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812 W. Wabash Avenue, Eureka, CA 95501-2138

Reference: 022043

June 15, 2022

Don Daniels
22985 El Camino Real
Santa Margarita, CA 93453

Subject: Soils Engineering Report for Proposed Single Family Residence at 495 Sea Court, Shelter Cove, Humboldt County, California

Dear Don Daniels:

Introduction

This report provides the results of SHN's investigation of geotechnical site conditions and geologic hazards for the proposed project at 495 Sea Court, in the community of Shelter Cove, Humboldt County, California. We understand the scope of the proposed improvements includes the development of a new single-family residential structure. This report contains the results of our field exploration and laboratory testing upon which our conclusions and recommendations are based. This report is intended to address all items requested in the soils report checklist provided on the Humboldt County Planning and Building Department's website (Humboldt County, 2008).

Project Description

The subject property is located in the coastal community of Shelter Cove in southern Humboldt County, California (Figure 1). The Assessor's parcel number (APN) and project site coordinates are as follows:

APN:	111-121-036
Latitude:	40.029671 N
Longitude:	-124.078449 W

The property on Sea Court is on the southwest side of Lower Pacific Drive and is on an undeveloped oceanfront parcel on the bluff top above the Pacific Ocean. The proposed project consists of the construction of a two-story, single-family residential structure. We anticipate the proposed single-family residential unit will be a wood-framed structure with an elevated primary living area, supported by, or partially supported on, concrete drilled piers. The lower level will be generally open and used for parking and may be slab-on-grade. The buildable area of the subject lot occurs on the planar bluff top surface. The lot extends to the southwest and encompasses a portion of the bedrock wave slope, which is not considered developable.

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SHN	Don Daniels Soils Engineering Report 495 Sea Court, Shelter Cove, California		Project Location Map SHN 022043
	June 2022	Figure1_ProjectLocationMap	Figure 1

Purpose and Scope of Work

The purpose of SHN's geotechnical investigation was to evaluate the physical and engineering properties of the site subsurface materials in the area of the proposed residence, in order to provide recommendations for the design and construction of the proposed residential structure. We also completed an assessment of geologic hazards in relation to the project to assess risk levels associated with geologic site conditions. The work described herein supplements previous geotechnical investigations completed by SHN at other nearby sites over the past several decades.

The scope of SHN's services consisted of a site visit and subsurface exploration at the residence site, geologic hazard analysis, review of aerial photos and published geologic maps, laboratory testing, engineering analysis, and preparation of this report containing the following items:

- A site map showing the location of the exploratory hand-auger borings
- Descriptions of soil, bedrock, and groundwater conditions interpreted based on our field exploration, laboratory testing, and review of existing geotechnical and geologic information
- Descriptions of geologic setting
- Discussion of geologic hazards
- Seismic design parameters in accordance with the applicable portions of the 2019 California Building Code (CBC; California Building Standards Commission, 2020) and American Society of Civil Engineers (ASCE) 7-16 Standard (ASCE, 2017), including the soil classification, seismic design category, and spectral response accelerations for design of the residential structure
- Recommendations for site preparation, fill material, placement and compaction requirements, and foundation support
- Assessment of foundation types and load-bearing soil conditions, including allowable bearing capacities and estimates of settlements for the proposed residential structure
- Recommendations for observation and testing during construction

Field Exploration

A project geologist from SHN conducted a site visit on March 17, 2022, to advance three exploratory hand-auger borings and observe existing site conditions. Borings HB-1, HB-2, and HB-3 (Figure 2) were advanced in the area of a proposed new single-family residence. All three hand borings met practical refusal on coarse gravels encountered at varying depths.

Hand-driven tube samples and bulk samples were collected of representative materials encountered in the borings. The subsurface materials encountered were logged and field classified in general accordance with the Manual-Visual Classification Method (ASTM-International [ASTM] D 2488). Final hand-auger boring logs, presented in Appendix 1, were prepared based on the field logs, examination of samples in the laboratory, and laboratory test results.

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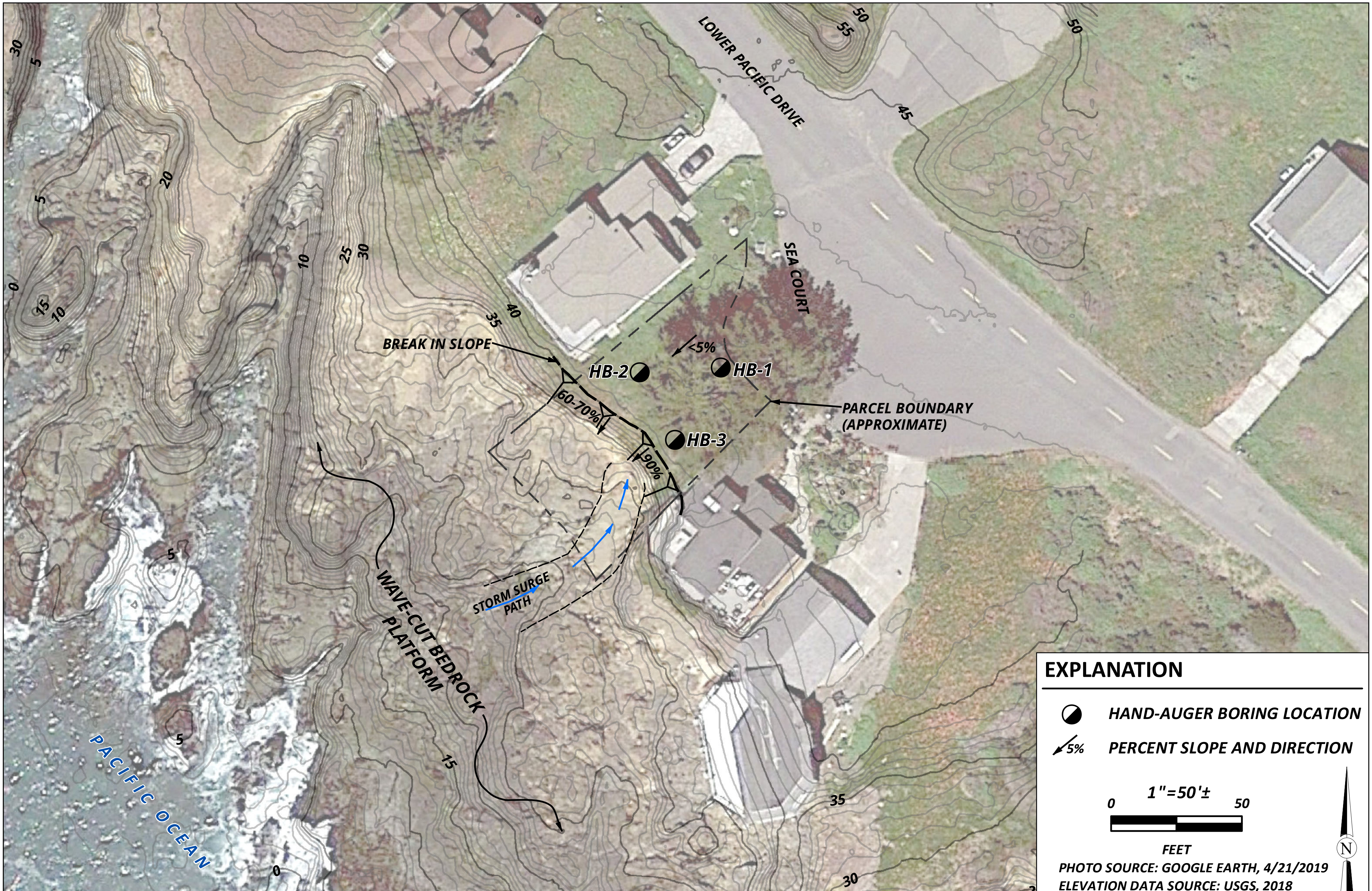


Don Daniels Soils Engineering Report 495 Sea Court, Shelter Cove, California		Site Map with Hand-Auger Boring Locations SHN 022043
June 2022	Figure2_SiteMap	Figure 2

Laboratory Testing

Selected soil samples collected from the subsurface investigation were tested in SHN's certified soils-testing laboratory in Eureka, California, to determine index properties and strength characteristics of the subsurface materials. Samples were tested for in-place moisture content and dry density, percent passing #200 sieve (percent fines), and Atterberg limits (liquid limit, plastic limit, and plasticity index). Test results are provided at the corresponding sample locations on the test pit logs in Appendix 1 and included as Appendix 2.

