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November 30, 2020

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Subject: **Geotechnical Investigation and Geologic Hazard Report**
Proposed Felisky Single Family Residence
79532 Ray Brown Road
Arch Cape, Clatsop County, Oregon
EEI Report No. 20-186-1

Dear Mr. Felisky:

Earth Engineers, Inc. (EEI) is pleased to transmit our Geotechnical Investigation and Geologic Hazard Report for the above referenced project. The attached report includes the results of field and laboratory testing, an evaluation of geologic hazards that may influence the proposed development, recommendations for foundation design, as well as recommendations for general site development.

We appreciate the opportunity to perform this geotechnical study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office.

Respectfully submitted,
Earth Engineers, Inc.

Troy Hull, P.E., G.E. Principal Geotechnical Engineer	Ken Andrieu, R.G. Senior Geologist	Jacqui Boyer Geotechnical Engineering Associate

Attachment: Geotechnical Investigation and Geologic Hazard Report

Distribution (electronic copy only):
Addressee
Andrew Montgomery, Terraforma (arch.terraforma@gmail.com)

Earth Engineers, Inc.

GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARD REPORT

for the

Proposed Felisky Single Family Residence
79532 Ray Brown Road
Arch Cape, Clatsop County, Oregon

Prepared for

Marc Felisky
14311 Nevers Road
Snohomish, Washington 98290

Prepared by

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EEI Report No. 20-186-1

November 30, 2020

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1.0 PROJECT INFORMATION

1.1 Project Authorization

Earth Engineers, Inc. (EEI) has completed a Geotechnical Investigation and Geologic Hazard Report for the proposed single family residence to be located at 79532 Ray Brown Road in Arch Cape, Clatsop County, Oregon. Our services were performed in general accordance with EEI Proposal No. 20-P321 dated October 21, 2020 and authorized by signing on October 23, 2020.

1.2 Project Description

Our current understanding of the project is based on the information provided via email to EEI Principal Geotechnical Engineer Troy Hull from the project architect Andrew Montgomery with Terraforma.

We were also provided the following documents via e-mail:

- An existing topographic site plan titled "Marc & Chance Felisky, Parcel 1, Partition Plat 1992-006, NW ¼ Section 31, T4N, R10W, W.M., Clatsop County, Oregon" prepared by S&F Land Services, dated October 1, 2020.
- Sheets A1, A2 and A2.1 of the drawing set titled "The Felisky Beach Home", prepared by Terraforma, dated October 5, 2020. See Figure 1 below.
- February 7, 2013 report by Mead Engineering for the property.
- March 26, 2013 report by Mead Engineering for the property. This appears to be an updated and more comprehensive report to replace their February 7 report.

Briefly, we understand the plan is to demolish the existing 3-story home and replace it with a new two-story home in its place, as shown in Figure 1 below. The proposed footprint of the new residence is approximately 3,370 square feet. It is our understanding that the project consists of relocating the existing septic field approximately 20 feet to the south.

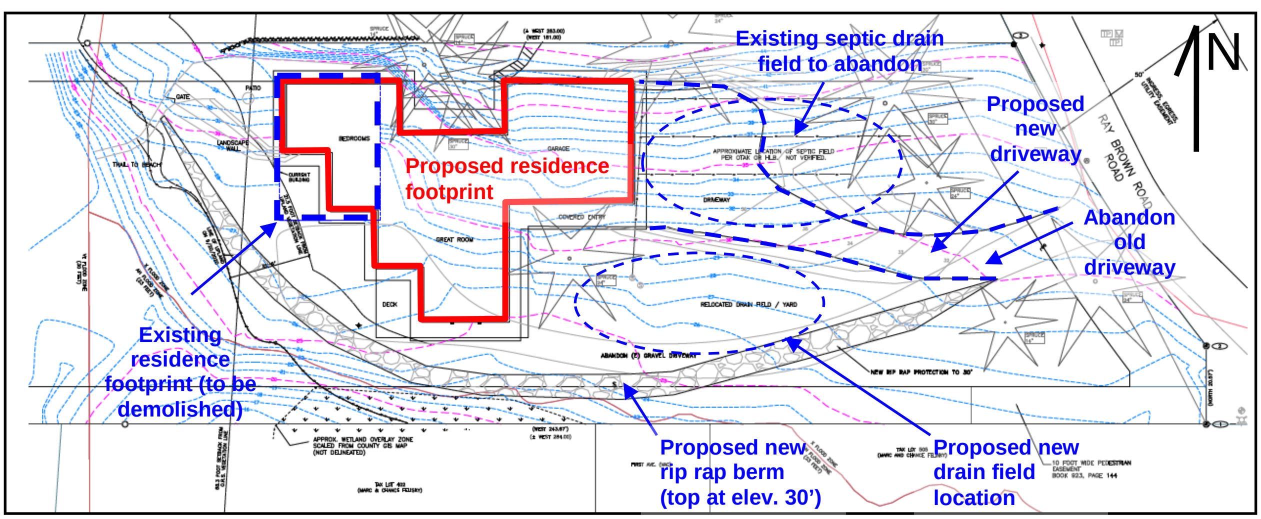


Figure 1: Site Plan provided on Sheet A1 of the drawing set referenced above.

Since the project is still in the preliminary stages, we have not been provided detailed construction drawings or foundation loading for the proposed home construction. For the purposes of this report, we are assuming typical residential foundation loads of 4 kips per linear foot for wall footings, 50 kips per column footing, and 150 psf for floor slabs.

We have been provided proposed grading information (refer to topographic information in Figure 1 above. The most notable grading issue is that it appears the downhill side of the proposed new driveway will have a retaining wall that will be about 5 to 6 feet tall to retain fill for the lower portion of the driveway. The uphill side of the driveway will have a retaining wall that will retain a cut into the slope that is up to about 7 feet tall. The only other significant grading issue is that a rip rap fill mound is planned to be installed on the west and south side of the house. The top of the fill mount is planned for elevation 30, which means the rip rap fill height will range from 0 to about 8 feet.

We have assumed that the home will be constructed in accordance with the 2017 Oregon Residential Specialty Code (ORSC) or 2019 Oregon Structural Specialty Code (OSSC), whichever is applicable.

As shown below, the site is located within the boundaries of a very large, ancient landslide as mapped by the Statewide Landslide Inventory Database of Oregon (https://gis.dogami.oregon.gov/maps/slido/)—as are most of the tens of residences in the Cove Beach/Falcon Cove subarea of Arch Cape.

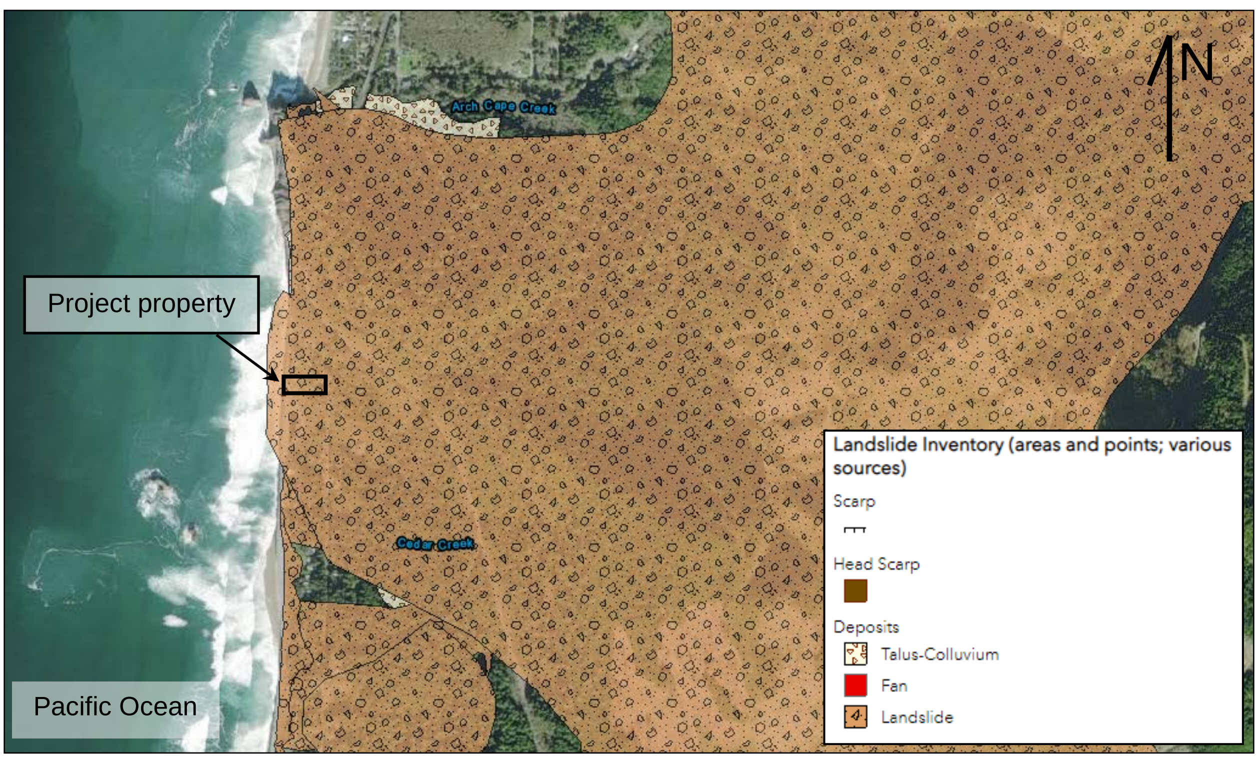


Figure 2: Property is located within a large slide mass (SLIDO 4.1).

Additionally, the Oregon HazVu: Statewide Geohazards Viewer (http://www.oregongeology.org/hazvu/) lists the site as being within a severe Cascadia earthquake expected shaking zone, a severe crustal expected earthquake shaking hazard zone, a moderate liquefaction hazard zone, a very high landslide (existing landslide) hazard area, a low to very high coastal erosion hazard zone, and fully within the statutory tsunami inundation line and partially within the effective FEMA 100 year flood zone.

As described above, the site is located in several geologically hazardous zones. Due to these hazards, Clatsop County requires a Geologic Hazard Report meeting the requirements of Clatsop County Land and Water Development and Use Ordinance (LWDUO), Section 4.040. See Section 3.0 of this report.

Additionally, the geotechnical engineering portion of this report has been prepared in general accordance with Clatsop County's LWDUO Section 4.044 ("Geotechnical Report Requirements").

1.3 Purpose and Scope of Services

The purpose of our services was to perform a geotechnical investigation of the lot in order to explore the subsurface conditions at the site to better define the soil, rock, and groundwater properties present and provide geotechnical recommendations for the proposed construction, as well as conduct a Geologic Hazard Assessment to meet the requirements of Clatsop County LWDUO Section 4.040 for properties located in geologic hazard areas.

Our site investigation consisted of advancing 2 drilled Standard Penetration Test (SPT) borings (B-1 and B-2) on the property using a T26 Beretta tracked drill rig subcontracted from PLi Systems of Hillsboro, Oregon. Boring B-1 was advanced to a depth of 9.5 feet below ground surface (bgs) located southwest of the existing residence. This boring was terminated shallower than planned due to practical drilling refusal (presumably on a floating boulder in the soil matrix). SPT soil samples from B-1 were collected at intervals of 2.5 feet to the terminal depths of the exploration. Boring B-2 was advanced to a depth of 31.5 feet bgs located south of the existing residence. SPT soil samples from B-2 were collected at intervals of 2.5 feet in the upper 15 feet and 5 foot intervals thereafter to the terminal depths of the exploration.

The soil samples were tested in the laboratory to determine the material's properties for our evaluation. Laboratory testing was accomplished in general accordance with ASTM procedures.

This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents recommendations regarding the following:

- A discussion of subsurface conditions encountered including pertinent soil and groundwater conditions.
- Slope stability assessment.
- Seismic design parameters in accordance with ASCE 7-16.
- A Geologic Hazard Assessment meeting the requirements of Clatsop County LWDUO Section 4.040.
- Geotechnical related recommendations for deep foundation design.

- Structural fill recommendations, including an evaluation of whether the in-situ soils can be used as structural fill.
- Retaining wall recommendations.
- Floor slab recommendations.
- General site earthwork recommendations, including temporary and permanent slopes, as well as site drainage.
- Other discussion on geotechnical issues that may impact the project.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 Site Location and Description

The subject property is located at 79532 Ray Brown Road, in Arch Cape, Clatsop County, Oregon. The property is approximately trapezoidal, and covers 0.75 acres. The property is bordered to the north by a single family residence at 79544 Ray Brown Road, which is owned by the Felisky Family Trust, to the east by Ray Brown Road, to the south by undeveloped property also owned by Marc and Chance Felisky, and to the west by the Pacific Ocean. The subject property is currently developed with a 3 story home (i.e. 2 stories plus a daylight basement). We have not been provided any drawings for the existing home that would indicate what type of foundation it is supported on (i.e. shallow versus deep foundation). See Figure 3 below for the project vicinity.

