjacent foundations, traffic loads or other loads. If additional surcharge loading is anticipated, we should be consulted to assist in evaluating their effects.

d. Drainage. We recommend that a backdrain be provided behind all retaining walls or that the walls be designed for full hydrostatic pressures. The backdrains should consist of four-inch diameter SDR 35 perforated pipe sloped to drain to outlets by gravity, and of clean, free-draining, Class II permeable material. The Class II permeable material should extend 12 inches horizontally from the back face of the wall and extend from the bottom of the wall to one foot below the finished ground surface. The upper 12 inches should be backfilled with compacted fine-grained soil to exclude surface water. We recommend that the ground surface behind retaining walls be sloped to drain. Under no circumstances should surface water be diverted into retaining wall backdrains. Where migration of moisture through walls would be detrimental, the walls should be waterproofed.

19. 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 Hembree Lane Subdivision located at 7842 Hembree Lane in Windsor, 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.

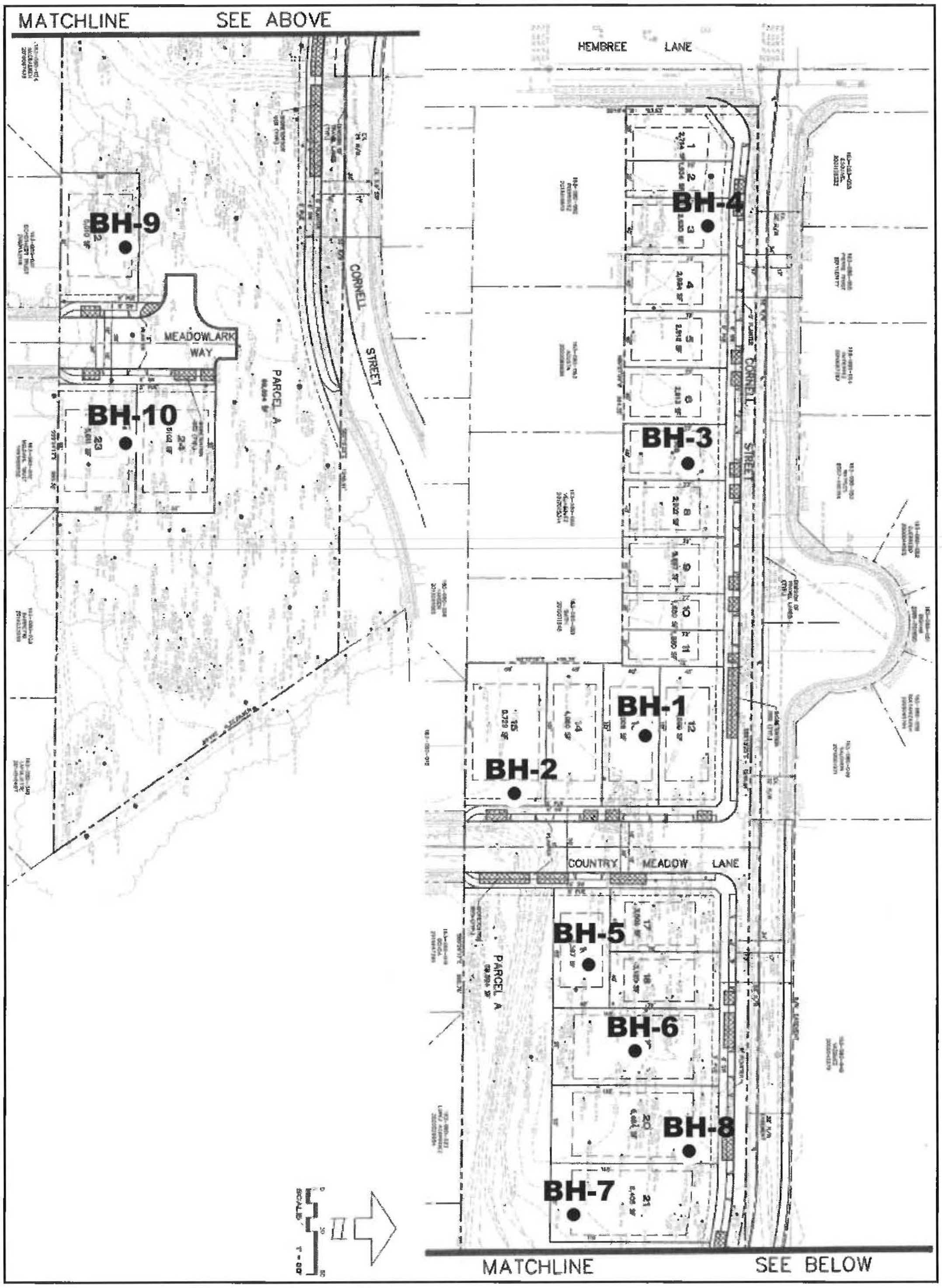
Soil deposits may vary in type, strength, and many other important properties between points of observation and exploration. Additionally, changes can occur in groundwater and soil moisture conditions due to seasonal variations or for other reasons. Therefore, it must be recognized that we do not and 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.

