Geologic Hazards and
Geotechnical Investigation
Tax Lots 6100 and 6199, Map 06-11-34DA
N.W. Jetty Avenue
Lincoln City, Oregon

Prepared for:
Ms. Dana Director
12716 Elk Rock Road
Lake Oswego, Oregon 97034

Project #Y204427 October 23, 2020

H.G. Schlicker & Associates, Inc.

H.G. Schlicker & Associates, Inc.
607 Main Street, Suite 200 · Oregon City, Oregon 97045
(503) 655-8113 · FAX (503) 655-8173

Project #Y204427 October 23, 2020

To: **Ms. Dana Director**
12716 Elk Rock Road
Lake Oswego, Oregon 97034

Subject: **Geologic Hazards and**
Geotechnical Investigation
Tax Lots 6100 and 6199, Map 06-11-34DA
N.W. Jetty Avenue
Lincoln City, Oregon

Dear Ms. Director:

The accompanying report presents the results of our geologic hazards and geotechnical investigation for the above subject site.

After you have reviewed our report, we would be pleased to discuss it and to answer any questions you might have.

This opportunity to be of service is sincerely appreciated. If we can be of any further assistance, please contact us.

H.G. SCHLICKER & ASSOCIATES, INC.

J. Douglas Gless, MSc, RG, CEG, LHG
President/Principal Engineering Geologist

JDG:aml

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

H.G. Schlicker & Associates, Inc.
607 Main Street, Suite 200 · Oregon City, Oregon 97045
(503) 655-8113 · FAX (503) 655-8173

Project #Y204427 October 23, 2020

To: **Ms. Dana Director**
12716 Elk Rock Road
Lake Oswego, Oregon 97034

Subject: **Geologic Hazards and**
Geotechnical Investigation
Tax Lots 6100 and 6199, Map 06-11-34DA
N.W. Jetty Avenue
Lincoln City, Oregon

Dear Ms. Director:

1.0 Introduction and General Information

At your request and authorization, a representative of H.G. Schlicker and Associates, Inc. (HGSA) visited the subject site on October 19, 2020, to complete a geologic hazards and geotechnical investigation of Tax Lots 6100 and 6199, Map 06-11-34DA, in the Roads End area of Lincoln City, Oregon (Figures 1 and 2; Appendix A). It is our understanding that you are interested in purchasing the property and constructing a single-family residence at the site.

This report addresses the engineering geology and geologic hazards at the site with respect to constructing an addition to the existing home. The scope of our work consisted of a site visit, site observations and measurements, hand augered borings, a slope profile, limited review of the geologic literature, interpretation of topographic maps, lidar, and aerial photographs, and preparation of this report, which provides our findings, conclusions, and recommendations.

2.0 Site Description

The site consists of two tax lots, Tax Lot 1600 and Tax lot 1699. Tax Lot 1600 consists of an approximately 0.13-acre lot approximately 50 feet wide, north to south, and 115 feet long east to west (Figures 2 and 3). Tax Lot 1699 is immediately west of Tax Lot 1600 and consists of an approximately 0.31-acre lot that is approximately 50 feet wide, north to south and extends west onto the beach approximately 260 to 270 feet.

The site is bounded by developed lots to the north and south, NW Jetty Avenue to its east, and the bluff, beach, and Pacific Ocean to its west. The site lies on an elevated marine terrace adjacent to an approximately 20-foot-high bluff. The area of the site east of the bluff is gently sloping to the north and west (Figures 3 and 4). The bluff along the western part of the site slopes down to the west at approximately 20 to 50 degrees, with an average slope angle of approximately 40 degrees.

The bluff slope is densely vegetated with European beachgrass, ferns, blackberry and brush. The area east of the bluff slope is generally vegetated with lawn grass, shore pine, blackberry, English ivy and brush (Appendix A).

An oceanfront protective structure is present at the site and exposed along the lower part of the bluff. The revetment consisted of 2 to 5-foot diameter basaltic boulders and was in general disrepair at the time of our site visit (Appendix A).

2.1 Proposed Development

Based on the information provided to us, you plan to construct a house at the site. We have provided geotechnical recommendations for design of a single-family residence in Sections 8.1 through 8.12 below. HGSA should be contacted to review development plans for the site. There will be additional charges for these services.

2.2 History of The Site and Surrounding Areas

The subject property has an oceanfront protective structure and lies in an area of bluffs that have generally been protected by oceanfront protective structures. Oceanfront protective structures extend approximately 50 feet north of the site to Roads End State Recreation Site and approximately 600 feet south of the subject site. We expect that this general stretch of coastline will have additional shore protection constructed as bluff recession continues in the future. According to the Oregon Coastal Atlas Ocean Shores Data Viewer (http://www.coastalatlas.net/oceanshores, accessed October 2020), the lot is eligible for a beachfront protective structure on the Goal 18 Eligibility Inventory. However, the potential to receive a permit for oceanfront protection is dependent upon meeting certain regulatory requirements in addition to the Goal 18 eligibility requirement.

2.3 Site Topography, Elevations, and Slopes

The area of the subject site east of the bluff is relatively flat. The bluff along the western part of the site slopes down to the west from approximately 20 to 50 degrees, with an average slope angle of approximately 40 degrees (Figures 3 and 4; Appendix A). Based on 2009 lidar data from DOGAMI, the upper marine terrace lies at an elevation of approximately 46 feet (NAVD 88), and the beach/dune junction is at an elevation of approximately 20 feet (Figure 3). Based on our review of historical aerial imagery and beach profile data, the elevation of the beach varies by a few feet to about 6 feet or more (NAVD 88).

2.4 Vegetation Cover

The bluff slope is densely vegetated with European beachgrass, ferns, blackberry and brush. The area east of the bluff slope is generally vegetated with lawn grass, shore pine, blackberry, English ivy and brush (Appendix A).

2.5 Subsurface Materials

Detailed descriptions and analyses of geology and subsurface materials at the site are provided in Sections 3.1 and 3.3 below. Marine terrace deposits exposed north of the site at Roads End State Recreation Site consist of tan to light brown, moist, medium dense to dense, friable, fine-grained, cross-bedded sand, overlain by silt fill soil.

2.6 Site Oceanfront Conditions

The site is located along an oceanfront bluff slope consisting primarily of marine terrace sands that have undergone recession as a result of wind and rain erosion, sloughing, and shallow landsliding. An oceanfront protective structure is present at the site and exposed along the lower part of the bluff. The revetment consisted of 2 to 5-foot diameter basaltic boulders and was in general disrepair at the time of our site visit. A detailed description of the fronting beach area is provided in Section 3.2, with oceanfront slope stability and erosion discussed in Section 4.0 below.

2.7 Drift Logs or Flotsam

At the time of our site visit, we observed a minor accumulation of driftwood and flotsam in the beach area at the site. Satellite imagery indicates that the accumulation of driftwood and flotsam in the vicinity is generally consistent with slightly greater accumulation in late spring.

2.8 Streams or Drainage and Influence on Beach Elevations

Logan Creek discharges onto the beach at the northern end of Roads End State Recreation Site approximately 500 feet north of the site. It does not significantly influence the beach elevation at the site.

2.9 Headland Proximity and Influence on Beach Sediment Transport and Elevations

Headlands are not present in this local section of the Oregon Coast and the Lincoln City oceanfront. The site lies within the Lincoln littoral cell. The sands within the Lincoln littoral cell are believed to have little or no transport beyond Cascade Head approximately 1.3 miles north of the site and Cape Foulweather approximately 12 miles to the south (Komar, 1997).

2.10 Shore Protection Structures

The subject property has an oceanfront protective structure and lies in an area of bluffs that have generally been protected by oceanfront protective structures. Oceanfront protective structures extend approximately 50 feet north of the site to Roads End State Recreation Site and approximately 600 feet south of the subject site.

2.11 Beach Access Pathways

A small pathway along the site's southern property boundary leads down the bluff slope to the beach. Public beach access is present at Roads End State Recreation Site approximately 100 feet north of the site.

2.12 Human Impacts and Influence on Site Resistance to Ocean Wave Attack

The existing riprap revetment increases the site resistance to ocean wave attack.

3.0 Geologic Mapping, Investigation and Descriptions

3.1 Geology

The site lies in an area mapped as Quaternary marine terrace deposits underlain by lower Eocene Nestucca Formation (Schlicker et al., 1973). The marine terrace deposits consist of semi-consolidated, uplifted beach sand overlain locally by fine-grained dune deposits. The uplifted marine terrace sediments are typically high-energy nearshore marine deposits capped by beach sand (Kelsey et al., 1996). The marine terrace deposits exposed north of the site consist of tan to light brown, moist, medium dense to dense, friable, fine-grained, cross-bedded sand. The underlying Nestucca Formation consists of fine-bedded tuffaceous siltstone and sandstone with ash and glauconitic sandstone interbeds. Locally, the Nestucca Formation is below the beach elevation.

3.2 Description of the Fronting Beach

3.2.1 Summer and Winter Average Beach Widths

The beach at the site has a width of approximately 100 feet to more than 300 feet in this area during the winter and summer, respectively, depending upon sand transport in any given year. The beach here is dynamic and frequently changes, primarily due to rip current formation and El Niño and La Niña ocean conditions. Typically, the beach is broad and dissipative in summer, becoming narrower and steeper in winter, particularly during prolonged storm cycles.