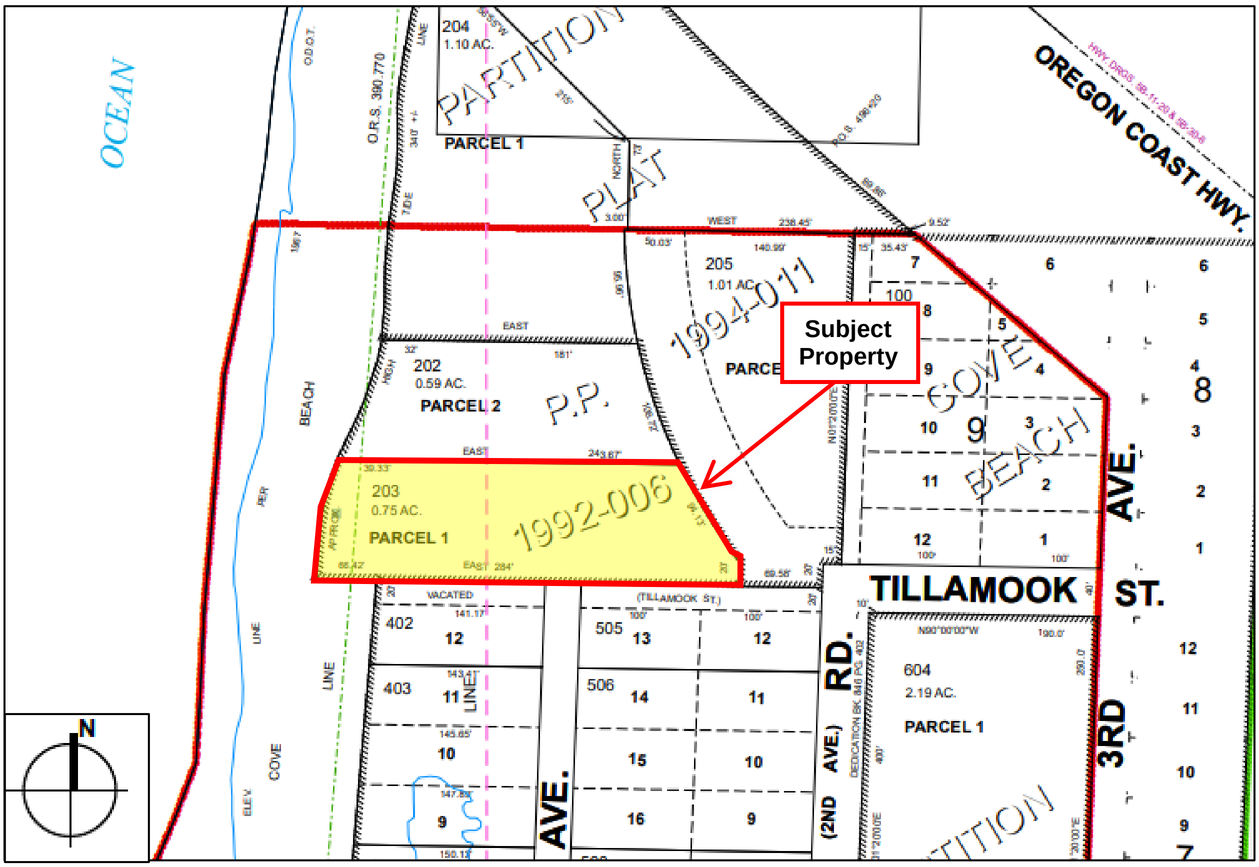


Figure 3: Project vicinity map showing the subject property (highlighted in yellow) and its surrounding area. Base map source: Clatsop County WebMaps (http://maps.co.clatsop.or.us/).

The property generally slopes down to the south from an elevation of 46 feet MSL at the northern edge of the property to an elevation of 16 feet MSL at the southwest. The surface is hummocky, as is typical in areas of landslide topography, with local slopes ranging from 10H:1V (Horizontal:Vertical) to 1.75H:1V. It should be noted that localized slopes in the area of the proposed residence are up to 2.75H:1V. The steepest slopes on the subject property are located west of the existing residence, sloping down to the west (down to the beach), where they are up to 1.75H:1V. See the topographic survey of the western portion of the lot below in Figure 4.

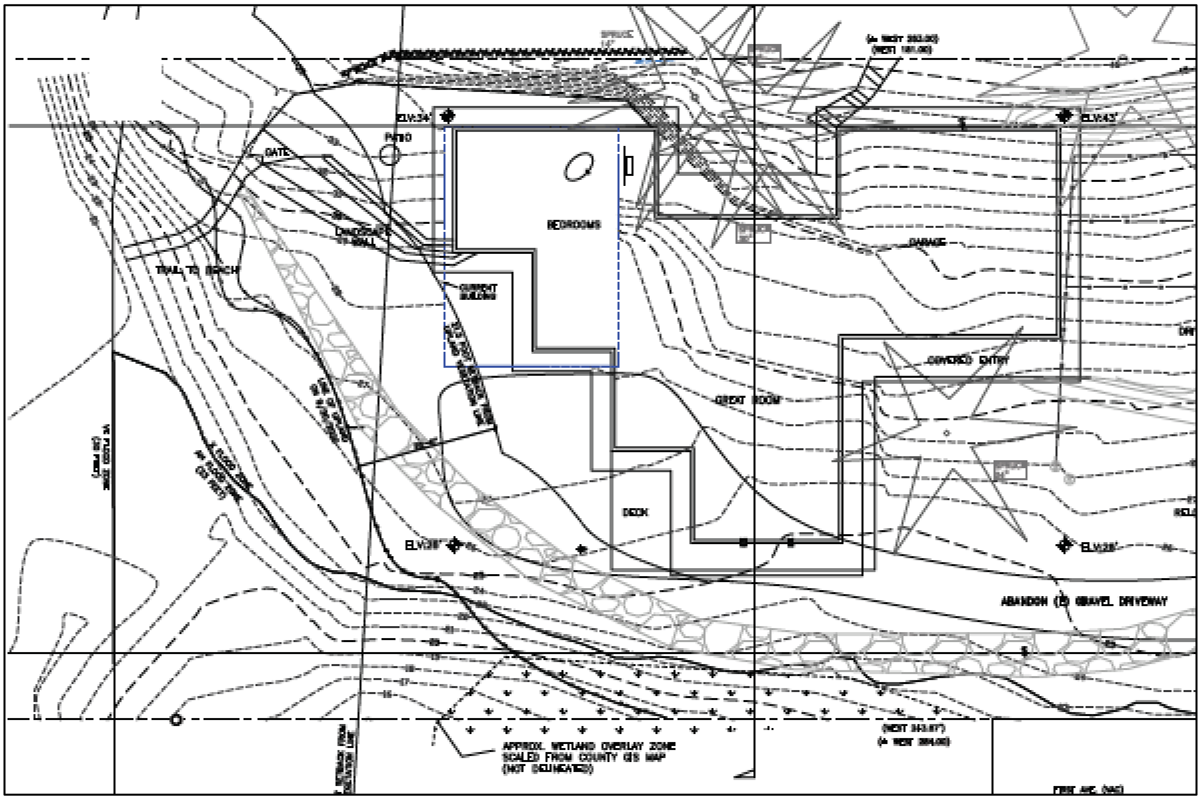


Figure 4: Topographic existing conditions showing location of proposed residence. Contours are marked on 1-foot intervals. Base drawing source: Sheet A1 referenced above.

As stated above, the subject property is currently developed with an existing residence. According to the Clatsop County website, the existing residence was constructed in 1994 and is 1,784 square feet. The two-story residence has a daylight basement and covered carport. As stated above, the existing residence is planned to be demolished to make way for a residence with a larger footprint 3,370 square feet in size. All of the additional footprint will be to the east and south (i.e. the new house will not be located any closer to the beach than the current house).

There are scattered trees on the subject property that are generally straight-trunked however some trees are slightly titled and bent. This indicated variable amounts of shallow soil creep, which is common in sloping areas with landslide deposits. See Photos 1 through 3 below.

There is an existing rockery retaining wall immediately north of the existing house. While we have not performed a detailed engineering evaluation of its stability, visually it appeared to be in good condition and stable at the time of our site investigation. It should be noted that rockery walls do require periodic maintenance over their life (i.e. restacking of boulders that shift). See Photo 4 below. Based on the layout of the new house, it appears that the eastern portion of the rockery wall will be removed. We assume the exterior wall of the new house will be designed to retain the slope where the rockery wall is removed.

Photo 1: Current site conditions showing the existing residence on the subject property and hummocky land features, facing west.

Photo 2: Current site conditions showing the existing residence on the subject property and hummocky land features, facing east.

Photo 3: Current site conditions showing a pistol butted tree on the subject property, facing south.

Photo 4: Rockery wall located north of the existing residence, facing east.

During our site reconnaissance, EEI Senior Geologist Ken Andrieu, R.G., walked along the beach immediately west of the subject property. The beach consisted of rounded basalt cobbles and exposed surfaces of landslide debris (i.e. silty sand with sandstone gravel and cobbles pieces) overlying what appeared to be intact weathered to decomposed sandstone (see Photo 5 below). Unprotected portions of the bluff display extensive signs of active erosion and mass wasting processes as evidenced by bare soil head scarps along the upper reaches of the bluff. See Figure 5 for an isometric view of the subject property.

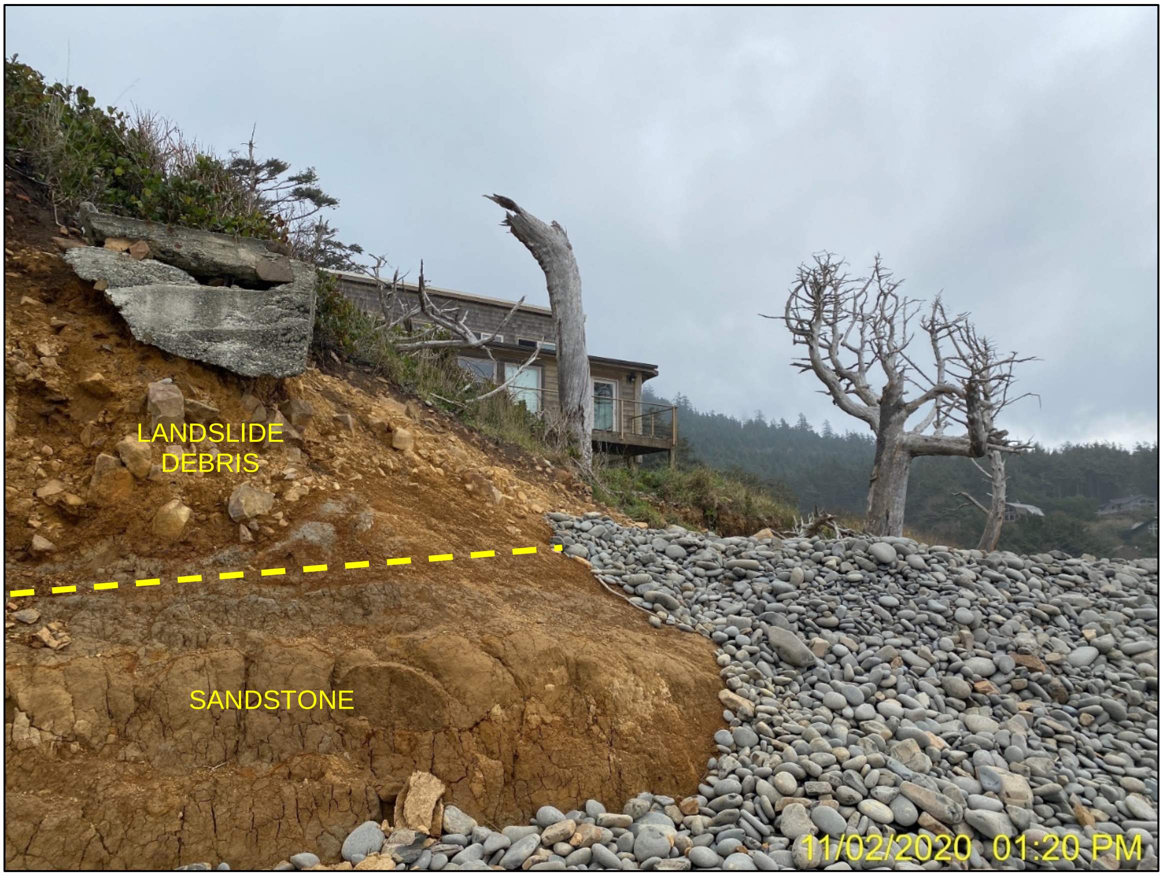


Photo 4: Aggressive erosion observed in exposed landslide deposit overlying intensely weathered sandstone which is mostly protected by the beach cobbles.

Figure 5: Isometric view of subject property and surrounding geography from Google Earth.

<u>2.2 Subsurface Materials</u>

Our site investigation consisted of advancing 2 drilled SPT borings (B-1 and B-2). B-1 was advanced to a depth of 9.5 feet below ground surface (bgs) and B-2 was advanced to a depth of 31.5 feet bgs using a T26 Beretta tracked drill rig subcontracted from PLi Systems of Hillsboro, Oregon. It should be noted that boring B-1 was advanced using hollow stem auger drill methods and B-2 was advanced using mud rotary drilling. Upon completion, the borings were backfilled with bentonite chips. The locations of our explorations are shown in Appendix B.

