PJC & Associates, Inc.
Consulting Engineers & Geologists

January 4, 2017 Job No. 7048.01

Tom Coyne
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Subject: Design Level Geotechnical Investigation
Proposed Foundation Upgrade
Jenner Inn - Tree House
9470 Riverside Drive
Jenner, California

Dear Tom:

PJC and Associates, Inc. (PJC) is pleased to submit the results of our design level geotechnical investigation for the proposed foundation upgrade project for the Jenner Inn's tree house structure located at 9470 Riverside Drive in Jenner, California. The approximate location of the site is shown on the Site Location Map, Plate 1. The site corresponds to latitude and longitunal coordinates of 38.4502°N and 123.1151°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, 2016, and your authorization to proceed dated March 21, 2016. This report presents our engineering opinions and recommendations regarding the geotechnical aspects of the design and construction of the proposed project. Based on the results of this study, it is our opinion that the project site can be developed from a geotechnical engineering standpoint provided the recommendations presented herein are incorporated in the design and carried out through construction.

1. PROJECT DESCRIPTION

Based on our review of the structural engineering plans prepared by Santos & Urrutia, dated September 26, 2016, and information provided by you, it is our understanding that the project will consist of constructing a new foundation for the southern half of the tree house structure at the Jenner Inn to allow for construction of a daylight basement. The structure is currently supported by shallow pier blocks. The northern portion of the structure, including the foundations, will remain "as-is". The upgraded building will consist of a split-level wood frame structure with raised wood-floors and a concrete slab-on-grade in the basement. Our scope of work on the project consisted of exploring the underlying soil and bedrock conditions underlying the site, laboratory testing, and performing

SCALE 1:24,000

REFERENCE: USGS DUNCAN MILLS, CALIFORNIA 7.5 MINUTE QUADRANGLE, PHOTOREVISED 1979.

PJC & Associates, Inc. Consulting Engineers & Geologists	SITE LOCATION MAP TREE HOUSE FOUNDATION UPGRADE JENNER INN - 9470 RIVERSIDE DRIVE JENNER, CALIFORNIA	PLATE 1
	Proj. No: 7048.01 Date: 01/17 App'd by: PJC	

geotechnical engineering analysis to develop on opinion on site preparation and new foundation design parameters.

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.

The project site is situated on moderately sloping terrain. We anticipate that site grading for the project will consist of cuts of up to six feet and less to access foundation areas and construct retaining walls for the basement. Aside from possible retaining wall backfills, we do not anticipate the placement of significant fill 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 three exploratory boreholes to depths between 3.1 and 14.0 feet below the existing ground surface to observe the soil, bedrock, and groundwater conditions at the site. Our staff geologist was on site to log the materials encountered in the boreholes and to obtain representative samples for visual classification and laboratory testing.

b. Laboratory observation and testing of representative samples obtained during the course of our field investigation to evaluate the engineering properties of the subsurface soils and bedrock 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, slope stability, etc.).

d. Perform engineering analyses to develop geotechnical recommendations for site preparation and earthwork, foundation type and design criteria, lateral earth pressures, settlement, slab-on-grade recommendations, retaining wall design criteria, 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 tree house structure is located less than a few hundred feet north of the main lodge building at the Jenner Inn property. The tree house building consists of a raised one-story wood-frame structure. The structure is bordered to the north by an access driveway.

b. Topography and Drainage. The property is situated on moderately sloping hillside near the base of a south facing hillside. According to the United States Geological Survey (USGS) Duncans Mills, California, 7.5 Minute Quadrangle Map (Topographic), the project site is situated near an elevation of 32 feet above mean sea level (MSL). Site drainage is provided by sheet flow and surface infiltration. Regional drainage is provided by the Russian River which is located approximately 350 feet south of the site. The mouth of the Russian River and the Pacific Ocean are located approximately 0.7 miles west of the site.

c. Geology. According to the California Division of Mines and Geology, Planning in Sonoma County, Special Report 120 (Special Report 120), the site has been mapped to be underlain by mélange bedrock of the Jurassic-Cretaceous Franciscan Complex (KJf_s). Bedrock units of the Franciscan Complex generally consist of sheared shale and sandstone with resistant blocks of chert, blueschist and greenstone, and generally less resistant serpentinite. As a result of intense tectonic processes, the bedrock units tend to be highly sheared, fractured and pervasively shattered. Consequently, the strength, degree of weathering and therefore the engineering properties of Franciscan Complex bedrock units can vary drastically over relatively short distances. In addition, according to a slope stability map presented in Special Report 120, the site is located near the lower edge of a massive landslide complex. A discussion of the mapped landslide complex is provided in Section 7 of this report.

d. 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 Alquist-Priolo Earthquake Fault Studies Zone. According to the computer fault modeling program *EQFAULT*, the three closest known active faults to the site are the San Andreas, the Point Reyes and the Rodgers Creek faults. The San Andreas fault is located 2.1 miles to the southwest, the Point Reyes fault is located 15.8 miles to the south, and the Rodgers Creek fault is located 19.7 miles east of the site. The estimated maximum site acceleration from an earthquake on the Maacama (South) fault is 0.671g.