3.2.2 Beach Sediment Median Grain Size

Beach sediment is primarily fine-grained to lesser medium-grained sand with cobbles exposed near the site in the back-beach area.

3.2.3 Summer and Winter Beach Elevations and Average Slopes

The beach slopes west at approximately 7 degrees in the winter and a few degrees in the summer. Based on our review of beach morphology monitoring data available for this section of Oregon's coast from 1997 to 2002, beach elevations varied by 0 to 6 feet from minimum to maximum, with a minor change at the beach-bluff junction (Allan and Hart, 2005). The beach elevation can change substantially associated with El Niño and La Niña events, with the sand being stripped off, exposing the wave-cut platform beneath. Topographic contours derived from 2009 lidar data provided by DOGAMI show the elevation above mean sea level of the beach-bluff junction west of the subject property as approximately 20 feet (NAVD 88) (Figure 3), which generally agrees with data from Allan and Hart (2005).

3.2.4 Rip Currents or Embayments

Rip currents and rip current embayments have formed frequently along this stretch of beach within the last decade, as evidenced by our review of historical aerial and satellite imagery.

3.2.5 Offshore Rock Outcrops and Sea Stacks

Offshore rock outcrops or sea stacks are not present near the site. Mapping by Priest and Allan (2004) shows Tertiary Cascade Head Basalt outcrops approximately 1.3 miles north and 1.7 miles south of the site.

3.2.6 Depth of Beach Sand to Bedrock

We did not observe any exposed bedrock on the beach during our site visit. However, we estimate sand and cobble depths along the beach at this time to be about 8 feet thick.

3.3 Subsurface Conditions

At the time of our site visit, we completed subsurface exploration by excavating five hand augered borings to depths up to approximately 4.5 feet. A Geologist from our office logged the borings and visually classified the soils encountered according to the Unified Soil Classification System (USCS). A detailed description of subsurface conditions is provided in Appendix B, and the approximate locations of the borings are shown on Figures 3 and 4.

In general, materials encountered in borings consisted of approximately 1 to 2 feet of loose silt fill soils, underlain by soft to medium stiff silt and variably cemented dense sand. Free groundwater was not encountered in the borings. Soils exposed in the upper bluff fronting Roads End State Recreation Site approximately 50 feet north of the site are similar to those encountered in our hand augered borings.

3.4 Structures

Structural deformation and faulting along the Oregon Coast are dominated by the Cascadia Subduction zone (CSZ), which is a convergent plate boundary extending for approximately 680 miles from northern Vancouver Island to northern California. This convergent plate boundary is defined by the subduction of the Juan de Fuca plate beneath the North America Plate and forms an offshore north-south trench approximately 60 miles west of the Oregon coast shoreline. A resulting deformation front consisting of north-south oriented reverse faults is present along the western edge of an accretionary wedge east of the trench, and a zone of margin-oblique folding and faulting extends from the trench to the Oregon Coast (Geomatrix, 1995).

The nearest fault is a westerly trending normal fault, approximately 1.3 miles north of the site (Schlicker et al., 1973). Several other generally parallel faults which trend in a southwesterly direction toward Siletz Bay have been mapped within 2.5 to 4 miles east and southeast of the site. These are normal faults with their upthrown sides to the northwest and cut Tertiary aged deposits with no indications of recent movement.

The nearest mapped potentially active faults are the Yaquina Head Fault, located approximately 23 miles south of the site, and the Yaquina Bay Fault, located approximately 26 miles south of the site. The Yaquina Head Fault is an east-trending oblique fault with left-lateral strike-slip and either contractional or extensional dip-slip offset components (Personius et al., 2003). It offsets the 80,000-year-old Newport marine terrace by approximately 5 feet, indicating a relatively low rate of slip, if still active (Schlicker et al., 1973; Personius et al., 2003). The Yaquina Bay Fault is a generally east-northeast trending oblique fault that also has left-lateral strike-slip and either contractional or extensional dip-slip offset components (Personius et al., 2003). This fault is believed to extend offshore for approximately 7 to 8 miles and may be a structurally controlling feature for the mouth of Yaquina Bay (Goldfinger et al., 1996; Geomatrix, 1995). At Yaquina Bay, a 125,000-year-old platform has been displaced approximately 223 feet up-on-the-north by the Yaquina Bay Fault. This fault has the largest component of vertical slip (as much as 2 feet per 1,000 years) of any active fault in coastal Oregon or Washington (Geomatrix, 1995). Although the age for the last movement of the Yaquina Bay Fault is not known, the fault also offsets 80,000-year-old marine terrace sediments.

4.0 Erosion and Slope Stability

The bluff along the western part of the site is approximately 20 feet high and slopes down to the west from 20 to 50 degrees. The lower part of the bluff is partially protected from ocean wave erosion by a riprap revetment, which is currently in relatively poor condition. The revetment has been disturbed by wave action and slopes down to the west at approximately 50 degrees, which is steeper than typical for a riprap revetment. We observed indications of ocean wave erosion and minor sloughing along the lower bluff (Appendix A). We anticipate that the bluff above the revetment will continue to erode and slough due to ocean wave erosion unless the riprap revetment is repaired.

Priest (1994) and Priest et al. (1994) determined the average annual erosion rate for adjacent bluff segments, which do not have riprap as 0.27 ± 0.34 feet per year. This erosion rate was calculated by measuring the distance from existing structures to the bluff compared to distances measured on a 1939 or 1967 vertical aerial photograph. Provided that the revetment at the base of the bluff is repaired and maintained, as needed, we anticipate that future recession caused by ocean wave erosion will be essentially zero. As discussed above, the bluff area above the revetment will likely experience continued ocean wave erosion until it is repaired.

Based on mapping completed by Priest and Allan (2004), the beach, bluff slope and upper bluff within approximately 20 feet of the bluff slope break lie within the Active Erosion Hazard Zone. The area within approximately 20 feet of east of the Active Erosion Hazard Zone lies within the High-Risk Erosion Hazard Zone. The next approximately 25 feet east lies in the Moderate-Risk Erosion Hazard Zone, and the easternmost part of the site along NW Jetty Avenue lies in the Low-Risk Erosion Hazard Zone. Coastal erosion hazard zone definitions and methodology are provided below.

The methodology provided by Priest and Allan (2004) defines four coastal erosion hazard zones for bluffs of Lincoln County, Oregon, as follows:

"*The basic techniques used here are modified from Gless and others (1998), Komar and others (1999), and Allan and Priest (2001). The zones are as follows:*

1) Active hazard zone: The zone of currently active mass movement, slope wash, and wave erosion.

2) The other three zones define high-, moderate-, and low-risk scenarios for expansion of the active hazard zone by bluff top retreat. Similar to the dune-backed shorelines, the three hazard zones depict decreasing levels of risk that they will become active in the future. These hazard zone boundaries are mapped as follows:

a. <u>High-risk hazard zone:</u> The boundary of the high-risk hazard zone will represent a best case for erosion. It will be assumed that erosion proceeds gradually at a mean erosion rate for 60 years, maintaining a slope at the angle of repose for talus of the bluff materials.

b. <u>Moderate-risk hazard zone:</u> The boundary of the moderate-risk hazard zone will be drawn at the mean distance between the high- and low-risk hazard zone boundaries.

c. <u>Low-risk hazard zone:</u> The low-risk hazard zone boundary represents a "worst case" for bluff erosion. The worst case is for a bluff to erode gradually at a maximum erosion rate for 100 years, maintaining its slope at the angle of repose for talus of the bluff materials. The bluff will then be assumed to suffer a maximum slope failure (slough or landslide). For bluffs composed of poorly consolidated or unconsolidated sand, another worst-case scenario will be mapped that assumes that the bluff face will reach a 2:1 slope as rain washes over it and sand creeps downward under the forces of gravity. For these sand bluffs, whichever method produces the most retreat will be adopted" (Priest and Allan, 2004).

It should be noted that mapping done for the 2004 study was intended for regional planning use, not for site-specific hazard identification.

4.1 Analyses of Erosion and Flooding Potential

4.1.1 DOGAMI Beach Monitoring Data

As discussed in Section 3.2.3 above, beach monitoring data for this section of Oregon's coast shows that beach elevations varied by several feet from minimum to maximum over the monitored period of 1997 to 2002 (Allan and Hart, 2005).

4.1.2 Human Activities Affecting Shoreline Erosion

The existing riprap revetment reduces the shoreline erosion at the site.

4.1.3 Mass Wasting

Weathering, landsliding, recession rates, and other erosional processes at this oceanfront site are discussed in Section 4.0 above and Section 4.2.3 below.

4.1.4 Erosion Potential From Wave Runup Beyond Mean Water Elevation

Coastal erosion rates and hazard zones (as referenced in Priest and Allan, 2004) are presented in Section 4.0 above. In the bluff-backed shoreline recession methodology applicable to the subject site, wave erosion at the bluff toe and associated parameters (rock composition, vegetative/protective cover, ballistics of debris, bluff slope angle of repose, etc.) are more critical to erosion zone and rate estimates than calculating wave runup elevation which changes with many variables such as changing beach elevations, presence of transient dunes, etc. Because of the existing riprap revetment and vegetative cover protecting the bluff slope, only minor erosion is expected with a high wave run-up event at this site, provided that the revetment is repaired and maintained, as needed. It is the chronic nature of the wave attack hazard that undercuts the toe of the bluff, creating bluff instability.