Geologic Setting

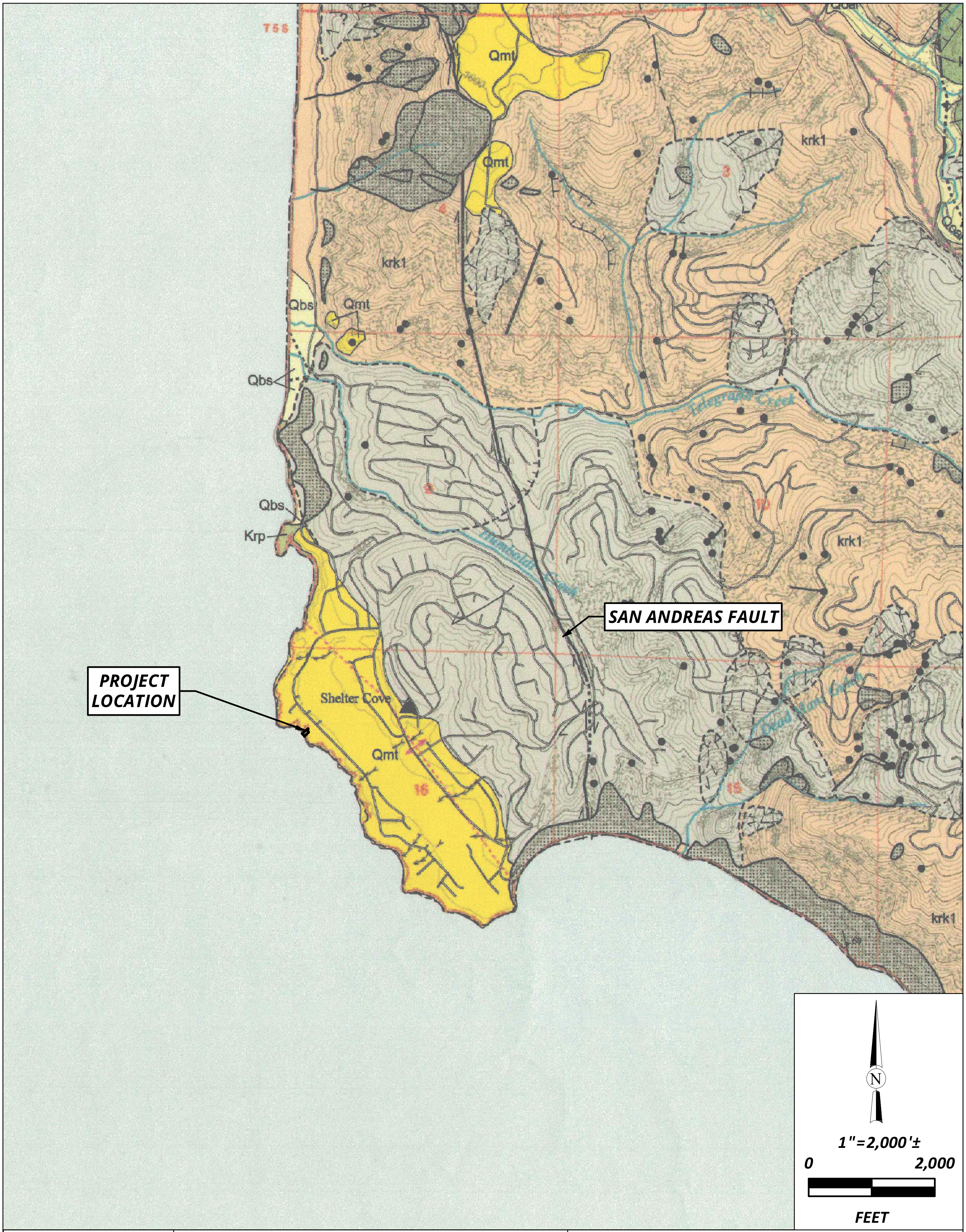
The subject property is in the coastal southern Humboldt County community of Shelter Cove, on a gently sloping marine terrace and wave-cut bedrock platform. The site is located on the coast in a complex and dynamic geologic environment, south of Cape Mendocino. Cape Mendocino, approximately 22 miles northwest of the project area, marks the intersection of three crustal plates known as the Mendocino Triple Junction (MTJ) and is characterized by active tectonic deformation and high rates of seismicity. The tectonics of the area to the southwest of the MTJ are predominately controlled by strike-slip faulting. The project parcel is approximately 4,400 feet west of the northern portion of the San Andreas fault. This section of the fault was observed to have ruptured during the great San Francisco earthquake in 1906 and, therefore, is considered active.

According to published geologic mapping, the site is mapped as being underlain by Quaternary-age marine terrace deposits consisting of sand and gravel deposited in a shallow marine setting on gently inclined wave-cut beaches (Davenport et al., 2002; Figure 3 and Figure 3a). Mapping indicates bedrock below the thin veneer of terrace deposits is igneous and sedimentary rocks of Point Delgada formation within the King Peak Terrane of the Franciscan Complex. This bedrock is described as basaltic rocks, sandstone, minor argillite, and mélange containing blueschist blocks. Bedrock in the hills to the east is mapped as Miocene to late Cretaceous mélange or folded argillite of the King Peak Terrane. Davenport et al., (2002) and McLaughlin et al. (2000) map the upland area of eastern Shelter Cove as Holocene and Pleistocene landslide deposits.

Marine terrace deposits were encountered in our exploratory borings at the subject site and are described below. Additionally, the bedrock platform, exposed on the wave slope, in the southwest part of the site is comprised of sandstone and basaltic bedrock.

Site Conditions

The project site is on a nearly-level surface with slopes that are generally less than 5 percent, and elevations averaging 40 feet above mean sea level (Figure 2). The subject parcel is elongate in the northeast direction and bordered on the northwest and southeast by residential lots that are developed with single-family residences. The parcel is approximately 127 feet long on the northwest side and 95 feet on the southeast side, and roughly 55 to 60 feet wide. The northeast side of the parcel is defined by Sea Court off Lower Pacific Drive and the southwest side of the parcel is on the bedrock platform.



Don Daniels
Soils Engineering Report
495 Sea Court, Shelter Cove, California

Geologic Map
(Davenport et al., 2002)
SHN 022043

June 2022 | Figure3_GeologicMap | Figure 3

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MAP UNITS

QUATERNARY AND LATE TERTIARY OVERLAP DEPOSITS

- Qscu — **Undifferentiated Stream Channel Deposits (Holocene)** - Unconsolidated sediments in active channels and flood plains.
- Qbs — **Beach Sand (Holocene)** - Marine-laid deposits of fine- to coarse-grained sand and gravel; may migrate seasonally.
- Qds — **Aeolian deposits (Holocene)** - Unconsolidated fine- to medium-grained, well-sorted sand.
- Qf — **Alluvial fan (Holocene)** - Characteristic fan-cone shapes at the mouths of eroding stream canyons; includes debris fans.
- Qal — **Alluvium (Holocene and Late Pleistocene?)** - Undifferentiated alluvial deposits of unconsolidated sand, gravel, silt, and lesser clay.
- Qc — **Colluvium (Holocene-Pleistocene)** - Talus and slope wash deposits.
- Qoal — **Older alluvium (Early Holocene and Pleistocene)** - Unconsolidated to weakly consolidated alluvial deposits above the active channel in broader canyons and valleys; vegetation is characteristically well-established.
- Qrt — **River terrace deposits (Holocene and Pleistocene)** - Dominantly sand and gravel with lesser silt and clay deposited during higher stream stands over flat-lying to gently inclined platforms.
- Qmt — **Marine terrace deposits (Quaternary)** - Sand and gravel deposited in a shallow marine setting on gently inclined wave-cut beaches.
- Qt — **Undifferentiated terrace deposits (Quaternary)** - Alluvial and/or (near the coast) shallow marine deposits preserved in erosional remnants of older platforms well above present stream level.
- QTw — **Overlap Deposits (Neogene)** - Wildcat Group-equivalent rocks; weakly lithified sandstone, mudstone, and minor conglomerate.

