Geologic Hazards and Geotechnical Investigation
Tax Lot 1100, Map 7-11-10AB
2715 NW Inlet Avenue
Lincoln City, Oregon

Prepared for:
William Christensen

Project #Y244738 July 17, 2024

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 #Y244738 July 17, 2024

To: **William Christensen**
14597 Kingscross Circle NW
Silverdale, WA 98383-7996

Subject: **Geologic Hazards and Geotechnical Investigation**
Tax Lot 1100, Map 7-11-10AB
2715 NW Inlet Avenue
Lincoln City, Oregon

Dear Mr. Christensen:

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.

Adam M. Large, MSc, RG, CEG
President/Principal Engineering Geologist

AML:mgb

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Dear Mr. Christensen:

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 June 19, 2024, to complete a geologic hazards and geotechnical investigation of Tax Lot 1100, Map 7-11-10AB, at 2715 NW Inlet Avenue in the Wecoma Beach area of Lincoln City, Oregon (Figures 1 and 2; Appendix A). It is our understanding that you are working with a contractor to construct an addition on the east side of the existing house and garage.

This report addresses the engineering geology and geologic hazards at the site with respect to constructing an addition at the site. The scope of our work consisted of a site visit, site observations and measurements, hand augered borings, a slope profile, a 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 lies in the Wecoma Beach area of Lincoln City, Oregon (Figure 1). The existing house is located adjacent to a steep oceanfront bluff slope on a marine terrace approximately 30 feet above the beach level. The site is bounded to the west by the beach and the Pacific Ocean, to its north and south by developed hotel/motel properties, and to its east by NW Inlet Avenue. The lot measures approximately 50 feet wide, north to south and up to approximately 360 feet long, east to west (Figure 2); however, the Statutory Vegetation Line is mapped approximately 290 feet west of the eastern property line, and the actual vegetation line at the base of the bluff is approximately 200 feet west of the eastern property line.

Vegetation surrounding the house at the site consists of lawn grass and ornamental shrubs. The vegetation on the bluff slope consists of salal, English ivy, grasses, and other brush (Appendix A).

2.1 Proposed Development

Based on the information provided to us, you plan to add an addition to the eastern portion of the house and the attached garage.

We have provided geotechnical recommendations for the design of an east-side addition in Sections 8.1 through 8.12 below. HGSA should be contacted to review development plans for the site. HGSA may also need to provide additional recommendations or investigation (if needed) as appropriate to the proposed development of the site. If any construction is to occur west of the footprint of the existing house, additional investigation and recommendations will be required. There will be additional charges for these services.

2.2 History of The Site and Surrounding Areas

According to Lincoln County records, the two-story house with a finished attic and attached garage was built in 1938. The closest portion of the western edge of the attached two-story deck framing is currently located approximately 33 feet east of the top of the upper bluff edge (Figure 4; Appendix A). The east side of the attached garage is approximately 77 feet east of the bluff edge.

Reportedly, portions of the existing foundation have been underpinned with helical piers. However, that work appears to have been completed by others and is not within the scope of this investigation.

An asphalt driveway provides access from NW Inlet Avenue to an older exposed aggregate concrete apron at the garage door (Appendix A).

The subject property has an oceanfront protective structure consisting of a riprap revetment in an area of bluffs that generally have oceanfront protective structures; however, the structures' age, type, and condition vary. Oceanfront protection continues to the lot immediately north and several lots to the south of the subject lot (Appendix A).

According to the Oregon Coastal Atlas Ocean Shores Data Viewer (http://www.coastalatlas.net/oceanshores, accessed July 2024), the lot appears eligible for a beachfront protective structure on the Goal 18 Eligibility Inventory.

2.3 Site Topography, Elevations, and Slopes

The area of the subject site east of the top of the bluff slope is gently sloping between approximately 1 and 6 degrees, and the elevation ranges between approximately 50 and 54 feet (NAVD 88) (Figures 3 and 4).

2.4 Vegetation Cover

The upper portion of the site around the house is vegetated with landscape plants and grass. The vegetation on the bluff slope consists of salal, English ivy, grasses, and other 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.

2.6 Site Oceanfront Conditions

The site is located along an oceanfront bluff slope consisting primarily of marine terrace sands and colluvium. The bluff at the site and many properties along this stretch of beach currently have oceanfront protective structures. An older riprap revetment consisting of breccia stone is present at the site fronted by a transient vegetated foredune. 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 did not observe a substantial accumulation of drift logs or flotsam in the beach area. Satellite imagery indicates that little accumulation of driftwood and flotsam occurs in the vicinity.

2.8 Streams or Drainage and Influence on Beach Elevations

We did not observe streams in the vicinity of the site. The nearest major stream is the D River, approximately 1.1 miles south of the site. Logan Creek discharges onto the beach approximately 1.6 miles north of the site at Roads End State Park.

Along this stretch of beach, culverts and smaller drainages convey water to the beach from the upland.

2.9 Headland Proximity and Influence on Beach Sediment Transport and Elevations

Headlands are present approximately 2.8 miles north of the site at the end of Logan Road. 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 north and Government Point, approximately 10.9 miles to the south (Komar, 1997).

Smaller rock outcrops and reefs near the shoreline appear to have influenced the seasonal/periodic formation of rip currents and embayments along this section of the coast.

2.10 Shore Protection Structures

A riprap revetment is present at the site and extends several hundred feet to the south of the site (Appendix A).

2.11 Beach Access Pathways

Public beach access is present approximately 290 feet south at the western end of NW 26th Street.

