Appendix D:
Geotechnical Report

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GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION
THE OAKS AT FOSS CREEK
111, 155, 157 CHIQUITA ROAD
HEALDSBURG, CALIFORNIA

JOB NO. 6959.01

PREPARED FOR:

DRG BUILDERS, INC.
3496 BUSKIRK AVE, #144
PLEASANT HILL, CA 94523

ATTENTION: MR. DOYLE HEATON

PREPARED BY:

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APRIL 12, 2016

PJC & Associates, Inc.
Consulting Engineers & Geologists

April 13, 2016 Job No. 6959.01

DRG Builders, Inc.
Attention: Doyle Heaton
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Subject: Design Level Geotechnical Investigation
"The Oaks at Foss Creek" Subdivision
111, 155, 157 Chiquita Road
Healdsburg, California

Dear Doyle:

PJC & Associates, Inc. (PJC) is pleased to submit this report presenting the results of our geotechnical investigation for the proposed "Oaks at Foss Creek" Subdivision located at 111, 155, 157 Chiquita Road in Healdsburg, California. Our services were completed in accordance with our proposal for geotechnical engineering services, dated January 26, 2016. This report presents our 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

RD:nb

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GEOTECHNICAL INVESTIGATION
PROPOSED "OAKS AT FOSS CREEK" SUBDIVISION
111, 155, 157 CHIQUITA ROAD
HEALDSBURG, CALIFORNIA
APN'S 089-013-012 & 013 & 014

1. PROJECT DESCRIPTION

Based on the tentative map and preliminary improvement plans prepared by Munselle Civil Engineering, dated December 16, 2015 and our discussions with Munselle Civil Engineering, the project will consist of constructing thirty-five single-family and three multi-family residential structures, a storm water retention basin, common areas and constructing approximately 1,500 feet of new asphalt paved roadways to provide access to the residences. The project will also include construction of new exterior flatwork, trails and installation of utilities. We anticipate that the residential buildings will consist of one or two-story, wood frame structures with joist supported, raised wood floors and concrete slab on grade floors in the garages.

Structural loading information was not available at the time of this investigation. For our analysis, we assume that dead plus live continuous wall loads will be 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 plan, the structures will be constructed near existing grade. Therefore, site grading will consist of cuts and fills of about three feet or less to achieve the desired pad and roadway grades and provide adequate gradients for site drainage.

2. SCOPE OF SERVICES

The purpose of this study was to provide geotechnical criteria for the design and construction of the proposed project. Specifically, the scope of our services consisted of the following:

a. Performing 11 exploratory borings (BH-1 through BH-9, BH-1a and BH-2a) to depths of between 5.5 and 50.5 feet below the existing ground surface to observe the soil and groundwater conditions underlying the site. Our civil engineer and staff geologist were on-site to observe the drilling of the boreholes, log the materials encountered and to obtain representative samples for visual classification and laboratory testing.

b. Perform laboratory tests on selected samples to assist in the evaluation of

the index and engineering properties of the subsurface conditions 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, ground rupture, settlement, lurching and lateral spreading, slope stability, 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, site drainage, pavement design criteria, support of concrete slabs-on-grade and construction considerations.

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

3. SITE CONDITIONS

a. Surface. The site is located on the north side of Chiquita Road in a partially developed residential and industrial area of northern Healdsburg. The generally trapezoidal shaped property comprises 6.9 acres of land and is bounded by Chiquita Road on the south, access roads and industrial structures of Simi Winery to the west and north, and Norton Slough to the east. At the time of our investigation, the site was generally covered with a low to moderate growth of perennial grasses, mature trees, and gravel parking areas and was occupied by several exiting single family homes and a storage yard. We observed an existing well in the vicinity of proposed lots 14 through 16.

b. Topography and Drainage. According to the USGS Geyserville, California 7.5 minute Quadrangle, the site lies in hills on the eastern margin of the Russian River Valley at an approximate elevation of 160 feet above mean sea level (MSL). The terrain is generally level.

Site drainage consists of sheet flow over the ground surface, and by surface infiltration. A seasonal drainage course bisects the site and joins Norton Slough on the eastern margin of the site. Regional drainage is provided by the Russian River, located approximately three miles south of the site.