FRANCISCAN COMPLEX*

Coastal Belt

Coastal Terrane (Pliocene to Late Cretaceous)

- co1 — **Melange** - Dominantly highly folded argillite and highly clayey, penetratively sheared rock that exhibits rounded, lumpy, and irregular, poorly incised topography.
- co2 — **Melange** - Subequal amounts of shattered sandstone and argillite with much clayey, penetratively sheared rock that exhibits generally irregular topography lacking well-incised sidehill drainages.
- co3 — **Broken sandstone and argillite** - Exhibits sharp-crested topography with a well-incised system of sidehill drainage.
- co4 — **Intact sandstone and argillite** - Exhibits sharp-crested topography with a regular, well-incised system of sidehill drainage.
- cob — **Basaltic rocks (Late Cretaceous)** - Pillow flows, tuffs, flow breccias, and intrusives present as rare blocks in melange.
- cols — **Limestone (Late Cretaceous)** - Pink to red, manganiferous, and containing pelagic foraminifers.
- m — **Blueschist (Jurassic?)**

King Range Terrane (Miocene to Late Cretaceous)

- Krp — **Igneous and sedimentary rocks of Point Delgada (Late Cretaceous)** - Basaltic rocks, sandstone, minor argillite, and melange containing rare blueschist blocks.
- krk1 — **Melange and (or) folded argillite of King Peak** - Thin-bedded, highly folded, predominantly argillitic sequences that exhibit subdued, irregular topography lacking a well-incised system of sidehill drainages.
- krk2 — **Highly folded, broken formation of King Peak** -Thick- to thin-bedded arkosic sandstone and calcareous argillite that exhibit sharp-crested topography with well incised but irregular sidehill drainages.
- krk3 — **Highly folded, largely unbroken rocks of King Peak** - Sandstone and argillite that exhibit sharp-crested topography with a regular, well-incised system of sidehill drainages.
- krl — **Limestone** - Red to white, locally with planktic or benthic foraminifers; present locally as melange blocks, and as interbeds in calcareous argillite.
- krc — **Chert** - Red to green, locally manganiferous, with radiolaria and diatoms.
- krb — **Basaltic rocks** - Tholeiitic and alkalic, present as rare blocks in melange.

Yager Terrane (Eocene to Paleocene)

- y1 — **Sheared and highly folded mudstone** - Includes minor rhythmically interbedded sandstone, locally with lenses of conglomerate. Exhibits irregular topography lacking a well-incised system of sidehill drainages.
- y2 — **Highly folded, broken mudstone, sandstone, and conglomeratic sandstone** - Exhibits topography with sharp ridge-crests and well-incised sidehill drainages.
- y3 — **Highly folded, little-broken sandstone, conglomerate, and mudstone** - Exhibits sharp-crested topography with a regular, well-incised system of sidehill drainages.
- Ycgl — **Conglomerate** - Polymict, well-rounded clasts that include volcanic, granitic, and less common metaclastic rocks.

Central Belt (Paleocene? to Jurassic)

- cm1 — **Melange** - Predominantly penetratively sheared, locally tuffaceous, scaly meta-argillite and less abundant blocks of metasandstone.
- cb1 — **Broken formation** - Bedded to massive, locally folded, rarely conglomeratic metasandstone and meta-argillite, with minor highly sheared rocks.
- cwr — **White Rock metasandstone (Paleocene and/or Late Cretaceous)** - Arkosic metasandstone and minor meta-argillite, thick-bedded to massive, shattered and sheared.
- cls — **Limestone (Late to Early Cretaceous)** - Red, pink, gray, or white foraminiferal limestone.
- bs — **Basaltic rocks (Cretaceous and Jurassic)** - Includes pillowed and non-pillowed flows, flow breccias, submarine tuff, and diabase.
- b — **Melange block** - Lithology unknown.
- sp — **Serpentinite** - Interleaved locally along faults.

* Franciscan Complex subdivisions from McLaughlin and others, 2000

MAP SYMBOLS

Lithologic contact: Solid where location is certain, dashed where approximately located or inferred, dotted where concealed, and queried where continuation or existence is uncertain.

Fault: Solid where location is certain, dashed where approximately located or inferred, dotted where concealed, and queried where continuation or existence is uncertain.

Thrust fault: Barbs on upper plate. Solid where location is certain, dashed where approximately located or inferred, dotted where concealed, and queried where continuation or existence is uncertain.

Lineament: Linear feature of unknown origin noted on aerial photographs.

Anticline: Solid where location is certain, dashed where approximately located or inferred, dotted where concealed, and queried where continuation or existence is uncertain.

Syncline: Solid where location is certain, dashed where approximately located or inferred, dotted where concealed, and queried where continuation or existence is uncertain.

Strike and dip of bedding
Strike and dip of bedding, top indicator observed
Strike and dip of overturned bedding
Strike and dip of overturned bedding, top indicator observed
Strike of vertical bedding
Strike of vertical bedding, top indicator observed
Strike and dip of foliation
Strike of vertical foliation
Strike and dip of joint

Watershed boundary
Subbasin boundary
County boundary
Public Land Survey System
Stream
Road, street or trail
City or town

ROCK SLIDE (ROTATIONAL / TRANSLATIONAL LANDSLIDE): Slope movement with bedrock as its primary source material. This class of failure includes rotational and translational landslides; relatively cohesive slide masses with failure planes that are deep-seated in comparison to those debris slides of similar areal extent. The slide plane is curved in a rotational slide. Movement along a planar joint or bedding surface may be referred to as translational. Complex versions with combinations of rotational heads and translational movement or earthflows downslope are common. indicates a scarp; arrows show direction of movement; queried where the presence of the slide is uncertain. Boundary is solid where historically active, dashed where dormant, queried where uncertain.

EARTHFLOW: Slow to rapid movement of mostly fine-grained soil with some rocky debris in a semi-viscous, highly plastic state. After initial failure, the mass may flow or creep seasonally in response to changes in groundwater level. These types of slope failures often include complexes of nested rotational slides and deeply incised gullies. Boundaries are usually indistinct. indicates a scarp; arrow indicates direction of movement. Queried where the presence of the slide is uncertain. Boundary is solid where historically active, dashed where dormant, queried where uncertain.

DEBRIS SLIDE: Mass of unconsolidated rock, colluvium, and coarse-grained soil that has moved slowly to rapidly downslope along a relatively steep, shallow, translational failure plane. Debris slides form steep, unvegetated scars in the head region and possibly irregular, hummocky deposits in the toe region. Scars commonly ravel and remain unvegetated for several seasons depending on slope aspect. Queried where the presence of the slide is uncertain. Boundary is solid where historically active, queried where uncertain.

DEBRIS FLOW / TORRENT TRACK: Long stretches of bare ground that have been scoured and eroded to bedrock by extremely rapid movement of water-laden debris. Debris flows are commonly triggered by debris sliding in the source area during high intensity rains. Debris is often deposited downslope as a tangled mass of organic material in a matrix of rock and soil; debris may be reworked and incorporated into subsequent events; lack of vegetation indicates recent activity. Queried where the presence of the slide is uncertain. Boundary is solid where historically active, dashed where dormant, queried where uncertain.