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

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

3.0 Geologic Mapping, Investigation and Descriptions

3.1 Geology

The site lies in an area that has been mapped as Quaternary marine terrace deposits, which consist of semi-consolidated, fine- to medium-grained, uplifted beach sand commonly overlain by fine-grained stabilized dune deposits (Schlicker et al., 1973). The marine terrace deposits generally consist of unconsolidated to semi-consolidated uplifted gravel, beach, and dune sand with local minor consolidated clay-rich paleosol, colluvium, debris flows, and alluvial interbeds, overlain locally by fine-grained dune deposits (Priest and Allan, 2004). The uplifted marine terrace sediments are typically high-energy nearshore marine deposits capped by beach sand (Kelsey et al., 1996).

The marine terrace deposits mantle wave-cut benches of tilted strata of lower Eocene Nestucca Formation. The Nestucca Formation consists of thin-bedded, tuffaceous siltstone and sandstone with ash and glauconitic sandstone interbeds (Schlicker et al., 1973). At the site, the Nestucca Formation is below beach level.

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 200 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 transporting sediment. 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 at the site is comprised of primarily fine-grained to lesser medium-grained sand.

3.2.3 Summer and Winter Beach Elevations and Average Slopes

The beach slopes west at approximately 2 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 approximately 4 feet from minimum to maximum (Allan and Hart, 2005). The beach elevation can change substantially associated with El Niño and La Niña events. Topographic contours derived from 2016 lidar data provided by NOAA 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

We did not observe rip currents or rip embayments in this area at the time of our site visit. Rip currents and embayments can form, however, as evidenced by our review of historical aerial imagery along this general area of the coast.

3.2.5 Offshore Rock Outcrops and Sea Stacks

Rock outcrops or sea stacks are not present in the immediate vicinity of the site. Mapping by Priest (1994) shows possible Basalt of Cascade Head outcrops near the shoreline approximately 1,000 feet and greater to the south-southwest of the site.

3.2.6 Depth of Beach Sand to Bedrock

During our site visit, we did not observe any exposed bedrock on the beach. However, we estimate sand and cobble depths along the beach at this time to be about 6 feet thick.

3.3 Subsurface Conditions

At the time of our site visit, we explored the subsurface by advancing two hand-augered borings up to a depth of approximately 5 feet below the ground surface (bgs), and we used a tile probe to estimate depths to dense material in areas closer to utilities and other existing improvements. The approximate location of the boring is shown on Figures 3 and 4. A geologist from our office visually classified the soils encountered according to the Unified Soil Classification System (USCS) as follows:

B-1	Depth (ft.)	USCS	Description
	0 – 3.0	ML (Fill)	SILT FILL; dark black, moist, loose. Organic rich with variable sand content throughout
	3.0 – 4.0	SM	Silty SAND; brown, wet, loose, with minor organics.
	4.0 – 5.0	SM	Silty SAND; brown, moist, loose to medium dense. With isolated seams of black silt. Free groundwater was not encountered.

B-2	Depth (ft.)	USCS	Description
	0 – 1.6	ML (Fill)	SILT FILL; dark black, moist, loose. Organic rich with variable sand content throughout. River rock landscaping at the surface.
	1.6 – 3.0	SM	Silty SAND; brown, moist, medium dense to dense. Non-cemented.

In general, we encountered loose sandy silt fill and organic-rich soils to depths up to approximately 3 feet, overlying loose to medium dense silty sand in the area southwest and southeast of the garage. We also probed the site with a tile probe to depths up to approximately 4 feet in areas of the proposed addition where the presence of utilities was suspected; we encountered highly variable resistance over a short distance.

3.4 Geologic 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).

Several other generally parallel faults that trend in a southwesterly direction toward Siletz Bay have been mapped within approximately 1 to 3 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 21 miles south of the site, and the Yaquina Bay Fault, located approximately 24 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 site is located on a marine terrace with a steep oceanfront bluff slope that formed due to ocean wave erosion and undergoes continuous wind and rain impacts, sloughing, and landsliding. Transient dunes that have formed at the beach-bluff junction can be eroded by a single storm.

The western portion of the site is mapped in an area of moderate to high landslide susceptibility based on the DOGAMI methodology (Burns, Mickelson, and Madin, 2016). However, the relatively short height of the bluff and the existing revetment protecting the base of the bluff slope from erosion reduces the potential for landsliding.

The site lies in a coastal area mapped as experiencing critical erosion of marine terraces and sediments (Schlicker et al., 1973). Priest et al. (1994) did not determine an erosion rate at the site due to the presence of the oceanfront protective structure. However, the erosion rate at unprotected oceanfront bluffs north of the site was determined to be 0.27 ± 0.34 feet per year.

This erosion rate was calculated by measuring the distance between existing structures and the bluff and compared to distances measured on a 1939 or 1967 vertical aerial photograph.

Coastal erosion is episodic, with some years of little to no erosion and other years of more severe erosion.

Based on mapping completed by Priest and Allan (2004), generally, the existing home lies in areas designated as Moderate and Low-Risk Coastal Erosion Hazard Zones. The area west of the home and east of the bluff edge appears to lie in the High Coastal Erosion Hazards Zone. The bluff slope and beach lie within the Active Coastal Erosion Hazard Zone. A site within the High-Risk Hazard Zone has a high probability that the area could be affected by active erosion in the next approximately 60 to 100 years; a site within the Moderate-Risk Hazard Zone has a moderate probability that the area could be affected by active erosion in the next approximately 60 to 100 years, and a site within the Low-Risk Hazard Zone has a low but significant probability that the area could be affected by active erosion in the next approximately 60 to 100 years (Priest and Allan, 2004). 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) <u>Active hazard zone:</u> 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 the 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

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 adverse effects of 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 4.1.8 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 the existing riprap protects the base of the bluff slope, only minor erosion, if any, is expected with a high wave run-up event at this site. The existing dense vegetation above and below the revetment indicates that it has not been recently overtopped. However, it is the chronic nature of the wave attack hazard that undercuts the toe of the bluff, creating bluff instability, and over an extended period of time, has the potential to undermine the revetment.