4. 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.

5. SUBSURFACE CONDITIONS

a. Soils and Bedrock. The subsurface conditions at the project site were investigated by drilling three exploratory boreholes (BH-1 through BH-3) at the site to depths between 3.1 and 14.0 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 and bedrock 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 boreholes encountered soil strata and bedrock at depths varying from three to eleven feet below the ground surface. At the surface, the exploratory boreholes encountered a sandy clay artificial fill deposit which extended one-half to two feet below the ground surface. The fill appeared dry to very moist, loosely to moderately compacted, and exhibited medium plasticity characteristics. Underlying the fill, BH-1 encountered a colluvial soil stratum which extended to five and one-half feet below the ground surface. The colluvium appeared moist, stiff, and exhibited medium plasticity characteristics. Underlying the colluvium in BH-1, and underlying the fill in BH-2 and BH-3, the boreholes encountered silty clay and gravelly clay residual soil strata which extended three to eleven feet below the ground surface. The residual soil strata appeared slightly moist to very moist, stiff to hard, and exhibited low to high plasticity characteristics. Underlying the residual soil strata, the boreholes encountered blueschist and shale bedrock which extended to the maximum depths explored. The bedrock appeared slightly hard to moderately hard, friable to weak, and highly weathered.

b. Groundwater. Groundwater was encountered in BH-1 at a depth of 9.5 feet below the ground surface at the time of our field exploration on April 6, 2016. No groundwater or seepage was encountered in BH-2 and BH-3. As with all hillside sites in Sonoma County, perched groundwater zones and seepage could develop at the site due to seasonal rainfall and would vary on the amount of rainfall received and time since it was received. We judge that seepage at the site, if it develops, would likely dissipate several weeks following seasonal rainfall.

6. 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 hazards which could result in damage to the proposed structure.

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.

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. According to the Association of Bay Area Governments (ABAG) the site is located in an area which is considered to have high liquefaction potential. However, the soils and bedrock encountered during our exploration are not considered to be prone to liquefaction. Therefore, it is judged that the risk of liquefaction at the site is low to non-existent.

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. Provided the planned retaining walls are constructed we do anticipate that lateral spreading or lurching is a serious concern for the project.

e. Tsunami. Based on the relative tsunami hazard map presented in Special Report 120, the site appears to be located in an area that may be inundated by tsunami waves with a run-up of 20 feet along the Pacific Coast and Russian River. However, contrary to Special Report 120, according to a Tsunami Inundation Map prepared by ABAG, the site appears to be located just outside of the Tsunami Inundation Zone.

f. Expansive Soils and Bedrock. Based on our findings and laboratory testing, the colluvium encountered in BH-1 exhibits medium plasticity characteristics (PI=24) and should be considered to be moderately expansive. However, the boreholes encountered high plasticity fill and residual soil strata which could be potentially. Therefore, the presence of potentially expansive soils should be considered during design and construction of the project. The bedrock does not appear to be expansive.

7. SLOPE STABILITY

a. Global Slope Stability. Based on our review of the California Divisions of Mines and Geology Special Report 120 slope stability map, the site appears to be located just outside the lower edge of a massive landslide complex. The mapped questionable landslide extends nearly one and one-half miles upslope of the site. The existing structure, driveway, fences, trees, etc. do not exhibit any obvious indications of distress from global slope movements. However, based on the results our investigation, we judge that the project site is located in an area which is considered to have a higher than normal risk for displacement and deformation resulting from earthquake-induced landsliding. We judge the recommendations and conclusions provided herein our geotechnical investigations are still suitable for construction of the proposed project. However, prior to construction of the proposed project, we judge that the owner must understand and accept the risk of potential displacement and deformation resulting from earthquake-induced landsliding.

b. Local Slope Stability. Based on our site reconnaissance and subsurface exploration, the project site appears locally stable. No obvious indications of slope instability such as local landsliding, debris flows, soil creep or areas experiencing extensive erosion were observed at the site during our site reconnaissance.

8. CONCLUSIONS

Based on the results of our investigation it is our professional opinion that the proposed 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 proposed project are the presence of an artificial fill deposit, compressible native surface soils and the presence of potentially expansive soils. Furthermore, the surface soils are potentially prone to downhill creep. These soils are not suitable for support of shallow foundations or non-structural concrete slabs-on-grade. We judge that the new foundations should extend into bedrock. We recommend that the new foundations should consist of drilled piers which extend into bedrock.