SMALL LANDSLIDE: Landslide too small to delineate at 1:24,000 scale (typically less than 1/5 acre in area or less than 150 feet in length).

DISRUPTED GROUND: Irregular ground surface caused by complex landsliding processes resulting in features that are indistinguishable or too small to delineate individually at 1:24,000 scale; also may include areas affected by downslope creep, expansive soils, and/or gully erosion. Boundaries are usually indistinct.

DEBRIS SLIDE SLOPE / SOURCE AREA: A geomorphic feature characterized by steep, usually well vegetated slopes that appear to have been sculpted by numerous debris slides and debris flows. Upper reaches (source areas) of these slopes are often tightly concave and very steep. Soil and colluvium atop bedrock may be disrupted by active debris slides and debris flows. Slopes near the angle of repose may be relatively stable except where weak bedding planes, bedrock joints and fractures parallel the slope.

INNER GORGE: A geomorphic feature consisting of steep slopes adjacent to channels. The gorge typically is created by accelerated downcutting in response to regional uplift. It is defined as an area of streambank between the channel and the first break in slope. Line is queried where uncertain, or broken into segments to represent a stretch of discontinuous inner gorge too small to accurately represent at 1:24,000 scale. One-sided hachures indicate inner gorge on one side of channel only; hachures point downslope.

GULLY: Distinct, narrow channel formed by erosion of soil or soft rock material by running water. Channels are larger and deeper than rills and usually carry water only during and immediately after heavy rain or following the melting of ice or snow. Arrows point downhill; line is queried where uncertain.

GEOLOGICAL NOTES

1) The landslides and geomorphic features were mapped from 1984 WAC aerial photographs, nominal scale 1:31,680, and 2000 WAC aerial photographs, nominal scale 1:24,000. Field verification of landslide and geomorphic features was very limited and mapping relied primarily on interpretation of aerial photographs.

2) The geology depicted on this map was modified from 1:100,000-scale source data (McLaughlin and others, 2000). Although the geology has been presented on this map at a scale of 1:24,000, the detail and accuracy of the bedrock and structural data are limited to the spatial resolution of the 1:100,000 scale in which the digital database was originally compiled.

3) Please see geologic report for full lithologic descriptions, geologic setting, methodologies and limitations.

4) Landslides shown on this map have been divided into groups based on the clarity of their morphology and inferred type of movement. The landslides are also classified according to the certainty of their existence as determined by analysis of aerial photographs. The various landslide designations are not intended to, nor should they be interpreted to imply, the relative stability of slopes involved. Please see Plate 2 for relative landslide potential of the study area.

5) The scale of this map limits the delineation of some features, and the map should not be substituted for site-specific studies.

6) Information on this map is not sufficient to serve as a substitute for the geologic and geotechnical site investigations required under Chapters 7.5 and 7.8 of Division 2 of the California Public Resources Code.

7) Historical mapping by CGS (Spittler, 1983 and 1984; DMG, 1999) was considered and incorporated using current interpretive protocols for identifying and classifying geomorphic features and/or landslides. Historical mapping added directly to the Mattole River Watershed database is referenced in the electronic database with a citation to the North Coast Watersheds Mapping, digital compilation DMG CD 99-002 (DMG, 1999).

8) All small landslides (depicted on the map as points) inferred from review of the 1984 and 2000 aerial photograph sets and those mapped on CGS Open-File Reports (Spittler, 1983 and 1984) are shown on the map.

9) Digital data shown on this map as well as additional landslide and fluvial geomorphology data are available from the following sources: on the CGS website at www.conservation.ca.gov/cgs, on compact disc from CGS (CD-ROM 2002-09), or on the North Coast Watershed Assessment Program website at www.ncwatershed.ca.gov.

The northeast two-thirds of the parcel is situated on the marine platform (upper surface), while the southwest third is an exposed wave-cut bedrock platform (lower surface).

A 6- to 8-foot-tall coastal bluff with slopes of about 60 to 70 percent and as much as 90 percent, marks the transition from the level terrace to the bedrock platform. The bluff is mildly arcuate and sharp-crested and is vegetated with ice plant that extends to the base of the slope. The bedrock platform extends approximately 160 feet to the southwest from the base of the terrace to the Pacific Ocean. The platform in the vicinity of the subject property is roughly 350 long, measured along the coastline, and between 150 and 160 feet wide as measured from the base of the terrace to the Pacific Ocean. The platform has surface elevations that range from 10 to 35 feet above mean sea level.

A broad, shallow storm surge path is in the southern corner of the parcel (Figure 2). A 5-foot-diameter cypress tree is along the southeastern property boundary.

Subsurface Conditions

The subsurface conditions of the upper surface at the location of the proposed residence, were explored using a hand auger. The borings (HB-1, HB-2, and HB-3; Figure 2) were advanced to depths ranging from 3 feet to 7.25 feet below ground surface (BGS) and terminated due to auger refusal on coarse gravels. Soils at the site consist of dark brown and yellowish-brown sandy silt (ML) and silty sand (SM) topsoil containing 5 to 10 percent coarse subrounded gravel. Sandy elastic silt (MH) with approximately 35 percent fine to coarse subrounded sand is present below the topsoil and was encountered at a depth of 5 feet BGS in HB-2. We interpret the sandy elastic silt (MH) to represent the late-Pleistocene marine terrace deposit. The bedrock below the site is sandstone and basalt of the Point Delgada formation. The soil profile described above was also observed in the bluff face sitting directly on the bedrock platform, and the bluff face, between 6 and 8 feet tall, represents the thickness of the topsoil and terrace deposits overlying the bedrock. We expect this relationship to extend beneath the building site.

Groundwater was not observed in any of our borings to the maximum depth explored of 7.25 feet BGS. Soil mottling was observed in the sandy elastic silt at a depth of 5 feet. Mottling within the soil profile represents the historical range of groundwater fluctuation. Groundwater likely perches on bedrock and flower lateral toward the bluff face. Groundwater levels are expected to fluctuate seasonally, on the order of several feet in elevation. However, we do not anticipate groundwater to be encountered during excavation of shallow foundations or site grading, assuming this work is conducted during the dry season.

Geologic Hazards

Potential geologic/geotechnical hazards that were assessed for the site include seismic ground shaking, surface fault rupture, seismically induced deformation (liquefaction and seismic settlement), tsunami hazard, sea-level rise, storm surge inundation, and slope stability. Other geologic hazards are not considered relevant at this site. The assessment of the potential geologic hazards is presented in this section.

Seismic Ground Shaking

The entire North Coast region is a seismically active area where strong seismic shaking presents a significant hazard. That hazard is present at the project site but is no greater than that present elsewhere in the region. The site is approximately 4,400 feet northwest of the zoned active trace of the San Andreas fault, in Shelter Cove (CGS, 2018). Historic surface ruptures were observed on this trace of the fault in the vicinity of Shelter Cove following the great San Francisco earthquake of 1906. The Little Salmon fault, also considered active by the state, is located approximately 37 miles to the north (CGS, 2018). Additionally, the Cascadia subduction zone is located roughly 36 miles to the northwest, offshore. Based on the proximity to these active faults, the site is anticipated to experience strong seismic ground shaking during an earthquake event.

Surface Fault Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2018). As mentioned above, the site is located within 4,400 feet of the San Andreas fault. The site is also west of the Kings Range Thrust fault (2 miles east) and the Whale Gulch fault (2.6 miles east). These faults are not considered active by the State of California. As no active fault is present in the immediate project vicinity, it is our opinion that the potential for surface rupture is low.