4.1.5 Frequency of Erosion-Inducing Processes

As discussed in Section 4.0 above, the average annual erosion rate for the unprotected area 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 2.15 ± 0.61 mm/year between 1967 and 2023 (NOAA Tides & Currents Sea Level Trends, http://tidesandcurrents.noaa.gov/sltrends, Accessed April 2023). 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) did not determine an erosion rate at the site but has determined the average annual erosion rate for unprotected areas of the bluff north and south of the site as 0.27 ± 0.34 feet per year.

4.2 Assessment of Potential Reactions to Erosion Episodes

4.2.1 Legal Restrictions of Shoreline Protective Structures

According to the Ocean Shores Viewer (http://www.coastalatlas.net/oceanshores/, accessed July 2024), the site appears to be Goal 18 eligible for a beachfront protective structure.

As noted in Section 2.0 above, the subject site has an oceanfront protective structure. The subject site and the lots to the north and south of the subject site were generally 'developed' before January 1, 1977; however, this is a legal issue that can have varying interpretations.

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, including the recommended oceanfront setback for foundations and 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.

When older revetments are severely damaged, replacing them with a new revetment using a modern design and configuration is recommended.

4.2.3 Annual Erosion Rate for the Property

The mapped annual erosion rate for the property is effectively 0 feet/yr due to the existing rip rap revetment. 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 (CSZ) 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 probability that a Cascadia earthquake and Tsunami will occur in the next 50 years ranges from 7 to 12 percent for a complete rupture affecting the entire fault zone, 16 to 22 percent for a partial rupture that impacts the Oregon and northern California coast, and, 37 to 43 percent chance for a partial rupture that would affect just the southern Oregon and northern California coast (OSU News and Research Communications, 2010; Goldfinger et al., 2012; OSSPAC, 2013; Allan and Gabel, 2022). 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, 2012).

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 to 400 years) followed by gaps of 700 to 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 to 2 meters (Leonard et al., 2004). More recent modeling by Witter et al. (2011) provides estimates for coastal subsidence that range from 3 to 6 feet for L (~9.0 Mw) to XXL

(~9.1 Mw) earthquakes and approximately 2 feet of subsidence along the central Oregon coast resulting from the most likely (M1) earthquake scenario (Allan and Gabel, 2022). With the lowering of the coast, significant coastal erosion of beaches, dunes, and bluffs, lasting years to decades, is anticipated as the coastline strives to reach a new equilibrium (Allan and Gabel, 2022). 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 that show the lowest relative hazard 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 expected amount of shaking to be felt at the site if a magnitude 9.0 CSZ earthquake occurs has been mapped as severe (DOGAMI Oregon HazVu website (https://gis.dogami.oregon.gov/maps/hazvu/), accessed July 2024). "Severe" is the second 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 #41041C0107E), the existing house and the upper bluff area lie in an area rated as Zone X, defined as an area of minimal flood hazard. The central and lower bluff slope and beach to the west lie in an area rated as Zone VE (EL 36), which is defined as a special flood hazard area with base flood elevations determined. Also known as coastal high-hazard areas, Zone VE flood zones are subject to the 1% annual chance (base) flood limits and wave effects 3 feet or greater.

Based on the Oregon Department of Geology and Mineral Industries mapping (DOGAMI, 2013), the upper bluff slope and existing house at the subject site lie within the tsunami inundation zone resulting from an approximately 9.0 and greater magnitude Cascadia Subduction Zone (CSZ) earthquake. The 2013 DOGAMI mapping is based upon 5 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 approximately 30-foot high bluff along the western part of the site is undergoing recession as a result of continuous ocean wave impacts, wind and rain erosion, sloughing and landsliding.
2. Uncontrolled fill and soft soils, at least 2 feet to approximately 4 feet thick or more, are present in the area proposed for the addition. The native dune sands at the site can be poorly consolidated.
3. The western portion of the subject site is mapped within a FEMA special flood hazard area and is subject to flooding.
4. There is an inherent regional risk of earthquakes and tsunamis 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 an

addition to the eastern portion of the existing house. Additional recommendations or modifications of the recommendations included herein may be needed depending on the proposed design(s). If modifications to the existing structure contribute substantially greater loads to the existing foundations, additional geotechnical investigation, analysis, foundation design and construction recommendations may be required.

2. The site and nearby vicinity along the bluff should be monitored for signs of ground movement, sloughing, increased bluff recession, and sudden/rapid erosion events, particularly during times of heavy precipitation, inclement or severe weather, and major storms. The condition of the riprap revetment at the base of the bluff should be monitored, maintained and repaired as needed.
3. Carefully control and maintain all stormwater drainage systems at the site. Plan sets should incorporate drainage and erosion control, as discussed in Sections 8.4, 8.5, 8.8, 8.9, 8.10, and 8.11 below.
4. Lincoln City may require a topographic survey performed by a licensed land surveyor to identify the bluff edge and determine the bluff setback's location to be used in planning, permitting, and construction.

The presence of fill and soft soil in the proposed construction area and uncertainties regarding the construction of the existing foundations indicate that modifications to the existing eastern foundations may be necessary to ensure adequate structural support for the proposed addition.

We recommend that the existing foundations be adequately supported during construction or excavation to prevent settlement. Temporary support of the existing structure may be necessary during excavation and construction.