4. 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 basins. 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 are now accreted to the continent (northern Coast Ranges). East-dipping thrust and reverse faults were believed to be the dominant controlling structures.

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

According to published geologic literature, the site has been mapped to be underlain by Quaternary alluvial deposits (Qa). These deposits have been described to consist of moderately to poorly sorted sand and gravel with silts and clays, typically found in small valleys, where separate alluvial fan, basin, terrace and active stream channel deposits exist in close proximity to one another. This was confirmed by our subsurface exploration. These deposits likely extend to a great depth below the site.

5. FAULTING

Geologic structures in the region are primarily controlled by northwest trending faults. No known active fault passes through the site. The site is not located in the State of California Earthquake Fault Studies Zone. Based on our research, the three closest known active faults to the site, with the potential to produce the largest accelerations, are the Maacama (South), the Rodgers Creek and the San Andreas faults. Table 1 outlines the nearest known active faults, their associated maximum credible magnitudes and the estimated maximum site accelerations due to potential earthquakes on those faults.

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance from Site (Miles)	Maximum Credible Earthquakes (Moment Magnitude)	Estimated Maximum Site Accelerations (g)
Maacama (South)	4.7	6.9	0.388
Rodgers Creek	8.8	7.0	0.281
San Andreas	20.1	7.9	0.249

Reference: "EQFAULT" Ver. 3.00, software program.

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 eleven exploratory boreholes (BH-1a, BH-2a, and BH-1 through BH-9) in the vicinity of the proposed structures to depths of between 5.5 and 50.5 feet below the existing ground surface. The boreholes were drilled to observe the soil and groundwater conditions, perform standard penetration testing (SPT) and to collect samples of the underlying soil deposits for visual examination and laboratory testing. The drilling and sampling procedures, descriptive borehole logs and laboratory procedures are included in Appendices A and B, respectively.

The exploratory boreholes generally encountered heterogeneous alluvial soil deposits and artificial fill consisting of clayey sands, sandy clays and clayey gravels that extended to the maximum depths explored. Based on the exploratory boreholes, it appears that the site is blanketed by a layer of porous and organic rich clayey sand topsoil which extends to an approximate depth of three to five feet below the existing ground surface. Artificial fill of variable composition and density was encountered in BH-2a, BH-7, BH-8 and BH-9, which extended to approximate depths of 1.5 to 2.5 feet below the existing ground surface The clayey sands varied from dry to moist, loose to very dense in relative density, and were generally

fine grained with occasional fine to coarse grained gravels. The sandy clays present varied from slightly moist to very moist, soft to hard and low to high plasticity. The clayey gravel appeared moist to saturated, dense and fine to coarse grained.

b. Groundwater. The groundwater table was encountered at a depth of eight and one half feet below the existing ground surface in BH-2, at 19 feet in BH-1a, and was observed at 24 feet below ground surface in an existing well at the property. Although a detailed groundwater table study was not performed, we anticipate that the groundwater table will fluctuate by many feet throughout the year.

8. GEOLOGIC HAZARDS AND SEISMIC CONSIDERATIONS

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

a. Surface Fault Rupture. Rupture of the ground surface typically occurs along active fault traces. Evidence of existing faults or previous fault-related ground displacement is not indicated in the geologic literature reviewed or during our field exploration. Therefore, we judge that the risk of surface fault rupture at the site is low.

b. Ground Shaking. The site has been subjected in the past to ground shaking by earthquakes on all of 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 very strong ground shaking during the design life of the project.

c. Liquefaction. According to the USGS Liquefaction Susceptibility Map (2006) for Sonoma County the site is located within an area considered to have a moderate susceptibility to liquefaction during or immediately following a significant seismic event. Liquefaction is a seismic hazard that occurs in saturated, loosely packed, predominantly granular soils found below the phreatic groundwater. In general, these loose materials experience a rapid, temporary loss in shear strength due to an increase in pore water pressure in response to strong earthquake ground shaking. Upon dissipation of pore water pressures following shaking, there is reduction in the void ratio of the impacted soils causing differential and erratic ground settlement. Loosely-packed, fine-grained sandy soils below the water level are most susceptible to liquefaction. However, case studies

have shown that soft silts and loose gravels with limited drainage paths are also susceptible to liquefaction. Bedrock materials and plastic clayey soils with a liquid limit greater than 32 are generally not known to be prone to liquefaction.

