GEOTECHNICAL REPORT

Ericksen Ave Development

568 Ericksen Avenue NE
Bainbridge Island, Washington

Prepared for: Wing Point Properties

Project No. 170194 • June 21, 2018

Aspect
CONSULTING

earth + water

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Aspect Consulting, LLC

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1 Introduction

1.1 General

This report summarizes Aspect Consulting, LLC's (Aspect) observations, conclusions, and recommendations made during a geotechnical evaluation for the multi-family residential project at 568 Ericksen Avenue NE on Bainbridge Island, Washington (Site). We performed our geotechnical engineering evaluation in accordance with our agreed upon scope of work dated April 10, 2017 and authorized by you on April 11, 2017.

1.2 Scope of Services

Our scope of services included a literature review, site reconnaissance, subsurface explorations, and geotechnical engineering and slope stability evaluations. This report includes:

- Site and project descriptions;
- Distribution and characteristics of subsurface soils and groundwater based on two drilled soil borings;
- Slope stability analyses and corresponding structure setback recommendations;
- Seismic design considerations in accordance with the current version of the International Building Code (IBC);
- Suitable foundation types and allowable bearing pressure(s);
- Stormwater recommendations; and
- Site preparation recommendations and general construction recommendations.

1.3 Project Understanding

The Site consists of Kitsap County Parcel No. 262502-2-051-2009 at 568 Ericksen Avenue NE in Bainbridge Island, Washington, as shown on Figure 1, *Site Location Map.* The Site is developed with one building used for office space along the west side of the Site, as shown on Figure 2, *Site Exploration Plan.*

Current project plans include demolishing the existing structure and constructing six single-family residences along the north boundary of the Site (Project). Two parking stalls will be located along the west side of the Site. Access will remain from the east side of Ericksen Avenue NE using a, new access driveway and parking along the south end of the Site. A stormwater detention vault is planned to be constructed under the access driveway.

The east-facing slope on the east side of the Site is approximately 64 feet tall with a greater than 40 percent steep slope, as shown on Figure 2. The slope is considered by City of Bainbridge Island (COBI) as a geologically hazardous area. This geotechnical engineering evaluation presents the results of our evaluation for the minimum setbacks and corresponding recommended buffer reduction from these steep slopes for the proposed Project, in accordance with the applicable components of the COBI's Municipal Code.

2 Site Conditions

2.1 Site Description

2.1.1 General

The Site is a trapezoid-shaped parcel, covering about 0.44 acres that extends from Ericksen Avenue NE towards a ravine that drains to the south into the Puget Sound, as shown on Figure 1. One building is located on the west side of the Site used for office space. The building was built in 1905 and has a 642-square-foot footprint. A pond is located on the east side of the Site near the top of the steep, east-facing slope. Access to the Site is from the east side of Ericksen Avenue NE.

The top of the steep, east-facing slope is at about Elevation 140 feet (above mean sea level, North American Vertical Datum 1988 [NAVD88]; COBI, 2017). The slope has a greater than 40 percent slope and the toe of the steep slope is at about Elevation 76 feet.

West of the steep slope, the upland developed portion of the Site slopes gently down from the northwest to the southeast with about 10 feet of Elevation loss.

2.2 Geologic Setting

2.2.1 Geology

The Site is located within the Puget Lowland, a broad area of tectonic subsidence flanked by two mountain ranges—the Cascades to the east and the Olympics to the west. The majority of the Puget Lowland is overlain by thick Pleistocene-age sediments resulting from repeated cycles of glacial and nonglacial deposition and erosion. During nonglacial cycles, the Puget Lowland was dominated by lowland forests and broad river valleys similar to the predevelopment Duwamish valley. During glacial cycles, ice sheets up to 3,000 feet thick occupied the Puget Lowland and carved out the deep marine waterways and river valleys, and sculpted the uplands. The existing ground surface morphology has been defined by the glacial processes associated with the Vashon Stade of the Fraser Glaciation Age, the most recent glacial activity in the Puget Lowland occurred about 12,000 to 20,000 years ago.

The geologic map of the Site vicinity shows the Site as underlain by the Pleistocene Pre-Vashon-age deposits (Qpv) and the overlying Vashon till (Qvt) unit just to the west (Haugerud, 2011). The Pre-Vashon deposits are described as a diamict of fluvial, lacustrine, and glacial origin. Vashon till was deposited during the most recent glacial advance and was overridden and compacted by glacial ice, creating a dense/hard configuration. The Qvt unit is generally described as a dense diamict of sand, gravel, and silt, and is colloquially referred to as "hardpan".