4.1.5 Frequency of Erosion-Inducing Processes

As discussed in Section 4.0 above, the average annual erosion rate for unprotected areas of the bluff north and south of the site is 0.27 ± 0.34 feet per year (Priest and Allan, 2004). Ocean wave, wind, and rain erosion are continuous and ongoing processes that impact bluff recession.

4.1.6 Bluff-Backed Shoreline Erosion Potential

Discussed in Section 4.0 above, including the methodology in Priest and Allan (2004).

4.1.7 Sea Level Rise

Information from NOAA's Garibaldi and Newport/South Beach monitoring stations provides an average sea level rise of approximately 2.18 ± 0.68 mm/year between 1967 and 2019 (NOAA Tides & Currents Sea Level Trends, http://tidesandcurrents.noaa.gov/sltrends). Global climate change can also influence rates of sea-level rise (refer to Section 7.0).

4.1.8 Estimated Annual Erosion Rate

A detailed discussion of recession and estimated erosion rates is in Section 4.0 above; Priest (1994) and Priest et al. (1994) have determined the average annual erosion rate for unprotected bluffs near the site as 0.27 ± 0.34 feet per year. However, provided that the revetment protecting the bluff is repaired and maintained, as needed, we anticipate that future recession caused by ocean wave erosion will be essentially zero. The rate used in our geologic hazard setback analysis was 0.35 feet per year.

4.2 Assessment of Potential Reactions to Erosion Episodes

4.2.1 Legal Restrictions of Shoreline Protective Structures

As noted in Section 2.0 above, the subject site has an oceanfront protective structure. Lots in the Roads End area were generally 'developed' before January 1, 1977; however, this is a legal issue that can have varying interpretations. According to the Ocean Shores Viewer (http://www.coastalatlas.net/oceanshores/, accessed October 2020), the site appears to be Goal 18 eligible for a beachfront protective structure.

4.2.2 Potential Reactions to Erosion Events and Future Erosion Control Measures

Site geologic hazards conclusions and development recommendations are presented in Section 8.0 below, which includes the recommended oceanfront setback for foundations along with a discussion of inherent risks to development in coastal areas with characteristics such as those at the site, as presented and analyzed in Section 4.0 above. Deep foundations, oceanfront protective structures, retaining walls, underpinning of foundations, vegetation management, relocation of structures, and bioengineering can all be potential reactions and control measures to erosion events.

4.2.3 Annual Erosion Rate for the Property

An average annual erosion rate of 0.35feet per year is used in the determination of oceanfront setbacks for the subject site. For further information, please refer to Sections 4.0 and 4.1.8 above.

5.0 Regional Seismic Hazards

Abundant evidence indicates that a series of geologically recent large earthquakes related to the Cascadia Subduction Zone have occurred along the coastline of the Pacific Northwest. Evidence suggests that more than 40 great earthquakes of magnitude 8 and larger have struck western Oregon during the last 10,000 years. The calculated odds that a Cascadia earthquake will occur in the next 50 years range from 7–15 percent for a great earthquake affecting the entire Pacific Northwest to about a 37 percent chance that the southern end of the Cascadia Subduction Zone will produce a major earthquake in the next 50 years (OSSPAC, 2013; OSU News and Research Communications, 2010; Goldfinger et al., 2012). Evidence suggests the last major earthquake occurred on January 26, 1700, and may have been of magnitude 8.9 to 9.0 (Clague et al., 2000; DOGAMI, 2013).

There is now increasing recognition that great earthquakes do not necessarily result in a complete rupture along the full 1,200 km fault length of the Cascadia subduction zone. Evidence in the paleorecords indicates that partial ruptures of the plate boundary have occurred due to smaller earthquakes with moment magnitudes (Mw) < 9 (Witter et al., 2003; Kelsey et al., 2005).

These partial segment ruptures appear to occur more frequently on the southern Oregon coast, as determined from paleotsunami studies. Furthermore, the records have documented that local tsunamis from Cascadia earthquakes recur in clusters (~250–400 years) followed by gaps of 700–1,300 years, with the highest tsunamis associated with earthquakes occurring at the beginning and end of a cluster (Allan et al., 2015).

These major earthquake events were accompanied by widespread subsidence of a few centimeters to 1–2 meters (Leonard et al., 2004). Tsunamis appear to have been associated with many of these earthquakes. In addition, settlement, liquefaction, and landsliding of some earth materials are believed to have been commonly associated with these seismic events.

Other earthquakes related to shallow crustal movements or earthquakes related to the Juan de Fuca plate have the potential to generate magnitude 6.0 to 7.5 earthquakes. The recurrence interval for these types of earthquakes is difficult to determine from present data, but estimates of 100 to 200 years have been given in the literature (Rogers et al., 1996).

Based on the 1999 Relative Earthquake Hazard Map of the Lincoln City area (Madin and Wang, 1999), the subject site lies in an area designated as Zone D, which represents areas having the lowest relative hazards associated with earthquakes. The degree of relative hazard was based on the factors of ground motion amplification, liquefaction, and slope instability, with slope instability being the most critical factor at the subject site.

The subject site is mapped in an area of very strong expected earthquake shaking during an earthquake in a 500-year period (DOGAMI Oregon HazVu website, accessed October 2020). "Very Strong" is the third-highest level of a six-level gradation from "Light" to "Violent" in this mapping system.

6.0 Flooding Hazards

Based on the 2019 Flood Insurance Rate Map (FIRM, Panel #41041C0020E), the eastern portion of the site lies in an area rated as Zone X, which is defined as an area determined to be outside the 0.2% annual chance floodplain. The bluff slope and beach area west of the site is rated as Zone VE (EL 39), which is defined as a coastal flood zone with velocity hazard (wave action), Base Flood Elevations determined.

Based on Oregon Department of Geology and Mineral Industries mapping (DOGAMI, 2013), the beach and bluff slope lie within the tsunami inundation zone resulting from an 8.7 and larger magnitude Cascadia Subduction Zone (CSZ) earthquake. The higher elevation portion of the site east of the bluff slope lies within the tsunami inundation zone resulting from a 9.0 and larger magnitude CSZ earthquake. The 2013 DOGAMI mapping is based upon five computer-modeled scenarios for shoreline tsunami inundation caused by potential CSZ earthquake events ranging in magnitude from approximately 8.7 to 9.1. The January 1700 earthquake event (discussed in Section 5.0 above) has been rated as an approximate 8.9 magnitude in DOGAMI's methodology. More distant earthquake source zones can also generate tsunamis.

7.0 Climate Change

According to most of the recent scientific studies, the Earth's climate is changing as the result of human activities, which are altering the chemical composition of the atmosphere through the buildup of greenhouse gases, primarily carbon dioxide, methane, nitrous oxide, and chlorofluorocarbons (EPA, 1998). Although there are uncertainties about exactly how the Earth's climate will respond to enhanced concentrations of greenhouse gases, scientific observations indicate that detectable changes are underway (EPA, 1998; Church and White, 2006). Global sea-level rise, caused by melting polar ice caps and ocean thermal expansion, could lead to flooding of low-lying coastal property, loss of coastal wetlands, erosion of beaches and bluffs, and saltwater contamination of drinking water. Global climate change and the resultant sea-level rise will likely impact the subject site through accelerated coastal erosion and more frequent and severe flooding. It can also lead to increased rainfall, which can result in an increase in landslide occurrence.

8.0 Conclusions and Recommendations

The main engineering geologic concerns at the site are:

1. The riprap revetment at the base of the slope along the beach is in a state of disrepair. The area of the bluff above the revetment will continue to experience ocean wave erosion and sloughing until the revetment is repaired. We can assist with the oceanfront protection permit application process. There will be an additional cost associated with this.

2. Fill, soft/loose, disturbed, and organic-rich soils approximately one to two feet deep or more are present at the site and will need to be removed from footing and slab areas prior to construction.

3. There is an inherent regional risk of earthquakes along the Oregon Coast, which could cause harm and damage structures. The site also lies within a mapped tsunami inundation hazard zone. A tsunami impacting the Lincoln City area could cause harm, loss of life, and damage to structures. These risks must be accepted by the owner, future owners, developers, and residents of the site.

The following recommendations should be adhered to during design and construction:

8.1 General Recommendations

1. HGSA will need to review a complete plan set for any proposed construction on the lot. The plans will need to incorporate the recommendations included herein. Please note that these recommendations are intended for the construction of a single-family house. Additional recommendations or modifications of the recommendations included herein may be needed depending on the proposed design(s).

2. Carefully control and maintain all stormwater drainage systems at the site. Plan sets should incorporate proper drainage and erosion control, as discussed in Sections 8.4, 8.5, 8.8, 8.9, 8.10, and 8.11 below.

3. A topographic survey performed by a licensed land surveyor will be required by Lincoln City to identify the bluff edge and determine the exact location of the bluff setback. Lincoln City will also require an infiltration test for on-site infiltration of stormwater.

Provided that all recommendations herein are adhered to, no adverse effects are anticipated on adjacent properties.

