**PJC & Associates, Inc.**
*Consulting Engineers & Geologists*

March 21, 2018 Job No. 8077.01

Vincent Chang
c/o: Pedersen Associates Landscape Architects
Attention: Pete Pedersen
PA@PedersenAssociates.com

Subject: Design Level Geotechnical Investigation
Proposed Residence and Detached Garage/Study
900 Highway 1
Bodega Bay, California

Dear Vincent:

PJC & Associates, Inc. (PJC) is pleased to submit this report which presents the results of our design level geotechnical investigation for the proposed residence and detached garage/study located at 900 Highway 1 in Bodega Bay, California. The approximate location of the site is shown on the Site Location Map, Plate 1. The site corresponds to the geographic latitudinal and longitudinal coordinates of 38.3305° north and 123.0447° west, according to GPS measurements performed at the site. Our services were completed in accordance with our proposal for geotechnical engineering services dated January 9, 2018, and your authorization to proceed with the work, dated January 9, 2018. 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 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 preliminary plans prepared by Pedersen Associates Landscape Architects, dated December 15, 2017, and the preliminary architectural plans prepared by JRP Architect dated December 15, 2017, it is our understanding that the proposed project will consist of constructing an approximately 3,754 square foot residence with an approximately 600 square foot detached garage/study at the 1.48 acre undeveloped lot. The residence will consist of a single-story structure with straw bale walls and joist-supported raised wood interior floors with concrete slab-on-grade covered entry and patio areas. The detached garage/study will consist of a two-story, wood frame structure with concrete slab-on-grade floors. We anticipate that the project will include the construction of exterior flatwork and retaining walls. The project will be serviced by underground municipal utilities.


SCALE: 1:24,000

REFERENCE: USGS BODEGA HEAD, CALIFORNIA 7.5 MINUTE QUADRANGLE, MAP REVISED 1987.

| PJC & Associates, Inc. Consulting Engineers & Geologists | SITE LOCATION MAP<br>PROPOSED RESIDENCE AND DETACHED GARAGE/STUDY<br>900 HIGHWAY 1<br>BODEGA BAY, CALIFORNIA | PLATE<br>1 |
|---|---|---|
| | Proj. No: 8077.01 Date: 1/18 App'd by: PJC | |

Structural foundation loading information was not available at the time of this report. 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.

Grading and drainage plans were unavailable at the time of this report. According to building cross-sections provided in the preliminary plans, we anticipate that site grading will consist of cuts up to approximately five feet and less, and fills of two to three feet and less to achieve the finished pad grades and provide adequate gradients for site drainage. The plans indicate that retaining walls will be required for the project.

## 2. SCOPE OF SERVICES

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

a. Drilling three exploratory boreholes to depths between 10.0 and 15.0 feet below the below the existing ground surface to observe the soil, bedrock, and groundwater conditions underlying the site. Our staff geologist performed the drilling, obtained representative samples for visual classification and laboratory testing and performed a surface reconnaissance of the site.

b. Laboratory observation and testing was performed on representative samples obtained during the course of the field investigation to evaluate the appropriate engineering characteristics of the soils and bedrock 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, expansive soils, etc.).

d. Perform engineering analyses to develop geotechnical recommendations for site preparation and grading, compaction requirements for subgrades and fills, foundation type(s) and design criteria, lateral earth pressures, support of concrete slabs-on-grade, retaining wall design criteria, site drainage and construction considerations.

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

## 3. SITE CONDITIONS

a. General. The project site is located east of Bodega Harbor. The 1.48 acre undeveloped property is located approximately 320 feet north of the intersection of Harbor View Way and Highway 1. At the time of our field investigation, pampas grass inhabited the northern margin of the property, with coyote brush at the southwestern portion of the property, and a Monterey Cyprus grove at the southeastern portion of the property. There is what appears to be the remnants of an abandoned northwest-southeast trending cut/fill earthen road south of the project site location. The property is bounded by Harbor View Way to the east, Highway 1 to the south, and vacant lots to the north and west.

b. Topography and Drainage. The project site is located on an uplifted marine terrace. The property in general gradually slopes to the southwest, towards Highway 1 and Bodega Harbor. According to USGS Bodega Head, California 7.5 Minute Quadrangle Topographic Map, the site is located at an approximate elevation of 66 feet above mean sea level (MSL). Regional drainage consists of sheet flow and surface infiltration that generally extends west and towards the Bodega Harbor, which is located approximately 525 feet southwest of the site. A low area at the northwestern margin of the property, nearby Highway 1 appears to be prone to spring and surface seepage conditions.

## 4. GEOLOGIC SETTING

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

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

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. Most important of these is the San Andreas fault itself, which was the loci of a major rupture in 1906.