2.2.2 Faults and Seismicity

The Site area is located within the Puget Lowland physiographic province, an area of active seismicity that is subject to earthquakes on shallow crustal faults and deeper subduction zone earthquakes. The Site area lies about 1 mile north of the Seattle fault zone, which consists of shallow crustal tectonic structures that are considered active (evidence for movement within the Holocene [since about 15,000 years ago]) and is believed to be capable of producing earthquakes of magnitude 7.3 or greater. The recurrence interval of earthquakes on this fault zone is believed to be on the order of a thousand years or more. The most recent large earthquake on the Seattle fault occurred about 1,100 years ago (Pratt et al., 2015). There are also several other shallow crustal faults in the region capable of producing earthquakes and strong ground shaking.

The Site area also lies within the zone of strong ground shaking from earthquakes associated with the Cascadia Subduction Zone (CSZ). Subduction zone earthquakes occur due to rupture between the subducting oceanic plate and the overlying continental plate. The CSZ can produce earthquakes up to magnitude 9.3 and the recurrence interval is thought to be on the order of about 500 years. A recent study estimates the most recent subduction zone earthquake occurred around 1700 (Atwater et al., 2015).

Deep intra-slab earthquakes, which occur from tensional rupture of the sinking oceanic plate, are also associated with the CSZ. An example of this type of seismicity is the 2001 Nisqually earthquake. Deep intra-slab earthquakes typically are magnitude 7.5 or less and occur approximately every 10 to 30 years.

2.3 Site Reconnaissance

An Aspect geologist performed a Site reconnaissance on April 20, 2017, to observe the existing conditions and, to the extent possible, identify geologic and landslide-related features. The Site reconnaissance was performed by traversing the slopes, noting visible features such as outcrops, scarps, cracks, springs, hummocks, vegetation, and the general geomorphology that may be indicative of ground movement.

The slope is approximately 64 feet tall and was measured to be approximately 35 to 37 degrees (70 to 75 percent). The level area of the Site at the top of the steep slope is landscaped with small trees, grass, and a row of bamboo near the crest of the slope. The top of the slope and the slope itself was vegetated with young to mature evergreens with forest undergrowth including ferns, ivy, salal, and a few young deciduous trees. A few of the evergreens showed a very minor bowed trunk, indicating minor past slope movement. Landscape debris was observed on the slope.

No water seepage or standing water was observed on the Site during our reconnaissance. A landscape pond was located about 20 feet west of the top of the slope.

2.4 Subsurface Conditions

2.4.1 Subsurface Explorations

Our subsurface explorations encountered Vashon till which was mapped on the geologic map just to the west of the Site (Haugerud et al., 2011). We inferred subsurface conditions at the Site using the completed field explorations, readily available geologic data, and our experience with the local geology.

On April 26, 2017, two borings—AB-01 and AB-02—were advanced to 21 and 51.4 feet below ground surface (bgs), respectively. The locations of the borings are shown on Figure 2.

Detailed descriptions of the subsurface conditions encountered in our explorations, as well as, the depths where characteristics of the soils changed, are on the soil boring logs presented in Appendix A. Soils were classified per the Unified Soil Classification System (USCS) in general accordance with the American Society for Testing and Materials (ASTM) D2488, Standard Practice for Description and Identification of Soils (Visual and Manual Procedure; ASTM, 2012). A key to the symbols and terms used on the logs is provided in Figure A-1. The depths on the logs where conditions changed may represent gradational variations between soil types and actual transitions may be more gradual.

A subsurface profile, A-A' (plan view on Figure 2; cross section Appendix B), was modeled using the computer model SLIDE (Rocscience, 2016). Based on our experience with the local geology, we interpreted between the two borings and extrapolated material types and contact depths (Appendix B).

2.4.2 Soil

The soil conditions observed during the subsurface exploration are summarized, from shallowest to deepest, as follows:

FILL

Fill refers to human-placed material. In boring AB-01, we encountered fill from the ground surface down to about 5 feet below ground surface (bgs). The fill consisted of very loose, moist, brown to dark brown, gravelly, silty SAND (SM).