Select soil samples were tested in the laboratory to determine material properties for our evaluation. Laboratory testing was accomplished in general accordance with ASTM procedures. The testing performed included moisture content tests (ASTM D2216) and fines content determinations (ASTM D1140). The test results have been included on the exploration logs in Appendix C.

In general, we encountered a surficial layer of topsoil overlying what we interpret to be variable landslide debris, overlying decomposed to less weathered sandstone with depth. Each individual stratum encountered is discussed in further detail below.

TOPSOIL

In B-1, the surficial layer encountered consisted of a moist to wet, dark brown sandy silt with rootlets and decomposed organics. The thickness of this stratum was 6 inches in B-1. It should be noted we did not collect a surface sample in B-2 and therefore we did not encounter a topsoil layer.

LANDSLIDE DEBRIS

Directly below the topsoil in B-1 and the surficial layer for B-2, we encountered what we interpreted to be landslide debris. The landslide debris was generally an orange-brown to brown to dark brown silty sand with gravel and cobble sized sandstone pieces. We also encountered charcoal and wood chips in this stratum. Laboratory tests conducted on samples obtained from this stratum results in moisture contents of 55 to 102 percent, and a fines content of 41 to 45 percent. It should be noted that the high moisture content readings are likely due to the presence of organics. Based on the SPT N_{60} values and drive probe testing we consider this stratum to be loose to medium dense. The landslide debris extended to the terminal depths of our exploration in B-1 due to refusal on what we interpreted to be a large piece of hard rock. The final depth of this stratum in B-2 was 15 feet bgs.

NATIVE SANDSTONE

Directly below the landslide debris described above, we encountered what we interpreted to be a native sandstone. It appears to be intensely weathered to decomposed sandstone. It sampled as a sand in our borings, indicating that it had very weak cementation, which was broken by the driving of the SPT sampler. This stratum was generally a gray to gray-brown sand with few to little silt, gravel and cobble. We also encountered a floating boulder at a depth of 15.5 feet bgs. We do not know if it was basalt or sandstone. Laboratory tests conducted on samples obtained from this stratum resulted in moisture contents of 26 to 39 percent and fines contents of 8 to 10 percent. Based on the SPT N_{60} values we consider this stratum to be dense to very dense. This stratum extended to the terminal depth of B-2 (i.e. 31.5 feet bgs).

The classifications noted above were made in accordance with the Unified Soil Classification System (USCS) as shown in Appendix D. The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The exploration logs included in Appendix C should be reviewed for specific information at specific locations. These records include soil descriptions, stratifications, and locations of the samples. The stratifications shown on the logs represent the conditions only at the actual exploration locations. As described, we encountered possible landslide debris in our explorations. It should be noted that the explorations performed are not adequate to accurately identify the full extent of existing landslide debris across the site. Consequently, the actual landslide debris extent may be much greater than that shown on the exploration logs and discussed herein. Variations may occur and should be expected between locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on these logs. The samples that were not altered by laboratory testing will be retained for 60 days from the date of this report and then will be discarded.

2.3 Groundwater Information

As stated above, groundwater was not encountered at the time of our exploration. It should be noted that it is difficult to distinguish a groundwater table during mud rotary drilling in B-2, however, we anticipate a groundwater table may be at a depth of approximately 27 to 28 feet given the elevation of our explorations in relation to sea level.

It should be noted that EEI previously issued a Geotechnical Investigation Report for a different client for a nearby property approximately 0.2 miles southeast of the subject property. During this investigation, we encountered groundwater at a depth of 15 feet bgs in our boring. This could potentially be a perched water table or a spring, as springs are common in sloping coastal sites. The elevation for this boring was approximately 75 feet above the subject property.

Note that the water table elevation can fluctuate seasonally and annually, especially during periods of extended wet or dry weather or from changes in land use.

3.0 GEOLOGIC HAZARD ASSESSMENT

3.1 Soil Survey

The United States Department of Agriculture (USDA) Soil Survey provides geographical information of the soils in Clatsop County as well as summarizing various properties of the soils. The USDA shows the native soils on the site mapped as Skipanon gravelly medial silt loam on 30 to 60 percent slopes.[1] This soil type is well-drained, occurs on hillslopes and mountain slopes and is derived from a parent material of mass movement deposits derived from a mixture of igneous and sedimentary rock types overlying sedimentary rock.

3.2 Geology

The site is located above a coastal bluff overlooking Cove Beach on the Oregon Coast north of Cape Falcon. The region is underlain by a framework of Miocene aged (23 to 5 million years ago) volcanic rocks and Oligocene (33 to 23 million years ago) to Miocene aged marine sedimentary deposits that have been deposited over a basement rock of Eocene-aged (54 to 33 million years ago) volcanic arc deposits. Overlying this framework are Quaternary–aged (1.8 million years ago to present) marine terrace deposits, beach and dune deposits and landslide deposits.

More specifically, Niem and Niem (1985)[2] maps the underlying geology at the western half of the subject property as the Miocene aged Astoria Formation from the Astoria Group. This formation is described as "marine sandstone and siltstone, including shelf, slope channel, deltaic and turbidite sandstone, and slope mudstone". See Figure 6 below.

[1] Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture. Web Soil Survey. Available online at http://websoilsurvey.nrcs.usda.gov/ accessed October 27, 2020.

[2] Niem, A.R., and Niem, W., 1985, Geologic map of the Astoria Basin, Clatsop and northernmost Tillamook Counties, northwest Oregon: Portland, Oregon, Oregon Department of Geology and Mineral Industries Oil and Gas Investigation Map OGI-14, Plate 1, scale 1:100,000.

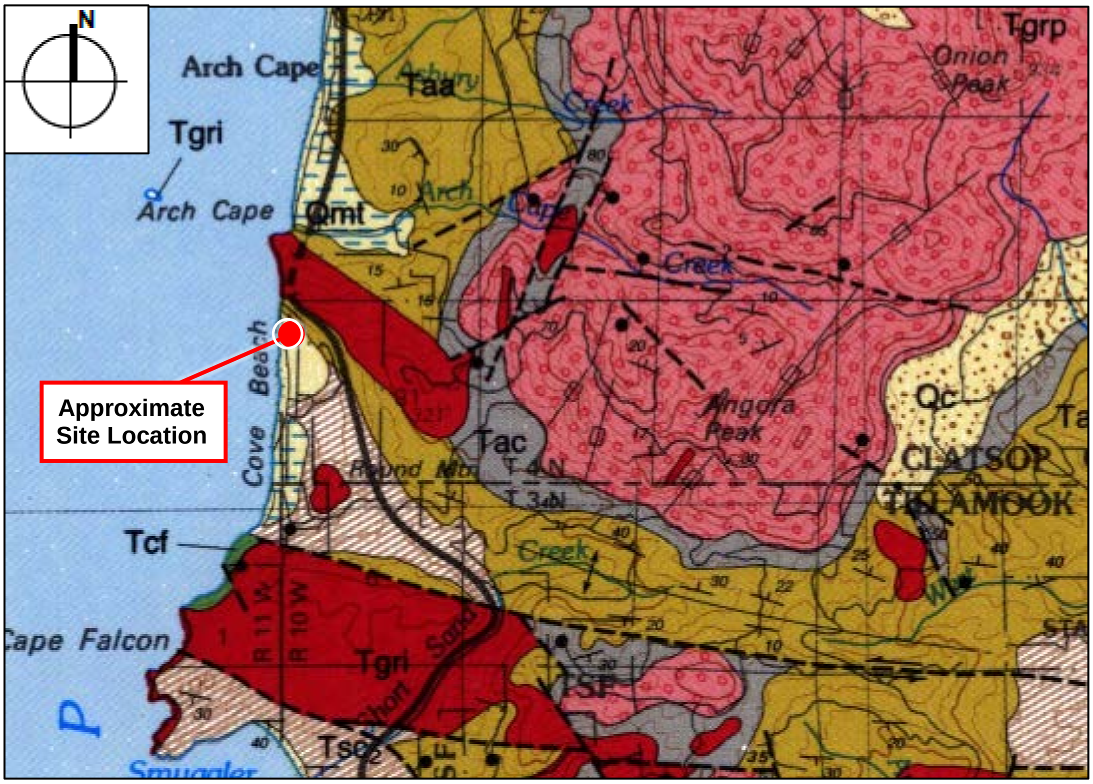


Figure 6: Geologic map of the subject property and its surrounding areas (source: Niem and Niem, 1985).

Schlicker and others (1972)[3] mapped an active landslide area to the east of Highway 101 near the subject property. See Figure 7 below. Active landslide areas are described as "areas where ground movement is continuous or periodic or areas in which historic (within about 100 years) movement has taken place. The area includes debris and rockfalls on the headlands, shallow slump failures along terraces fronting the ocean and bays, and areas of local slump in upland areas". The underlying bedrock unit in the active landslide area is the Miocene aged Astoria Formation (Tma), while the area in the immediate vicinity of the subject property is mapped as Pleistocene aged marine terrace deposits (Qmt) and Pleistocene and Holocene aged beach sand (bs).

As stated above in Section 2.1, the hummocky terrain and slightly bent and tilted trunks of some of the remaining pine trees are consistent with sloping properties on ancient landslide deposits. We did not observe signs of recent or active landslides from our reconnaissance of the immediate area, aside from active beach erosion at the toe of the bluff. Based on our observations of exposed and subsurface soils, as well as the geomorphic features of the site and nearby properties, it is our opinion that the site is likely located on a very large, ancient landslide. The reactivation of the landslide debris could be caused by an earthquake, significant wet winter weather saturating and lubricating the landslide material, ocean storms that that erode the toe of the landslide debris formation, or a combination thereof. It is our opinion that the risk to this property of being impacted

[3] Schlicker and others, 1972. Environmental Geology of the Coastal Region of Tillamook and Clatsop Counties, Oregon, Oregon Department of Geology and Mineral Industries, Bulletin 74.

by a landslide in the future is similar to numerous other properties that have already been developed in the Cove Beach area.

The upper, roughly 15 feet of soft soil is at risk of localized shallow landsliding or slope creep. Adding the weight of a home to this soil layer could increase that risk. As such, to mitigate this risk, the house would need to be supported on a deep foundation that extends through this potentially unstable soil layer. We discuss foundations in more detail in Section 4 of this report.

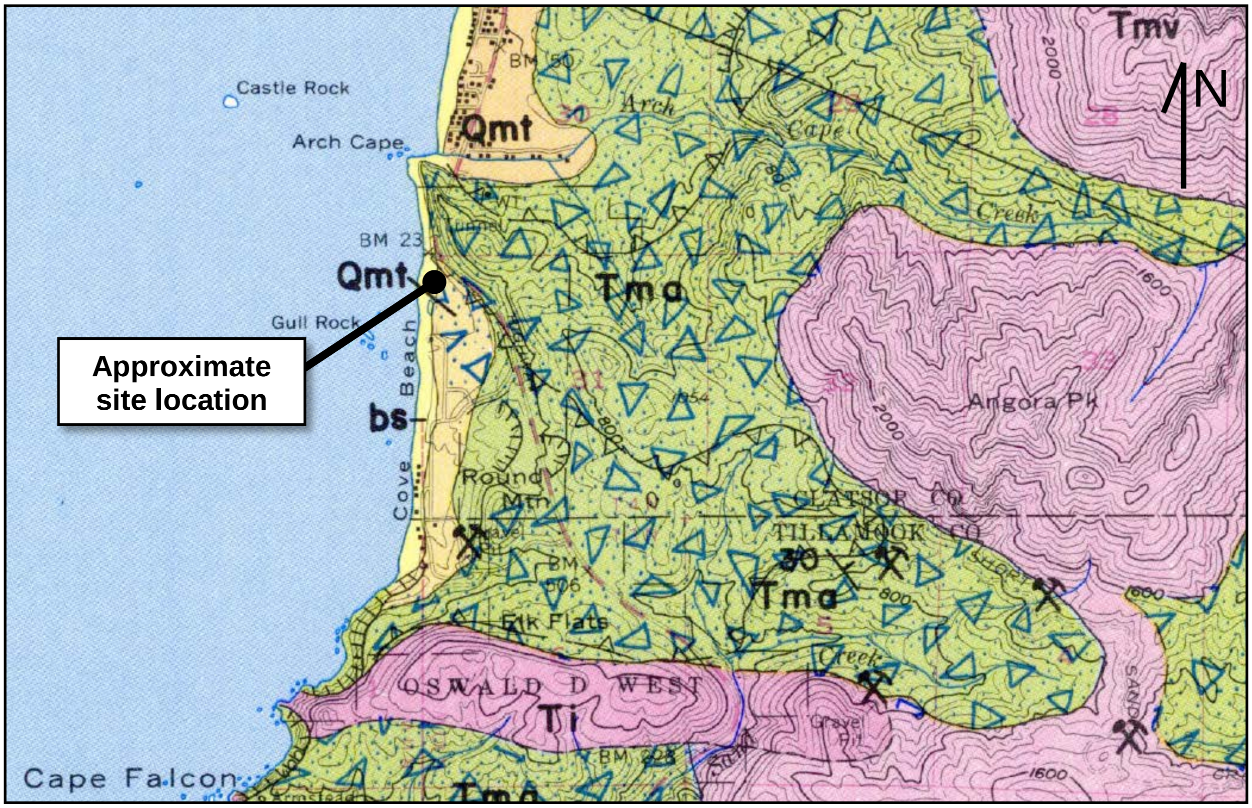


Figure 7: Geologic map of the area; the large green area with triangles is mapped as ancient landslide material (source: Schlicker and others, 1972).