The occurrence of this phenomenon is dependent on many complex factors including the intensity and duration of ground shaking, groundwater level 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, two of our boreholes (BH-1a and BH-2a) were drilled to depths of 50.5 feet below the existing ground surface. Our boreholes encountered both granular and cohesive soils below the groundwater level at the site. BH-1a encountered medium stiff to very stiff cohesive soil strata to a depth of 18 feet below the ground surface overlying dense granular soils present from 18 feet to 32 feet below the ground surface, and 39 to 49 feet below the ground surface. Underlying the dense granular soils in BH-1a, cohesive, stiff clayey soils were present from 32 feet to 39 feet and 49 to 50.5 feet below the ground surface. The free groundwater level in BH-1a, at the time of drilling on February 10, 2016, was found to be 19 feet below the ground surface. In BH-2a, the borehole encountered medium dense granular soil from 2.5 to 6.5 feet below the ground surface, underlain by very stiff to hard cohesive soil strata to a depth of 50.5 feet below ground surface. Free groundwater was not encountered in BH-2a at the time of drilling on February 24, 2016. As discussed previously, groundwater levels fluctuate seasonally. Our field exploration did not encounter loose, saturated, granular soil stratums. The site is generally underlain by cohesive soils, and granular soils which are sufficiently dense, contain significant quantities of fines, and are generally not considered susceptible to liquefaction or densification. Therefore, we judge that the risk of soil liquefaction or densification at the site is low.

d. Densification. Densification is a phenomenon where loose, non-saturated granular soils experience void reduction and subsequent settlement due to earthquake induced ground shaking. Non-cemented, cohesionless soils, such as loose sands or gravels above the groundwater level, are susceptible to this type of settlement. Loose and dry cohesionless soils were not encountered in our boreholes. Therefore, we judge the potential for seismically-induced 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 slope face. Lurching is an action which produces cracks or fissures parallel to an unsupported slope face, such as streams or banks, when the

earthquake motion is at right angles to them. Norton Slough crosses through the property and has exposed banks of approximately 10 feet in height. However, due to the presence of relatively strong cohesive soils and dense granular soils, we judge that the potential for lateral spreading and lurching at the site is low.

f. Artificial Fill and Disturbed Surface Soil. In general, the soil exposed at the ground surface is highly variable in composition. The surface soil consists of varying thicknesses of artificial fill and weak, disturbed natural soil. The ground surface appears moderately hard; however, our subsurface exploration indicates the natural surface soil across the project site is weak, porous, compressible and locally disturbed by previous earthwork activities. The depth of weak and compressible native soils and variable density artificial fill is approximately three feet.

g. Expansive Soils. Based on laboratory analysis, the near surface clayey sand and sandy clay soils vary from low to high plasticity (PI=11, 37, 39). However, based on our experience and expansion index testing of a composite sample from the upper three feet, in general, the surface and near surface soils have very low expansion potential (EI=6).

h. Corrosive Soils. Based on corrosion potential testing performed on a composite sample obtained from the upper four feet of material encountered in the boreholes, the site soils are mildly acidic (pH 6.0), and should not have a significant adverse impact on concrete, reinforcing steel, mortar, grout or cement. Standard Caltrans times to perforation for galvanized steel are as follows:

Sample	18 Gauge	12 Gauge
Bulk 1	21 years	46 years

The calculated average pitting rates are approximately 0.125 mm/year, thus time to pitting to a 2 mm depth is approximately 16 years. If desired, steel longevity could be improved by using an increased gauge or more resistant steels and/or by taking other actions, such as the use of cathodic protection, use of a polymer coating, use of bonded plastic coated steels, or use of plastic, fiberglass or concrete assets.

9. CONCLUSIONS AND RECOMMENDATIONS

Based upon results of our investigation, it is our professional opinion 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. The primary geotechnical consideration in design and construction of the project is the presence of weak and compressible surface and near surface soils.