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

8.2 Site Preparation and Foundation Setbacks

All new footing and slab areas shall be stripped of all organic, disturbed, and loose/soft soils, existing fills, and debris. We anticipate that non-organic, suitable sandy soils will be encountered at depths of 2 to 4 feet; however, depths may vary.

Sandy soils within the footprint of the proposed construction should be moisture-conditioned and compacted to a dense state utilizing a vibratory compactor. Over-excavation and replacement with structural fill may be necessary to achieve the desired final grades.

We have calculated a setback using an average annual erosion rate of 0.44 ft/yr, which consists of the mapped erosion rate for unprotected bluff plus approximately ½ of the error determined by Priest and Allan (2004); Lincoln City's required additional 5 feet was added to this. The setback for new shallow foundations, as measured east from the upper edge of the bluff slope, should be a minimum of 30 horizontal feet, as shown on Figures 3 and 4. New foundations more than approximately 30 feet and greater from the upper bluff edge can utilize standard continuous and isolated shallow spread footings on dense sandy soils or structural fill placed on these sandy soils.

Please note that 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

All new footing areas should be stripped of all organic and loose/soft soils and existing fills. Footings bearing on 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.	

We recommend any additions be constructed with an elevated floor and crawlspace design. For conventional light-frame construction*, our recommended minimum widths and embedment depths for continuous footings are as follows:

MINIMUM FOOTING WIDTHS & EMBEDMENT DEPTHS		
Number of Stories	One	Two
Minimum Footing Width	12 inches	15 inches
Minimum Exterior Footing Embedment Depth [a]	15 inches	18 inches
Minimum Interior Footing Embedment Depth [b]	6 inches	6 inches
[a] If foundations will be placed along or immediately adjacent to slopes steeper than 3H:1V, foundation embedments will need to be a minimum of 24 inches, as approved by a representative of our firm. [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.		

*Please contact us for additional recommendations if brick veneer, hollow concrete masonry, or solid concrete or masonry wall construction is incorporated into the design of the house.

Isolated footings should meet Section R403.1.7 of the 2023 Oregon Residential Specialty Code (ORSC) requirements.

Deck footings should meet or exceed the minimum sizes set forth in Table R507.3.1 of the 2023 ORSC.

Reportedly, portions of the existing foundation are underpinned with a deep foundation consisting of helical piers. Based on our characterization of the subsurface materials in the area of the proposed attached addition, we have provided recommendations for new shallow isolated and/or continuous footings. However, from an engineering geologic perspective, there is an increased risk of non-earthquake differential settlement when portions of a foundation bear in potentially two different materials. We defer to the structural engineer and architect to provide feedback regarding the designed total and differential settlement tolerances and if any unusual loading patterns resulting in large differential loads or eccentrically loaded new footings are incorporated into the design.

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	95% 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 95 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. An underslab drainage system is recommended for all slabs, as per the architect's recommendations. Where floor coverings are planned, slabs should also be underlain by a suitable moisture barrier.

8.5 Retaining Walls

Due to the flat topography of the eastern portion of the site around the existing house, we do not anticipate any free-standing retaining walls will be proposed as part of the project. Please contact us to provide recommendations for retaining walls if they are incorporated into the design.

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 = 1.326 g
Site Coefficient	F_a = 1.2
Design Spectral Response Acceleration at Short Periods	S_{DS} = 1.061 g

8.7 Structural Fills

Structural fill supporting loads should consist of imported, granular material, free of organics and deleterious materials, and contain no particles greater than 1 inch in diameter so that nuclear methods (ASTM D6938) can be easily used for field density testing. Fill should contain less than 5% of material passing the 200 mesh sieve based on the minus ¾ inch fraction and a washed sieve analysis. Structural fill should be placed in lifts not exceeding 8 inches and compacted to a minimum of 95% of the maximum dry density as determined by ASTM D1557. All areas to receive fill should be stripped of all organic soils, organic debris and existing fill to a depth approved by a representative of HGSA.

STRUCTURAL FILL	
Compaction Requirements	95% ASTM D1557, compacted in 8-inch lifts maximum, at or near the optimum moisture content

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 D6938 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

We do not anticipate any permanent cut slopes related to the proposed development around the existing house. However, temporary unsupported cut and fill slopes less than 8 feet in height should be sloped no steeper than 1 horizontal to 1 vertical (1H:1V). If temporary slopes greater than 9 feet high are desired, or if 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.

Permanent unsupported cut and fill slopes shall be constructed no steeper than 2 horizontal to 1 vertical (2H:1V).

8.11 Drainage

Surface water should be diverted from new and existing 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 any new perimeter footings and sloped a minimum of 2 percent to a gravity outlet. A suitable perimeter footing drain system would consist of a 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 the installation of a drain that is tied into the perimeter footing drain and tightlined to an approved disposal point. All crawlspaces will need to be vented as per ORSC requirements.

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.

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 B). 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 B). 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 and therefore, a review of the site and/or this report may be necessary as time passes to assure its accuracy and adequacy.

The boring logs 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 boring locations. Also, the passage of time may result in a change in the soil and groundwater conditions at the site.

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.