3.3 Seismicity

Oregon's position at the western margin of the North American Plate and its location relative to the Pacific and Juan de Fuca plates have had a major impact on the geologic development of the state. The interaction of the three plates has created a complex set of stress regimes that influence the tectonic activity of the state. The western part of Oregon is heavily impacted by the influence of the active subduction zone formed by the Juan de Fuca Oceanic Plate converging upon and subducting beneath the North American Continental Plate off the Oregon coastline.

The Cascadia Subduction Zone, located approximately 100 kilometers off of the Oregon and Washington coasts, is a potential source of earthquakes large enough to cause significant ground shaking at the subject site. Research over the last several years has shown that this offshore fault zone has repeatedly produced large earthquakes, on average, every 300 to 700 years. It is generally understood that the last great Cascadia Subduction Zone earthquake occurred about 300 years ago, in 1700 AD. Although researchers do not necessarily agree on the likely magnitude, it is widely believed that an earthquake moment magnitude (M_w) of 8.5 to 9.5 is possible. The duration of strong ground shaking is estimated to be greater than 1 minute, with minor shaking lasting on the order of several minutes.

Additionally, earthquakes resulting from movement in upper plate local faults are considered a possibility. Crustal earthquakes are relatively shallow, occurring within 10 to 20 kilometers of the surface. Oregon has experienced at least two significant crustal earthquakes in the past decade—the Scotts Mills (Mt. Angel) earthquake (M_w 5.6) on March 25, 1993 and the Klamath Falls earthquake (M_w 5.9) on September 20, 1993. Based on limited data available in Oregon, it would be reasonable to assume a M_w 6.0 to 6.5 crustal earthquake may occur in Oregon every 500 years (recurrence rate of 10 percent in 50 years). There are no mapped crustal faults in the immediate vicinity of the property, but there is a marine crustal fault running east to west approximately 7 miles to the northwest of the property[4].

In accordance with Section 1613.2.2 of the 2019 OSSC and Table 20.3-1 of ASCE 7-16 we recommend a Site Class D (stiff soil) with an average standard penetration resistance between 15 and 50 blows per foot when considering the average of the upper 100 feet of bearing material beneath the foundations. Inputting our recommended Site Class as well as the site latitude and longitude into the Structural Engineers Association of California (SEAOC) – OSHPD Seismic Design Maps website (http://seismicmaps.org) which is based on the United States Geological Survey, we obtained the seismic design parameters shown in Table 1 below.

Table 1: Seismic Design Parameter Recommendations (ASCE 7-16)

PARAMETER	RECOMMENDATION
Site Class	D
S_s	1.312g
S_1	0.688g
F_a	1.00
F_v	Null – See Section 11.4.8
S_{MS} (= S_s x F_a)	1.312g
S_{M1} (= S_1 x F_v)	Null – See Section 11.4.8
S_{DS} (=2/3 x S_s x F_a)	0.875g
Design PGA (= S_{DS} / 2.5)	0.350g
MCE_G PGA	0.660g
F_{PGA}	1.100
PGA_M (= MCE_G PGA x F_{PGA})	0.726g

Note: Site latitude = 45.7956108, longitude = -123.9673994

The return interval for the ground motions reported in the table above is 2 percent probability of exceedance in 50 years.

[4] USGS U.S. Quaternary Faults Interactive Map, https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf.

Per Section 11.4.8 of ASCE 7-16 a site-specific seismic site response is required for structures on Site Class D and E sites with S_1 greater than or equal to 0.2g. The S_1 value for this site is greater than 0.2g as shown in Table 1 above. Therefore, a site response analysis is required as part of the design phase. However, Section 11.4.8 does provide an exception for not requiring a site response analysis (reference Sections 11.4.8.1, 11.4.8.2 and 11.4.8.3). The project Structural Engineer should determine if the proposed building will meet any of the exceptions—if the building does not meet the exception requirements then EEI should be retained to perform a site-specific site response analysis.

We understand a Supplement 1 dated December 12, 2018 has been issued for ASCE 7-16 to correct some issues in the original publication. One of the corrections in the Supplement pertains to Table 11.4-2 (see table below) for determining the value of the Long-Period Site Coefficient, F_V, which is then used to calculate the value of T_S. The T_S value is needed for one of the exceptions in Section 11.4.8. Without the correction in Supplement 1, it would not be possible to determine F_V and calculate T_s. Based on Supplement 1, the F_V value may be determined from the following corrected table.

Table 2: Long-Period Site Coefficient, F_V (corrected Table 11.4-2 in ASCE 7-16).

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
Site Class	S_1<=0.1	S_1<=0.2	S_1<=0.3	S_1<=0.4	S_1<=0.5	S_1>=0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	**2.2**[a]	**2.0**[a]	**1.9**[a]	**1.8**[a]	**1.7**[a]
E	4.2	**3.3**[a]	**2.8**[a]	**2.4**[a]	**2.2**[a]	**2.0**[a]
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: use linear interpolation for intermediate values of S_1.

[a] See requirements for site-specific ground motions in Section 11.4.8. These values of F_V shall be used only for calculation of T_S.

3.4 Geologic Hazards

The Oregon Department of Geology and Mineral Resources (DOGAMI) maps various geologic hazards, such as 100-year flooding, earthquake ground shaking, costal erosion, tsunamis, and landslides.[5] This service, generally referred to as Oregon's HazVu, shows the geologic hazards associated with development of this region of the site to include the following:

- Severe Cascadia earthquake expected shaking
- Severe crustal expected earthquake shaking
- Moderate potential of liquefaction due to earthquake shaking

[5] Oregon HazVu: Statewide Geohazards Viewer, available online at: http://www.oregongeology.org/sub/hazvu/ accessed 10/27/20

- Very high potential for landslide susceptibility
- Low to very high coastal erosion hazard
- Statutory tsunami inundation line
- Effective FEMA 100-year flood zone

Figures 8 through 13 below show mapping of the geologic hazards as presented by Oregon's HazVu.

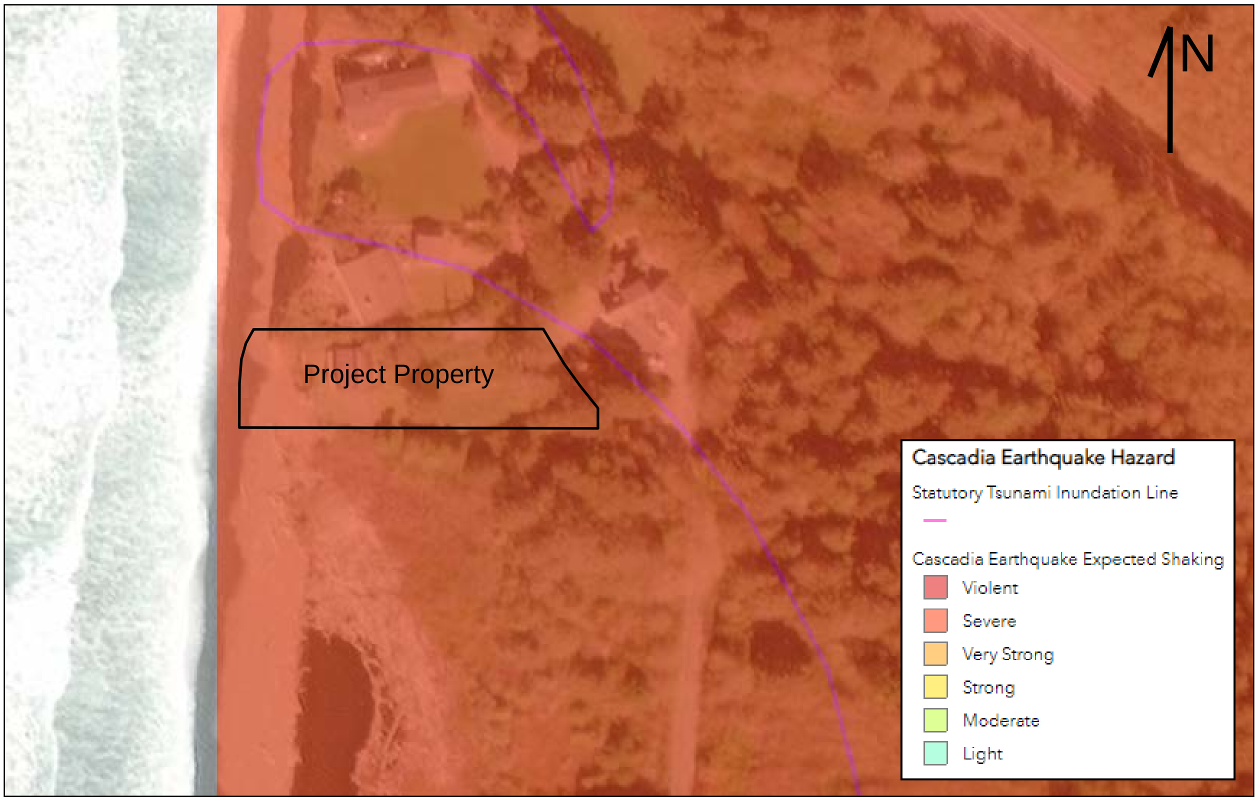


Figure 8: HazVu map showing the Cascadia earthquake expected shaking hazard zones and the statutory tsunami inundation line.

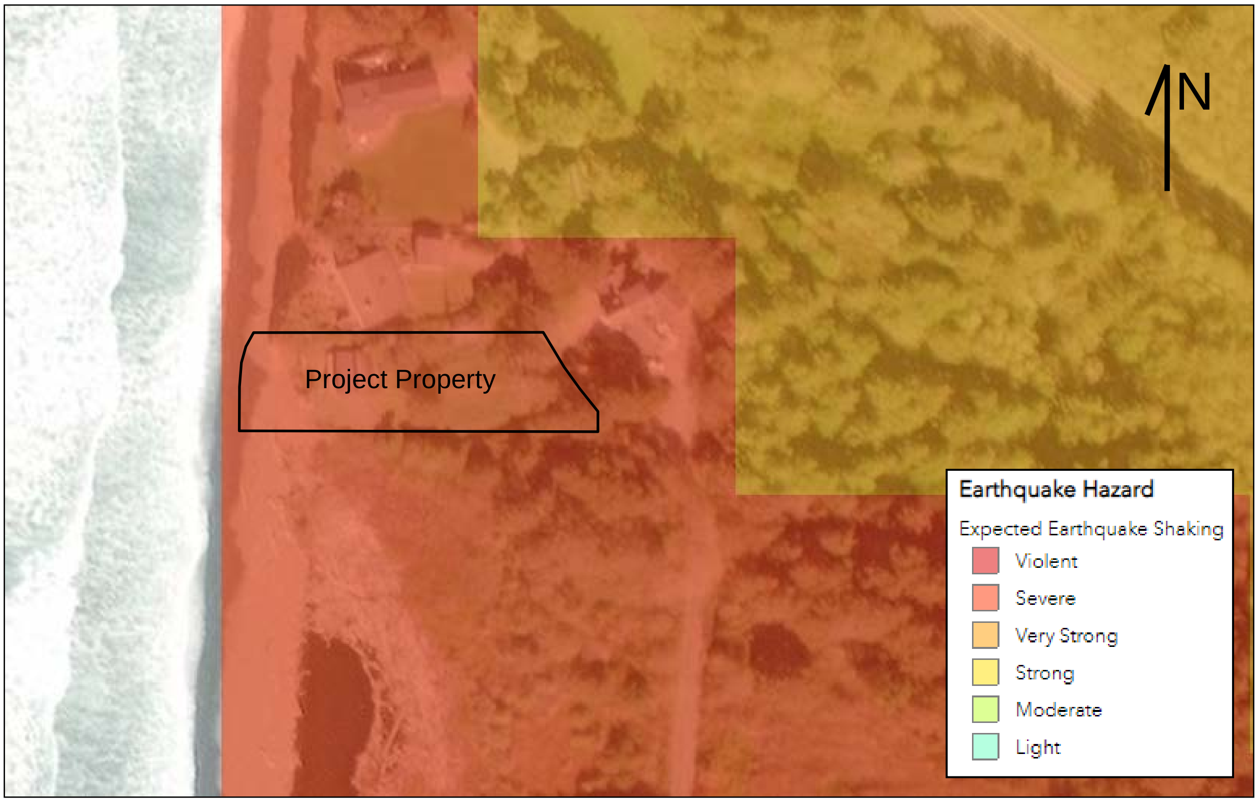


Figure 9: HazVu map showing the crustal earthquake expected shaking hazard zones.