8.2 Site Preparation and Foundation Setbacks

It is anticipated that excavations at the site can be completed using conventional earth moving equipment. Unsuitable organic-rich, soft, and fill soils should be completely removed from all building areas (refer to Section 8.3 below).

Any tree stumps, including the root systems, should be removed from beneath footing, slab and pavement areas, and the resulting holes backfilled with compacted non-organic structural backfill placed in lifts not exceeding 8 inches and compacted to a dry density of at least 92 percent of the Modified Proctor maximum dry density (ASTM D1557).

If wet weather grading is unavoidable due to construction schedules or wet soil conditions are encountered, stabilization of the subgrade soils with aggregate may become necessary. The use of clean, well-graded 1½ inch minus crushed rock fill (containing less than 5 percent material passing the No. 200 sieve) is recommended. The thickness of the applied granular fill should be sufficient to stabilize the subgrade soils. The use of geotextiles may reduce the applied thickness of granular fills.

To help mitigate future recession of the bluff caused by erosion and landsliding, we recommend that shallow foundations be set back a <u>minimum</u> of approximately 26 feet east of the upper bluff edge. The western edge of foundations using this setback from the upper bluff slope will vary from north to south due to the upper bluff's general south-

southwest trend. This setback would allow room on the subject property to mitigate slope issues should bluff slope erosion occur in the future. We have determined this oceanfront setback based on an average annual erosion rate of 0.35 ft/yr for 60 years and have added Lincoln City's required additional 5 feet.

Please note, the Oregon Coast is a dynamic and energetic environment. Most of the coastline is currently eroding and will continue to erode in the future. Most structures built near ocean bluffs will eventually be undermined by erosion and landsliding. The setback recommendations presented in this report are based on past average erosion rates as determined from aerial photography and past and current geologic conditions and processes. These setbacks are intended to protect the structure(s) from bluff recession for 60 years. Geologic conditions and the rates of geologic processes can change in the future. Setbacks greater than our recommended minimum setbacks would provide the proposed structure with greater anticipated life and lower risk from some geologic hazards.

8.3 Soil Bearing Capacities for Shallow Foundations

Individual and/or continuous spread footings should bear in undisturbed, native, non-organic, medium-stiff/dense to stiff/dense soils, or properly engineered and compacted granular fill placed on these soils. All footing areas should be stripped of all organic and loose/soft soils and existing fills. We anticipate that non-organic, stiff soils will be encountered at depths of approximately 1 to 2 feet. However, depths may vary.

Footings bearing in undisturbed, native, non-organic, firm soils or properly compacted structural fill placed on these soils may be designed for the following:

ALLOWABLE SOIL BEARING CAPACITIES	
Allowable Dead Plus Live Load Bearing Capacity [a]	1,500 psf
Passive Resistance	200 psf/ft embedment depth
Lateral Sliding Coefficient	0.30
[a] Allowable bearing capacity may be increased by one-third for short-term wind or seismic loads.	

Our recommended minimum footing widths and embedment depths are as follows:

MINIMUM FOOTING WIDTHS & EMBEDMENT DEPTHS			
Number of Stories	One	Two	Three
Minimum Footing Width	12 inches	15 inches	18 inches
Minimum Exterior Footing Embedment Depth [a]	12 inches	18 inches	24 inches
Minimum Interior Footing Embedment Depth [b]	6 inches	6 inches	6 inches
[a] All footings shall be embedded as specified above, or extend below the frost line as per Table R301.2(1) of the 2017 ORSC, whichever provides greater embedment. [b] Interior footings shall be embedded a minimum of 6 inches below the lowest adjacent finished grade, or as otherwise recommended by our firm. In general, interior footings placed on sloping or benched ground shall be embedded or set back from cut slopes in such a manner as to provide a minimum horizontal distance between the foundation component and face of the slope of one foot per every foot of elevation change.			

8.4 Slabs-On-Ground

All areas beneath slabs should be excavated a minimum of 6 inches into native, non-organic, firm soils. The exposed subgrade in the slab excavation should be cut smooth, without loose or disturbed soil and rock remaining in the excavation.

SLABS-ON-GROUND	
Minimum thickness of 3/4 inch minus crushed rock beneath slabs	6 inches
Compaction Requirements	92% ASTM D1557, compacted in 8-inch lifts maximum

The slab excavation should then be backfilled with a minimum of 6 inches of ¾ inch minus, clean, free-draining, crushed rock placed in 8-inch lifts maximum, which are compacted to 92 percent of the Modified Proctor (ASTM D1557). Reinforcing of the slab is recommended, and the slab should be fully waterproofed in accordance with structural design considerations. Slab thickness and reinforcing should be determined in accordance with structural considerations. Where floor coverings are planned, slabs should also be underlain by a suitable moisture barrier.

8.5 Retaining Walls

For static conditions, free-standing retaining walls using free-draining granular backfill should be designed for a lateral active earth pressure expressed as an equivalent fluid weight (EFW) of 35 pounds per cubic foot, assuming level backfill. An EFW of 45 pounds per cubic foot should be used, assuming sloping backfill of 2H:1V.

At-rest retaining walls should be designed for a lateral at-rest pressure expressed as an equivalent fluid weight of 60 pounds per cubic foot, assuming level backfill behind the wall equal to a distance of at least half of the height of the wall. Walls need to be fully drained to prevent the build-up of hydrostatic pressures.

The EFWs herein assume static conditions and no surcharge loads from vehicles or structures. If surcharge loads will be applied to the retaining walls, forces on the walls resulting from these loads will need to be added to the pressures given above. For seismic loading, a unit pseudostatic force equal to 12.6 pcf $(H)^2$, where H is the height of the wall in feet, should be added to the static lateral earth pressure. The location of the pseudostatic force can be assumed to act at a distance of 0.6H above the base of the wall.

RETAINING WALL EARTH PRESSURE PARAMETERS	
Static Case, Active Wall (level backfill/grades)	35 pcf [a]
Static Case, Active Wall (2H:1V backfill/grades)	45 pcf [a]
Static Case, At-Rest Wall (level backfill/grades)	60 pcf [a]
Seismic Loading (level backfill/grades)	12.6 pcf $(H)^2$ [b]

[a] Earth pressure expressed as an equivalent fluid weight (EFW).
[b] Seismic loading expressed as a pseudostatic force, where H is the height of the wall in feet. The location of the pseudostatic force can be assumed to act at a distance of 0.6H above the base of the wall.

Free-draining granular backfill for walls should be placed in 8-inch horizontal lifts and machine compacted to 90 percent of the maximum dry density as determined by ASTM D1557. Compaction within 2 feet of the wall should be accomplished with light weight hand-operated compaction equipment to avoid applying additional lateral pressure on the walls. Drainage of the retaining wall should consist of slotted drains placed at the base of the wall on the backfilled side and backfilled with free-draining crushed rock (less than 5% passing the 200-mesh sieve using a washed sieve method) protected by non-woven filter fabric (Mirafi® 140N or equivalent) placed between the native soil and the backfill. Filter fabric protected free-draining crushed rock should extend to within 2 feet of the ground surface behind the wall, and the filter fabric should be overlapped at the top per the manufacturer's recommendations. All walls should be fully drained to prevent the build-up of hydrostatic pressures. All retaining walls should have a minimum of 2 feet of embedment at the toe or be designed without passive resistance. The EFWs provided herein assume that free-draining material (less than 5% passing the 200-mesh sieve on a wet sieve analysis) will be used for the retaining wall backfill.

8.6 Seismic Requirements

The structure and all structural elements should be designed to meet current Oregon Residential Specialty Code (ORSC) seismic requirements. Based on our knowledge of subsurface conditions at the site and our analysis using the guidelines recommended in the ORSC, the structure should be designed to meet the following seismic parameters:

SEISMIC DESIGN PARAMETERS	
Site Class	D
Seismic Design Category	D_2
Mapped Spectral Response Acceleration for Short Periods	S_S = 1.317 g
Site Coefficients	F_a = 1.200 F_v = 1.700
Design Spectral Response Acceleration at Short Periods	S_{DS} = 1.054 g

8.7 Structural Fills

Structural fills should consist of imported, crushed granular material, free of organics and deleterious materials, and contain no particles greater than 1½ inches in diameter so that nuclear methods (ASTM D2922 & ASTM D3017) can be easily used for field density and moisture testing. All areas to receive fill should be stripped of all soft soils, organic soils, organic debris, existing fill, and disturbed soils.

STRUCTURAL FILL	
Compaction Requirements	92% ASTM D1557, compacted in 8-inch lifts maximum, at or near the optimum moisture content.
Benching Requirements [a]	Slopes steeper than 5H:1V that are to receive fill shall be benched. Fills shall not be placed along slopes steeper than 3H:1V, unless approved by H.G. Schlicker & Associates, Inc.
[a] Benches shall be cut into native, non-organic, firm soils. Benches shall be a minimum of 6 feet wide with side cuts no steeper than 1H:1V and no higher than 6 feet. The lowest bench shall be keyed in a minimum of 2 feet into native, non-organic, firm soils.	