According to California Division of Mines and Geology, Geology for Planning in Sonoma County, Special Report 120, the site is underlain by Quaternary marine terrace deposits (Qtd). These near shore marine sediments were deposited in a series of terraces that have been uplifted since Pleistocene time (approximately 1.8 million years ago). The terraces were formed from fluctuating sea levels caused by the advances and retreats of glaciers, which were characteristic during that time. The terraces have been slowly uplifted towards the east to form the characteristic bench and slope topography that extends several miles inland in some locations. Marine terraces sediments generally consist of unconsolidated silt, clay sand and gravel. Our subsurface exploration identified that the marine terrace deposits are underlain by sandstone bedrock of the Cretaceous to Jurassic Franciscan Complex (KJfss). Bedrock units of this portion of the Franciscan Complex are generally comprised of graywacke type sandstone and shale with lesser greenstone, conglomerate, and chert. Typical Franciscan Complex bedrock units are highly fractured and shattered. No Franciscan Complex bedrock surface exposures were observed at the property. However, we have observed Franciscan Complex bedrock exposed in nearby road cuts along Highway 1 within approximately ½ mile of the project site.

## 5. FAULTING

a. General. The San Andreas Fault has been mapped to be located approximately 500 feet west of the project site. The San Andreas Fault Zone has long been recognized as the major active fault along the Pacific Coast of the United States. It has been the focus for many earthquakes in historical times, the most famous being the April 1906 San Francisco earthquake. The fault zone is over 650 miles long, up to six miles wide, and extends from Shelter Cove in Humboldt County to the Salton Sea in Southern California. The boundaries of the fault zone are not straight, parallel lines, as is commonly thought. In the vicinity of Bodega Head, the San Andreas Fault Zones maximum width is about two miles.

The San Andreas Fault is a right lateral, strike-slip fault which has been active since approximately Early to Middle Tertiary time. Horizontal displacement along the fault has amounted to possibly hundreds of miles during that time. The amount of vertical movement is unknown. It is unlikely that there has been movement along the entire fault zone during any single earthquake. During the April 1906 earthquake, movement was reported along a continuous line from Point Arena, Mendocino County, to a point near San Juan Bautista, San Benito County, a distance of about 190 miles.

b. Alquist-Priolo Earthquake Fault Zone Map. Based on our review of the Bodega Head Quadrangle, Alquist-Priolo Earthquake Fault Zone Map, the project site is located in the State designated, Alquist-Priolo Earthquake Fault Zone. The Alquist-Priolo Earthquake Zoning Act, formerly called the Alquist-Priolo Special Studies Zones Act, was signed into California law on December 22, 1972. Under this Act, earthquake fault zones were delineated along known active faults. An active fault is one that has shown evidence of surface displacement within Holocene time (the last approximately 11,000 years). According to the Alquist-Priolo Earthquake Fault Zone Bodega Head Map, an approximately located, active fault traces exists within approximately 500 feet west of the project site, roughly along the Bodega Bay Harbor shoreline. Special studies are required in these zones for structures constructed for human occupancy. However, according to the Act, an exemption to this requirement is "a single family wood or steel framed dwelling or barn not exceeding two stories when such dwelling is not a part of a development of four or more dwellings." This exemption appears to apply to the project, however this should be confirmed by PRMD.

c. EQFAULT. According to the computer software fault modeling program EQFAULT, the four closest known active faults to the site are the San Andreas, the Point Reyes, the Rodgers Creek, and the Maacama (South) faults. The San Andreas fault is located approximately 500 west of the site, the Point Reyes fault is located 9.9 miles southwest of the site, the Rodgers Creek fault is located 20.2 miles northeast of the site and the Maacama (South) fault is located 25.5 miles northeast of the site. Table 1 outlines the nearest known active faults and their associated maximum magnitudes which are predicted to occur on those faults.

**TABLE 1**
**CLOSEST KNOWN ACTIVE FAULTS**

| Fault Name | Distance from Site | Maximum Earthquakes (Moment Magnitude |
|---|---|---|
| San Andreas | ~500 feet | 7.9 |
| Point Reyes | 9.9 miles | 6.8 |
| Rodgers Creek | 20.2 miles | 7.0 |
| Maacama (South) | 25.5 miles | 6.9 |

Reference-Blake, "EQFUALT" Version 3.00, software program.

## 6. SEISMICITY

Measurements of movement along the San Andreas Fault in the vicinity of Hollister using specially designed "creep recorders," indicate an annual movement of perhaps one-half inch. This continuing movement is horizontal, with the western block being displaced to the north relative to the eastern block. The April 1906 earthquake caused an average horizontal movement of five to ten feet in the vicinity of Bodega Bay, with a maximum horizontal displacement of about 21 feet reported near Olema, Marin County. Vertical displacement of as much as two or three feet is believed to have occurred in places, with the western block uplifted relative to the eastern block.

Most historic earthquakes originating on the San Andreas Fault in the northern California area have been relatively small. However, the earthquake occurring in 1906 had its epicenter near Olema, Marin County, and movement along this segment resulted in up to 21 feet of reported right-lateral displacement. Within the Fort Ross area, north of Bodega Bay, horizontal surface displacements of 12 to 15 feet were reported. Vertical displacements were generally less than two feet.

During the lifetime of the proposed project, it is possible that future damaging earthquakes could occur on any one of the previously discussed faults, most notably the San Andreas Fault. In general, the intensity of ground shaking at the site will depend on 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.