The relative density of this coarse-grained deposit was very loose with N-values about 1 blow per foot (bpf).

COLLUVIUM

Colluvium is generally loose to medium dense soil that mantels hillsides due to accumulating soil creep, slope wash, and sloughing. We encountered colluvium along the steep east facing slope as well as in boring AB-02 from the ground surface down to about 5.5 feet bgs. The colluvium consisted of loose, moist, brown, gravelly, silty SAND (SM) with trace rootlets and moderate iron-oxide staining.

The relative density of this coarse-grained deposit was loose with N-values about 8 bpf.

VASHON TILL (Qvt)

Vashon till was encountered below fill in AB-01 and below colluvium in AB-02. With increased depth, the deposit became light gray in color. Vashon till at the Site consisted of layers of moist to very moist, light gray, very gravelly, silty SAND (SM) to sandy SILT (ML) with layers of cleaner sand. In AB-01 the upper portion of the till was weathered light brown due to iron-oxide staining.

The relative density of this fine-grained deposit was very dense with N-values greater than 50 bpf.

2.4.3 Groundwater

Groundwater was not encountered in boring AB-01. Groundwater seepage was measured in boring AB-02 about 41.7 feet bgs during drilling in a sandy deposit.

Samples within the upper 10 feet of both borings were very moist indicating a potential perched condition. A perched groundwater condition occurs when surface water percolates into the shallow subsurface and collects on relatively impermeable materials. In this case, the fill material, colluvium, and weathered Vashon till are considered low permeability, while the unweathered Vashon till unit is essentially impermeable. Groundwater conditions at the Site will vary with fluctuations in precipitation, Site usage (such as irrigation), and off-Site land use.

3 Conclusions and Recommendations

3.1 General

We conclude that the proposed Project will not negatively affect the stability of the geologically hazardous areas located at and near the Site, provided the recommendations herein are incorporated in design and construction. The key findings and conclusions include:

- The subsurface conditions at the Site consist of fill and colluvium near the top of the slope and Vashon till extending to the base of the slope. The Vashon till is not susceptible to deep-seated landsliding and can readily support the loads from the proposed residences;
- Shallow colluvial landslides are possible on the steep, east-facing slope; however, if best management practices are followed for steep slopes, the risk will be mitigated;
- Slide planes and/or subsurface layering that may indicate or be preferable for global slope instability were not encountered in the borings;
- Based on our slope stability analysis, we recommend a reduced minimum 20-foot buffer plus the 15-foot building setback for a total of 35 feet from the top of the steep slope;
- Shallow spread footing foundations are appropriate for the proposed residence foundations at the recommended setback from the steep slope;
- If fill, colluvium, and loose weathered Vashon till is encountered in the area of the planned footing subgrade, it should be removed and replaced with structural fill;
- Stormwater should be directed and/or conveyed away from the foundation and from the steep slope into the proposed stormwater detention vault; and
- Excavation with conventional equipment is feasible over the majority of the Site.

The grading and final development plans for the Project had not been completed at the time this report was prepared. Thus, we have not evaluated the impacts of Site grading on the stability of the existing slope. Once they are complete, Aspect should be engaged to review the Project plans and update our recommendations if necessary.

3.2 Seismic Considerations

3.2.1 Seismic Hazards

3.2.1.1 Ground Shaking

There is a high potential for low to moderate levels of shaking from all identified seismic sources to occur during the life of the proposed Project. The USGS National Seismic

Hazards Mapping Program interactive de-aggregation tool attributes over 46 percent of the seismic hazard at the Site to deep intra-slab earthquakes (USGS, 2014); less than 40 percent of seismic hazards at the Site are attributed to the Seattle fault zone earthquakes. Due to the lengthy recurrence intervals between large seismic events, the potential for strong ground shaking is considered low during the life of the proposed Project but must be considered for design of the structure, as required by the current building code.

3.2.1.2 Liquefaction and Lateral Spread

Liquefaction occurs when loose, saturated, and relatively cohesion-less soil deposits temporarily lose strength from seismic shaking. The primary factors controlling the onset of liquefaction include intensity and duration of strong ground motion, characteristics of subsurface soil, *in situ* stress conditions, and the depth to groundwater.

The Washington Department of Natural Resources (DNR) maps the Site as having a very low susceptibility to liquefaction (DNR, 2004). Given the relative density/consistency and composition of the soils expected below ground water, soil liquefaction is not a design consideration at this Site.