Proper test frequency and earthwork documentation usually require daily observation during stripping, rough grading, and placement of structural fill. Field density testing should generally conform to ASTM D2922 and D3017, or D1556. To minimize the number of field and laboratory tests, fill materials should be from a single source and of a consistent character. Structural fill should be approved and periodically observed by HGSA and tested by a qualified testing firm. Test results will need to be reviewed and approved by HGSA. We recommend that at least three density tests be performed for every 18 inches or every 200 cubic yards of fill placed, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor schedule the testing. Relatively more testing is typically necessary on smaller projects.

8.8 Groundwater

Groundwater may be encountered in excavations. If groundwater is encountered, unwatering of the excavation is required and should be the contractor's responsibility. This can typically be accomplished by pumping from one or more sumps, or daylighting excavations to drain.

8.9 Erosion Control

Vegetation should be removed only as necessary, and exposed areas should be replanted following construction. Disturbed ground surfaces exposed during the wet season November 1 through April 30) should be temporarily planted with grasses or protected with erosion control blankets or hydromulch.

Temporary sediment fences should be installed downslope of any disturbed areas of the site until permanent vegetation cover can be established. Unless approved by HGSA, the oceanfront slope should remain undisturbed.

Exposed sloping areas steeper than 3 horizontal to 1 vertical (3H:1V) should be protected with a straw erosion control blanket (North American Green S150 or equivalent) to provide erosion protection until permanent vegetation can be established. Erosion control blankets should be installed as per the manufacturer's recommendations.

8.10 Cut and Fill Slopes

Temporary unsupported cut and fill slopes less than 9 feet in height should be sloped no steeper than 1½ horizontal to 1 vertical (1½H:1V). If temporary slopes greater than 9 feet high are desired, or water seepage is encountered in cuts, HGSA should be contacted to provide additional recommendations. Temporary cuts in excess of 4 feet high and steeper than 1H:1V will likely require appropriate shoring to provide for worker safety, per OSHA regulations. Temporary cuts should be protected from inclement weather by covering them with plastic sheeting to help prevent erosion and/or failure.

If the cut slope recommendations presented below cannot be achieved due to construction and/or property line constraints, temporary or permanent retention of cut slopes may be required, as determined by a representative of HGSA.

TEMPORARY AND PERMANENT CUTS	
Temporary Cuts	1½H:1V (maximum) [a]
Permanent Cuts	2H:1V (maximum) [a]
[a] All cuts greater than 9 feet high, or cuts where water seepage is encountered, shall be approved by a representative of H.G. Schlicker & Associates, Inc.	

Permanent unsupported cut and fill slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V). Fill slopes steeper than 2H:1V should be retained or be mechanically reinforced using geogrids or other suitable products as approved by HGSA. Areas that slope steeper than 5H:1V and are to receive fill should be benched. Benches should be cut into native, non-organic, firm soil. The lowest bench should be keyed a minimum of 2 feet into native, firm soil, and be a minimum of 6 feet wide.

8.11 Drainage

Surface water should be diverted from building foundations and walls to approved disposal points by grading the ground surface to slope away a minimum of 2 percent for 6 feet towards a suitable gravity outlet to prevent ponding near the structures. Permanent subsurface drainage of the building perimeter is recommended to prevent extreme seasonal variation in moisture content of subgrade materials and subjection of foundations and slabs to hydrostatic pressures.

Footing drains should be installed adjacent to the perimeter footings and sloped a minimum of 2.0 percent to a gravity outlet. A suitable perimeter footing drain system would consist of 4-inch diameter, perforated PVC pipe (typical) embedded below and adjacent to the bottom of footings, and backfilled with approved drain rock. The type of pipe to be utilized may depend on building agency requirements and should be verified prior to construction. HGSA also recommends lining the drainage trench excavation with a geotextile filter such as Mirafi® 140N or equivalent to increase the life of the drainage system. The perimeter drain excavation should be constructed in a manner that prevents undermining of foundation or slab components or any disturbance to supporting soils.

In addition to the perimeter foundation drain system, drainage of any crawlspace areas is required. Each crawlspace should be graded to a low point for installation of a drain that is tied into the perimeter footing drain and tightlined to an approved disposal point.

All roof drains should be collected and tightlined in a separate system independent of the footing drains, or an approved backflow prevention device shall be used. All roof and footing drains should be discharged to an approved disposal point. If water will be discharged to the ground surface, we recommend that energy dissipaters, such as splash blocks or a rock apron, be utilized at all pipe outfall locations. Water collected on the site should not be concentrated and discharged to adjacent properties. Water should not be disposed of along the bluff slope unless piped to the toe of the slope.

8.12 Plan Review and Site Observations

We should be provided the opportunity to review all site development, foundation, drainage, and grading plans prior to construction to assure conformance with the intent of our recommendations (Appendix C). The plans, details, and specifications should clearly show that the above recommendations have been implemented into the design.

A representative of HGSA should observe foundation setbacks and site foundation excavations prior to placing structural fill, forming and pouring concrete (Appendix C). Please provide us with at least five (5) days' notice prior to any needed site observations. There will be additional costs for these services.

9.0 Limitations

The Oregon Coast is a dynamic environment with inherent, unavoidable risks to development. Landsliding, erosion, tsunamis, storms, earthquakes, and other natural events can cause severe impacts to structures built within this environment and can be detrimental to the health and welfare of those who choose to place themselves within this environment. The client is warned that, although this report is intended to identify the geologic hazards causing these risks, the scientific and engineering communities' knowledge and understanding of geologic hazards processes is not complete. This report pertains to the subject site only and is not applicable to adjacent sites, nor is it valid for types of development other than that to which it refers. Geologic conditions, including materials, processes, and rates, can change with time. Therefore a review of the site and/or this report may be necessary as time passes to assure its accuracy and adequacy.

The hand augered borings and related information depict generalized subsurface conditions only at these specific locations and at the particular time the subsurface exploration was completed. Soil and groundwater conditions at other locations may differ from the conditions at these locations.

Our investigation was based on engineering geological reconnaissance and a limited review of published information. The data presented in this report are believed to be representative of the site. The conclusions herein are professional opinions derived in accordance with current standards of professional practice, budget, and time constraints. No warranty is expressed or implied. Site-specific performance of this site during a seismic event has not been evaluated. If you would like us to do so, please contact us. This report may only be copied in its entirety.

10.0 Disclosure

H.G. Schlicker & Associates, Inc. and the undersigned Certified Engineering Geologist have no financial interest in the subject site, the project, or the Client's organization.

11.0 References

Allan, J. C. and Hart, R., 2005, A geographical information system (GIS) data set of beach morphodynamics derived from 1997, 1998, and 2002 LIDAR data for the central to northern Oregon coast: Technical Report to the Oregon Department of Land Conservation and Development: Oregon Department of Geology and Mineral Industries, Open-File Report O-05-09, 16 pages.

Allan, J. C., Ruggiero, P., Cohn, N., Garcia, G., O'Brien, F. E., Serafin, K., Stimely, L. L. and Roberts, J. T., 2015, Coastal Flood Hazard Study, Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, Open-File Report O-15-06, 351 p.

Church, J. A., and White, N. J., 2006, A 20th-century acceleration in global sea-level rise: Geophysical Research Letters, v. 22, LO1601, 4 p.

Clague, J. J., Atwater, B. F., Wang, K., Wang, Y., and Wong, I., 2000, Penrose Conference 2000 - Great Cascadia Earthquake Tricentennial, Programs Summary, and Abstracts: Oregon Department of Geology and Mineral Industries, Special Paper 33, 156 p.

DOGAMI, 2013, Tsunami inundation maps for Lincoln City North, Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, TIM-Linc-01, maps.

EPA, 1998, Climate Change and Oregon; Environmental Protection Agency, EPA 236-98-007u, 4 p.

Geomatrix Consultants, 1995, Seismic design mapping, State of Oregon, final report: Prepared for the Oregon Department of Transportation, Project No. 2442.

Goldfinger, C., Kulm, L. D., Yeats, R. S., Appelgate, B., MacKay, M. E., and Cochrane, G. R., 1996, Active strike-slip faulting and folding of the Cascadia Subduction-Zone plate boundary and forearc in central and northern Oregon: U.S. Geological Survey Professional Paper 1560, p. 223-256.

Goldfinger, C., Nelson, C. H., Morey, A. E., Johnson, J. E., Patton, J. R., Karabanov, E., Gutiérrez-Pastor, J., Eriksson, A. T., Gràcia, E., Dunhill, G., Enkin, R. J., Dallimore, A., and Vallier, T., 2012, Turbidite event history—Methods and implications for Holocene paleoseismicity of the Cascadia subduction zone: U.S. Geological Survey Professional Paper 1661–F, 170 p.

Kelsey, H. M., Nelson, A. R., Hemphill-Haley, E., and Witter, R. C., 2005, Tsunami history of an Oregon coastal lake reveals a 4600 yr record of great earthquakes on the Cascadia subduction zone: Geological Society of America Bulletin, v. 117, no. 7/8, p. 1009-1032.

Kelsey, H. M., Ticknor, R. L., Bockheim, J. G., and Mitchell, C. E., 1996, Quaternary upper plate deformation in coastal Oregon: Geological Society of America Bulletin, v. 108, no. 7, p. 843-860.

Komar, P. D., 1997, The Pacific Northwest coast: living with the shores of Oregon and Washington. Durham: Duke University Press.