The boreholes encountered weak and compressible soil and artificial fill of variable density and composition extending to depths of approximately three 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 engineered fill, foundations or concrete slabs-on-grade when their moisture content 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 soils is impeded by engineered fill, slabs or foundations. 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. Therefore, the weak and compressible native soils and artificial fill should be subexcavated and recompacted to provide adequate support for foundations, slabs-on-grade and new pavements. Provided the weak and compressible soils are properly subexcavated and recompacted, conventional spread footings and conventional concrete slab-on-grade interior floors may be used.

The near surface sandy clay soils encountered at depths of two to four feet below the existing ground surface are potentially expansive. Expansive soils experience volumetric changes due to changes in moisture content. The volumetric changes can cause irregular and differential ground movement that could result in differential movement and cracking of shallow foundations, concrete slabs-on-grade, and pavements and to the exterior and interior finishes of the structures. Expansive clay soils must not be allowed within the upper 36 inches of building pads and 18 inches of pavement subgrade.

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

10. SITE GRADING AND EARTHWORK

a. General. Based on the preliminary grading plan and site topography we anticipate grading for the project will consist of minor cuts and fills of approximately three feet and less to achieve the desired finish grades and provide adequate gradients for site drainage.

Depending on the final design, it will likely be necessary to improve the near surface site soils by overexcavation and recompaction. All weak and compressible native soils and any artificial fill underlying building pads, areas where existing grade will be maintained, or that are scheduled to receive fill should be subexcavated and recompacted.

b. Demolition and Stripping. Any existing structures should be demolished and removed from the site. The site should be stripped of surface vegetation, surface debris, old fills, and surface soils containing organic matter. We anticipate the average depth of stripping will be about 4 to 6 inches; these materials should be removed off site, or stockpiled for later use in landscape areas, if desired. If underground utilities pass through the site, we recommend that these utilities be removed in their entirety or rerouted where they exist outside an imaginary plane sloped two horizontal to one vertical (2H:1V) from the outside bottom edge of the nearest foundation element. Any existing wells, septic systems or leach fields should be abandoned and plugged according to regulations set forth by the Sonoma County Health Department. Voids left from the removal of utilities or other obstructions should be replaced with compacted engineered fill under the observation of the project geotechnical engineer.

c. Excavation/Removal of Weak Soils. Where planned cuts do not remove existing weak and compressible soils beneath building foundations, interior slabs and adjacent exterior concrete walkways or driveways within five feet of the building, and in roadway or structural areas where existing grade is to be maintained or fills are to be constructed, the weak and compressible native soils should be subexcavated to an approximate depth of 36 inches below planned structures and 18 inches below pavement subgrade, and replaced as properly compacted engineered fill. The actual depth of subexcavation and recompaction could vary depending on the conditions exposed and should be determined by the geotechnical engineer in the field during construction. The lateral extent of the subexcavation should extend at least five feet beyond perimeter foundation edges of structures, and at least three feet beyond pavement edges. If present, any artificial fill should also be removed until firm native soils are exposed. The exposed surface should be prepared and compacted according to the following sections of this report. If expansive clay soils are encountered within 36 inches of building pad grades and within 18 inches of pavement

subgrade, they must be removed and replaced with properly compacted low to non-expansive engineered fill.

The bottom of the subexcavations should be scarified to a minimum depth of eight inches, moisture conditioned to at or within two percent of the optimum moisture content, and compacted to at least 90 percent of the materials relative maximum dry density as determined by ASTM D-1557 test procedures. The subexcavated material should be approved by the geotechnical engineer prior to use as fill. All fill material should be placed and compacted in accordance with the recommendations presented in Table 2.

d. Fill Placement & Compaction. In general the on-site materials may be used as fill. Excavated material to be used for the construction of site fills should not contain organic material or man-made debris, have a low to non-expansive potential, should have no rock or similar irreducible material with a maximum dimension greater than eight inches, and should be approved by the geotechnical engineer before use. Rocks greater than eight inches can be placed in deeper sections of fill, if approved by the geotechnical engineer during grading.

All fill material should be placed and compacted in accordance with the recommendations presented in Table 2. All fills should be placed in lifts no greater than eight inches in loose thickness.