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

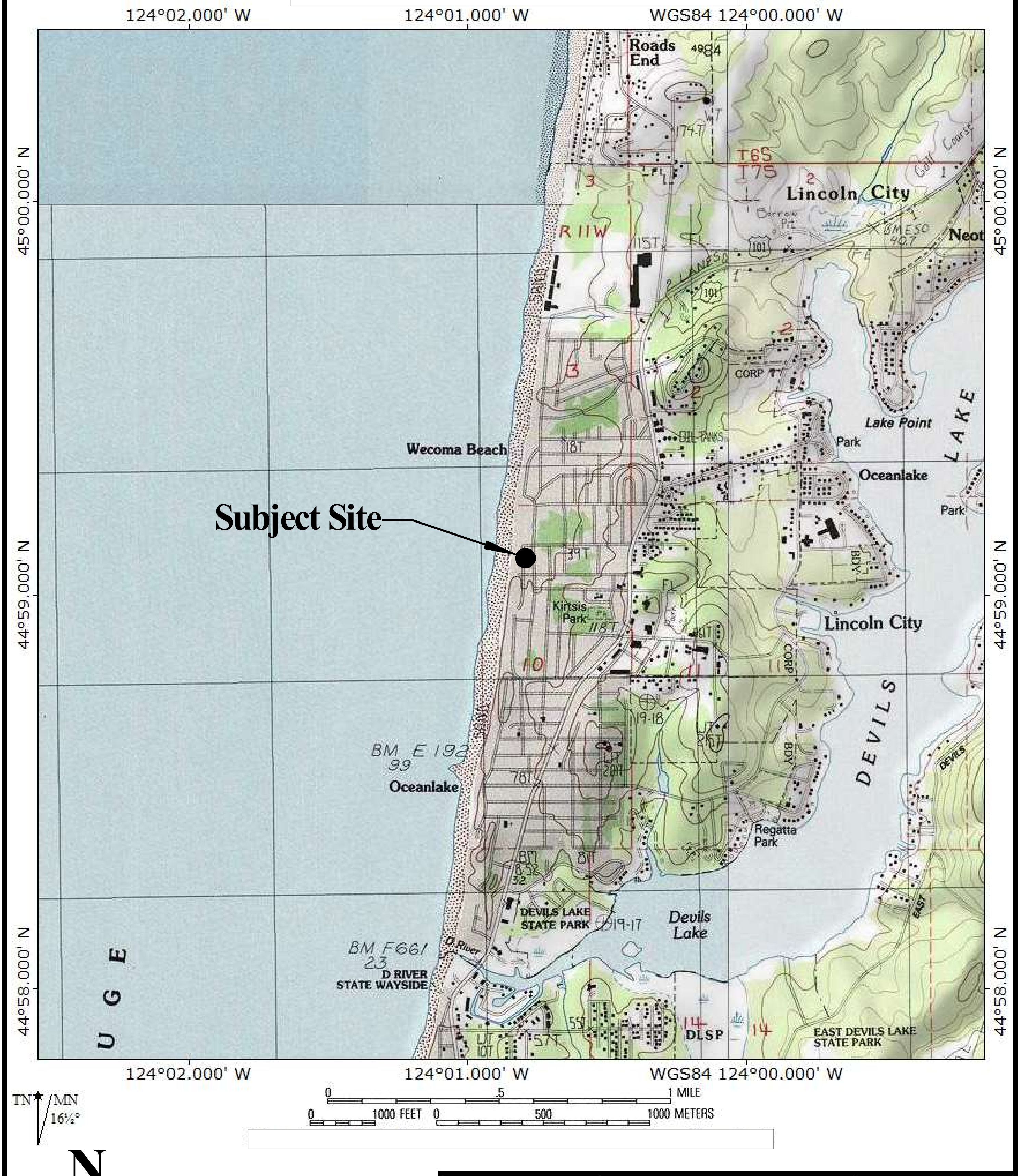
Respectfully submitted,

H.G. SCHLICKER AND ASSOCIATES, INC.

EXPIRES: 12/31/2024

Adam M. Large, MSc, RG, CEG
President/Principal Engineering Geologist

AML:mgb



Date: 07/17/2024	**Project #Y244738**	Prepared by: MGB
Scale: 1" = 2,000'		Approved by: AML