7. SUBSURFACE CONDITIONS

a. Soils and Bedrock. The subsurface conditions at the site were investigated by drilling three exploratory boreholes (BH-1 through BH-3) to depths between 10.0 and 15.0 feet below the existing ground surface. The approximate borehole locations are shown on the Borehole Location Plan, Plate 2. The boreholes were drilled to observe the soil, bedrock, and groundwater conditions and to collect samples of the underlying soils and bedrock 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 encountered 3.0 to 3.5 feet of topsoil consisting of a loose to medium dense clayey sand soil deposit. The topsoil appeared moist and fine to coarse grained with organics. Underlying the topsoil, the boreholes encountered a series of sandy clay terrace deposits followed by Franciscan Complex sandstone bedrock. The sandy clay terrace deposits extended to depths between 5.5 and 11.0 feet below the ground surface and appeared moist, stiff to hard, to soft and exhibited low to medium plasticity characteristics. The sandstone bedrock extended to the furthest depths explored and appeared slightly hard, friable, and highly weathered.

b. Groundwater. Groundwater was encountered in BH-1 at a depth of 10.0 feet during our field investigation on January 25, 2018. We also observed an area experiencing spring and surface seepage less than a few hundred feet northwest of the project site, close to Highway 1. Subsurface seepage within the soil strata and between the soils strata and bedrock may occur at the site during and following prolonged rainfall. Based on the subsurface conditions encountered, we judge that such conditions, if they develop, would likely dissipate following seasonal rainfall.

## 8. GEOLOGIC AND SEISMIC CONSIDERATIONS

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

a. Surface Fault Rupture. Rupture of the ground surface is expected to occur along known active fault traces. An active fault is one that has shown evidence of surface displacement within Holocene time (the last approximately 11,000 years). According to the Alquist-Priolo Earthquake Fault Zone Bodega Head Map, an approximately located, active fault traces exists approximately 500 feet west of the project site, roughly along the Bodega Bay Harbor shoreline. Special studies are required in these zones for structures constructed for human occupancy. However, according to the Act, an exemption to this requirement is "a single family wood or steel framed dwelling or barn not exceeding two stories when such dwelling is not a part of a development of four or more dwellings." This exemption appears to apply to the project, however this should be confirmed by PRMD. However, due to the close proximity to active faults, it should be considered that the risk of ground rupture at the site is high. The owner should understand and accept the risk of building in an active fault zone.

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. Based on this data and the anticipated life expectance of the project, it is judged that there is a high potential that the site will be subjected to very strong seismic shaking. The severity of the shaking depends on many complex factors. Among these factors are the moment magnitude, focal depth, distance from the causative fault, source mechanism, duration of shaking, high bedrock accelerations, type of surficial deposits, topography and design, type and quality of building construction.

c. Liquefaction. The Bay Area Government Maps (ABAG) indicates that the site is considered low in liquefaction potential. The soils and bedrock underlying the site are not at risk of liquefaction. Therefore, we judge that the risk of soil liquefaction at the site is 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. There are no exposed faces or creek embankments adjacent to the site. Therefore, we judge that the potential for lateral spreading and lurching at the site is low.

e. Expansive Soils and Bedrock. Based on our field and laboratory observations (PI=13) the site soils generally exhibit low plasticity characteristics. Therefore, the site soils are not considered to be expansive. Furthermore, the bedrock is not considered to be prone to shrink and swell cycles. Soil shrink and swell cycles do not appear to be a serious concern for the project.

f. Slope Stability. The California Division of Mines and Geology Special Report 120 landslides and relative slope stability maps the site as being underlain with soils and rock that is relatively unstable on slopes greater than 15% (category C & Bf). Areas mapped in this slope stability category generally contain abundant landslides. Furthermore, we judge slopes at the site exceeding 15 percent could be prone to soil creep.

g. Tsunamis. A tsunami is a series of waves propagated by the sudden displacement of a column of water. According to ABAG, the property appears to be located above and outside the potential tsunami inundation area.

## 9. CONCLUSIONS

Based on our investigation, we judge that the site is suitable for development from a geotechnical engineering standpoint, provided the recommendations presented in this report are incorporated into the design and carried out through construction. The primary geotechnical concern in design and construction of the project is the presence of weak and compressible topsoil that extends to depths between 3.0 and 3.5 feet below existing grade.

Weak and compressible soils may appear hard and strong when dry. However, they could potentially collapse under the load of foundations, engineered fill, and concrete slabs 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 structures. The detrimental effects of such movements can be significantly reduced by extending foundations below the weak zone. Below the topsoil, the soils gain strength and are only slightly compressible for the anticipated loads of construction. We recommend that foundation support be derived from the firm soils below the topsoil deposit. Planned cuts will likely remove the majority of weak soils in some of the foundation areas. We recommend that spread footing foundations should extend at least 18 inches into firm native soils as determined by the geotechnical engineer in the field. For estimating purposes, excavations on the order of 42 inches below the existing ground surface should be expected. The actual foundation depth should be determined by the geotechnical engineer in the field. As an alternative to the deep footings, we judge the structures could also be supported by drilled, cast-in-place pier and grade beam foundation system.