It is recommended that any import fill to be used on site 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

TABLE 2
SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations*
General Engineered Fill (Native)	In lifts, a maximum of eight inches in loose thickness, compact to at least 90 percent relative compaction at or within two percent of the optimum moisture content.
General Engineered Fill (Low to Non-Expansive Import)	In lifts, a maximum of eight inches in loose thickness, compact to at least 90 percent relative compaction at or within two percent of the optimum moisture content.
Trenches**	Compact to at least 90 percent relative compaction at or within two percent of the optimum moisture content.
Pavement Areas	Compact the top eight inches of the subgrade and the entire aggregate base section to at least 95 percent relative compaction at or within two percent of the optimum moisture content.

* All compaction requirements stated in this report refer to dry density and moisture content relationships obtained through the laboratory standard described by the most recent addition of ASTM D 1557.
** Depths below finished subgrade elevations.

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 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 the on-site soils. Special and relatively expensive construction procedures should be anticipated if grading must be completed during the winter and early spring.

11. FOUNDATIONS: CONVENTIONAL SPREAD FOOTINGS

a. Vertical Loads. Provided the building pads are prepared according to the recommendations of this report, the structures may be supported by conventional, minimum depth spread footings. All footings should be reinforced. The recommended soil bearing pressures, depth of embedment and minimum width of spread footings are presented in Table 3. The bearing values provided have been calculated assuming that all footings uniformly bear on at least 18 inches of low to non-expansive engineered fill.

TABLE 3
FOUNDATION DESIGN CRITERIA

Footing Type	Bearing Pressure (psf)*	Minimum Embedment (in)**	Minimum Width (in)
Continuous wall	2,000	12	12
Isolated Column	2,500	12	18

*Dead plus live load
** Below lowest adjacent grade

The allowable soil bearing pressures are net values. The weight of the foundation and backfill over the foundation may be neglected when computing dead loads. The allowable soil 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 using friction or passive pressure. A friction factor of 0.35 is considered appropriate between the bottom of concrete structures and the engineered fill. A passive pressure equivalent to that exerted by a fluid weighing 350 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 undisturbed soil. 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 to close all cracks 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. Maximum settlements of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be less than one inch. Differential settlements between similarly loaded, adjacent footings

are expected to be less than one-half of one inch. The majority of the settlement is expected to occur during construction and the placement of dead loads.

12. CONVENTIONAL CONCRETE SLABS-ON-GRADE

We anticipate that concrete slabs-on-grade may be used in living areas. We consider concrete slabs in living areas as critical slabs. Settlement and excessive cracking is not acceptable. Therefore, the interior slabs in living areas should be supported entirely on compacted engineered fill of uniform thickness or the slabs should be structurally designed to span areas of non-uniform support. If grading does not remove the weak and compressible soils, non-critical, conventional slabs, such as garage floors and exterior flatwork, may be placed on these soils in their existing condition, if some slab settlement and cracking are acceptable to the owner. If this is not acceptable, these slabs should also be treated as critical slabs.

All concrete slab-on-grade subgrade should be scarified to a depth of eight inches, moisture conditioned and rolled or compacted to produce a firm and unyielding surface. The slab subgrade should not be allowed to dry before placing concrete.

Slabs-on-grade should be at least five inches thick and underlain by a four-inch layer of compacted clean gravel or crushed rock. 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. We recommend that the gravel be placed as soon as possible after preparation of the subgrade to prevent drying of the subgrade soils. If the subgrade is allowed to dry out prior to slab-on-grade construction, the subgrade soil should be moisture conditioned by sprinkling before slab-on-grade construction.

The rock will serve as a capillary break; however, moisture may accumulate in the base course. Therefore, a plastic vapor barrier of at least ten-mil thickness should be provided over the rock where moisture protection is desired. To aid in curing the concrete and to protect the vapor barrier against puncture, the vapor barrier should be covered by a two-inch layer of moistened sand. To control cracking, the slabs should be reinforced as determined by the project structural engineer. Special care should be taken to insure that reinforcement is placed at the slab mid-height. Exterior slabs should be carefully separated from foundations with felt paper or other positive and low friction material.

We should be retained to review the slab-on-grade subgrade preparation, to review the actual soil conditions exposed, and provide modifications in the field, if necessary.