Leonard, L. J., Hyndman, R. D., and Mazzotti, S., 2004, Coseismic subsidence in the 1700 great Cascadia earthquake: Coastal estimates versus elastic dislocation models: Geological Society of America Bulletin, May/June 2004, v. 116, no. 5/6, pp. 655–670.

Madin, I. P., and Wang, Z., 1999, Relative earthquake hazard maps for selected urban areas in western Oregon: Oregon Department of Geology and Mineral Industries, Interpretive Map Series IMS-10.

Oregon Seismic Safety Policy Advisory Commission (OSSPAC), February 2013, The Oregon Resilience Plan: Reducing Risk and Improving Recovery for the next Cascadia Earthquake and Tsunami—Report to the 77th Legislative Assembly: State of Oregon Office of Emergency Management, 341 p.

OSU News and Research Communications, May 24, 2010, Odds are 1-in-3 that a huge quake will hit Northwest in next 50 years: Oregon State University, Corvallishttp://oregonstate.edu/ua/ncs/archives/2010/may/odds-huge-quake-Northwest-next-50-years

Personius, S. F., Dart, R. L., Bradley, L-A, Haller, K. M., 2003, Map and data for Quaternary faults and folds in Oregon: U.S. Geological Survey, Open-File Report 03-095, 556 p., map.

Priest, G. R., and Allan, J. C., 2004, Evaluation of Coastal Erosion Hazard Zones Along Dune and Bluff Backed Shorelines in Lincoln County, Oregon: Cascade Head to Seal Rock, Technical Report to Lincoln County: Oregon Department of Geology and Mineral Industries, Open-File Report O-04-09, 202 pages.

Priest, G. R., 1994, Chronic geologic hazard map of the Lincoln City-Wecoma Beach Area, Coastal Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, Open-File Report O-94-14, map.

Priest, G. R., Saul, I., and Diebenow, J., 1994, Explanation of chronic geologic hazard maps and erosion rate database, coastal Lincoln County, Oregon: Salmon River to Seal Rock: Oregon Department of Geology and Mineral Industries, Open-File Report 0-94-11, 45 p.

Rogers, A. M., Walsh, T. J., Kockelman, J., and Priest, G. R., 1996, Earthquake hazards in the Pacific Northwest - an overview: U.S. Geological Survey, Professional Paper 1560, p. 1- 54.

Schlicker, H. G., Deacon, R. J., Olcott, G. W., and Beaulieu, J. D., 1973, Engineering geology of Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, Bulletin 81.

Witter, R. C., Kelsey, H. M., and Hemphill-Haley, E., 2003, Great Cascadia earthquakes and tsunamis of the past 6700 years, Coquille River estuary, southern coastal Oregon: Geological Society of America Bulletin, v. 115, p. 1289-1306.

It has been our pleasure to serve you. If you have any questions concerning this report or the site, please contact us.

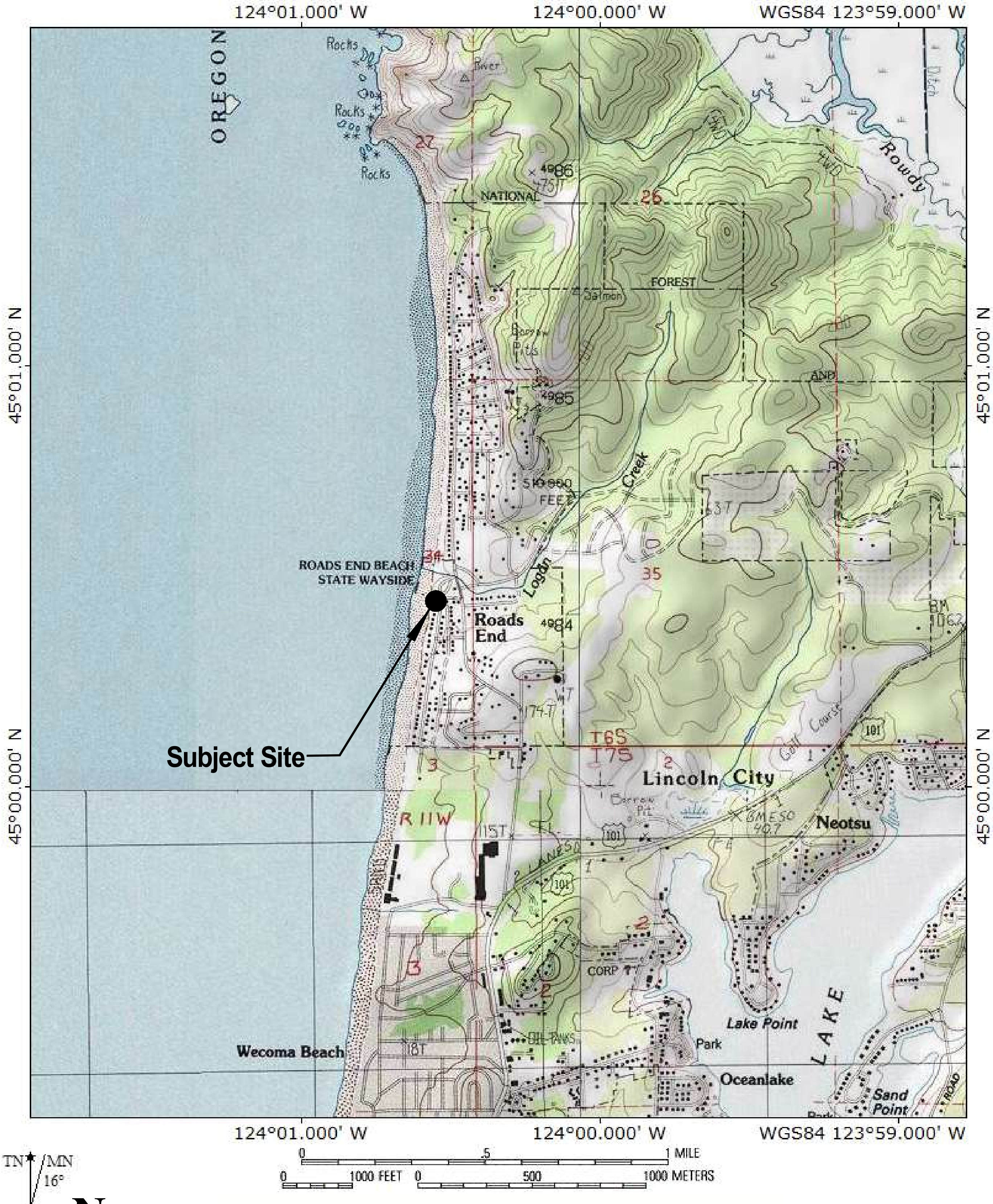
Respectfully submitted,

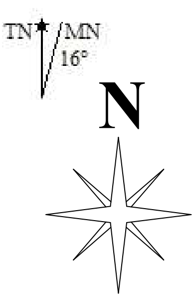
H.G. SCHLICKER AND ASSOCIATES, INC.