We anticipate that concrete slabs-on-grade will be used in the garage/exterior flatwork. Due to the weak soils, we recommend that slabs be underlain by at least 18 inches of low to non expansive compacted engineered fill and that a slab subdrain be installed below the slabs. The engineered fill should extend at least three feet beyond the garage and exterior flatwork. For optimum performance, total engineered fill thicknesses should not vary by more than two feet across the slabs.

Detailed geotechnical engineering recommendations for use in design and construction of the project are presented in the subsequent sections of this report.

## 10. EARTHWORK AND GRADING

a. Stripping. 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 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 or septic systems not included in the project should be abandoned in accordance with the requirements of the 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.

b. <u>Excavation and Compaction</u>. Following site stripping, excavations should be performed to achieve finish grade or prepare areas to receive fill. Where fill is required, the weak soils should be subexcavated and firm soils exposed, as determined by the geotechnical engineer on site during grading. We anticipate subexcavation depths up to 3.5 feet could be required to reach firm soils, where cutting is not performed. A level bench extending the width of the fill should be excavated. The exposed surface should be scarified to a depth of eight inches, moisture conditioned to two 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 1557-09 laboratory compaction test procedures. Potentially expansive soils, if encountered during grading, must not be placed within 18 inches of the garage and exterior slabs. Additional expansion laboratory testing could be required during construction. The low to non-expansive site soils free of organics and rocks larger than four inches in size, may be considered suitable for use as engineered fill.

The fill material should be spread in eight-inch thick loose lifts, moisture conditioned to two percent over the optimum moisture content and compacted to at least 90 percent of the maximum dry density of the materials. Imported fill, if required, should be evaluated and approved by the geotechnical engineer before importation. For optimum performance, total engineered fill thicknesses should not vary by more than two feet across the building pad and slabs. The lateral extent of the low to non-expansive fill should be a minimum of three feet beyond the edges of the garage and exterior concrete flatwork.

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 two percent over 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 two percent over 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.

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.

11. FOUNDATION OPTION: DEEPENED SPREAD FOOTINGS

a. Vertical Loads. The proposed residence and detached garage/study may be supported on deepened spread footings gaining support in the firm native soils below the topsoils provided they extend a minimum of 18 inches into firm native soils. For estimating purposes excavations on the order of 42 inches below the existing ground surface should be expected. 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 Tables 3. The bearing values provided have been calculated assuming that all footings bear on firm native soils, as determined by the geotechnical engineer on site during construction.

**TABLE 3**
**FOUNDATION DESIGN CRITERIA**

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

* Dead plus live load.
**At least 18" into firm native soils.

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, only the bottom 18 inches should be used for passive resistance.

Footing concrete should be placed neat against firm soils. 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 be less than one inch. Differential settlement between similarly loaded, adjacent footings is expected to be less than one-half inch. The majority of the settlement is expected to occur during construction and placement of dead loads.

## 12. RESIDENCE FOUNDATION OPTION -DRILLED PIERS

a. Vertical Loads. As an alternative to the deep footings, the proposed new residence could be supported on drilled, cast-in-place concrete piers with a minimum diameter of 12 inches spaced at least three pier diameters center to center. All piers should be reinforced. The piers will derive their support through peripheral friction. The piers should extend at least 10 feet below the finished ground surface, regardless of structural loads.

All perimeter piers and piers carrying interior continuous wall loads should be connected with grade beams or tie beams. The grade beams should be designed to span from pier to pier to support the structural loads.

The piers may be designed using an allowable dead plus live skin friction of 650 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 or earthquakes can be resisted by the piers through a combination of cantilever action and passive resistance of the soil surrounding the pier. A passive equivalent fluid pressure of 300 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 during pier drilling, 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.

## 13. NON-STRUCTURAL CONCRETE SLABS-ON-GRADE

Non-structural slabs-on-grade may be used for the garage slab and exterior flatwork provided the slabs are underlain by 18 inches of compacted low to non expansive engineered fill. The engineered fill should extend at least three feet beyond the garage and exterior flatwork. For optimum performance, total engineered fill thicknesses should not vary by more than two feet across the building pad and slabs.

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

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. The gravel should be moistened slightly prior to placing concrete. Control joints should be provided to induce and control cracking. Exterior slabs and the garage slab should not be tied to foundations. We recommend that slabs be reinforced to reduce cracking due to thermal and curling stresses. However, some cosmetic cracking will likely occur. Special care should be taken to insure that reinforcement is placed at the slab mid-height.

## 14. SEISMIC DESIGN

Based on criteria presented in the 2016 edition of the California Building Code (CBC) and ASCE (American Society of Civil Engineers) STANDARD ASCE/SEI 7-13, the following minimum criteria should be used in seismic design:

| | | |
|---|---|---|
| a. | Site Class: | C |
| b. | Mapped Acceleration Parameters: | $S_s$ = 2.606<br>$S_1$ = 1.251 |
| c. | Spectral Response Acceleration Parameters: | $S_{MS}$ = 2.606<br>$S_{M1}$ = 1.627 |
| d. | Design Spectral Acceleration Parameters: | $S_{DS}$ = 1.737<br>$S_{D1}$ = 1.084 |

15. RETAINING WALLS

a. Static Lateral Earth Pressures. 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.