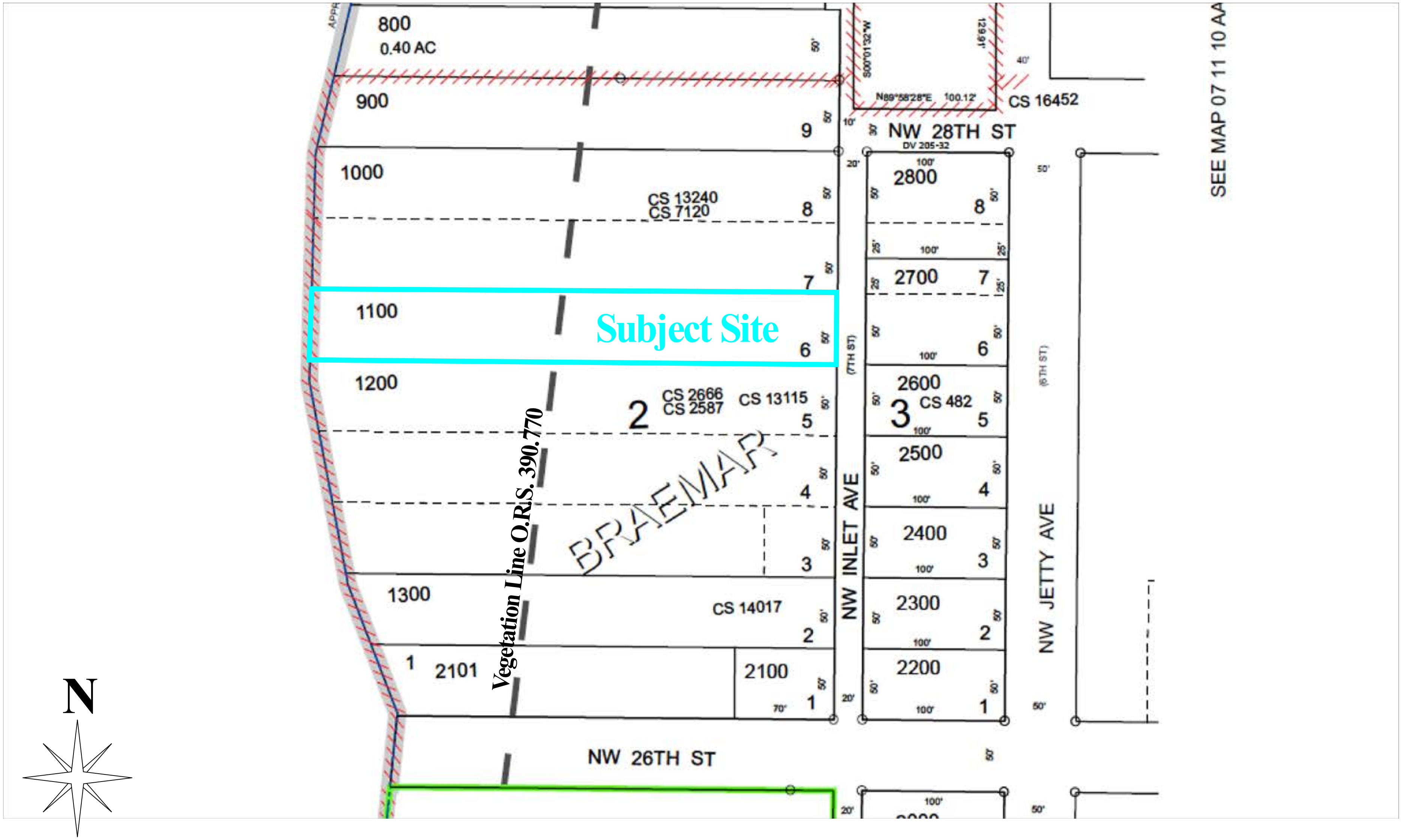
Location Map

Tax Lot 1100, Map 7-11-10AB

2715 NW Inlet Avenue, Lincoln City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 1



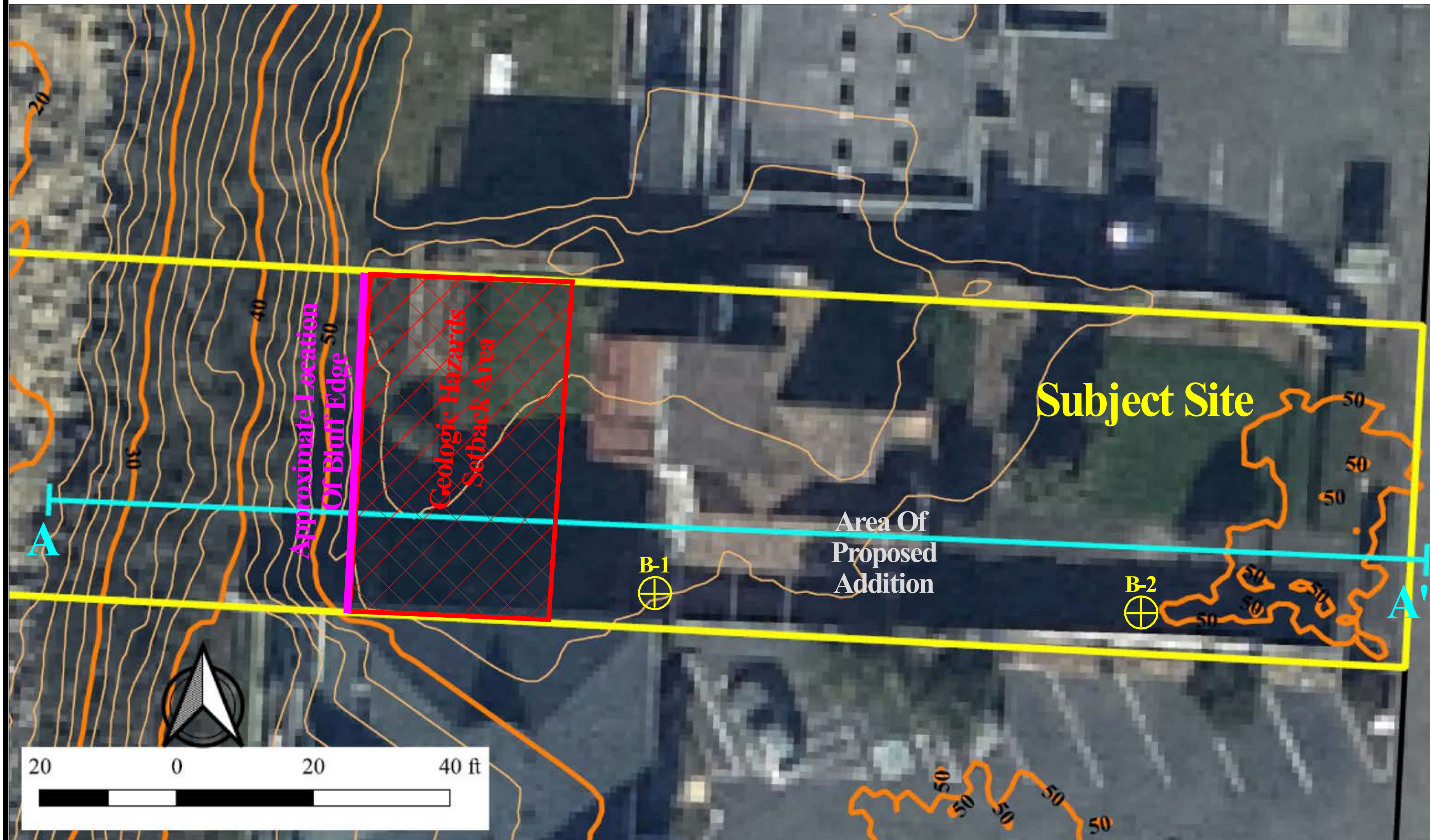
Date: 007/17/2024	Project #Y244738	Prepared by: MGB
Scale: 1" = 100'		Approved by: AML

Plat Map
Tax Lot 1100, Map 7-11-10AB
2715 NW Inlet Avenue, Lincoln City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 2

Modified from the Lincoln County assessor's plat
06-11-27DD, Lincoln City
All locations and dimensions are approximate.