20. 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 observing grading and earthwork, approving foundation excavations, approving slab subgrade, approving subgrade/baserock preparation, and approving the installation of drainage parameters.

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.



EXPLANATION

APPROXIMATE SCALE: 1"=120'

● BOREHOLE LOCATION AND DESIGNATION

REFERENCE: PRELIMINARY SITE PLAN TITLED "HEMBREE LANE", DRAWN BY CIVIL DESIGN CONSULTANTS, INC. DATED FEBRUARY 2022.

PJC & Associates, Inc. Consulting Engineers & Geologists	BOREHOLE LOCATION PLAN PROPOSED HEMBREE LANE SUBDIVISION 7842 HEMBREE LANE WINDSOR, CALIFORNIA	PLATE 2
	Proj. No: 10801.01 Date: 5/2022 App'd by: PJC	

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-1

CLIENT Falcon Point Associates LLC
PROJECT NAME Proposed Hembree Lane Subdivision
JOB NUMBER 10801.01
LOCATION 7842 Hembree Lane, Windsor, California
DATE STARTED 3/25/22 **COMPLETED** 3/25/22
GROUND ELEVATION ______ **HOLE SIZE** 4"
DRILLING CONTRACTOR Pearson Drilling
GROUND WATER LEVELS:
DRILLING METHOD B-53 Hollow Stem Auger with 140lb hammer
▽ **AT TIME OF DRILLING** 19.00 ft
LOGGED BY SS **CHECKED BY** PJC
▼ **AT END OF DRILLING** 25.00 ft
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		0.0' - 2.25'; SANDY SILT (ML); mottled orange and light gray, moist, very stiff, low plasticity, few organics, trace pebbles (TOPSOIL).										
			MC		13	3.5	96	15	20	18	2	
		2.25' - 6.0'; CLAYEY SAND (SC); light yellowish brown, slightly moist to moist, dense, fine to coarse grained sand, few fine grained subrounded gravels (ALLUVIUM).										
			MC		43		108	14				
5			SPT		31			9				
		6.0' - 30.0'; SANDY CLAY (CL); grayish brown, moist to saturated, very stiff to hard, low plasticity, with thin clayey sand seams (ALLUVIUM).	SPT		15			11				
10			MC		42	4.5+	102	18				
15			MC		27	4.5+	97	22				
	▽											
20												

(Continued Next Page)

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-1

CLIENT Falcon Point Associates LLC **PROJECT NAME** Proposed Hembree Lane Subdivision

JOB NUMBER 10801.01 **LOCATION** 7842 Hembree Lane, Windsor, California

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 (%)
20		6.0' - 30.0'; SANDY CLAY (CL); grayish brown, moist to saturated, very stiff to hard, low plasticity, with thin clayey sand seams (ALLUVIUM). *(continued)*	MC		32	4.5+	102	22				
25		▼ (groundwater)										
			MC		24	4.0	85	31				
30			MC		25		91	28				

Bottom of borehole at 30.0 feet.