13. SEISMIC DESIGN

Based on criteria presented in the 2013 edition of the California Building Code (CBC) and ASCE (American Society of Civil Engineers) STANDARD ASCE/SEI 7-10, the following minimum criteria should be used in seismic design:

a.	Site Class:	D
b.	Mapped Acceleration Parameters:	$S_s = 1.51$ $S_1 = 0.60$
c.	Site Adjusted Spectral Response Acceleration Parameters:	$S_{MS} = 1.51$ $S_{M1} = 0.90$
d.	Design Spectral Acceleration Parameters:	$S_{DS} = 1.01$ $S_{D1} = 0.602$

14. 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 on-site or imported 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 or pavements.

15. ASPHALTIC CONCRETE PAVEMENTS

Resistance Values (R-Value) of 40 and 21 were determined in the laboratory for the surface and near surface soils at the site. For our analysis, a reduced R-value of 20 was used. Pavement thickness was computed from Chapter 633 of the 2012 edition of Caltrans Highway Design Manual and is based on R-value testing performed on representative samples of the onsite soils in the general vicinity of

the proposed paved areas and a pavement design 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 or the County of Sonoma. Pavement design sections are presented in Table 4.

Any artificial fill or weak soils encountered during grading should be completely removed and recompacted in planned paved areas under the direction of the geotechnical engineer in the field during construction. Prior to placement of the aggregate base material, the top eight inches of the pavement subgrade should be scarified, moisture conditioned to within two percent of the optimum moisture content, and compacted to at least 95 percent relative compaction. Aggregate base materials should be spread in thin layers, moisture conditioned, and compacted to at least 95 percent relative compaction to form a firm and unyielding base.

Potentially expansive soils were encountered during our subsurface exploration and could be present beneath planned paved areas. Expansive soils could experience volumetric changes with changes in the moisture content. The resulting volumetric changes could cause intolerable differential movement that could cause cracking to pavements. Cracking would be prevalent along the edges of the pavement where cyclic wetting and drying occur. Expansive materials must not be used as fill in paved areas. If they are encountered at subgrade, they should be replaced with a low to non-expansive material. If expansive materials are placed under pavements, then cracking of pavements will occur. The owner must understand the potential and accept the risk that he will incur additional costs to repair damaged pavements due to the expansive clays. This condition should be further evaluated when subgrade is exposed.

TABLE 4
PAVEMENT SECTION DESIGN RECOMMENDATIONS
(Subgrade R-Value=20)

Traffic Index	Asphaltic Concrete (in)	Class II Aggregate Base (in)
4.0	2.5	6.0
5.0	3.0	7.0
6.0	3.0	10.0
7.0	3.5	12.0

The material and methods used should conform to the requirements of the City of Healdsburg specifications or the current edition of the Caltrans Standard Specifications, except that compaction requirements for the soil subgrade and aggregate baserock should be based on ASTM D-1557. 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. The following are alternatives which can be taken to extend the life of the pavement.

a. Construct a moisture 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. The actual depth could be modified based on Expansion Index testing (EI) when the actual subgrade conditions are exposed.

b. Extend concrete curbs at least six inches below the bottom of the aggregate base layer.

c. Install pavement subdrains beneath the aggregate base layer. Subdrains should consist of four-inch diameter SDR 35 perforated pipe sloped to drain to outlets by gravity, encased in clean, free-draining, three-quarter to one and one-half inch crushed rock or gravel.

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.

16. DRAINAGE

All final grades should be provided with positive gradients away from foundations to provide rapid removal of surface water runoff to an adequate discharge point. No ponding of water should be allowed adjacent to the foundations.

The use of continuous roof gutters and downspouts is recommended to reduce the possibility of soil saturation adjacent to the structure. 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 structure onto an erosion resistant surface. Roof downspouts and surface drains must be maintained entirely separate from other drains. The outlets should discharge into closed conduits and away from the structure.

17. 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 "Oaks at Foss Creek" Subdivision located at 111, 155, 157 Chiquita Road in Healdsburg, 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.

17. 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 foundation excavations, surfaces prepared to receive fill, field density testing of engineered fill, and installation of the drainage facilities. 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.