The plans indicate that the southern half of the structure will be supported by drilled piers. It appears the northern half of the structure will remain "as-is". For optimal building performance, we recommend that the entire structure should be underpinned by new foundations. If the existing foundations are to remain, then differential settlement may occur between the dissimilar foundation types and the differential movement could exceed tolerable limits. The owner should understand the potential and accept the risk and structural distress.

The plans indicate that the project will include the construction of a new slab-on-grade floor. We recommend that new slab-on-grade floors be supported on bedrock or underlain by at least 24 inches of low to non-expansive engineered fill. If a 24-inch thick layer of engineered fill is not practicable below the slab, we recommend that the slab should be structurally designed. We recommend that a subdrain should be installed below the slab-on-grade floor.

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

9. SITE GRADING AND EARTHWORK

a. Demolition and Stripping. Existing structures to be removed should be demolished and removed off site. Existing structures to remain and exposed by excavation should be underpinned. We recommend that structural areas be stripped of 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 any other 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. Voids left from the removal of utilities or other obstructions should be replaced with compacted engineered fill under the observation of the project geotechnical engineer.

b. <u>Excavation and Compaction</u>. Following site stripping, excavation should be performed to achieve finish grade. Where new fills are required, the existing fill and weak soils should be subexcavated and bedrock 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 1557-09 laboratory compaction test procedures. Highly plastic clays must not be placed within 24 inches of slab grade. Additional expansive soil laboratory testing could be required during construction. The fill material should be spread in eight-inch thick loose lifts, moisture conditioned to within two percent of the optimum moisture content, and compacted to 90 percent of the maximum dry density of the materials. Imported fill, should be evaluated and approved by the geotechnical engineer before importation.

c. <u>Cut and Fill Slopes</u>. Generally, it is recommended that cut and fill slopes be constructed at an inclination no steeper than 2H:1V. Temporary slopes should be graded to slopes 3/4H:1V or flatter. This should be evaluated by the geotechnical engineer in the field during construction. Potentially unstable subsurface conditions, such as adverse bedding, joint planes, zones of weakness, clay zones, or exposed seepage, may require either flatter slopes than specified above or retaining structures. It is recommended that a geotechnical engineer observe the cut slopes and provide final recommendations for the control of adverse conditions during grading operations, if encountered. During the rainy season, the cut slopes should be checked for springs or seepage areas. The surfaces of the cut slopes should be treated as needed in order to minimize the probability of slumping and erosion.

d. <u>Mitigations to Reduce Potential Foundation Undermining</u>. The project will require excavations near the existing structure up to six feet which will be supported by new retaining walls. An evaluation of the existing foundations is beyond the scope of our work on this project. To reduce potential undermining of existing foundations, we recommend that excavations within five feet of the existing footings should be underpinned. PJC can assist with design of the underpinning during construction.

All site preparation and fill placement should be observed by a representative of PJC. It is important that during the stripping, subexcavation and grading/scarifying 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.

10. FOUNDATIONS: DRILLED CAST-IN-PLACE PIERS

a. Vertical Loads. Due to the presence of weak and compressible surface soils prone to downhill creep, the new foundations should consist of drilled, cast-in-place concrete piers with a minimum diameter of 16 inches spaced at least three pier diameters center to center. All piers should be reinforced. The piers will derive their support through peripheral friction. Perimeter and interior piers should extend at least 12 feet below the existing ground surface and at least eight feet into firm residual soils and/or bedrock. The piers should be designed and reinforced to resist active lateral soil pressures generated from soil creep. The creep forces used in design should be equal to an active pressure of 50 pounds per square foot per one foot of depth (psf/ft) acting on two pier diameters. The height of the active creep zone should be considered to be three feet.

All piers should be connected with grade beams or tie beams, or the interior slab-on-grade. The grade beams or tie beams should be spaced no further than 15 feet apart in both directions. The beams or slab should be designed to span from pier to pier to support the structural loads and to resist the stresses imposed by the creep forces. The beams and slab should be reinforced as determined by the project structural engineer.

The piers may be designed using an allowable dead plus live skin friction of 700 pounds per square foot (psf). The top three feet should be neglected for vertical capacity. This value may be increased by one-third for short duration wind and seismic loads. A value of one-half the vertical capacity of the pier should be used to resist uplift forces. End bearing should be neglected because of difficulty in cleaning out small diameter pier holes and the uncertainty of mobilizing skin friction and end bearing simultaneously.

b. Lateral Loads. Lateral loads resulting from wind, soil creep or earthquakes can be resisted by the piers through a combination of cantilever action and passive resistance of the soil and/or bedrock surrounding the pier. A passive pressure of 350 pounds per square foot per one foot of depth acting on two pier diameters should be used. The upper three feet should be neglected for passive resistance.

c. Settlement. The maximum and differential settlements of the piers is estimated to be small and within tolerable limits.