EXPIRES: 10/31/2021

J. Douglas Gless, MSc, RG, CEG, LHG
President/Principal Engineering Geologist

JDG:aml

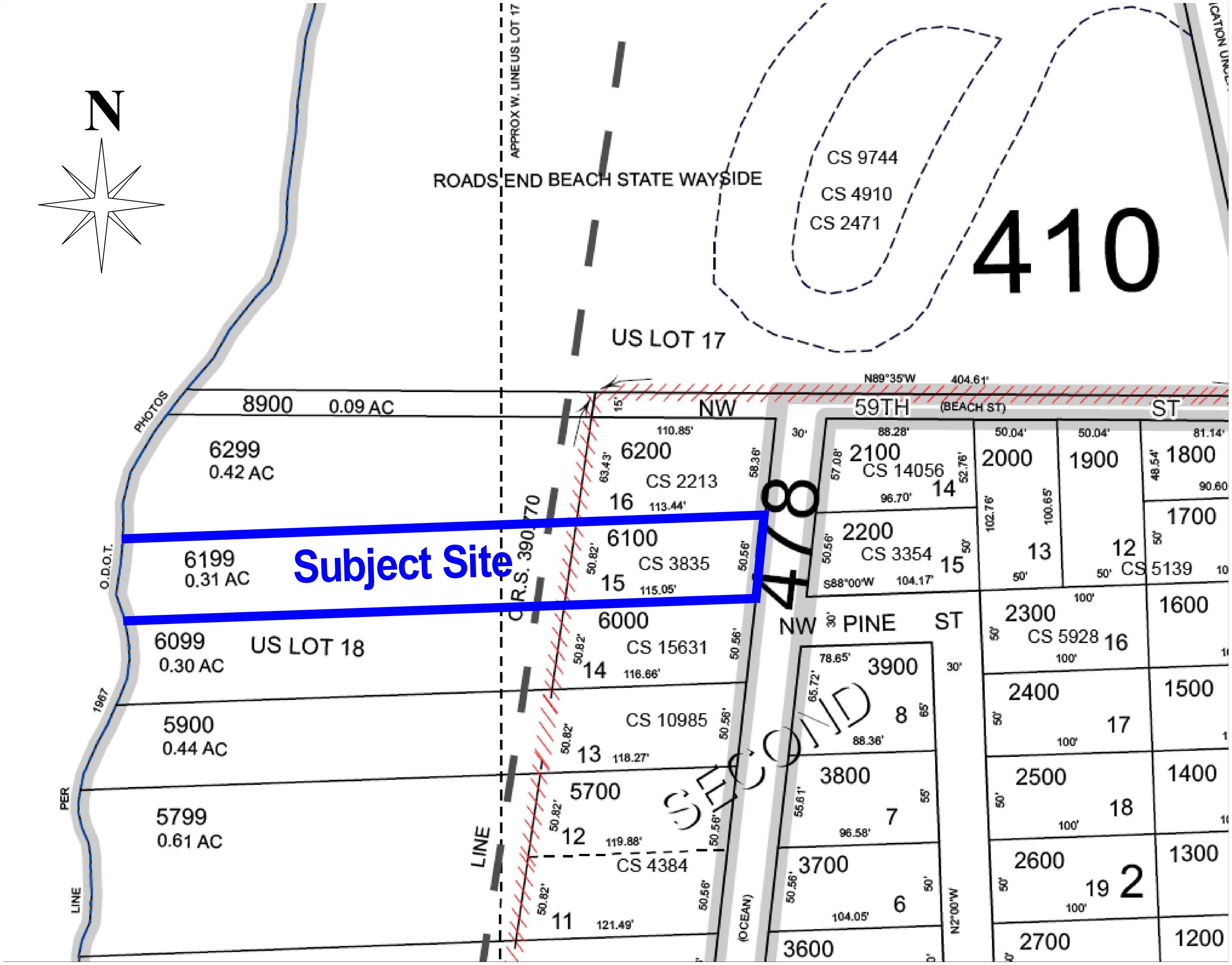




Date: 10/23/2020	**Project #Y204427**	Prepared by: AML
Scale: 1" = 2000'		Approved by: JDG

Location Map
Tax Lots 6100 and 6199, Map 6-11-34DA
N.W. Jetty Avenue, Lincoln City, Oregon

H.G. Schlicker & Associates, Inc. | **Figure 1**



Date: 10/23/2020	Project #Y204427	Prepared by: AML
Scale: 1" = 50'		Approved by: JDG

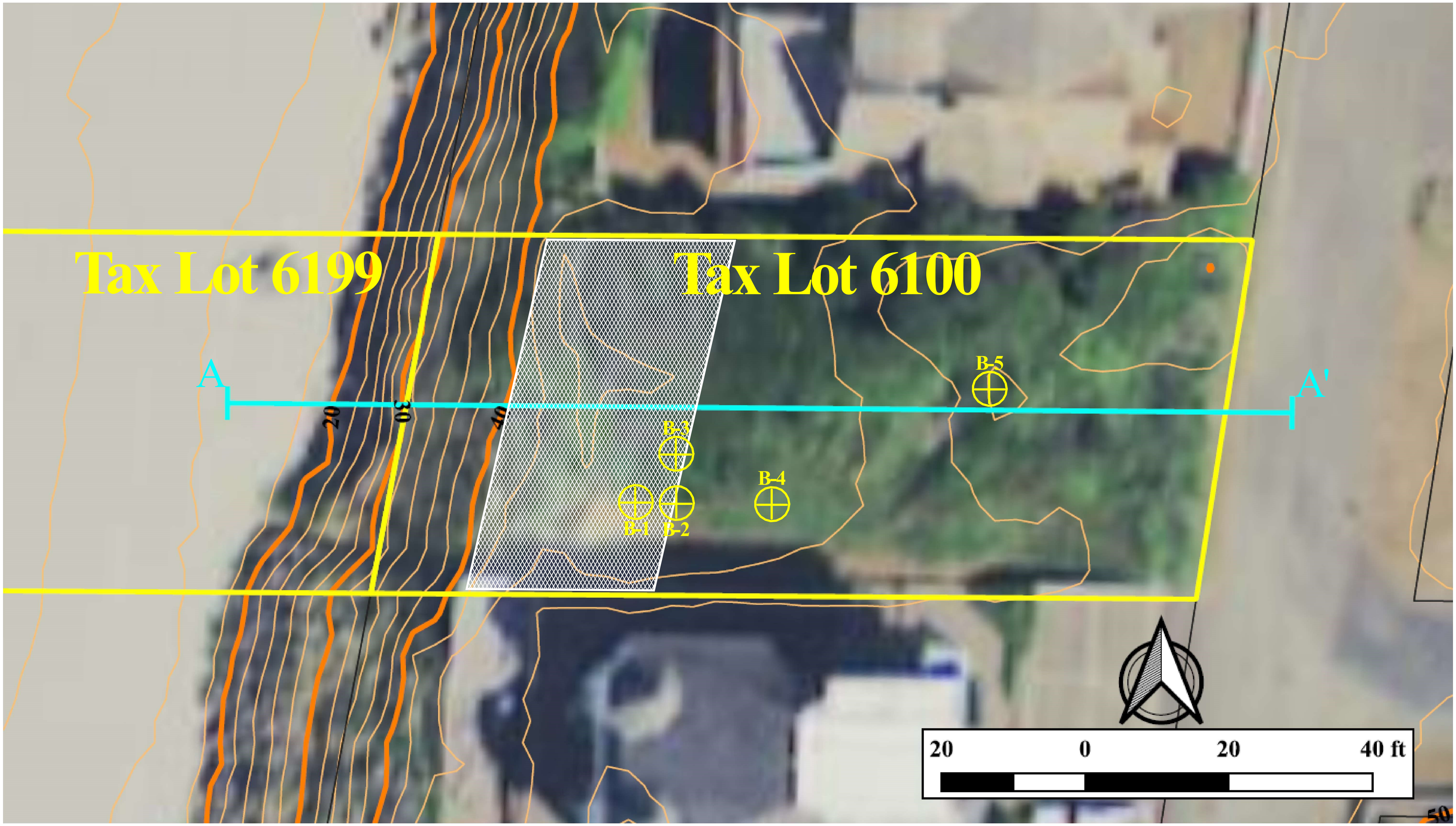
Plat Map

Tax Lots 6100 and 6199, Map 6-11-34DA
N.W. Jetty Avenue, Lincoln City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 2

Modified from the Lincoln County assessor's plat, 6-11-34DA
All locations and dimensions are approximate.



B-1

= Approximate location of boring

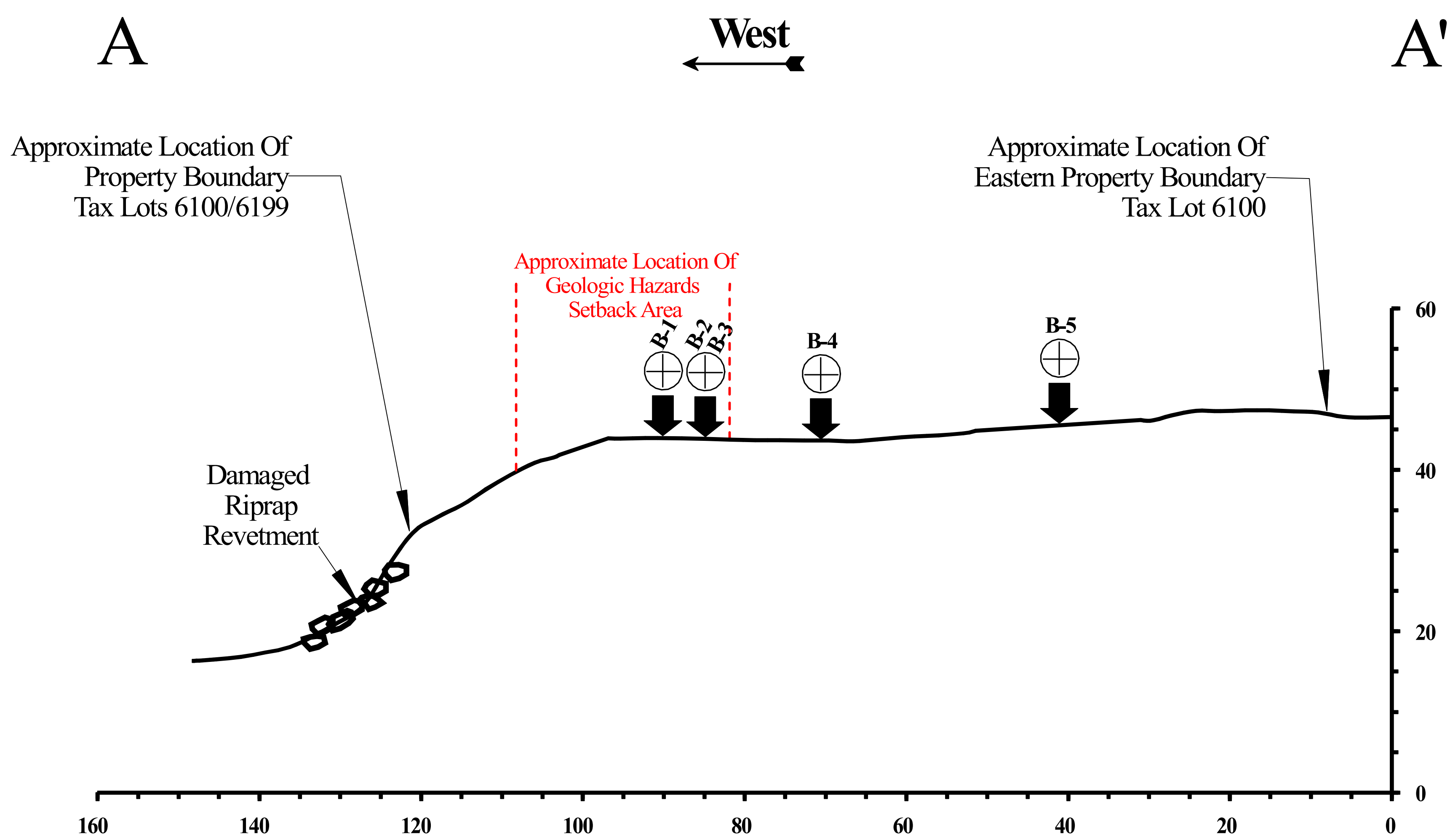
= Geologic Hazards Setback Area

A A'

= Approximate trend of profile line

Imagery provided Google. Topographic data derived from 2009 OLC North Coast lidar provided by DOGAMI. All locations and dimensions are approximate. Slope break and setback location should be confirmed in the field.