Figure 10: HazVu map showing the earthquake liquefaction hazard zones.

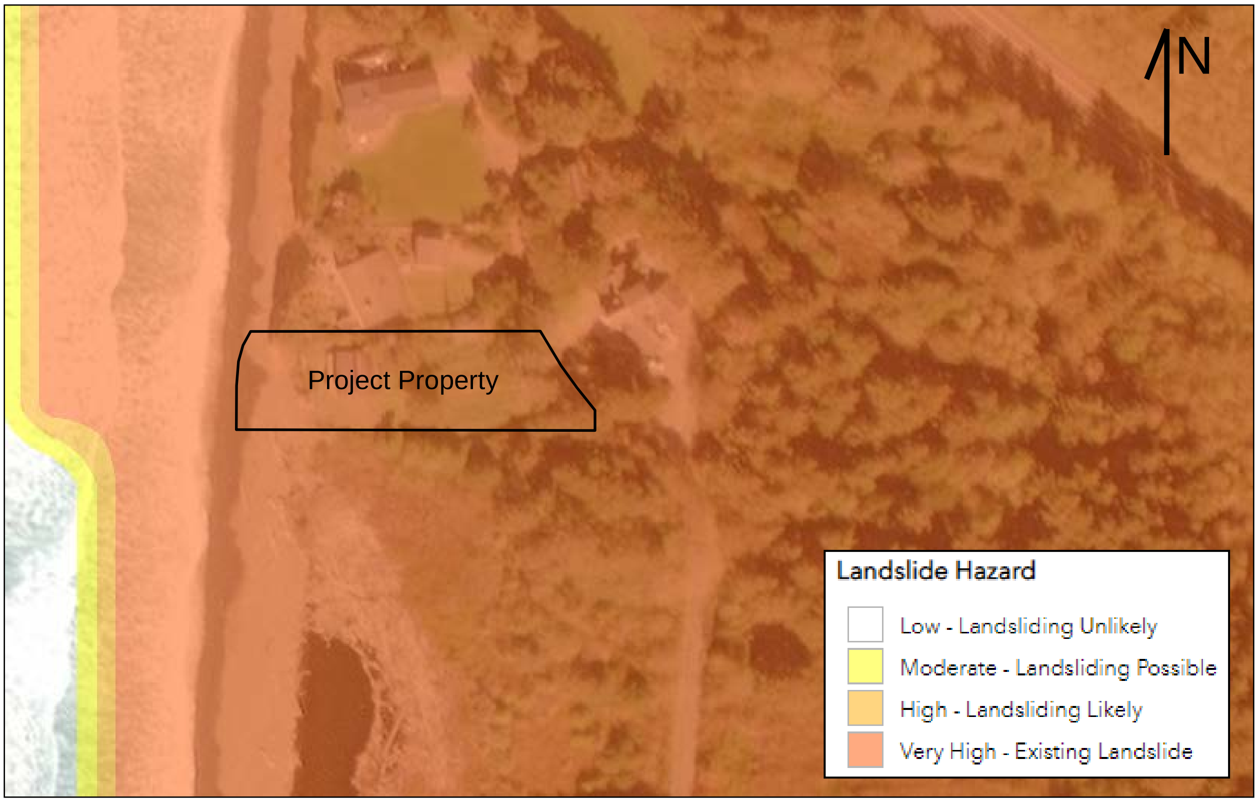


Figure 11: HazVu map showing the landslide hazard zones.

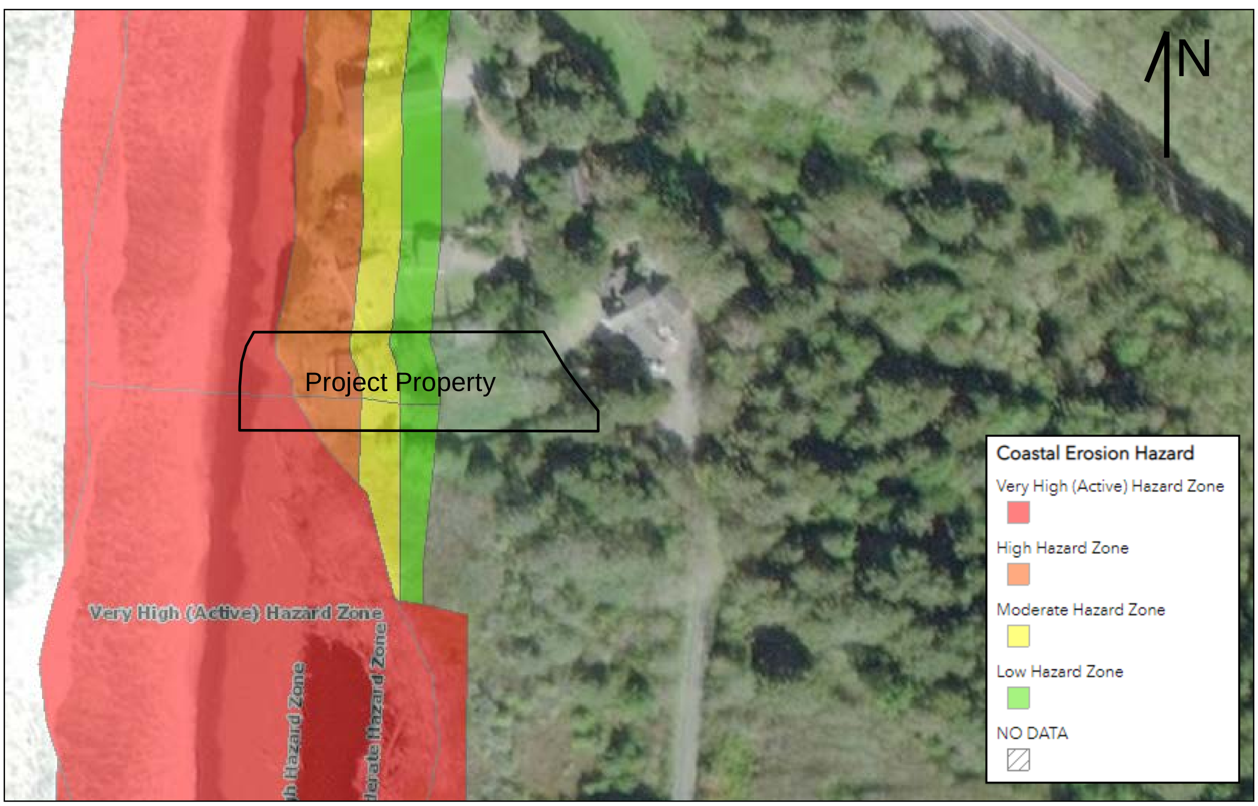


Figure 12: HazVu map showing the coastal erosion hazard zones.

Figure 13: HazVu map showing the flood hazard zones.

Based on our site reconnaissance, subsurface explorations, and office research, we consider the site to have the following geologic hazards:

- Reactivation of the ancient landslide debris.
- Minor shallow soil creep.
- Potential settlement associated with loose, organic, near surface soils.
- Tsunami inundation.
- Flooding.
- Coastal erosion.

Although the site is located in an area mapped as an active landslide, and has landslide deposits observable within our subsurface explorations, we believe that this landslide (as a single mass) is currently inactive. However, it is very normal/typical for small portions of the overall landslide mass to reactivate. A major seismic event could reactivate the slide mass, although to what degree is not known. We do not believe this property is at any greater risk from this hazard than the other numerous existing developed lots in the neighborhood.

As shown in Figure 12 above, the western portion of the site is mapped within a coastal erosion hazard area, ranging from low to high risk of coastal erosion. Much of the exposed bluff along Cove Beach shows signs of active and aggressive coastal erosion. The aggressive, active erosion is mostly limited to the landslide deposit which is generally at or above the elevation of the existing house, while the shoreline at lower elevations appears relatively stable (i.e. more resistive sandstone stratum) and protected by the beach cobbles. Although we do not believe that the proposed house is at immediate risk from coastal erosion, it could recede back towards the home gradually over time. We envision that it would occur in several sequences that would allow for addressing the issue before it ever reached the house. In addition, the house structure would be protected from erosion if supported on a foundation that bears directly on the more stable sandstone stratum.

As stated above, we did not encounter a clearly identifiable water table in our explorations. And the stratum below a depth of about 15 feet is a sandstone material. As such, we do not consider the soils encountered to be liquefiable.

In summary, it is our professional opinion that the proposed residential development on this property is feasible subject to the recommendations presented in this report. Primary considerations should be made to limiting the placement of fill to raise site grades, and maintaining adequate site surface and subsurface drainage. Vegetation should also be maintained to prevent excessive erosion, and should only be removed where needed to complete the proposed construction. Additionally, the house foundation should be engineered with the idea of resisting the effects of earthquake shaking and landslide reactivation. These recommendations are discussed in more detail in Section 4 below. Ultimately, owning a home in this area means there is an acceptance of risk that the property is located within a very large ancient landslide area that could reactivate at some time in the future, possibly en masse due to a Cascadia Subduction Zone earthquake event.

4.0 EVALUATION AND FOUNDATION RECOMMENDATIONS

4.1 Geotechnical Discussion

Based on our subsurface investigation and geologic hazard research, it is our professional opinion that the primary factors impacting the proposed development include the following:

1. **Location of property in a mapped landslide.** As stated above the property is located in a mapped ancient landslide that is considered to be currently inactive. It is our professional opinion that while the slide mass as a whole appears to be inactive there is risk that at some point in the future this slide mass could reactivate. However, this risk is no greater or less than any other properties located within this slide mass. In order to provide life-safety protection should the slide mass locally reactivate, we recommend the house be supported on a deep foundation system all tied together with an integrated, gridded system of rigid grade beams. In addition, we are recommending tiebacks in the easterly direction to provide lateral resistance to the grade beam system. We considered recommending a conventional shallow foundation that extends through the landslide debris layer and bears on the stable sandstone layer in order to mitigate future landslide risk. However, the sandstone stratum was encountered at a depth of about 15 feet in our boring, which is too deep to make a shallow foundation work. In addition, even if a shallow foundation could practically be extended to sandstone, the landslide material moving from the east would generate to much pressure on the face of the footings to resist.

   This deep foundation and tieback system may not totally eliminate the risk of some damage to the house should the slide locally re-activate but would hold the house to together long enough for the occupants to exit safely (i.e. life-safety is protected).

   As stated above, we encountered dense to very dense, weathered to decomposed sandstone at approximately 15 feet bgs in our boring, B-2. As such we anticipate installing the drilled, grouted micropiles somewhere between about 20 and 25 feet bgs (i.e. into the sandstone layer). It should be noted that we considered driven pin piles, however due to the presence of boulders and cobbles, it is our professional opinion that the pin piles could hit premature refusal and have trouble achieving the minimum recommended embedment depth into the sandstone. This concern is backed up by the fact that our boring B-1 encountered practical drilling refusal at a depth of 9.5 due to an apparent boulder.

2. **Slope stability** – As stated above, the surface of the property is hummocky as is typical in landslide topography. The localized slopes observed were up to 1.75H:1V in the area immediately west of the proposed residence. These slopes could be prone to shallow soil movement (sloughing, and slow, shallow soil creep). Our deep foundation and tieback recommendations below will address this slope issue.

3. **Presence of weak/compressible soils** – As discussed above, loose landslide debris soils were encountered in our explorations extending to a depth of about 15 feet bgs. As such, it's possible that there could be loose soils located within the proposed residence footprint.

However, we do not anticipate the need to remove the soft soils as the foundations will be pile supported and tied together with non-load bearing grade beams.

4. **Erosion hazard on subject property –** As stated above, the western portion of the site is mapped within a coastal erosion hazard area, ranging from low to high risk of coastal erosion. Much of the exposed bluff along Cove Beach shows signs of active and aggressive coastal erosion. The aggressive, active erosion is limited to the landslide deposit which is generally at or above the elevation of the existing house, while the shoreline at lower elevations is relatively stable and protected by the beach cobbles. Should the erosion patterns change and become more aggressive, the recommended deep foundation system will mitigate the risk to the proposed home of undermining due to severe erosion and provide time to repair the eroded area. It is our professional opinion that the proposed construction will not increase the risk of erosion on the subject property, given the drainage recommendations are followed in Section 5.2 below.

5. **Demolition of the existing residence** – The proposed construction includes demolition of the existing residence located on the subject property. We recommend that the demolition of the existing home include removal of all foundations, fill soils, floor slabs (if any), and underground utilities that are to be abandoned. Elements should be removed until clearly definable native soils are present. The void created from the demolition (if any) should be backfilled with structural fill as described in Section 4.3 below.

6. **Proposed relocation of the septic drainage field** – As stated above, is our understanding based on the plans provided to us that the existing septic field is planned to be relocated approximately 20 feet to the south. From a geotechnical standpoint, this is acceptable.