Liquefaction

Liquefaction is the transformation of a saturated granular material from a solid to a liquefied state as a result of increased pore pressure and decreased effective stress. This sudden loss of soil shear strength is caused by cyclic loading from a seismic event. Post-earthquake studies indicate that liquefaction occurrence is not random. Rather, it occurs in predictable locations where specific soil conditions exist. Generally, in order for liquefaction to occur, the following soil conditions are needed:

- Non-plastic granular soils (sand, silty sand, sandy silt, silt, and some gravels)
- Saturation (specifically, below the water table)
- Low relative density (standard penetration test [SPT] soil density/blow counts [$(N_1)_{60}$] less than 30, usually associated with materials of young geologic age)

The adverse effects of liquefaction include localized ground settlement, ground cracking and expulsion of water and sand (sand boils), the partial or complete loss of bearing and confining forces used to support loads, and lateral spreading.

Susceptibility to liquefaction decreases with increasing geologic age, due to the effects of weathering, and the degree of densification, compaction, and/or cementation. Based on the published results of geotechnical testing and post-earthquake studies, the susceptibility of sediments to liquefaction can be directly correlated to the type, origin, and age of the deposits. Geologic materials most susceptible to liquefaction are geologically recent (that is, late Holocene age) sand- and silt-rich deposits, located adjacent to streams, rivers, bays, or ocean shorelines. Youd and Hoose (1978) estimated liquefaction susceptibility of Pleistocene-age terraces as “low.”

As the subject site is located on a late Pleistocene marine terrace underlain by shallow bedrock, the potential for liquefaction is considered low.

Tsunami Hazard

The elevation of the subject property is between 30 and 40 feet above mean sea level (MSL), rising to an elevation of approximately 43 feet at Sea Court. According to the California Geological Survey (CGS) Humboldt County Tsunami Hazard Area Maps (CGS and OES, 2021), the subject property is subject to tsunami inundation (Figure 4). The inundation hazard boundary is drawn to include Lower Pacific Drive and the residential lots across the street from the subject site. The site may be subjected to tsunami inundation under extreme conditions.

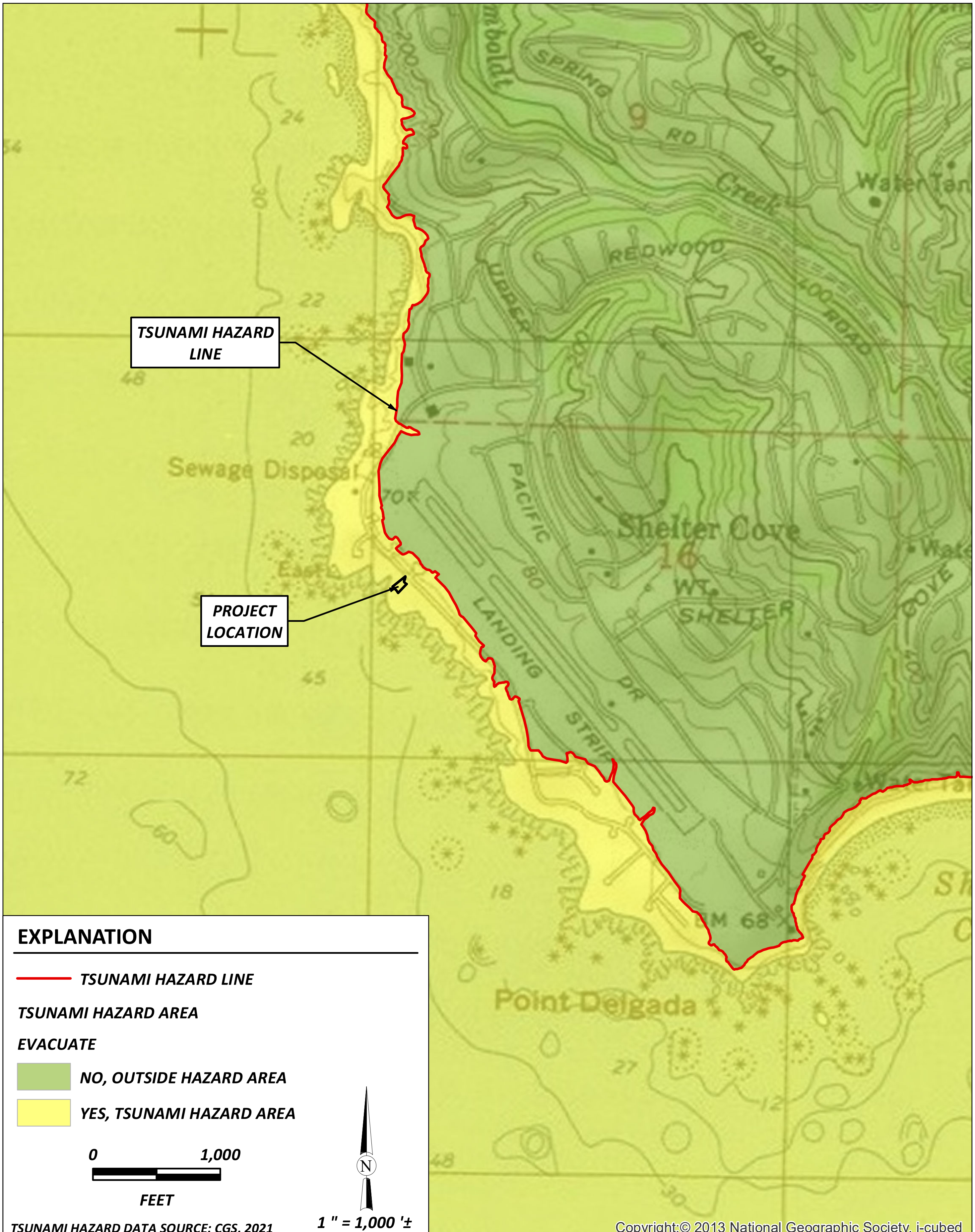
Sea Level Rise

There will be no immediate impact associated with sea level rise (SLR) in the next few decades (the economic lifespan of the proposed structure); elevation of the site is greater than 30 feet. However, sea level rise of several feet over coming decades would increase the hazard for both tsunami and storm wave inundation.

Storm Waves

On January 16, 2019, large storm waves occurred during a high tide and resulted in inundation of some of the ocean front lots on Lower Pacific Drive (southeast of the subject site) causing damage to the seaward side of at least two existing residences. Damages were restricted to structures located southeast of the subject site that are constructed on the bluff edge, at slightly lower elevations. The inundation had nominal impact on the bluff at the site and did not result in bluff retreat or erosion of the bluff face. Storm wave impact on the subject lot is effectively reduced by the presence of the broad, inclined bedrock platform along the coastline that extends into the surf zone and dissipates wave energy.

We expect storm wave hazard to increase at the site when large waves occur coincident with high tides. The exposure to this hazard will increase in the event of substantial sea level rise.



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SHN	Don Daniels Soils Engineering Report 495 Sea Court, Shelter Cove, California	Tsunami Hazard Map SHN 022043
June 2022	Figure4_TsunamiMap	Figure 4

Slope Stability

The subject property is located on a nearly-level marine terrace surface and wave-cut bedrock platform. A 6- to 8-foot tall, vegetated bluff marks the edge of the marine terrace before transitioning to the bedrock platform, below. Slopes on the bluff range from 60 to 70 percent and as much as 90 percent. The wave-cut platform extends roughly 160 feet from the base of the bluff to the ocean with elevations ranging from 10 to 35 feet above mean sea level. The platform is comprised of erosion-resistant basalt and sandstone bedrock, which extends along the coastline to the south for an additional 300 feet. The overall geometry of the bluff is planar to mildly arcuate and becomes more arcuate in the southern portion of the parcel, directly above the storm surge path as shown on Figure 2. The surge path is roughly 1 to 2 feet lower in elevation than the surrounding bedrock and is between 15 to 20 feet wide. This area provides a pathway for water to concentrate and increase velocity, which subsequently increases erosive power to the base of the bluff. The steepest slope on the bluff lies directly above this surge path with a gradient of up to 90 percent. The entire bedrock platform likely becomes inundated during rare events such as extremely high tides combined with storm swells (as described above) or the rare tsunami.