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PJC & Associates, Inc.

Consulting Engineers & Geologists

CLIENT Falcon Point Associates LLC **PROJECT NAME** Proposed Hembree Lane Subdivision
JOB NUMBER 10801.01 **LOCATION** 7842 Hembree Lane, Windsor, California
DATE STARTED 3/25/22 **COMPLETED** 3/25/22 **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 SS **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		0.0' - 2.0'; SANDY SILT (ML); grayish brown, moist, very stiff, low plasticity, porous with organics (TOPSOIL).	MC		10	3.0	99	15				
		2.0' - 4.5'; SANDY CLAY (CL); grayish brown, moist, stiff, low plasticity (ALLUVIUM).	SPT		12			19	31	20	11	
5		4.5' - 8.0'; CLAYEY SAND (SC); light gray, moist, medium dense, fine to coarse grained sand, high plasticity clay fines, partially cemented (ALLUVIUM).	MC		23		107	16				
10		8.0' - 10.0'; SANDY CLAY (CL); light grayish brown, moist, hard, low plasticity (ALLUVIUM).	MC		44	4.5+	106	16				

Bottom of borehole at 10.0 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-3

CLIENT Falcon Point Associates LLC **PROJECT NAME** Proposed Hembree Lane Subdivision

JOB NUMBER 10801.01 **LOCATION** 7842 Hembree Lane, Windsor, California

DATE STARTED 3/25/22 **COMPLETED** 3/25/22 **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 SS **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		0.0' - 2.25'; SANDY SILT (ML); light grayish brown with orange mottling, moist, very stiff, low plasticity, few pores (TOPSOIL).										
			MC		19	3.0	100	14				
		2.25' - 8.0'; CLAYEY SAND (SC); light yellowish brown, moist, medium dense to dense, fine to coarse grained sand, high plasticity fines, few gravels (ALLUVIUM).										
			SPT		20			16				26
5												
			MC		48		106	12				
		8.0' - 11.5'; SANDY CLAY (CL); light grayish brown, moist, hard, low plasticity (ALLUVIUM).										
10												
			MC		63	4.5+	109	15				

Bottom of borehole at 11.5 feet.

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PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-4

CLIENT Falcon Point Associates LLC **PROJECT NAME** Proposed Hembree Lane Subdivision

JOB NUMBER 10801.01 **LOCATION** 7842 Hembree Lane, Windsor, California

DATE STARTED 3/25/22 **COMPLETED** 3/25/22 **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 SS **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	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		0.0' - 1.5'; SANDY SILT (ML); light grayish brown, slightly moist, medium stiff, low plasticity, trace pores, thin clay seam at 1.25' (TOPSOIL).	MC		65	0.5	88	6				
		1.5' - 2.5'; GRAVELLY SAND (SW); grayish brown, slightly moist, very dense, fine to coarse grained sand, fine grained subrounded gravel, moderately cemented (ALLUVIUM).										
		2.5' - 10.0'; SANDY CLAY (CL); light brown, moist to very moist, hard, low plasticity, moderately cemented (ALLUVIUM).	SPT		47			19	36	25	11	
5			MC		78	4.5+	111	13				
10			SPT		34			24				

Bottom of borehole at 10.0 feet.

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PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-5

CLIENT Falcon Point Associates LLC
PROJECT NAME Proposed Hembree Lane Subdivision
JOB NUMBER 10801.01
LOCATION 7842 Hembree Lane, Windsor, California
DATE STARTED 4/7/22 **COMPLETED** 4/7/22
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** 5.00 ft
LOGGED BY DGN **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		0.0' - 3.25'; SANDY SILT (ML); light gray, moist, hard, low plasticity, few organics, trace pebbles (TOPSOIL).										
			MC		16	4.5+	104	12				
		3.35' - 15.5'; CLAYEY SAND WITH GRAVEL (SC); moderate brown with orange mottling, moist to saturated, medium dense, fine to coarse grained sand, fine to coarse grained subrounded gravel (ALLUVIUM).	MC		15		96	21				
5	▽											
			MC		15		100	20				
10			MC		22		102	21				
15			SPT		17			23				16