Date: 10/23/2020 Scale: 1" = 20'	**Project #Y204427**	Prepared by: AML Approved by: JDG
Site Topographic Map Tax Lots 6100 and 6199, Map 6-11-34DA N.W. Jetty Avenue, Lincoln City, Oregon		
H.G. Schlicker & Associates, Inc.		**Figure 3**



B-1

= Approximate location of borehole

All dimensions, elevations and locations are approximate.
Slope profile derived and modified from 2009 OLC North Coast Lidar provided by DOGAMI.

Date: 10/23/2020 Scale: 1" = 20'	**Project #Y204427**	Prepared by: AML Approved by: JDG
Slope Profile, A-A' Tax Lots 6100 and 6199, Map 6-11-34DA N.W. Jetty Avenue, Lincoln City, Oregon		
H.G. Schlicker & Associates, Inc.		**Figure 4**

Appendix A
– Site Photographs –

Photo 1 – Westerly view of the site from N.W. Jetty Avenue.

Photo 2 – Easterly view of the site from the beach.

Photo 3 – Northerly view along the beach and the Pacific Ocean with Cascade Head in the distance.

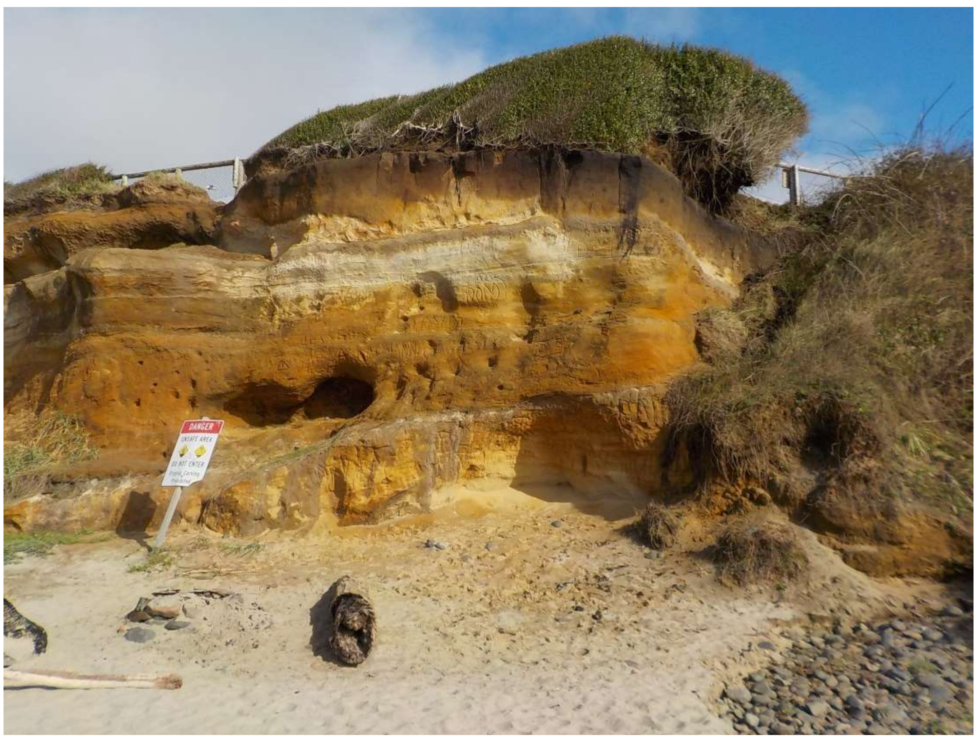


Photo 4 – View of the marine terrace sand and overlying soil exposed at Roads End State Recreation Site.


Photo 5 – View of soils encountered in boring B-4.


Photo 6 – View of the damaged riprap revetment exposed at the base of the bluff slope.

Appendix B
– Boring Logs –

Project #Y204427

HAND AUGERED BORING LOG EXPLANATION

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS), ASTM D2487			
MAJOR DIVISIONS		GROUP SYMBOL *	GROUP NAME
COARSE-GRAINED SOILS	GRAVELS	GW	Well-graded gravel
		GP	Poorly-graded gravel
		GM	Silty gravel
		GC	Clayey gravel
	SANDS	SW	Well-graded sand
		SP	Poorly-graded sand
		SM	Silty sand
		SC	Clayey sand
FINE-GRAINED SOILS	SILTS AND CLAYS Liquid Limits Less than 50	ML	Silt with low plasticity
		CL	Clay with low plasticity
		OL	Organic silt or organic clay with low plasticity
	SILTS AND CLAYS Liquid Limits 50 or more	MH	Silt with high plasticity
		CH	Clay with high plasticity
		OH	Organic silt or organic clay with high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, Muck, and other highly organic soils.

* NOTE: the symbol RK (not within the USCS system) is used in our logs to denote rock materials.

BORING LOGS

B-1

Depth (ft.)	USCS	Description
0 – 0.5	ML (Fill)	Sandy SILT FILL; dark reddish brown, wet, loose. With numerous small roots and sub-angular basaltic rock fragments to approximately 2" diameter.
		Boring met refusal at approximately 6" below ground surface on rock clasts. Free groundwater was not encountered.

B-2

Depth (ft.)	USCS	Description
0 – 0.5	ML (Fill)	Sandy SILT FILL; dark reddish brown, wet, loose. With numerous small roots and sub-angular basaltic rock fragments to approximately 2" diameter.
		Boring met refusal at approximately 6" below ground surface on rock clasts. Free groundwater was not encountered.

B-3

Depth (ft.)	USCS	Description
0 – 0.5	ML (Fill)	Sandy SILT FILL; dark reddish brown, wet, loose. With numerous small roots and sub-angular basaltic rock fragments to approximately 2" diameter.
		Boring met refusal at approximately 6" below ground surface on rock clasts. Free groundwater was not encountered.

B-4

Depth (ft.)	USCS	Description
0 – 1.5	ML (Fill)	Sandy SILT FILL; dark reddish brown, wet, loose. With numerous small roots near the surface and sub-angular basaltic rock fragments to approximately 2" diameter.
1.5 – 1.75	SP (Fill)	Slightly Silty SAND FILL; light brown/tan, wet, loose.

BORING LOGS (continued)

B-4 (continued)

Depth (ft.)	USCS	Description
1.75 – 2.25	ML	Sandy SILT;dark brown, wet, soft to medium stiff. With decaying organic material.
2.25 – 3.5	ML	Sandy SILT; brown, wet, medium dense.
3.5 – 4.0	SM	SILTY SAND; light brown, wet, medium dense.
4.0 – 4.5	SP	SAND; Tan, wet, medium dense to dense. Slightly to moderately cemented with orange iron staining.
		Boring terminated at approximately 4.5' below ground surface in moderately cemented sand. Free groundwater was not encountered.

B-5

Depth (ft.)	USCS	Description
0 –0.75	ML (Fill)	Sandy SILT FILL; dark reddish brown, wet, loose. With numerous small roots near the surface and sub-angular basaltic rock fragments to approximately 2" diameter.
0.75– 1.0	SP (Fill)	Slightly Silty SAND FILL; light brown/tan, wet, loose.
1.0 – 2.0	ML	Sandy SILT;dark brown, wet, soft to medium stiff. With decaying organic material.
2.0 – 3.0	ML	Sandy SILT; brown, wet, medium dense.
3.0 – 3.5	SP	SAND; Orange, wet, medium dense to dense. Moderately cemented with orange iron staining.
		Boring terminated at approximately 3.5' below ground surface in moderately cemented sand. Free groundwater was not encountered.

Appendix C

– Checklist of Recommended Plan Reviews and Site Observations –

Project #Y204427

APPENDIX C

Checklist of Recommended Plan Reviews and Site Observations

To Be Completed by a Representative of H.G. Schlicker & Associates, Inc.

Item No.	Date Done	Procedure	Timing
1*		Review site development, foundation, drainage, grading, and erosion control plans.	Prior to permitting and construction.
2*		Observe foundation excavations.	Following excavation of foundations, and prior to placing fill, forming and pouring concrete. **
3*		Review Proctor (ASTM D1557) and field density test results for all fills placed at the site.	During construction.

* There will be additional charges for these services.

** Please provide us with at least 5 days' notice prior to all site observations.