7. **Proposed wave protection** – It is our understanding that the client would like to install a rip rap berm to an elevation of 30 feet (i.e. increase the height of the current rip rap) upland of the vegetation line to protect the home from damage from high waves. These kinds of structures when located in an area that is actively eroded by wave action can have the effect of deflecting wave energy and thus increasing the rate erosion adjacent to the ends of the structure. From a geotechnical standpoint, the construction of this structure is acceptable provided it is located upland of the vegetation line and is not adjacent to areas of active erosion. It should be noted that this structure will not protect the house or property from tsunami level events and may not prevent all future storm events from reaching the house structure.

   Note that the berm weight will be placed on the existing landslide debris layer. Adding weight to landslide debris is usually discouraged. In this case, the greatest thickness of berm fill will be to the south. Our primary concern with adding weight to the landslide debris is to the west. But in this area, generally only up to about 4 feet of berm fill will be placed over a limited area. Ultimately, we cannot guarantee that the weight of the berm fill won't activate the landslide mass, but if it does, it would be on a limited basis and wouldn't affect the stability of the house as it will be supported on piles and tiebacks.

In summary, assuming that the risks outlined above are acceptable to the property owner, this site appears to be developable provided our recommendations in this report are followed.

4.2 Site Preparation

As stated above, the proposed construction includes demolition of the existing residence located on the subject property. In addition, site preparation will require tree clearing and grubbing prior to the commencement of excavation in the areas that have not yet been cleared. Topsoil, vegetation, roots, and any other deleterious soils will then need to be stripped from beneath the building area. Topsoil thickness in our explorations was about 6 inches, however it is not unusual for topsoil thickness to vary across the site. As stated above, we do not anticipate the need to remove the soft soils as the foundations will be pile supported. A representative of the Geotechnical Engineer should determine the depth of removal at the time of construction. All excavated material should be removed from the site. No new fills should be placed to permanently raise site grades.

Any utilities present beneath the proposed construction will need to be located and rerouted as necessary and any abandoned pipes or utility conduits should be removed to inhibit the potential for subsurface erosion. Utility trench excavations should be backfilled with properly compacted structural fill in accordance with Section 4.3.

4.3 Structural Fill

Due to the presence of landslide debris, we do not recommend placing structural fill to raise site grades (other than the planned rip rap berm). If site grades are raised, it will add weight to the slope and could cause potential landslide movement. The fill recommendations below are meant to address any small amount of structural backfill needed under and beside the footings, and the backfill of any utility trenches.

Structural fill should be free of organics or other deleterious materials, have a maximum particle size less than 3 inches, be relatively well graded, and have a liquid limit less than 45 and plasticity index less than 25. In our professional opinion, the on-site soils are generally not suitable as structural fill due to its high moisture content. Instead, we recommend the use of imported, well graded, crushed rock gravel. We recommend all structural fill be moisture conditioned to within 3 percentage points below and 2 percentage points above optimum moisture as determined by ASTM D698 (standard proctor). If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying.

Fill should be placed in relatively uniform horizontal lifts on the prepared subgrade which has been stripped of deleterious materials (i.e. topsoil and fill) and approved by the Geotechnical Engineer or his representative. Each loose lift should be about 1-foot thick. The type of compaction equipment used will ultimately determine the maximum lift thickness. Structural fill should be compacted to at least 95 percent of standard proctor maximum dry density as determined by

ASTM D698. Each lift of compacted engineered fill should be tested by a representative of the Geotechnical Engineer prior to placement of subsequent lifts.

4.4 Foundation Recommendations

Because the proposed residence will be located on a layer of ancient landslide debris, we recommend it be supported on a deep foundation system that transfers the weight of the house through the landslide debris and down to the stable, native, weathered to decomposed sandstone layer first encountered at about 15 feet in our boring. If the house were to be supported on a conventional shallow foundation bearing on the landslide debris stratum, the weight from the house could activate the ancient landslide material.

We considered several deep foundation options including helical piers, driven piles, auger-cast piles, drilled piers, and drilled micropiles. Our past experience working in these ancient landslide debris soils in the Cove Beach area indicates that the pile system selected should be drilled, as the landslide debris is a soil unit that contains a lot of gravel- and cobble-sized rock particles in it. Helical piers would not be able to achieve the embedment depth due to rock particle obstruction as well. Driven piles would require a very large pile driving hammer to pound the piles through the rock obstructions, which could create stability issues within the hillside. Auger-cast piles are not appropriate given the presence of rock particles in the soil layer, and drilled piers would require a very large rig with a lot of torque to be able to drill through the rocky soil. Therefore, we are recommending the new foundation for the proposed residence be supported by drilled and grouted micropiles.

Note that the deep foundation approach is intended to mitigate undermining of the house that results from the reactivation of the landslide debris to the west and south of the house. This foundation system is not intended to mitigate the reactivation of the larger landslide mass that covers this property as well as other properties in the area (i.e. landslide reactivation coming from east of the house).

4.4.1 Micropile Recommendations

Once the site has been properly prepared as discussed above we recommend the house be supported by 4.5-inch diameter minimum, drilled and grouted micropiles. This is a similar system that we used successfully to support a nearby house on Elk Flat Road (approximately 1 mile south of the subject property). The center bar should consist of a Grade 150, #10 Williams solid all-thread bar. The bar should be epoxy coated or metalized for corrosion protection. There are 2 types of epoxy coating: (1) post-fabricated, which is green in color, and (2) pre-fabricated, which is purple or gray in color. The pre-fabricated (purple or gray) epoxy coating provides better adhesion and abrasion resistance, which is what we recommend be used on this project because the micropiles are permanent and are a critical element to supporting the home.

Based on our experience on the nearby project, we expect that drilling will be very difficult and that the holes will need to be temporarily cased the full depth to prevent caving. Additionally, the contractor should anticipate that there may be significant grout loss during installation at some of the micropile locations and that some micropile drilling/grouting may need to occur twice. On the Elk Flat Road project, the micropile contractor was able to use a woven grout sock inside of the casing to reduce the grout loss. We recommend a post-grout tube be installed in each micropile to allow for post-grouting, if determined to be necessary.

Based on our boring information, we recommended that the micropiles be embedded at least 10 feet into the dense sandstone stratum, which was first encountered in our boring at a depth of 15 feet below existing grade. As such, we anticipate the micropiles to be about 25 to 30 feet long. Please note that the depth to the native stratum can vary across the site and the micropile lengths discussed in this paragraph are for preliminary construction budgeting purposes only. The actual micropile lengths will need to be determined during construction.

Based on a minimum ultimate strength of about 188 kips for the Grade 150 #10 center bar, we recommended the allowable compressive capacity of the micropiles be planned for 113 kips (i.e. 60 percent of the minimum ultimate strength) and that at least one micropile be load tested as detailed in Section 4.4.2 below. Because the load test would only be conducted to a Factor of Safety of about 133 percent, we recommend the target load test settlement be limited to 2/3-inch instead of 1-inch.

The quantity, spacing, and location of the micropiles should be specified by the Structural Engineer. In addition, the Structural Engineer should design the micropile connection to the new house foundation.

Provided our recommendations above are followed, we anticipate that total and differential settlement will be less than 1 inch and ½-inch between columns, respectively.

Lateral loads may be resisted by passive earth pressures acting against the face of the concrete footings based on an equivalent fluid density of 300 pounds per cubic foot (pcf) for footings poured "neat" against in-situ soils, or properly backfilled with structural fill. This is an ultimate value - we recommend a factor of safety of 1.5 be applied to the equivalent fluid pressure, which is appropriate due to the amount of movement required to develop full passive resistance. Because the foundations will be underpinned and the soil beneath the bottoms of footings can settle away from the concrete, we do not recommend using lateral frictional resistance between the footings and subgrade to resist lateral loads. Additional lateral resistance will be achieved by using tiebacks as discussed in Section 4.4.3 below.

A representative of the Geotechnical Engineer should be present during micropile installation and load testing.

4.4.2 Micropile Axial Compressive Load Testing

Our experience has shown that actual allowable compressive pile capacities can vary from our calculated values. As such, we recommend one axial compressive load test be performed to confirm our design recommendations. Load testing should be conducted in general accordance with ASTM D1143, “Piles Under Static Axial Compressive Loads”. We recommend pile load testing be conducted on the first production micropile (before any additional micropiles are installed) so that adjustments can be made to the design (i.e. spacing, length) if the actual test load achieved is less than what is estimated in this report. The load test should be taken to 80 percent of the minimum ultimate strength of the center bar (i.e. 150 kips for a Grade 150 #10 Williams solid bar). Since we do not plan to test the micropile to failure, a production pile may be used as the test micropile. The micropile load test should be conducted by the contractor under the supervision of the Geotechnical Engineer. The untested micropiles should be installed following the same procedures that will be used to install production pile that is load tested. The contractor may elect to substitute a tension test (ASTM D3689) for a compression test.

4.4.3 Tieback Recommendations

We recommend that the house be laterally supported by at least 4 tiebacks to mitigate geotechnical slope creep, because micropiles have very little lateral load capacity. The tiebacks were designed based on the following assumed parameters:

- 4.5 inch minimum borehole diameter.
- Grade 150, #10 Williams solid all-thread center bar. This bar has a yield strength (i.e. ultimate load capacity) of 188 kips and should only be load tested to 80 percent of the yield strength (i.e. 150.4 kips).
- Grout with a 28-day minimum compressive strength of 5,000 psi.
- Ultimate soil-grout bond strength of at least 40 psi (assumes pressure grouting method will be used).
- Locate the bonded length of the tiebacks into the native, weathered to decomposed sandstone stratum (i.e. 15 feet deep). All of the tieback within the landslide debris soil zone should be unbonded.
- 35 degree installation angle down from horizontal.
- All tiebacks to be pull tested and then locked off at a pre-stress load to be determined by EEI. Additional tieback load testing recommendations are contained in Section 4.4.4 below.

Based on the above assumed parameters, we recommend the tiebacks be constructed as follows:

Table 3: Tieback Recommendations.

Tieback Installation Angle (degrees)	Estimated Tieback Unbonded Length (feet)	Tieback Bonded Length (feet)	Estimated Total Tieback Length (feet)[1]	Tieback Design Load Capacity (kips)	Tieback Pull Test Load (kips)
35	26	10	36	113	150

Notes:

1. The tieback length is dependent upon the soil-grout bond strength, which has been assumed and will need to be confirmed with load testing. The actual bond length and total tieback length may be more or less than reported in Table 3.

The tiebacks should be generally equally spaced along the base of the west-facing exterior wall (i.e. embedded within the grade beam). We recommend the Structural Engineer locate the tiebacks at wall intersections, where the walls are typically most rigid. We can assist the Structural Engineering in locating the tiebacks upon request.

Due to the corrosive nature of the coast, we recommend the tieback center bars be protected from corrosion with epoxy coating or metalizing. Care will need to be taken when handling the bar during installation not to damage the epoxy.

All tiebacks to be pull tested and locked off at a pre-stress load to be determined by EEI. Additional tieback load testing recommendations are contained in Section 4.4.4 below.

4.4.4 Tieback Load Testing Recommendations

All of the tiebacks should be proof tested to 133 percent of the 113 kip design load (i.e. 150 kips) in accordance with the following load intervals: Alignment Load (AL), 0.25 design load (DL), 0.50 DL, 0.75 DL, 1.00 DL, 1.25 DL, 1.33 DL, AL, and the Lockoff Load. The alignment load should be no greater than 5 percent of the design load.

Proof test readings shall be taken immediately after reading each load increment, except at 1.0 DL, 1.25 DL and 1.33 DL. At these final 3 load increments, readings shall be taken at 1, 2, 3, 4, 5, 6 and 10 minutes. If the total creep movement exceeds 0.040 inches between 1 and 10 minutes (i.e. 1 log cycle), then the test load shall be maintained for an additional 50 minutes, with recordings at 20, 30, 40 50 and 60 minutes. The movement between 6 and 60 minutes (i.e. one log cycle) should not be greater than 0.080 inches. EEI's Geotechnical Engineer should ultimately evaluate the proof test results to verify the anchors will achieve their designed capacity without excessive movement.

The lockoff load should be minimized to reduce the amount of vertical pre-stress acting on the footings. In other words, the tieback pre-stress (i.e. lockoff) load should not significantly increase the vertical loading acting on the footings. Preliminarily we anticipate a lockoff load on the order of about 10 kips. This should be confirmed by the Geotechnical Engineer once the tieback proof load testing is conducted.

4.5 Floor Slab Recommendations

For the purposes of this report, we have assumed that maximum floor slab loads will not exceed 150 psf. Based on the existing soil conditions, the design of the floor slab can be based on a subgrade modulus (k) of 100 pci. This subgrade modulus value represents an anticipated value which would be obtained in a standard in-situ plate test with a 1-foot square plate. Use of this subgrade modulus for design or other on-grade structural elements should include appropriate modification based on dimensions as necessary.

In order to fully mitigate the risk of settlement, the concrete floor slab would need to be tied into the grade beams and supported on the micropiles recommended above (i.e. designed as a structural floor slab). However, if a conventional, less expensive floor slab-on-grade is preferred, to at least partially mitigate the risk of potential settlement, the floor slab should be supported on at least 12-inches of properly compacted crushed rock gravel structural fill overlying the existing (firm) soils. The structural fill recommendations are outlined in Section 4.3 above. The floor slabs should have an adequate number of joints to reduce cracking resulting from any differential movement and shrinkage. Ultimately, to address the risk of loss of ground support beneath floor slabs, it may be more prudent to have a crawl space, rather than floor slabs on grade where possible.