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 Hembree Lane Subdivision
JOB NUMBER 10801.01
LOCATION 7842 Hembree Lane, Windsor, California
DATE STARTED 4/7/22
COMPLETED 4/7/22
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 DGN
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		0.0' - 4.0'; SANDY SILT (ML); light gray, moist, very stiff, low plasticity, trace fine grained subrounded gravel (TOPSOIL).										
			MC		8	3.0	93	18				
			MC		26	3.0	108	17				
5		4.0' - 7.0'; CLAYEY SAND WITH GRAVEL (SC); light brown, moist, medium dense, fine to coarse grained sand, fine to coarse grained subrounded gravel (ALLUVIUM).										
			MC		27		109	15				
		7.0' - 10.0'; GRAVELLY SAND WITH CLAY (SW); light brown, moist, medium dense, fine to coarse grained sand, fine to coarse grained subrounded gravel (ALLUVIUM).										
10			MC		26		107	16				

Bottom of borehole at 10.0 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-7

CLIENT Falcon Point Associates LLC
PROJECT NAME Proposed Hembree Lane Subdivision
JOB NUMBER 10801.01
LOCATION 7842 Hembree Lane, Windsor, California
DATE STARTED 4/7/22 **COMPLETED** 4/7/22
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 DGN **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		0.0' - 3.0'; SANDY SILT (ML); light gray, moist, hard, low plasticity, trace fine grained subrounded gravel (TOPSOIL).										
			MC		26	4.5+	102	19				
		3.0' - 6.0'; CLAYEY SAND (SC); light yellowish gray, moist, dense, few fine grained subrounded gravels, moderately cemented (ALLUVIUM).	MC		48		107	13				
5												
		6.0' - 11.5'; CLAYEY SAND WITH GRAVEL (SC); light gray, moist, medium dense to dense, fine to coarse grained sand, few fine grained subrounded gravels (ALLUVIUM).	MC		23		107	16				
10												
			MC		38		113	14				

Bottom of borehole at 11.5 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-8

CLIENT Falcon Point Associates LLC

PROJECT NAME Proposed Hembree Lane Subdivision

JOB NUMBER 10801.01

LOCATION 7842 Hembree Lane, Windsor, California

DATE STARTED 4/7/22

COMPLETED 4/7/22

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 DGN

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		0.0' - 3.5'; SANDY SILT (ML); light brown, moist to very moist, very stiff, low plasticity (TOPSOIL).										
			MC		15	3.0 3.25	93 94	18 24				
		3.5' - 5.0'; SANDY SILT (ML); light brown, moist, hard, low plasticity (ALLUVIUM).										
5			MC		47	4.5+	96	19				

Bottom of borehole at 5.0 feet.

PJC & Associates, Inc.

Consulting Engineers & Geologists

BORING NUMBER BH-9

CLIENT Falcon Point Associates LLC
PROJECT NAME Proposed Hembree Lane Subdivision
JOB NUMBER 10801.01
LOCATION 7842 Hembree Lane, Windsor, California
DATE STARTED 4/7/22 **COMPLETED** 4/7/22
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** 12.00 ft
LOGGED BY DGN **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		0.0' - 6.0'; SANDY SILT (ML); light gray, moist to very moist, hard, low plasticity (TOPSOIL).										
			MC		15	4.5+	95	23				
			MC		50	4.5+	88	14				
5												
		6.0' - 9.0'; CLAYEY SAND WITH GRAVEL (SC); light brown, moist, medium dense, fine to coarse grained sand, fine to coarse grained subangular gravel (ALLUVIUM).	SPT		25			13				
		9.0' - 14.0'; SANDY CLAY (CL); light gray, very moist to saturated, very stiff, low plasticity (ALLUVIUM).										
10			SPT		15			25				
	∇											
		14.0' - 17.0'; CLAYEY SAND (SC); moderate brown, saturated, medium dense to dense, fine to coarse grained sand (ALLUVIUM).										
15			MC		13		81	23				13
			SPT		35			11				

Bottom of borehole at 17.0 feet.