Due to the erosion-resistant nature of the bedrock platform that provides a buffer of roughly 160 feet from the base of the bluff to the ocean, the bluff retreat process at this location appears to be occurring at a relatively low rate.

Presently, the most active coastal erosion process involves minor sloughing and surface erosion of the bluff face in the terrace sediments. These processes have resulted in effective thinning of the overlying marine terrace sediments nearest the ocean. Over the past approximately 50 years, less than 5 feet of bluff retreat appears to have resulted from these processes in the project vicinity. Although the bluff has demonstrated limited retreat since the 1947 air photo dataset, we understand the terrace sands and silts along the coastal bluff edge may be subject to erosion and failure during very strong, long duration earthquakes and/or by wave action generated during extreme storm events.

Seismic Design Parameters

Based on the subsurface conditions encountered at our exploration locations, laboratory test results, and our interpretation of soil conditions within 100 feet of the ground surface, we classify the site as a Site Class C consisting of a “Very Dense Soil Soft Rock” in accordance with Chapter 20 of ASCE 7-16 (ASCE, 2017). On this basis, the mapped and design spectral response accelerations were determined using the Structural Engineers Association of California (SEAOC, 2019) and California Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps (Accessed 5/4/22) in conjunction with the site class and the site coordinates for the project site (40.0296923° N, -124.0786187° W). Calculated values for ASCE 7-16 are presented in Table 1.

**Table 1. ASCE 7-16 Spectral Acceleration Parameters
Shelter Cove, Humboldt County, California**

Parameter	0.2 Second	1 Second
Maximum Considered Earthquake Spectral Acceleration (MCE_R)	S_S = 2.309	S_1 = 0.965
Site Class	C	
Site amplification factor	F_a = 1.2	F_v = 1.4
Site-modified spectral acceleration	S_{MS} = 2.77	S_{M1} = 1.35
Numeric seismic design value	S_{DS} = 1.847	S_{D1} = 0.9
Seismic Design Category (SDC)	E	
MCE_G peak ground acceleration (PGA)	0.987	
Site amplification factor at PGA (F_{PGA})	1.2	

Discussion and Conclusions

Based on the results of our geotechnical investigation, we conclude the residential site can be developed as planned for the proposed construction, provided the geotechnical recommendations contained herein are incorporated into the project design. The main geotechnical engineering consideration affecting design and construction of the project are the upper 2 feet of soft surficial soils.

The site is likely to experience strong seismic ground shaking resulting from earthquakes on active faults in the region during the design life of the proposed structure. The intensity of ground shaking from earthquakes will depend on several factors, including the distance from the site to the earthquake focus, the magnitude and duration of the earthquake, and the response of the underlying soil and bedrock. At a minimum, it will be necessary to design and construct the proposed residential structure in accordance with the earthquake-resistant provisions of the governing code.

Recommendations

Coastal Bluff Setback

We recognize the presence of the coastal bluff in the southwestern portion of the subject property and the associated hazards. Upon review of aerial photos dating back to 1947, it appears there has been no discernable change in the location or configuration of the bedrock platform or coastal bluff at the project site. The approximately 160 feet of erosion-resistant bedrock extending seaward from the base of the coastal bluff appears to be the main controlling factor in the low bluff retreat rates. We take retreat of the bluff to be a result of erosion during the most extreme erosive events (a tsunami for example); therefore, development should occur on the terrace surface, inland of the coastal bluff and definitely not on the wave-cut platform. Due to the lack of observed bluff retreat since 1947, we recommend that placement of the load-bearing foundation elements for the proposed structure be placed no less than 5 feet back from the top of bluff.

A setback of 5 feet from the break in slope, which we recommended to mitigate the possibility of slope failure, may not adequately address the hazards associated with storm surf or tsunamis. In order to mitigate the inundation hazard, increase the setback from the bluff or incorporate appropriate design accommodations to reinforce the seaward side of the structure. Additionally, we recommend maintaining vegetation along the bluff edge and face to aid in erosion prevention and not allowing concentrated drainage flow over the edge.

Site Preparation and Grading

Areas to be developed should be cleared of trees (except those noted to remain), debris, organics (grasses/shrubs and root systems), organic topsoil, loose soil, and any other unsuitable material. Where the removal of large trees is required, it will be necessary to remove all major root systems, then backfill the excavations with properly placed fill compacted to at least 90 percent relative compaction[1]. Site preparation operations should extend at least 5 feet beyond the footprint of the planned structure foundations. The cleared vegetation and debris should be removed from the site, but the strippings can be stockpiled for reuse in landscape areas. Any vegetation and organic topsoil with more than 2 percent organic material by dry weight should be removed. The Geotechnical Engineer should observe and approve the prepared site prior to any excavation, subgrade preparation, and placement of fill or improvements.

We expect that the site soils will be excavatable with conventional grading and trenching equipment. If grading commences in the winter or spring, or after a period of excessive rainfall, it is likely that the surficial soils will become saturated due to the presence of fine-grained material. Wet or saturated soil may cause difficulties in access with grading and trenching equipment and difficulties in loading, spreading, and compaction of fill material. Moisture conditioning and/or aerating of the site soils may be required. The time required for drying can be reduced by disking, ripping, or otherwise aerating the soil. To avoid supporting the new garage slab-on-grade, if used, on materials with variable support characteristics, we recommend that over-excavation be performed across the entire footprint of the new structure to a minimum depth of 24 inches below the existing ground surface. The limits of over-excavation should extend at least 3 feet horizontally from the edges of planned slab. Additional over-excavation may be required to remove soft, wet, yielding, or otherwise unsuitable material. The depth and extent of any additional over-excavation should be approved in the field by the Geotechnical Engineer, or their qualified representative, prior to placement of engineered fill.

The exposed over-excavation subgrade should be level and should be scarified to a minimum depth of 6 inches, moisture conditioned or aerated, and compacted to at least 90 percent relative compaction. The approved surface may then be brought to pad grade with placement of properly compacted engineered fill compacted to a minimum of 90 percent relative compaction.

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1 Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557 compaction test procedure. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.

The Contractor shall be responsible for the stability of all temporary excavations and should comply with applicable Occupational Safety and Health Administration (OSHA) regulations (California Construction Safety Orders, Title 8). The Contractor should periodically monitor all open cuts for evidence of incipient stability failures.

New Engineered Fills

Fill placed in areas to support proposed slabs-on-grade should meet the requirements for select engineered fill. Engineered fill should have less than 2 percent by dry weight of vegetation and deleterious material and should meet the gradation requirements presented in Table 2.

Table 2. Fill Gradation Criteria

Sieve Designation	Percent Passing by Dry Weight
3-inch (50 mm)[a]	100
1½-inch (37.5 mm)	90 minimum
¾-inch (19 mm)	70 minimum
No. 4 (4.75 mm)	60 minimum
No. 200 (75 μm)[b]	5 minimum, 30 maximum

a. mm: millimeters
b. μm: micrometers

Fine-grained soil with a liquid limit greater than 40 and a plasticity index greater than 15 should not be used as engineered fill. If clayey soils do not meet the plasticity requirements, mixing of the clayey soils with sandier soils may be required. Crushing and/or removal of rock particles greater than 3 inches in size will be required. In addition, we do not recommend using river-run material as engineered fill; crushed, angular material is preferred.

Engineered fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to a minimum of 90 percent relative compaction. The geotechnical engineer should approve all fill prior to placement.

A qualified field technician should be present to observe fill placement and perform field density tests in accordance with ASTM D 6938 at random locations throughout each lift to verify the specified compaction is being achieved. Less frequent testing may be acceptable for agricultural facilities in remote locations; however, the owner should be aware that reduced levels of testing are associated with increased levels of risk.