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

Active Pressure (level backfill) (5H:1V or less)....................45 psf/ft
At Rest Pressure (level backfill) (5H:1V or less)....................60 psf/ft

b. Lateral Earth Pressures from Surcharge Loads. Retaining walls should be designed to resist additional induced lateral earth pressures due to traffic surcharge loads. Retaining walls free to rotate on the top should be designed to resist additional active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation or supporting compacted engineered fill, they should be designed for additional "at rest" lateral earth pressures.

Retaining walls should be designed to resist the following additional earth pressures generated from vehicular surcharge loads (rectangular distribution):

Active Pressure (level backfill) (5H:1V or less)..............80 psf
At Rest Pressure (level backfill) (5H:1V or less).............110 psf

The use of heavy, multi-ton compaction equipment such as large sheepsfoot rollers should not be allowed within a distance equal to one-half of the total wall height from the back face of retaining walls or the walls should be designed for additional induced lateral earth pressures. Retaining walls free to rotate on the top should be designed to resist additional active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation or supporting compacted engineered fill, they should be designed for additional "at rest" lateral earth pressures.

c. Pseudostatic Pressure. For retaining 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:

$P_E =$ 26.0 $H^2$ (level condition) (5H:1V or less)

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 pressures 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, 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 one foot below the finished ground surface. The upper 12 inches should be backfilled with compacted fine-grained soil to exclude surface water. 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.

## 16. DRAINAGE

a. Surface Drainage. 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.

b. Slab-On-Grade Subdrains. 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.

c. Foundation Subdrains. We recommend that foundation subdrains be placed adjacent to all foundations. Where structures have an interior crawl space, foundation drains should extend at least 12 inches below the bottom of the interior crawl space grade. 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 ¾ to 1½-inch drain rock. A 4-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 to within 6 inches of the finished surface with drain rock. The upper 6 inches should consist of compacted soil to reduce surface water inclusion. We recommend that a drainage filter cloth such as Mirafi 140N be placed between the soil and the drain rock.

## 17. LIMITATIONS

The data, information, interpretations and recommendations in this report are presented solely as bases and guides for the geotechnical design of the proposed residence and detached garage/study located at 900 Highway 1 in Bodega Bay, California. The conclusions and professional opinions presented herein were developed in accordance with generally accepted geotechnical engineering principles and practices. As with all geotechnical reports, the opinions expressed here are subject to revisions in light of new information, which may be developed in the future, and no warranties are either expressed or implied.

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 purpose 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 and approved in writing. This report and the drawings contained herein are intended only for the design of the proposed project. 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 the 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 PJC does not and cannot have complete knowledge of the subsurface conditions underlying the subject site. The criteria presented are based upon the findings at the points of exploration and upon interpretative data, including interpolation and extrapolation of information obtained at points of observation.

## 18. ADDITIONAL SERVICES

Upon completion of the project plans, they should be reviewed by our firm to verify that the design is consistent with the recommendations of this report. During the course of this investigation, several assumptions were made regarding building loads and development concepts. Should our assumptions differ significantly from the final intent of the project designers, our office should be notified of the changes to assess any potential need for revised recommendations. Observation and testing services should be provided by PJC to verify that the intent of the plans and specifications is carried out during construction; these services should include observing the foundation excavations, field density testing of fill, approving slab subgrade, and observing the 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 the responsibility for items that they are not notified to observe.