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Consulting Engineers & Geologists

BORING NUMBER BH-10

CLIENT Falcon Point Associates LLC
PROJECT NAME Proposed Hembree Lane Subdivision
JOB NUMBER 10801.01
LOCATION 7842 Hembree Lane, Windsor, California
DATE STARTED 4/7/22
COMPLETED 4/7/22
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 DGN
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		0.0' - 2.0'; SANDY SILT (ML); moderate brown, very moist, medium stiff, low plasticity (TOPSOIL).										
		2.0' - 4.0'; SANDY SILT (ML); light brown, moist, hard, low plasticity (ALLUVIUM).	MC		60	4.5+	100	19				
5		4.0' - 5.5'; CLAYEY SAND WITH GRAVEL (SC); light brown, moist, dense, fine to coarse grained sand, fine to coarse grained subangualr gravel (ALLUVIUM).	MC		45		101	18				

Bottom of borehole at 5.5 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
[filled box] "Undisturbed" Sample
[boxed X] Bulk or Disturbed Sample
[empty box] 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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USCS SOIL CLASSIFICATION KEY
PROPOSED HEMBREE LANE SUBDIVISION
7842 HEMBREE LANE
WINDSOR, CALIFORNIA

PLATE 13

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APPENDIX A
FIELD INVESTIGATION

1. INTRODUCTION

The field program performed for this study consisted of drilling eight exploratory boreholes (BH-1 through BH-10) at the project site. The exploration was completed on April 7, 2022. 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 12.

2. BOREHOLES

The boreholes were advanced using a portable powered drill rig with solid stem flight augers. The drilling was performed under the observation of a certified engineering geologist and staff geologist of PJC who maintained a continuous log of the subsurface conditions and obtained samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System, as explained in 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 stratums using a 70 lbs pound hammer falling 30 inches to obtain an indication in the field of the density of the materials and to allow visual examination of at least a portion of the soil column. 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 log. All samples collected were labeled and transported to PJC's office for examination and laboratory testing.

APPENDIX B
LABORATORY INVESTIGATION

1. INTRODUCTION

This appendix includes a discussion of test procedures and results of the laboratory investigation performed for the proposed project. The investigation program was carried out by employing, whenever practical, currently accepted test procedures of the American Society of Testing and Materials (ASTM). Disturbed and undisturbed samples used in the laboratory investigation were obtained during the course of the field investigation as described in Appendix A of this report. Identification of each sample is by borehole number and depth.

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 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, sieve analysis, and Atterberg Limits testing.

a. Natural Water Content and Dry Density. The natural water content and dry density of selected samples was determined. The samples were extruded, visually classified and accurately measured to obtain 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° C. After drying, the weight of the sample was determined and the moisture content and dry density calculated.