Prior to placing the structural fill, the exposed subgrade surface should be prepared as discussed in Section 4.2. In addition, we recommend a proof-roll utilizing a fully loaded, dual axle dump truck or water truck in order to identify any unstable areas that should be removed prior to structural fill placement. The proofroll should be observed by a representative of the Geotechnical Engineer. If the subgrade cannot be accessed with a dump truck, then the subgrade will need to be visually evaluated by a representative of the Geotechnical Engineer by soil probing. If fill is required, the structural fill should be placed on the prepared subgrade after it has been approved by the Geotechnical Engineer.

The 12-inch thick crushed rock structural fill should provide a capillary break to limit migration of moisture through the slab. If additional protection against moisture vapor is desired, a moisture vapor retarding membrane may also be incorporated into the design. Factors such as cost, special considerations for construction, and the floor coverings suggest that decisions on the use of vapor retarding membranes be made by the project design team, the contractor and the owner.

4.6 Retaining Wall Recommendations

While we are not aware of any specific retaining wall details for the project, we are providing these general recommendations for preliminary planning purposes. Once more detailed plans are known about retaining walls, we should be provided the drawings so that we can update our recommendations if necessary.

Retaining wall footings should be designed in general accordance with the recommendations contained in Section 4.4 above (i.e. supported on drilled and grouted micropiles). For insignificant landscape retaining walls not greater than 4 feet tall, where excessive movement due to ground movement is acceptable, they may be supported on conventional shallow foundations designed for an allowable soil bearing capacity of up to 1,500 pounds per square foot.

Lateral earth pressures on walls, which are not restrained at the top, may be calculated on the basis of an “active” equivalent fluid pressure of 35 pcf for level backfill, and 60 pcf for sloping backfill with a maximum 2H:1V slope. Lateral earth pressures on walls that are restrained from yielding at the top (i.e. stem walls) may be calculated on the basis of an “at-rest” equivalent fluid pressure of 55 pcf for level backfill, and 90 pcf for sloping backfill with a maximum 2H:1V slope. The stated equivalent fluid pressures do not include surcharge loads, such as foundation, vehicle, equipment, etc., adjacent to walls, hydrostatic pressure buildup, or earthquake loading.

Surcharge loads on walls should be calculated based on the attached calculations/formulas shown in Appendix E.

For seismic loading on retaining walls with level backfill, new research indicates that the seismic load is to be applied at 1/3 H of the wall instead of 2/3 H, where H is the height of the wall[6]. We recommend that a Mononobe-Okabe earthquake thrust per linear foot of 10.9 psf*H^2 be applied at 1/3 H for level backfill. For a maximum 2H:1V slope we recommend 29.2 psf*H^2 be applied. This assumes a combination of granular backfill and soil retained by the walls.

Any minor amount of backfill for retaining walls should be select granular material, such as sand or crushed rock with a maximum particle size between ¾ and 1 ½ inches, having less than 5 percent material passing the No. 200 sieve. Because of their silt/clay content, the native soils do not meet this requirement, and it will be necessary to import material to the project for structure backfill. Silty soils can be used for the last 18 to 24 inches of backfill, thus acting as a seal to the granular backfill. All backfill behind retaining walls should be moisture conditioned to within ± 2 percent of optimum moisture content, and compacted to a minimum of 92 percent of the material's maximum dry density as determined in accordance with ASTM D698. Fill materials should be placed in layers that, when compacted, do not exceed about 8 inches. Care in the placement and compaction of fill behind retaining walls must be taken in order to insure that undue lateral loads are not placed on the walls.

An adequate subsurface drain system will need to be designed and installed behind retaining walls to prevent hydrostatic buildup. A waterproofing system should be designed for any basement walls where moisture intrusion is not desirable.

[6] Lew, M., et al (2010). "Seismic Earth Pressures on Deep Building Basements," SEAOC 2010 Convention Proceedings, Indian Wells, CA.

5.0 CONSTRUCTION CONSIDERATIONS

EEI should be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project. EEI cannot accept any responsibility for any conditions that deviate from those described in this report, nor for the performance of the foundations if not engaged to also provide construction observation for this project.

5.1 Moisture Sensitive Soils/Weather Related Concerns

The upper soils encountered at this site are expected to be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils that become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

Exposed fine grained soils can be extremely sensitive to moisture and should be protected with a layer granular fill (at least 2 inches thick) if the excavations are to be left open during periods of wet weather.

5.2 Drainage, Groundwater, and Stormwater Considerations

Water should not be allowed to collect in the foundation excavations or on prepared subgrades for the concrete slabs (i.e. driveway, patio, etc.) during construction. Positive site drainage should be maintained throughout construction activities. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff.

The site grading plan should be developed to provide rapid drainage of surface water away from the building areas and to inhibit infiltration of surface water around the perimeter of the building and beneath the slabs. The grades should be sloped away from the building area.

Because this site is located within ancient landslide debris, we strongly recommend that stormwater be hard piped to the City or County's existing stormwater system (i.e. the existing public ditches), or to the large drainage area to the south.

5.3 Excavations

Based on our past experience in the area, vertical cut slopes in the ancient slide debris may at first appear to be stable. However, over time (typically a few days), the soils may "relax" and slough. As such, the contractor should take care when excavating into these soils and we strongly recommend that they either use temporary shoring, or lay the excavated slopes back. Once the site soils are exposed, we can consult with the contractor to determine a safe layback slope angle. We can also provide temporary shoring recommendations, if requested.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. EEI does not assume responsibility for construction site safety or the contractor's compliance with local, state, and federal safety or other regulations.

6.0 SUMMARY FINDINGS AND CONCLUSIONS

We are providing this section of our report to facilitate the review of the anticipated building permit per Clatsop County Land and Water Development and Use Ordinance (LWDUO), Section 4.040. Our recommendations provided throughout the above sections of this report are summarized below in reference to Clatsop County Land and Water Development and Use Ordinance (LWDUO), Section 4.044.

(1) For areas identified in Section 4.044, the geotechnical report shall:

(A) Identify the hazards to life, public and private property which may be caused by mass movement (landsliding and sloughing), soil erosion or deposition, and earthquakes: The geologic hazards are identified and discussed in Sections 3.4 and 4.1.

(B) Identify the hazards to life, public and private property, and the natural environment which may be caused by the proposed use and other human activities: We do not expect an increase in hazards to life, public and private property, and the natural environment if our recommendations in Sections 4 and 5 are followed.

(C) Describe how the proposed development or use will be adequately protected from geologic hazards, including landsliding and sloughing, soil erosion or deposition, and earthquakes: The geologic hazards and geotechnical concerns for the development are addressed in Section 3.4 and 4.1.

(D) Describe how the proposed development is designed to minimize the adverse effects it might have on the site and adjacent areas: This is described in Section 4.1.

(2) For areas identified in Section 4.042 (2), and in addition to the standards identified in Section 4.044 (1), the geotechnical report shall identify the hazards to life, public and private property which may be caused by wind erosion or accretion, wave undercutting (erosion), and ocean overtopping (flooding, including tsunami): The applicable hazards are identified and discussed in Sections 3.4 and 4.1.

(3) For areas identified in Section 4.042 (1) and 4.042 (2), the geotechnical report shall describe how the proposed development provides for temporary and permanent stabilization and the planned maintenance of new and existing vegetation. Existing stabilizing vegetation, particularly trees, shall not be removed on slopes of 20% or greater: This is addressed in Section 3.4 and 4.2.

(4) For areas identified in Section 4.042 (1) and 4.042 (2), the geotechnical report shall be prepared in conformance with the document "Clatsop County – Geotechnical Report Content Standards": This report has been prepared with the above standards in mind.

(5) For areas identified in Section 4.042 (3), the geotechnical report shall be prepared by a certified engineering geologist, soils engineer, or civil engineer. Geotechnical reports

prepared for areas identified in Section 4.042 (3) shall incorporate specific construction and structural recommendations to address the soil characteristics of the site. Where pertinent, the discussion of specific construction and structural recommendations shall include: site preparation such as compaction or replacement of existing soils, bearing loads and the corresponding amount of settlement, steps to be taken with respect to ground and surface water, special foundation requirements, and foundation recommendations based on bearing capacity, design criteria, and the effect of adjacent loads: The above considerations are detailed in Sections 3 and 4 of this report

(6) For all areas identified in Section 4.042, the geotechnical report shall be prepared in conformance with the document "Clatsop County – Geotechnical Report Content Standards": Refer to item (4) above

7.0 REPORT LIMITATIONS

The subject development is located on a bluff fronting the Pacific Ocean. This property is subject to very dynamic forces (i.e. powerful winter storms, ocean currents, and earthquakes). The conditions of the subject property could change drastically in the future due to these forces and cannot be entirely predicted, nor can they be fully mitigated. These risks are common to other similar properties in the area, which have already been developed with similar residential homes. Determining the worst possible cases and then designing a structure to eliminate the risk from those cases is typically not economically feasible. Therefore, in accepting the recommendations herein, the owner must assume some risk that the reasonable worst-case conditions, described above, may be exceeded.

As is standard practice in the geotechnical industry, the conclusions contained in our report are considered preliminary because they are based on assumptions made about the soil, rock, and groundwater conditions exposed at the site during our subsurface investigation. A more complete extent of the actual subsurface conditions can only be identified when they are exposed during construction. Therefore, EEI should be retained as your consultant during construction to observe the actual conditions and to provide our final conclusions. If a different geotechnical consultant is retained to perform geotechnical inspection during construction then they should be relied upon to provide final design conclusions and recommendations, and should assume the role of geotechnical engineer of record, as is the typical procedure required by the governing jurisdiction.

The geotechnical recommendations presented in this report are based on the available project information, and the subsurface materials described in this report. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, EEI should be notified immediately to determine if changes in the foundation recommendations are required. If EEI is not retained to review these changes, we will not be responsible for the impact of those conditions on the project.

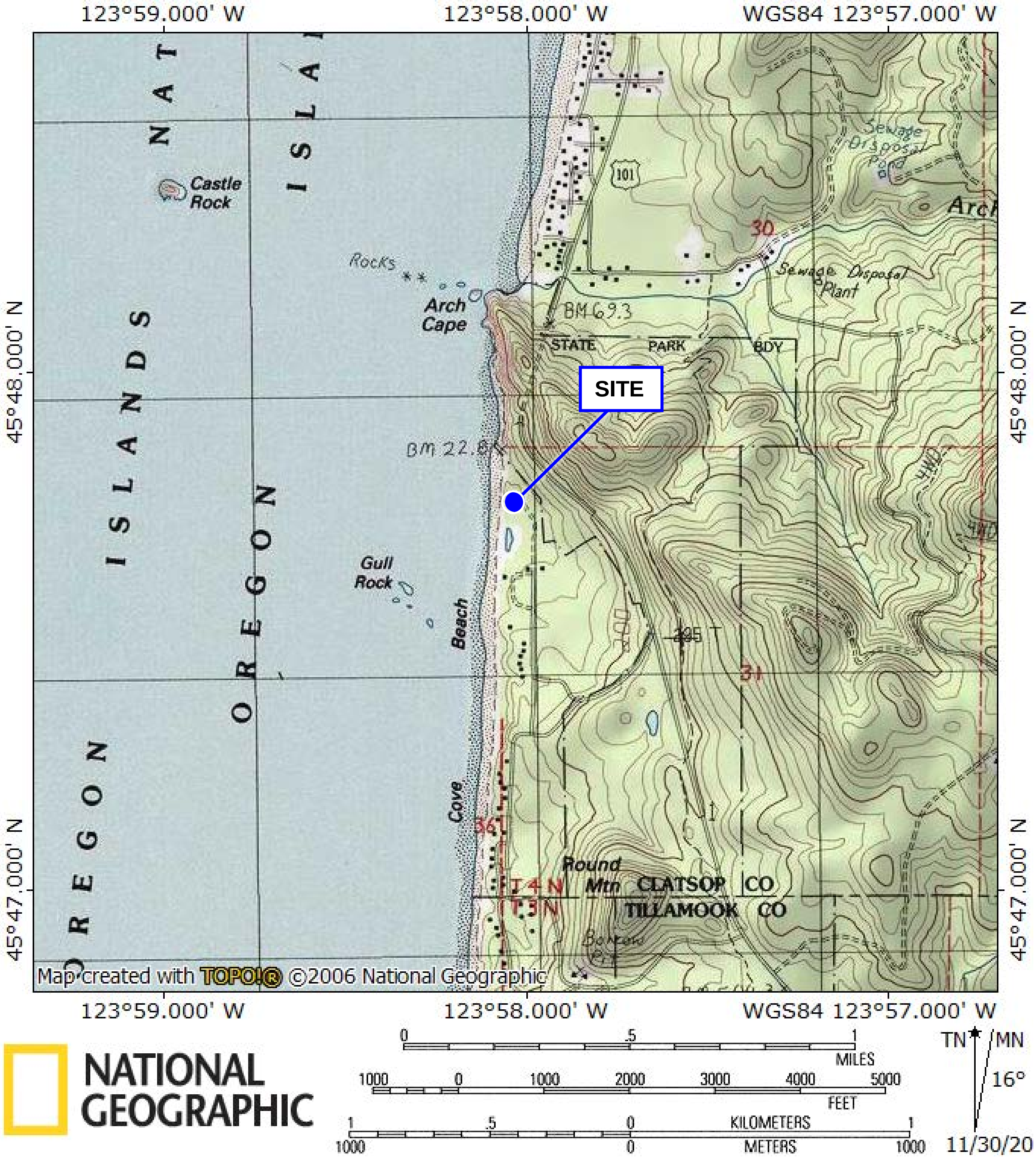
The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents.