If groundwater is encountered, it may be necessary to de-water the holes and/or place the concrete by the tremie method. If caving soils are encountered, it may be necessary to case the holes. Hard drilling may be necessary to achieve the required depths.

11. NON-STRUCTURAL CONCRETE SLABS-ON-GRADE FLOORS

Non-structural slabs may be used, provided they are supported on bedrock. If soft and/or expansive soils are encountered, we recommend that the new slab-on-grade floors be underlain by 24 inches of low to non-expansive compacted engineered fill. If a 24-inch thick layer of engineered fill is not practicable below the slab, we recommend that the slab should be structurally designed.

All slab subgrades should be moisture conditioned and rolled to produce a firm and unyielding subgrade. The slab subgrade should not be allowed to dry. Conventional 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.

We recommend that an impermeable membrane 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 non-structural slabs should be cast and maintained separate of existing foundations. A slab underdrain should be provided under the slab as presented on Plate 1b.

12. STRUCTURAL CONCRETE SLABS-ON-GRADE

If the 24-inch thick layer of engineered fill is not practicable below the slab, we recommend that the slab should be structurally designed. Structural slabs should be at least eight inches thick and reinforced as determined by the project structural engineer. Structural slabs should be supported by spread footings, and designed to span from pier to pier capable of supporting the anticipated design loads. The slab subgrade should be firm and unyielding and maintained in a moist condition at all times.

All slabs should be supported on at least four inches of clean gravel or crushed rock to provide a capillary break and provide uniform support for the slab. 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.

We recommend that an impermeable membrane at least 15 mils thick be placed over the drain rock to prevent migration of moisture vapor through the concrete slab. Control joints should be provided to induce and control cracking. A slab underdrain should be provided under the slab as shown on Plate 1b.

13. RETAINING WALLS

Retaining 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. The walls should be supported on drilled piers designed to the geotechnical criteria presented in this report.

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

Active Pressure (level backfill)	40 psf/ft	(pounds per square foot per depth)
Active Pressure (sloping backfill)	50 psf/ft	
At-Rest Pressure (level backfill)	55 psf/ft	
At-Rest Pressure (sloping backfill)	65 psf/ft	

The horizontal pseudostatic force acting upon retaining walls greater than six feet in height from earthquakes should be calculated from the following equation.

$P = 28.2 \times H^2$ (level backfill)

P = horizontal pseudostatic force acting upon the wall (lbs)

H = height of the wall (ft)

The location of the pseudostatic force is assumed to act at a distance of 0.67H above the base of the wall.

These pressures do not consider surcharge loads resulting from adjacent foundations, traffic loads or other loads. If additional surcharge loading is anticipated, we can assist in evaluating their effects. The walls should be completely waterproofed.

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, three-quarter to one and one-half inch crushed rock or gravel. The crushed rock or gravel should extend 12 inches horizontally from the back face of the wall and extend from the bottom of the wall to two feet below the finished ground surface. The upper 12 inches should be backfilled with compacted fine-grained soil to exclude surface water. Expansive clay soils should not be used for backfill soils.

A Mirafi 140N filter cloth should be placed between the on-site native material and the drain rock to prevent clogging. If Class 2 permeable drain rock is used, the filter fabric may be omitted.

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.

14. DRAINAGE

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

b. Slab Subdrain. We recommend that slab subdrains should be constructed below the slab-on-grade floor areas. Slab subdrain trenches should be constructed at a maximum of 20 foot intervals. The bottom of the trench should be sloped to drain by gravity. The bottom of the trench should be lined with a few inches of three-quarter to one and one-half inch drain rock or Class II permeable material. A four-inch diameter, SDR-35 perforated pipe, with holes down and sloped to drain, should be placed on top of the thin layer of drain rock. The trench should then be backfilled with compacted drain rock. We recommend that a drainage filter cloth such as Mirafi 140N be placed between the soil and the drain rock. The filter cloth can be omitted if Class II permeable material is used in lieu of the clean 3/4" drain rock. Surface drains must be maintained entirely separate from subdrains. A schematic detail is presented on Plate 1b.

15. 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: C

b. Mapped Acceleration Parameters: $S_s = 2.236$
$S_1 = 1.068$

c. Spectral Response Acceleration Parameters: $S_{Ms} = 2.234$
$S_{M1} = 1.388$

d. Design Spectral Acceleration Parameters: $S_{Ds} = 1.490$
$S_{D1} = 0.925$

16. 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 foundation upgrade project for the Jenner Inn's tree house structure located at 9470 Riverside Drive in Jenner, 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 and bedrock formations 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 is 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 grading and earthwork, approving foundation excavations, installation of helical piers, and approving the installation of 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.

Sincerely,

PJC & ASSOCIATES, INC.

Patrick J. Conway
Geotechnical Engineer
GE 2303, California

PJC: sms