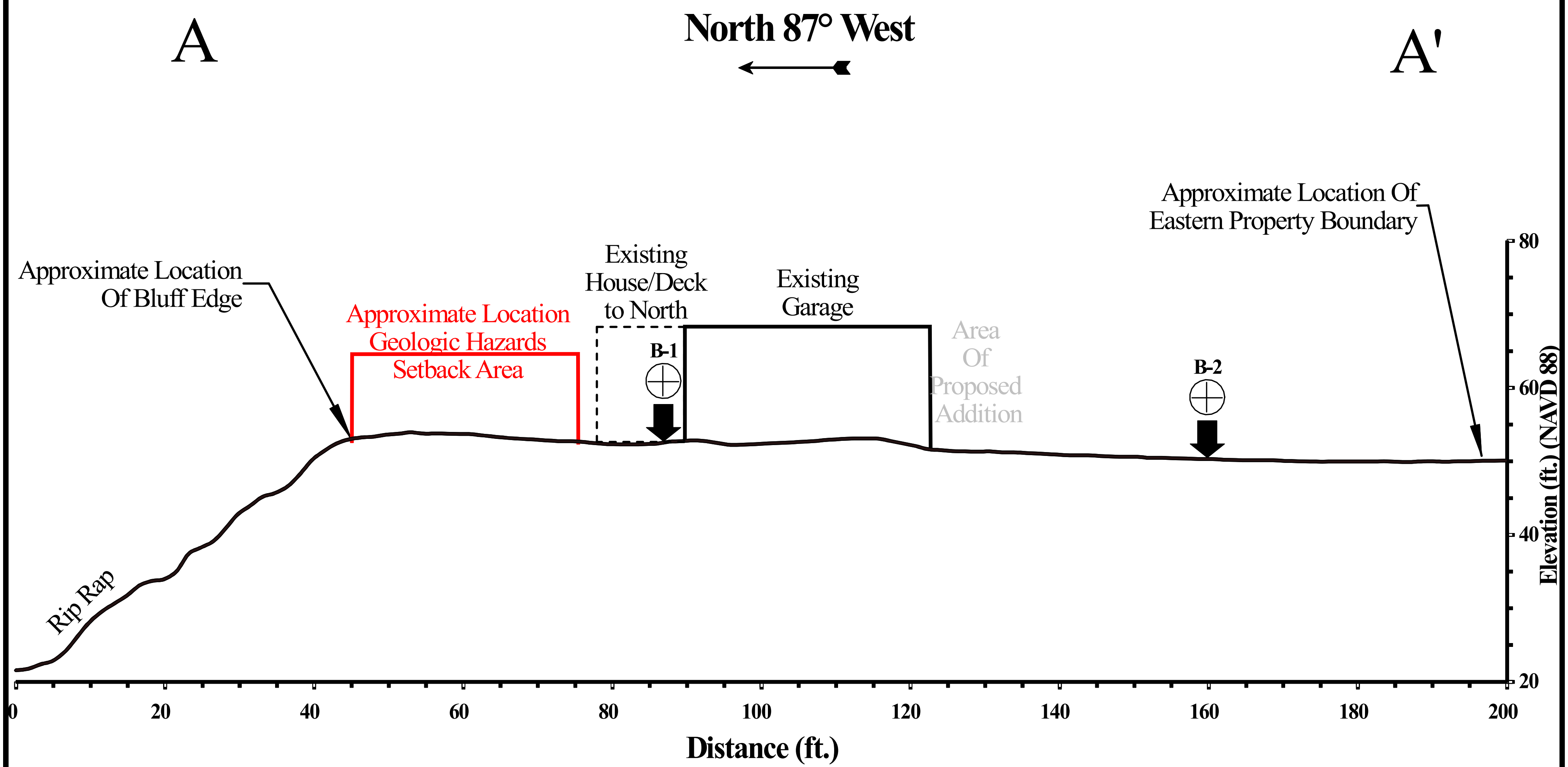


A A' = Approximate trend of profile line

B-1 = Approximate location of hand augered boring

Imagery provided NOAA Topographic data derived from west coast 2016 el nino dem data from NOAA.
All locations and dimensions are approximate.

Date: 07/17/2024	Project #Y244738	Prepared by:MGB Approved by: AML
Site Topographic Map Tax Lot 1100, Map 7-11-10AB 2715 NW Inlet Avenue, Lincoln City, Oregon		
H.G. Schlicker & Associates, Inc.		**Figure 3**



B-1

= Approximate location of boring

All dimensions, elevations and locations are approximate.
Slope profile modifed from west coast 2016 el nino dem from NOAA and
2009 OLC North Coast Lidar data from DOGAMI.