This report has been prepared for the exclusive use of Marc Felisky for the specific application to the proposed single family residence to be located at 79532 Ray Brown Road in Arch Cape, Clatsop County, Oregon. EEI does not authorize the use of the advice herein nor the reliance upon the report by third parties without prior written authorization by EEI.

APPENDICES

APPENDIX A – SITE LOCATION PLAN



Earth Engineers, Inc.	**Proposed Felisky Single Family Residence 79532 Ray Brown Road Arch Cape, Clatsop County, Oregon**	**Report No. 20-186-1** **November 30, 2020**

APPENDIX B – EXPLORATION LOCATION PLAN

Legend

= approximate drilled boring location

Source: Google Earth

Earth Engineers, Inc.	**Proposed Felisky Single Family Residence** **79532 Ray Brown Road** **Arch Cape, Clatsop County, Oregon**	**Report No. 20-186-1**	**November 30, 2020**

Earth Engineers, Inc.

Appendix C: Boring B-1

Sheet 1 of 1

Client: Marc Felisky
Project: Proposed Felisky Residence
Site Address: 79532 Ray Brown Road
Cove Beach, Oregon
Location of Exploration: See Appendix B
Logged By: Jacqui Boyer

Report Number: 20-186-1
Drilling Contractor: PLi Systems
Drilling Method: Hollow Stem Auger w/ auto SPT
Drilling Equipment: T26 Beretta
Approximate Ground Surface Elevation (ft msl): 27
Date of Exploration: 11/5/2020

Depth (ft)	Water Level	Geologic Description of Soil and Rock Strata (Lithology)	Sample Number	Blows per 6 Inches	N_{60} value	Moisture Content (%)	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Pocket Pen (tsf)	Remarks
0		Topsoil - dark brown sandy silt with rootlets, wet									
		Sand (SM) - brown to dark brown, silty sand with gravel and cobble, organics, charcoal and wood chips, moist, loose to medium dense	SPT-1	2, 5, 9	18	62					possible landslide debris
			SPT-2	1, 1, 2	4	61	45			0.25	
6		Sand (SM) - orange brown to dark brown, silty sand with gravel and cobble, wood pieces, moist, loose to medium dense	SPT-3	2, 4, 3	9	52	41			0.5	possible landslide debris auger scraping on rock
8			SPT-4	11, 9, 10	25	76	33			0.75	refusal on boulders

Notes : Boring terminated at a depth of approximately 9.5 feet below ground surface (bgs), terminated due to refusal on rock. Groundwater was not encountered at the time of our exploration. Boring backfilled with bentonite chips on 11/5/2020. N60 values reported are based on a SPT hammer energy correction factor of 1.388 (i.e. 83.3/60), reference "Report of SPT Hammer Energies" prepared by GeoDesign Inc. dated 3/12/2018. Approximate elevations from Google Earth.

Appendix C: Boring B-2

Earth Engineers, Inc.

Sheet 1 of 1

Client: Marc Felisky
Project: Proposed Felisky Residence
Site Address: 79532 Ray Brown Road
Cove Beach, Oregon
Location of Exploration: See Appendix B
Logged By: Jacqui Boyer

Report Number: 20-186-1
Drilling Contractor: PLi Systems
Drilling Method: Mud Rotary
Drilling Equipment: T26 Beretta
Approximate Ground Surface Elevation (ft msl): 28
Date of Exploration: 11/5/2020

Depth (ft)	Water Level	Lithology: Geologic Description of Soil and Rock Strata	Sample Number	Blows per 6 Inches	N_{60} value	Moisture Content (%)	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Pocket Pen (tsf)	Remarks
0		Sand (SM) - brown to dark brown, silty sand with gravel and cobble, organics, charcoal and wood chips, wet, loose									surface sample not taken possible landslide debris
3			SPT-1	3 1 1	3						no sample retained in split spoon
5.5			SPT-2	2 1 5	8	102					
8			SPT-3	4 3 3	8	103	41				
10.5			SPT-4	3 2 4	8	87				1	
13			SPT-5	5 4 4	10	55	43				
15		Sand (SW) - gray to gray-brown, sand with few to little silt, few gravel and cobble, moist to wet, dense to very dense	SPT-6	6 39 30	89	39					boulder encountered causing SPT hammer to bounce possible sandstone
20			SPT-7	8 14 17	40	28	10				
25			SPT-8	13 19 21	52	26					
30			SPT-9	7 12 15	35	30	8				

Notes : Boring terminated at a depth of approximately 31.5 feet below ground surface (bgs). Groundwater was not encountered at the time of our exploration. Boring backfilled with bentonite chips on 11/5/2020. N60 values reported are based on a SPT hammer energy correction factor of 1.388 (i.e. 83.3/60), reference "Report of SPT Hammer Energies" prepared by GeoDesign Inc. dated 3/12/2018. Approximate elevations from Google Earth.

APPENDIX D: SOIL CLASSIFICATION LEGEND

APPARENT CONSISTENCY OF COHESIVE SOILS (PECK, HANSON & THORNBURN 1974, AASHTO 1988)				
Descriptor	**SPT N_{60} (blows/foot)***	**Pocket Penetrometer, Qp (tsf)**	**Torvane (tsf)**	**Field Approximation**
Very Soft	< 2	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	2 – 4	0.25 – 0.50	0.12 – 0.25	Easily penetrated several inches by thumb
Medium Stiff	5 – 8	0.50 – 1.0	0.25 – 0.50	Penetrated several inches by thumb w/moderate effort
Stiff	9 – 15	1.0 – 2.0	0.50 – 1.0	Readily indented by thumbnail
Very Stiff	16 – 30	2.0 – 4.0	1.0 – 2.0	Indented by thumb but penetrated only with great effort
Hard	> 30	> 4.0	> 2.0	Indented by thumbnail with difficulty

* Using SPT N_{60} is considered a crude approximation for cohesive soils.

APPARENT DENSITY OF COHESIONLESS SOILS (AASHTO 1988)	
Descriptor	**SPT N_{60} Value (blows/foot)**
Very Loose	0 – 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

MOISTURE (ASTM D2488-06)	
Descriptor	**Criteria**
Dry	Absence of moisture, dusty, dry to the touch, well below optimum moisture content (per ASTM D698 or D1557)
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table, well above optimum moisture content (per ASTM D698 or D1557)

PERCENT OR PROPORTION OF SOILS (ASTM D2488-06)	
Descriptor	**Criteria**
Trace	Particles are present but estimated < 5%
Few	5 – 10%
Little	15 – 25%
Some	30 – 45%
Mostly	50 – 100%
Percentages are estimated to nearest 5% in the field. Use "about" unless percentages are based on laboratory testing.	

SOIL PARTICLE SIZE (ASTM D2488-06)	
Descriptor	**Size**
Boulder	> 12 inches
Cobble	3 to 12 inches
Gravel - Coarse Fine	¾ inch to 3 inches No. 4 sieve to ¾ inch
Sand - Coarse Medium Fine	No. 10 to No. 4 sieve (4.75mm) No. 40 to No. 10 sieve (2mm) No. 200 to No. 40 sieve (.425mm)
Silt and Clay ("fines")	Passing No. 200 sieve (0.075mm)

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2488)				
Major Division			**Group Symbol**	**Description**
Coarse Grained Soils (more than 50% retained on #200 sieve)	**Gravel** (50% or more retained on No. 4 sieve)	Clean Gravel	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels and gravel-sand-silt mixtures
			GC	Clayey gravels and gravel-sand-clay mixtures
	Sand (> 50% passing No. 4 sieve)	Clean sand	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sand with fines	SM	Silty sands and sand-silt mixtures
			SC	Clayey sands and sand-clay mixtures
Fine Grained Soils (50% or more passing #200 sieve)	**Silt and Clay** (liquid limit < 50)		ML	Inorganic silts, rock flour and clayey silts
			CL	Inorganic clays of low-medium plasticity, gravelly, sandy & lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silt and Clay (liquid limit > 50)		MH	Inorganic silts and clayey silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

GRAPHIC SYMBOL LEGEND		
GRAB		Grab sample
SPT		Standard Penetration Test (2" OD), ASTM D1586
ST		Shelby Tube, ASTM D1587 (pushed)
DM		Dames and Moore ring sampler (3.25" OD and 140-pound hammer)
CORE		Rock coring

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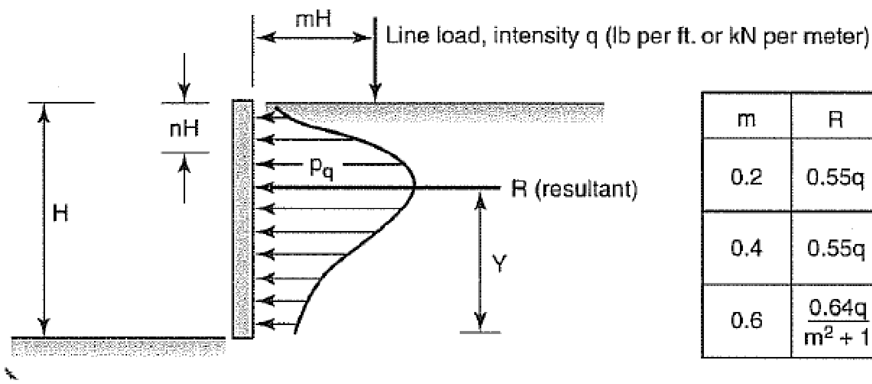
APPENDIX E: SURCHARGE-INDUCED LATERAL EARTH PRESSURES FOR WALL DESIGN

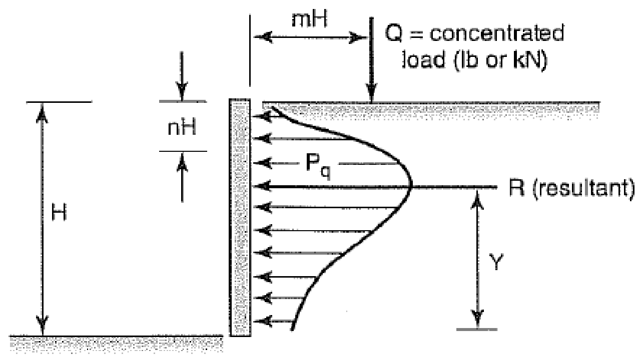
LINE LOAD (applicable for retaining walls not exceeding 20 feet in height):



m	R	Y
0.2	0.55q	0.60 H
0.4	0.55q	0.58 H
0.6	$\frac{0.64q}{m^2 + 1}$	0.52 H

Figure 16-28 Pressure distribution against vertical wall resulting from line load of intensity *q*.

CONCENTRATED POINT LOAD (applicable for retaining walls not exceeding 20 feet in height):



m	R	Y
0.2	$0.78 \frac{Q}{H}$	0.59 H
0.4	$0.78 \frac{Q}{H}$	0.59 H
0.6	$0.48 \frac{Q}{H}$	0.48 H

Figure 16-27 Pressure distribution against vertical wall resulting from point load, *Q*.

AREAL LOAD:

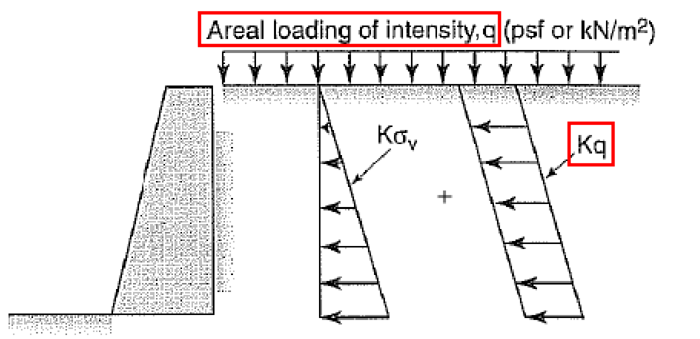
Figure 16-26 Influence of areal loading on wall pressures.

use K=0.4 for active condition (i.e. top of wall allowed to deflect laterally)

use K=0.9 for at-rest condition (i.e. top of wall not allowed to deflect laterally)

Resultant, R = K * q * H

Where H = wall height (feet)



Source of Figures: McCarthy, D.F., 1998, "Essentials of Soil Mechanics and foundations, Basic Geotechnics, Fifth Edition."

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Proposed Felisky Single Family Residence
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Report No. 20-186-1

November 30, 2020