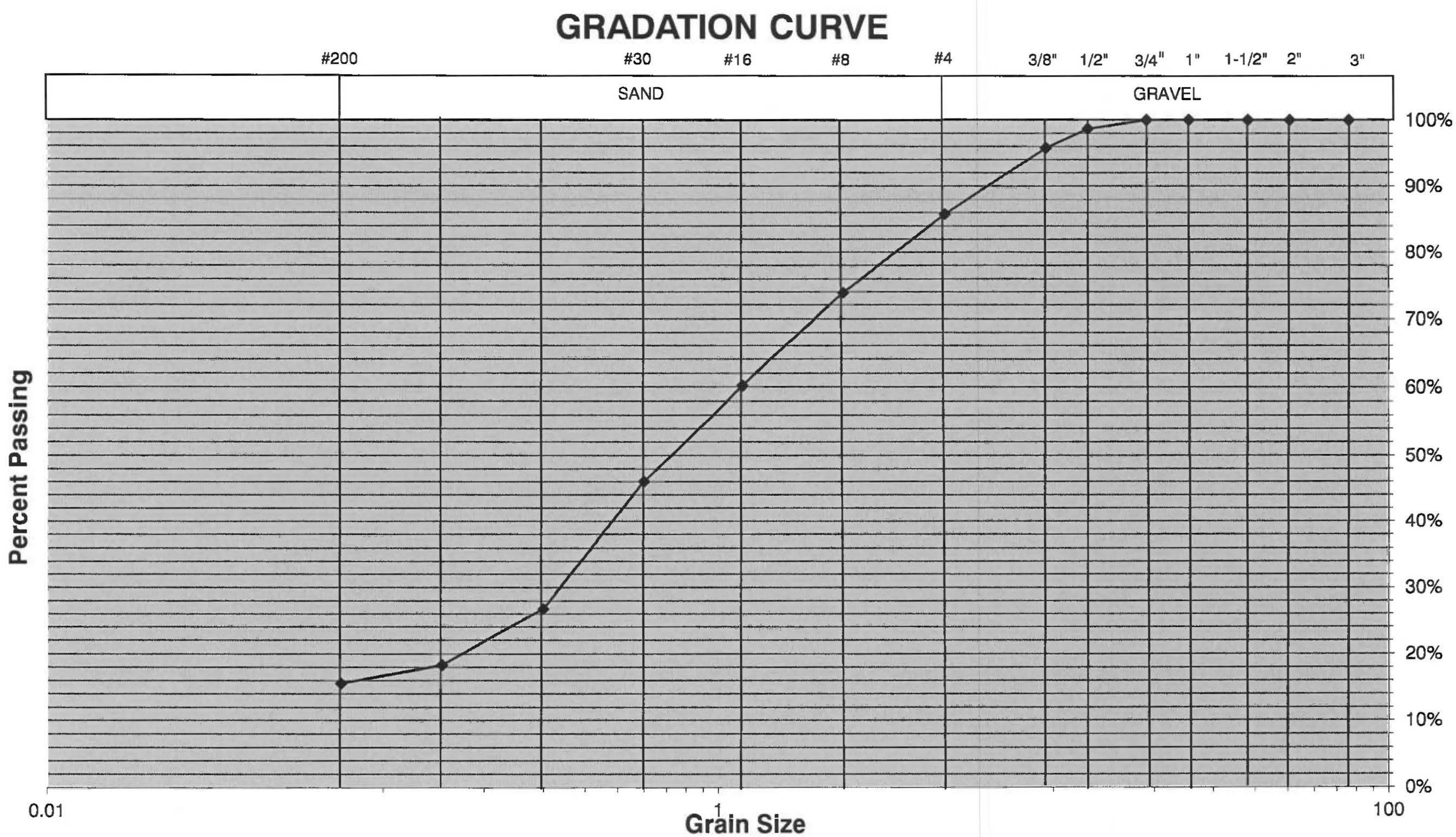
b. Sieve Analysis. The gradation characteristics of selected samples were determined in accordance with ASTM D422. The samples were soaked in water until individual soil particles were separated and then washed on the No. 200 mesh sieve. That portion of the material retained on the No. 200 mesh sieve was oven-dried and then mechanically sieved. The grain-size distribution tests are presented on the borehole logs and Plates 14 and 15.

c. Atterberg Limits Determination. Liquid and plastic limits were determined on selected samples in accordance with ASTM D 4318-83. 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. Pocket Penetrometer. 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 of these test are indicated on the borehole logs.



Sample: BH-5 at 14.5 to 15.5 feet; Moderate brown with orange mottling Clayey Sand With Gravel (SC).