Surface Drainage Control

Surface drainage should be planned to prevent ponding and enable water to drain away from foundations, slabs-on-grade, edges of pavements and tops of slopes, and towards suitable collection or discharge facilities. A positive surface drainage of at least 4 percent is recommended within 10 feet of all building foundations in unpaved areas. In paved areas, a positive surface drainage of at least 2 percent is recommended to allow for rapid removal of surface water. Roof drainage systems should be planned to direct rainwater away from building foundations.

Concentrated water should not be discharged onto bare ground or slopes but should be carried in pipes or lined channels to suitable disposal points. Because onsite soils generally have a moderate potential for erosion, we recommend that approved temporary and permanent erosion control measures be implemented to limit erosion and comply with applicable County of Humboldt regulations.

The use of water-intensive landscaping around the perimeter of structures should be avoided to reduce the amount of water introduced to the subgrade. Irrigation of landscaping around structures should be limited to drip or bubbler-type systems. Trees with large roots should also be avoided since they can dry out the soil beneath foundations and cause settlement. The purpose of these recommendations is to avoid large differential moisture changes adjacent to foundations, which have been known to cause large differential movement over short horizontal distances in expansive soils, resulting in cracking of slabs and architectural damage.

Utility Trench Backfill

Unless concrete bedding is required around utilities, bedding should consist of sand having a sand equivalent (SE) of at least 30. The bedding should extend from 6 inches below to 1 foot above the conduit or pipe. Sand bedding should not be jetted or ponded into place and should be mechanically compacted to a minimum of 90 percent relative compaction.

In areas to support improvements (such as adjacent-to-structure slabs-on-grade), backfill placed above the bedding in utility trenches (including culvert and sprinkler lines) should be properly placed and adequately compacted to minimize settlement and provide a stable subgrade. If possible, the trench backfill should be compacted following rough grading but prior to final grading and compaction. Onsite inorganic soils meeting the requirements for engineered fill may be used as trench backfill. Backfill consisting of onsite soils should be placed in layers not exceeding 8 inches in loose thickness, moisture-conditioned, and compacted to at least 90 percent relative compaction as described for engineered fill. Trench backfill need only be compacted to 85 percent relative compaction in landscape areas.

Drilled Pier Foundations

We understand the intent is to utilize a drilled pier foundation to support the main residential structure. As such, we recommend that the proposed residential structure be supported on drilled, reinforced, cast-in-place, concrete friction pier and grade beam foundations gaining support by end bearing in the underlying bedrock. Drilled piers should be at least 18 inches in diameter and be bottomed a minimum of 8 feet deep and socketed a minimum of 2 feet into the underling bedrock, whichever is deeper. Spacing of the piers should be determined, as required, by the load distribution, but minimum spacing should not be less than 3 pier diameters, center to center.

The piers should be designed using an allowable bearing capacity of 6,000 pounds per square foot (psf) for dead plus long-term live loads (10.6 kips for 18-inch-diameter end bearing pier). This value may be increased by one-third for total loads that include transient wind or seismic forces.

To resist lateral loads, the passive resistance of the soil can be used. A soil passive pressure equivalent to a fluid weighing 350 pounds per cubic foot (pcf) can be used below the upper 2 feet of the pier. Passive pressure can be assumed to act against the lateral projected area of the pier described by the vertical dimension times twice the pier diameter.

Successful completion of drilled pier foundations requires good construction procedures. Drilled pier excavations should be constructed by a skilled operator using techniques that allow end bearing by specifying equipment that will remove all slough from drilled holes, the reinforcing steel to be placed, and the concrete poured in a continuous manner to reduce the time that excavations remain open (particularly near the contact with the overlying soils).

If temporary casing is used, we recommend its removal from the hole as concrete is being placed. The bottom of the casing should be maintained below the top of the concrete during casing withdrawal and concrete placement. The casing should not be withdrawn until sufficient quantities of concrete have been placed into the excavation to balance the groundwater head outside the casing, if groundwater is encountered.

Grade beams should be incorporated between piers as required by the structural engineer. Perimeter grade beams for the residential structure, should extend at least 6 inches below bottom of slab subgrade to help reduce the potential for infiltration of surface runoff under the at-grade portions of these structures. Pier and grade beam reinforcing should be determined by the project structural engineer based on the preceding design criteria and structural requirements.

Garage Slabs-on-grade

Concrete garage slabs-on-grade should be supported by engineered fill prepared in accordance with our recommendations for earthwork.

To reduce water vapor transmission upward through floor slabs, concrete slabs-on-grade should be constructed on a minimum 4-inch-thick layer of capillary break material covered with a vapor retarder. The capillary break material should be free-draining, clean gravel or rock, such as, No. 4 by ¾-inch pea gravel or permeable aggregate complying with Caltrans Standard Specification, Section 68, Class 1, Type B Permeable Material. The vapor retarder should be at least 10 mil in thickness and meet the material requirements for Class C vapor retarders presented in ASTM E1745, and should be installed according to ASTM E1643. These installation requirements include overlapping seams by 6 inches, taping seams, and sealing penetrations in the vapor retarder.

Moisture vapor transmission is a specialty field, and we suggest that qualified experts be contacted to assist in the design and construction of measures related to moisture transmission through slabs-on-grade.

The American Concrete Institute (ACI) Committee document "Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials" (ACI 302.2R-06) provides guidelines for reducing moisture migration through slabs-on-grade. This document advises that concrete slabs be cast directly on the vapor retarder (ACI 302.2R-06, Section 9.3) and provides guidelines for selecting vapor permeance, tensile strength, and puncture resistance. When casting the slab directly on the vapor retarder, a reduced joint spacing, low shrinkage mix design, or other appropriate measures should be used to control slab curl. The ACI guide also notes that a maximum water-cement ratio of 0.5 has yielded satisfactory performance on many slab-on-grade projects. Water-reducing admixtures may be useful in achieving workability at low water-cement ratios. Control joints should be provided at appropriate intervals to control the location of shrinkage cracks. After proper curing, the slab should be allowed to dry and then should be tested to check that the moisture transmission rate is appropriate for the intended floor covering.

For exterior flatwork and other slabs-on-grade where water vapor transmission through slabs is not a concern, the vapor barrier and capillary break material described in this section may be omitted. However, a minimum of 4 inches of Class 2 Aggregate Base rock, compacted to a minimum of 90 percent relative compaction, should be provided beneath exterior flatwork and other slabs-on-grade where vapor transmission is not a concern.

It is important that the subgrade be moist and free of desiccation cracks at the time the slab is cast. Recommendations for slab reinforcement, strength, thickness, control and construction joints, and so on, should be provided by others. Although cracks in concrete slabs are common and should be expected, the following measures may help to reduce cracking of slabs.

Slabs should be cast using concrete with a maximum slump of 4 inches or less. Add a water-reducing agent or plasticizer to the concrete to increase slump while maintaining a low water-cement ratio to reduce concrete shrinkage. (Concrete having a high water-cement ratio is a major cause of concrete cracking.) Control joints should be provided at appropriate intervals to control the location of shrinkage cracks.

Closure

The analyses, conclusions, and recommendations contained in this report are based on site conditions that we observed at the time of our investigation, data from our subsurface explorations, our current understanding of proposed project elements, and on our experience with similar projects in similar geotechnical environments. We have assumed the information obtained from our limited subsurface explorations is representative of subsurface conditions throughout the areas of proposed development addressed in this report.

We have assumed, in preparing our recommendations, that SHN will be retained to review those portions of the plans and specifications that pertain to earthwork and foundations. The purpose of this review is to confirm that our earthwork and foundation recommendations have been properly interpreted and implemented during design. If we are not provided this opportunity for review of the plans and specifications, our recommendations could be misinterpreted.