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

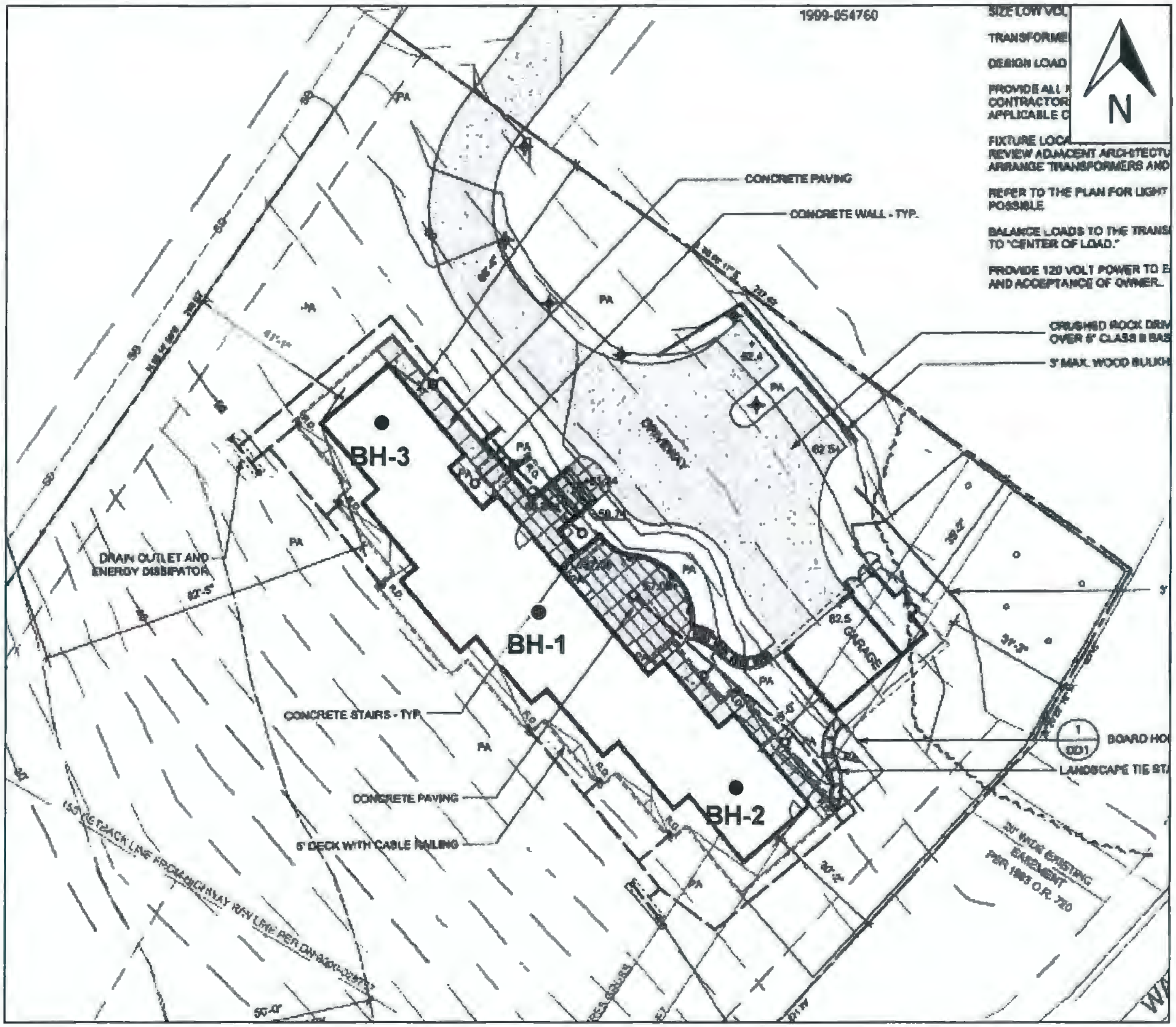
Sincerely,

PJC & ASSOCIATES, INC.

Patrick J. Conway
Geotechnical Engineer
GE 2303, California

REGISTERED PROFESSIONAL ENGINEER
PATRICK J. CONWAY
No. 2303
Exp. 3-31-18
GEOTECHNICAL
STATE OF CALIFORNIA

PJC:ljc:sms




NO SCALE

EXPLANATION

● BOREHOLE LOCATION AND DESIGNATION

REFERENCE: SITE PLAN TITLED "CHANG RESIDENCE," PREPARED BY PEDERSEN ASSOCIATES LANDSCAPE ARCHITECTS, DATED DECEMBER 15, 2017.

| | | |
|---|---|---|
| PJC & Associates, Inc.<br>Consulting Engineers & Geologists | BOREHOLE LOCATION PLAN<br>PROPOSED RESIDENCE AND DETACHED GARAGE/STUDY<br>900 HIGHWAY 1<br>BODEGA BAY, CALIFORNIA | PLATE<br>2 |
| | Proj. No: 8077.01 Date: 1/18 App'd by: PJC | |

# PJC & Associates, Inc.

*Consulting Engineers & Geologists*

## BORING NUMBER BH-1



CLIENT Vincent Chang

PROJECT NAME Proposed Residence and Detached Garage/Study

JOB NUMBER 8077.01

LOCATION 900 Highway 1 Bodega Bay, California

DATE STARTED 1/25/18 COMPLETED 1/25/18 GROUND ELEVATION HOLE SIZE 4"

DRILLING CONTRACTOR Lone Pine Drilling

GROUND WATER LEVELS:

DRILLING METHOD 6X6 with 140lb. Hammer

AT TIME OF DRILLING ---

LOGGED BY L.C. CHECKED BY PJC

AT END OF DRILLING ---

NOTES

AFTER DRILLING 10.00 ft

| 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' - 3.5'; CLAYEY SAND (SC); moderate brown, moist, medium dense, fine to coarse grained, trace organics (TOPSOIL) | | | | | | | | | | |
| | | | MC | | 9 | | 107 | 14 | | | | |
| | | 3.5' - 6.5'; SANDY CLAY (CL); moderate brown, moist, stiff to hard, low plasticity, clay content increases with depth (TERRACE DEPOSIT) | SPT | | 11 | | | 13<br>14 | 27 | 14 | 13 | |
| 5 | | | MC | | 37 | 4.5 | 113 | 17 | | | | |
| | | 6.5' - 8.0'; SANDY CLAY (CL); orangish moderate brown, moist, hard, medium plasticity, trace subangular sandstone gravels, with sand lenses (TERRACE DEPOSIT) | | | | | | | | | | |
| | | 8.0' - 15.0'; SANDSTONE (KJfss) slightly hard, friable, highly weathered (BEDROCK) | MC | | 39 | | 107 | 19 | | | | |
| 10 | | | | | | | | | | | | |
| | | | MC | | 53 | | 97 | 29 | | | | |
| 15 | | | SPT | | 27 | | | 4 | | | | |

Bottom of borehole at 15.0 feet.