SIEVE ANALYSIS
PROPOSED HEMBREE LANE SUBDIVISION
7842 HEMBREE LANE
WINDSOR, CALIFORNIA

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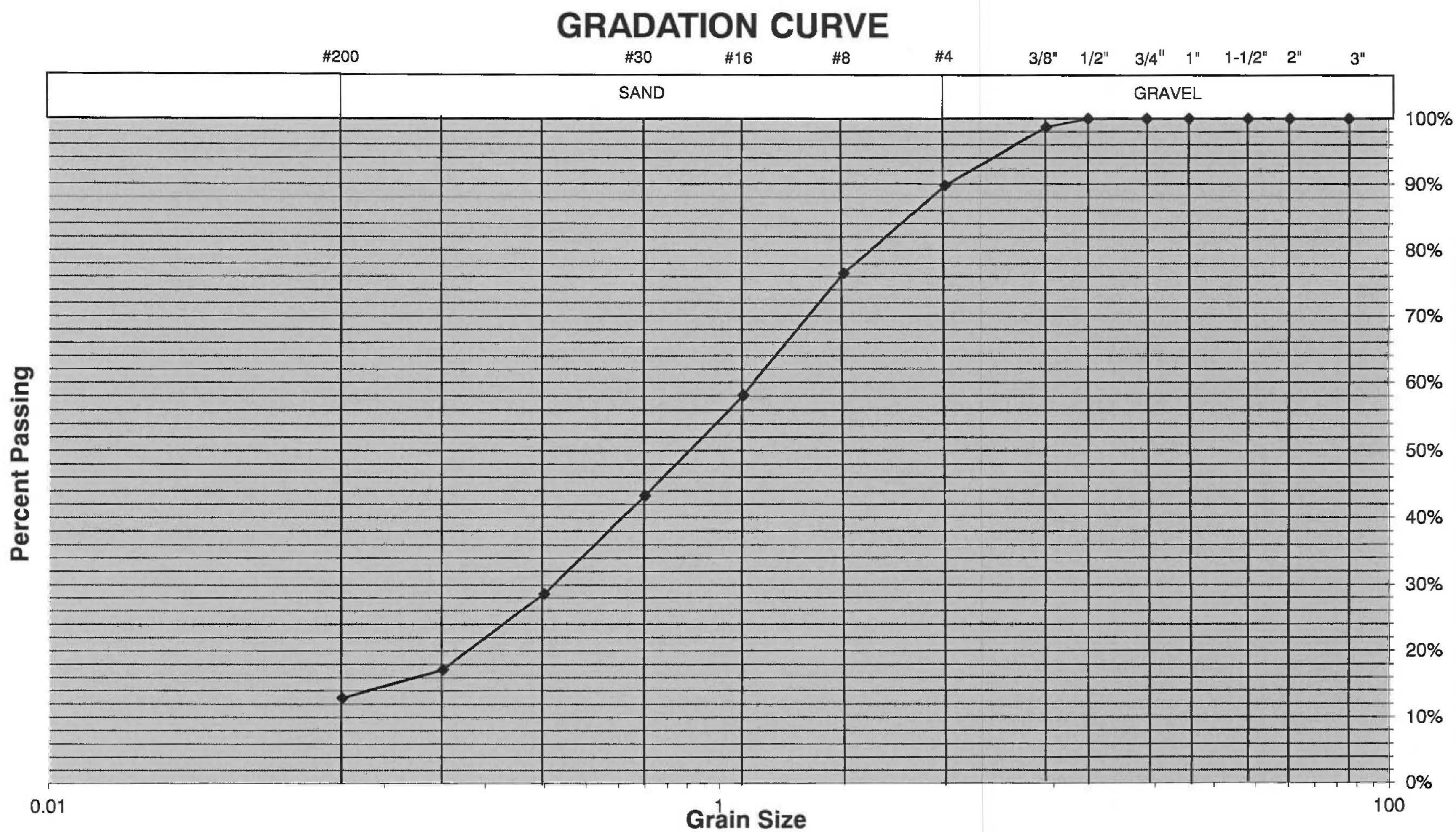

Sample: BH-9 at 15.0 feet; Moderate brown Clayey Sand (SC).

SIEVE ANALYSIS
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PLATE 15

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APPENDIX C
REFERENCES

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