We recommend a representative of our firm confirm site conditions during the construction phase. If subsurface conditions differ significantly from those disclosed by our investigation, we should be given the opportunity to re-evaluate the applicability of our conclusions and recommendations. Some alteration of recommendations may be appropriate. If the scope of the proposed construction, including the proposed structural location, changes from that described in this report, our recommendations should also be reviewed.

If there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we should review our report to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. This report is applicable only to the project and site studied.

The conclusions and recommendations presented in this report are professional opinions derived in accordance with current standards of professional practice. Our recommendations are based on the assumption that design of the improvements will conform to their intent. No warranty is expressed or implied.

Please call us at 707-441-8855 if you have any questions.

Sincerely,

SHN

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Appendices: 1. Hand-Auger Boring Logs
2. Laboratory Results

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Hand-Auger Boring Logs

1

KEY TO SYMBOLS

CLIENT Don Daniels
PROJECT NAME Daniels Geotechnical Investigation
PROJECT NUMBER 022043
PROJECT LOCATION 495 Sea Court, Shelter Cove, California

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

MH: USCS Elastic Silt

MLS: USCS Sandy Silt

SM: USCS Silty Sand

SAMPLER SYMBOLS

Auger Cuttings

Undisturbed Sample

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
▽ Water Level at Time Drilling, or as Shown
▼ Water Level at End of Drilling, or as Shown
▼ Water Level After 24 Hours, or as Shown

BORING NUMBER HB-1

CLIENT Don Daniels
PROJECT NAME Daniels Geotechnical Investigation
PROJECT NUMBER 022043
PROJECT LOCATION 495 Sea Court, Shelter Cove, California
DATE STARTED 3/17/22 **COMPLETED** 3/17/22
GROUND ELEVATION **HOLE SIZE** 4 inches
DRILLING CONTRACTOR SHN
GROUNDWATER DEPTH
DRILLING METHOD Hand Auger
AT TIME OF DRILLING ---
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
NOTES Boring terminated at 3' due to auger refusal on coarse gravel.

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		SILTY SAND (SM), moist, dark brown, fine to coarse rounded sand, 5-10% gravel, non-plastic fines, low dry strength some fine to medium roots at 0.75', intermixed with yellowish-brown silty sand, bioturbated (TOPSOIL).										
			AU									46
2.5		Hand-drive tube sample at 2'. Few medium to coarse roots.	UD		6	>4.5	82	26				

Refusal at 3.0 feet.
Bottom of borehole at 3.0 feet.

BORING NUMBER HB-2

CLIENT Don Daniels
PROJECT NAME Daniels Geotechnical Investigation
PROJECT NUMBER 022043
PROJECT LOCATION 495 Sea Court, Shelter Cove, California
DATE STARTED 3/17/22 **COMPLETED** 3/17/22
GROUND ELEVATION **HOLE SIZE** 4 inches
DRILLING CONTRACTOR SHN
GROUNDWATER DEPTH
DRILLING METHOD Hand Auger
∇ **AT TIME OF DRILLING** ---
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
NOTES Boring terminated at 7.25' due to auger refusal.

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		SANDY SILT (ML), moist, yellowish-brown and dark brown, fine to coarse rounded sand, 5-10% coarse subrounded gravel, non-cemented, non-plastic (TOPSOIL).										
			AU									54
		Hand-drive tube sample at 2'.	UD		9	>4.5	82	27				
2.5												
		Few coarse roots at 3'.										
			AU									
		Hand-drive tube sample at 4'.	UD		7	>4.5	87	29				
5.0		SANDY ELASTIC SILT (MH), moist, dark yellowish-brown, fine to coarse subrounded sand (~35%), medium plasticity, low to medium dry strength, mottled, contains angular cemented iron-stained coarse sand-size clasts (MARINE TERRACE).	AU						52	40	12	

Refusal at 7.3 feet.
Bottom of borehole at 7.3 feet.

BORING NUMBER HB-3

CLIENT Don Daniels
PROJECT NAME Daniels Geotechnical Investigation
PROJECT NUMBER 022043
PROJECT LOCATION 495 Sea Court, Shelter Cove, California
DATE STARTED 3/17/22 **COMPLETED** 3/17/22
GROUND ELEVATION **HOLE SIZE** 4 inches
DRILLING CONTRACTOR SHN
GROUNDWATER DEPTH
DRILLING METHOD Hand Auger
▽ **AT TIME OF DRILLING** ---
LOGGED BY P. Sundberg **CHECKED BY** J. Dailey
NOTES Boring terminated at 4.5' due to auger refusal.

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		SANDY SILT (ML), moist, dark brown, fine to coarse rounded sand, low plasticity, non-cemented, 5-10% coarse subangular gravel.										
		Grades yellowish-brown and dark brown (bioturbated).										
		Grades dark brown.										
2.5			AU									56

Refusal at 4.5 feet.
Bottom of borehole at 4.5 feet.

Laboratory Results

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DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	Daniels Geo	Project Number:	022043
Performed By:	KEW	Date:	3/29/2022
Checked By:	ALG	Date:	4/8/2022
Project Manager:	PRS		

Lab Sample Number	22-329	22-331	22-333		
Boring Label	HB1	HB2	HB2		
Sample Depth (ft)	2-2.5	2-2.5	4-4.5'		
Diameter of Cylinder, in	2.38	2.38	2.38		
Total Length of Cylinder, in.	7.43	7.43	7.43		
Length of Empty Cylinder A, in.	0.00	0.00	0.00		
Length of Empty Cylinder B, in.	2.50	1.50	2.62		
Length of Cylinder Filled, in	4.93	5.93	4.81		
Volume of Sample, in^3	21.93	26.38	21.40		
Volume of Sample, cc.	359.41	432.31	350.66		

Pan #	ss2	ss6	ss9		
Weight of Wet Soil and Pan	783.1	918.2	822.2		
Weight of Dry Soil and Pan	662.6	765.0	682.1		
Weight of Water	120.5	153.2	140.1		
Weight of Pan	193.5	196.0	196.5		
Weight of Dry Soil	469.1	569.0	485.6		
Percent Moisture	25.7	26.9	28.9		
Dry Density, g/cc	1.31	1.32	1.38		
Dry Density, lb/ft^3	81.5	82.2	86.5		

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PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	Daniels Geo	Project Number:	022043
Performed By:	KEW	Date:	3/29/2022
Checked By:	ALG	Date:	4/8/2022
Project Manager:	PRS		

Lab Sample Number	22-328	22-330	22-335		
Boring Label	HB-1	HB-2	HB-3		
Sample Depth	1.5-2	1.5-2	2.5-3		
Pan Number	ss7	ss8	ss10		
Dry Weight of Soil & Pan	408.9	444.2	419.3		
Pan Weight	193.3	193.0	195.4		
Weight of Dry Soil	215.6	251.2	223.9		
Soil Weight Retained on #200&Pan	310.0	309.3	295.0		
Soil Weight Passing #200	98.9	134.9	124.3		
Percent Passing #200	46	54	56		

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LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME:	**Daniels GEO**	JOB #:	**022043**	LAB SAMPLE #:	**22-334**
SAMPLE ID:	**HB-2, 5'-5.5'**	PERFORMED BY:	**KEW**	DATE:	3/29/2022
PROJECT MANAGER:	**PRS**	CHECKED BY:	**ALG**	DATE:	4/8/22

LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	7	8	9	10	11
B	PAN WT. (g)	15.560	15.470	15.830	15.470	15.500
C	WT. WET SOIL & PAN (g)	23.690	23.140	21.200	20.080	20.250
D	WT. DRY SOIL & PAN (g)	21.350	20.980	19.900	18.500	18.300
E	WT. WATER (C-D)	2.340	2.160	1.300	1.580	1.950
F	WT. DRY SOIL (D-B)	5.790	5.510	4.070	3.030	2.800
G	BLOW COUNT	--	--	29	23	18
H	MOISTURE CONTENT (E/F*100)	40.4	39.2	31.9	52.1	69.6

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
52	**12**	40