ORIGINAL GEOTECH BH COLUMNS - GINT STD US.GDT - 3/21/18 14:43 - C:\USERS\PUBLIC\DOCUMENTS\BENTLEY\GINT\PROJECTS\8077.01 900 HIGHWAY 1.GPJ

# PJC & Associates, Inc.

*Consulting Engineers & Geologists*

## BORING NUMBER BH-2



CLIENT Vincent Chang

PROJECT NAME Proposed Residence and Detached Garage/Study

JOB NUMBER 8077.01

LOCATION 900 Highway 1 Bodega Bay, California

DATE STARTED 1/25/18

COMPLETED 1/25/18

GROUND ELEVATION

HOLE SIZE 4"

DRILLING CONTRACTOR Lone Pine Drilling

GROUND WATER LEVELS:

DRILLING METHOD 6X6 with 140lb. Hammer

AT TIME OF DRILLING ---

LOGGED BY L.C.

CHECKED BY PJC

AT END OF DRILLING ---

NOTES

AFTER DRILLING --- Not Encountered

| 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'; CLAYEY SAND (SC); moderate brown, moist, loose, fine to coarse grained (TOPSOIL) | | | | | | | | | | |
| | | | MC | | 8 | | | | | | | |
| | | 3.0' - 6.0'; SANDY CLAY (CL); moderate brown, moist, hard, low plasticity, trace subangular sandstone gravels (TERRACE DEPOSIT) | | | | | | | | | | |
| 5 | | | MC | | 32 | | | | | | | |
| | | 6.0' - 10.0'; SANDSTONE (KJfss); orangish brown, slightly hard, friable, highly weathered (BEDROCK) | | | | | | | | | | |
| | | | MC | | 50 | | 105 | 17 | | | | |
| 10 | | | SPT | | 24 | | | | | | | |

Bottom of borehole at 10.0 feet.

# PJC & Associates, Inc.

*Consulting Engineers & Geologists*

## BORING NUMBER BH-3



CLIENT Vincent Chang

PROJECT NAME Proposed Residence and Detached Garage/Study

JOB NUMBER 8077.01

LOCATION 900 Highway 1 Bodega Bay, California

DATE STARTED 1/25/18

COMPLETED 1/25/18

GROUND ELEVATION

HOLE SIZE 4"

DRILLING CONTRACTOR Lone Pine Drilling

GROUND WATER LEVELS:

DRILLING METHOD 6X6 with 140lb. Hammer

AT TIME OF DRILLING ---

LOGGED BY L.C.

CHECKED BY PJC

AT END OF DRILLING ---

NOTES

AFTER DRILLING --- Not Encountered

| 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'; CLAYEY SAND (SC); dark brown, moist, loose, fine to coarse grained, trace roots, porous (TOPSOIL) | | | | | | | | | | |
| | | | MC | | 10 | | | | | | | |
| | | 3.25' - 5.5'; SANDY CLAY (CL); moderate brown, moist, stiff to hard, low plasticity (TERRACE DEPOSIT) | | | | | | | | | | |
| 5 | | | | | | | | | | | | |
| | | 5.5' - 11.0'; SANDY CLAY (CL); orangish brown, moist to very moist, stiff to soft, medium plasticity (TERRACE DEPOSIT) | | | | | | | | | | |
| | | | MC | | 36 | 1.75 | 114 | 17 | | | | |
| 10 | | | MC | | 12 | .47(U) | 104 | 22 | | | | |
| | | 11.0' - 12.5'; SANDSTONE (KJfss); slightly hard, friable, highly weathered (BEDROCK) | | | | | | | | | | |
| | | | MC | | 37 | | 110 | 19 | | | | |

Bottom of borehole at 12.5 feet.