Date: 07/17/2024	**Project #Y244738**	Prepared by: MGB
Scale: 1" = 20'		Approved by: AML

Slope Profile, A-A'
Tax Lot 1100, Map 7-11-10AB
2715 NW Inlet Avenue, Lincoln City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 4

Appendix A
– Site Photographs –

Photo 1 – Southwesterly view of 2715 NW Inlet Avenue from near Inlet Avenue.

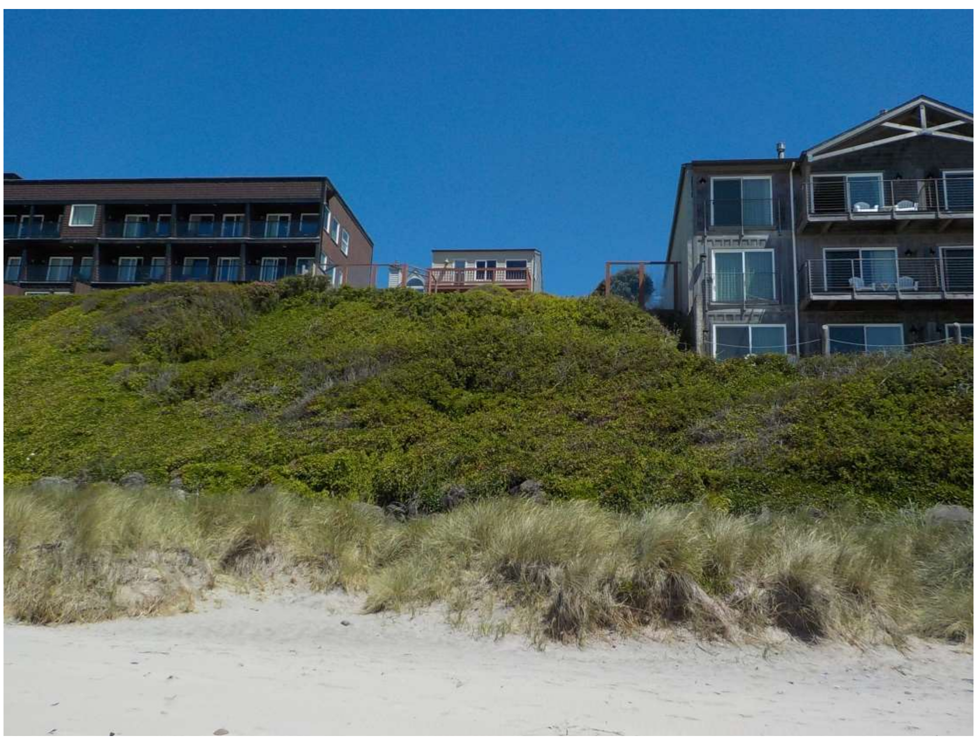


Photo 2 – Easterly view of the site from the beach.


Photo 3 – Close-up view of the existing rip rap stone along the base of the bluff.


Photo 4 – Northerly view of the bluff slope from near the bluff edge.

Photo 5 – View of the east side of the garage and driveway.

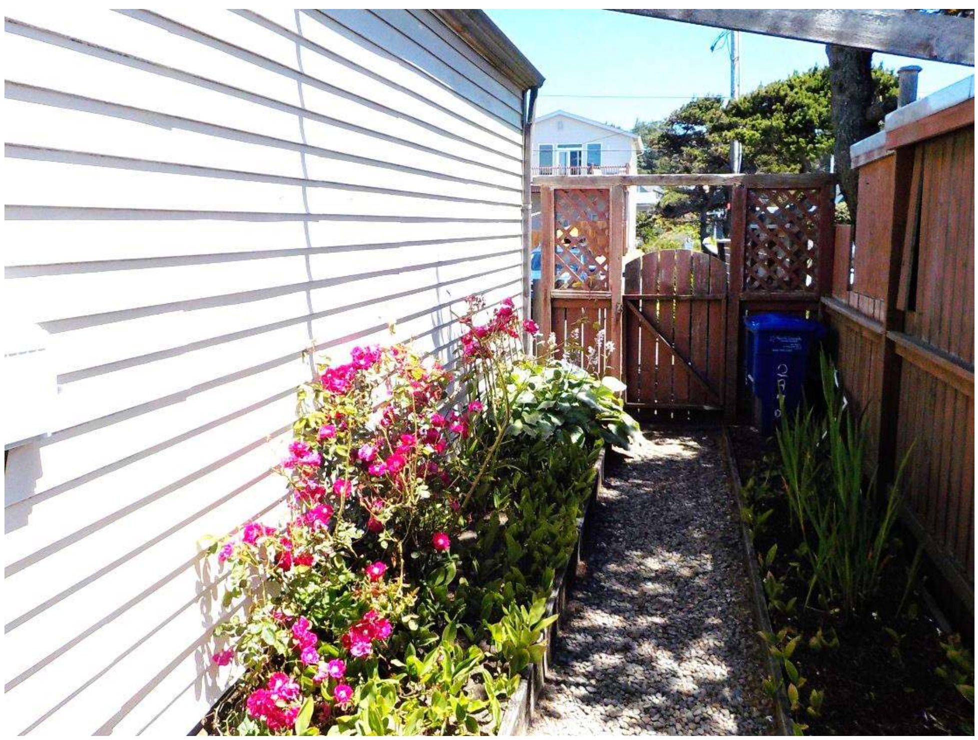


Photo 6 – View along the south side of the garage.


Photo 7 – Close-up view of the fill and loose sandy soils encountered in boring B-1.


Photo 8 – Close-up view of medium-dense sandy soils encountered in boring B-2.

Appendix B
– Checklist of Recommended Plan Reviews and Site Observations –

Project #Y244738

APPENDIX B
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 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 fill 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.