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

## KEY TO TEST DATA

LL — Liquid Limit (in %)
PL — Plastic Limit (in %)
G — Specific Gravity
SA — Sieve Analysis
Consol — Consolidation
■ "Undisturbed" Sample
☒ Bulk or Disturbed Sample
☐ No Sample Recovery

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

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

PJC & Associates, Inc.
Consulting Engineers & Geologists

USCS SOIL CLASSIFICATION KEY
PROPOSED RESIDENCE AND DETACHED GARAGE/STUDY
900 HIGHWAY 1
BODEGA BAY, CALIFORNIA

PLATE 6

Proj. No: 8077.01 Date: 1/18 App'd by: PJC

## ROCK TYPES

| | | |
|---|---|---|
| CONGLOMERATE | SHALE | METAMORPHIC ROCKS<br>HYDROTHERMALLY-ALTERED ROCKS |
| SANDSTONE | SHEARED SHALE MELANGE | IGNEOUS ROCKS |
| META-SANDSTONE . | CHERT | |

### BEDDING THICKNESS

| | |
|---|---|
| MASSIVE | Greater than 6 feet |
| THICKLY BEDDED | 2 to 6 feet |
| MEDIUM BEDDED | 8 to 24 inches |
| THINLY BEDDED | 2-1/2 to 8 inches |
| VERY THINLY BEDDED | 3/4 to 2-1/2 inches |
| CLOSELY LAMINATED | 1/4 to 3/4 inches |
| VERY CLOSELY LAMINATED | Less than 1/4 inch |

### JOINT, FRACTURE, OR SHEAR SPACING

| | |
|---|---|
| VERY WIDELY SPACED | Greater than 6 feet |
| WIDELY SPACED | 2 to 6 feet |
| MODERATELY WIDELY SPACED | 8 to 24 inches |
| CLOSELY SPACED | 2-1/2 to 8 inches |
| VERY CLOSELY SPACED | 3/4 to 2-1/2 inches |
| EXTREMELY CLOSELY SPACED | Less than 3/4 inch |

### HARDNESS

Soft - pliable; can be dug by hand

Slightly Hard - can be gouged deeply or carved with a pocket knife

Moderately Hard - can be readily scratched by a knife blade; scratch leaves heavy trace of dust and is readily visible after the powder has been blown away

Hard - can be scratched with difficulty; scratch produces little powder and is often faintly visible

Very Hard - cannot be scratched with pocket knife, leaves a metallic streak

### STRENGTH

Plastic - capable of being molded by hand

Friable - crumbles by rubbing with fingers

Weak - an unfractured specimen of such material will crumble under light hammer blows

Moderately Strong - specimen will withstand a few heavy hammer blows before breaking

Strong - specimen will withstand a few heavy ringing hammer blows and usually yields large fragments

Very Strong - rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

### DEGREE OF WEATHERING

Highly Weathered - abundant fractures coated with oxides, carbonates, sulphates, mud, etd., through discoloration, rock disintegration, mineral decomposition

Moderately Weathered - some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

Slightly Weathered - a few strained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

Fresh - unaffected by weathering agents, no appreciable change with depth.

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BEDROCK CLASSIFICATION KEY
PROPOSED RESIDENCE AND DETACHED GARAGE/STUDY
900 HIGHWAY 1
BODEGA BAY, CALIFORNIA

PLATE 7

Proj. No: 8077.01 Date: 1/18 App'd by: PJC

# APPENDIX A
# FIELD INVESTIGATION

## 1. INTRODUCTION

The field program performed for this study consisted of drilling three exploratory boreholes (BH-1 through BH-3) at the proposed building site. The exploration was completed on January 25, 2018. 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 5.

## 2. BOREHOLES

The boreholes were advanced using a portable powered drill rig with solid stem flight augers. The drilling was performed by a staff geologists of PJC who maintained a continuous log of the soil conditions and obtained samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System, as explained in Plate 6. The bedrock is described according to Plate 7.

Relatively undisturbed and disturbed samples were obtained from the exploratory boreholes. A 2.43-inch I.D. California Modified sampler and a 1.375-inch Standard Penetrometer sampler were driven into the underlying soil using a 70 pound hammer falling 30 inches to obtain an indication in the field of the density of the soil and to allow visual examination of at least a portion of the soil column. The number of blows required to drive the sampler at six-inch increments was recorded on each 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 currently accepted test procedures of the American Society of Testing and Materials (ASTM). Undisturbed and disturbed 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 and dry density, pocket penetrometer, and Atterberg Limits testing.

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

b. Pocket Penetrometer. Pocket Penetrometer tests were performed on cohesive stratums encountered during excavation. 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 (psf).

c. Atterberg Limits Determination. Liquid and plastic limits were determined on selected samples in accordance with ASTM D 4318-83.

d. <u>Unconfined Compression Test</u>. Unconfined compression tests were performed on intact samples obtained from the boreholes. The unconfined compression test is determined by axial loading the sample under a slow constant strain rate until failure is obtained. Failure stress is defined as the maximum stress at peak strain. The results of these tests are presented on the borehole logs.

# APPENDIX C
# REFERENCES

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

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

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

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

5. USGS Bodega Head California Quadrangle 7.5-Minute Topographic Map, Dated 1979.

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

7. California Building Code (CBC), 2016 edition.

8. "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," California Department of Conservation Division of Mines and Geology, Dated February 1998.

9. "EQFAULT" Version 3.0, computer program.

10. Preliminary Grading and Materials Plans titled, "Chang Residence," Sheets DD-0 through DD-2, prepared by Pedersen Associates, dated December 15, 2017.

11. Architectural plans titled, "Chang Residence," Sheets PD-1 through PD-3, prepared by JRP architect, dated December 15, 2017.