3.2.1.3 Seismically Induced Landslides

The presence of glacially-consolidated deposits underlying the relatively shallow fill, colluvium, and weathered Vashon till at the Site presents a low to moderate risk of seismically induced landslides. Future seismic events may cause movement within the surficial soils near and on the face of the slope. Further discussion and analyses of Site stability are presented in Section 3.3.2, *Slope Stability Analyses,* of this report and Appendix B.

3.2.1.4 Surficial Ground Rupture

The nearest known active fault trace is an unnamed fault, which is associated with the Seattle fault zone and located approximately 2,000 feet southwest of the Site (Gower et al., 1985). Due to the suspected long recurrence interval and the proximity of the Site from the mapped fault trace, the potential for surficial ground rupture at the Site is considered low during the expected life of the Project. Because there is no practical design mitigation for surface fault rupture, this is not a design consideration.

3.2.2 Seismic Design Criteria

Inertial seismic forces are expected to affect the Site and structures. Appropriate design of structures in accordance with the current version of the International Building Code (IBC), with State of Washington amendments, will mitigate seismic hazards.

The IBC requires design for a "Maximum Considered Earthquake (MCE)" with a 2 percent probability of exceedance (PE) in 50 years (2,475-year return period; IBC, 2015). The USGS has completed probabilistic ground motion studies and maps for Washington (USGS, 2014).

Current IBC design methodologies express the effects of site-specific subsurface conditions on the ground motion response in terms of the "site class." The site class can be correlated to the average standard penetration resistance (SPT) in the upper 100 feet of the soil profile. Based on the results of our subsurface exploration program Seismic Site Class C is appropriate for IBC code-based seismic design.

Based on the latitude and longitude of the Site (47.62895°N, -122.51665°W), the mapped maximum considered earthquake spectral response accelerations for short period (S_s) = 1.432g and for 1-second period (S_1) = 0.563g. Site coefficients for this Site are F_a = 1.0, F_v = 1.3. The maximum considered earthquake spectral response accelerations adjusted for Site class effects are S_{ds} = 0.954g, S_{d1} = 0.488g.

Using methods presented in the IBC and American Society of Engineers (ASCE) Standard 7-10, *Minimum Design Loads for Buildings and Other Structures*, we calculated a Site Class-adjusted peak ground acceleration (PGA_M) of 0.66g during the MCE. A pseudo-static coefficient (K_h) equal to half of the PGA was used in the seismic slope stability analyses.

3.3 Landslide and Steep Slope Considerations

3.3.1 Landslide and Erosion Hazards

The City maps a geologically hazardous area along the east-facing, slope east of the Site: greater than 40 percent steep slopes (COBI, 2017).

Three types of landslides (Varnes, 1978) are common on steep slopes and bluffs near the Site area: topples, deep-seated rotational slides, and surficial landslides. Based on the Site topography and lack of vertical relief, topples are not anticipated and are not a relevant landslide at the Site. Deep-seated rotational slides and surficial landslides are relevant to the Site and are described in further detail below.

Landslides may be triggered by natural causes such as precipitation, freeze-thaw cycles, seismic event, or be man-made (e.g., broken water pipes or stormwater flow).

3.3.1.1 Deep-Seated Rotational Landslides

Rotational landslides consist of deep-seated failures that typically involve slip along a curved surface(s). Rotational landslides may transport large masses of semi-intact soil downslope, resulting in alternating steep headscarps along the upper portion of the failure, with more gently sloping benches composed of displaced soil.

No rotational landslides have been mapped or inventoried on or adjacent to the Site vicinity (McKenna et al., 2008; Haugerud et al., 2011; Ecology, 1979). Additionally, no evidence of recent or ancient rotational landslide activity was apparent at or in the immediate vicinity of the Site. No weak zones or bedding plans were observed in the subsurface investigation of the glacially-overridden deposits. The geologic map (Haugard, 2011) did indicate a landslide within the ravine about 200 feet south of the Site.

3.3.1.2 Surficial Landslides

Surficial or debris landslides consist of sliding of the weathered colluvial soil layer and overlying vegetation that typically mantles steep slopes in the Puget Sound region. Surficial slides commonly result from a significant increase in the moisture content within the upper weathered soil layer on slopes. Increased moisture typically results from periods of extended, heavy precipitation, groundwater seepage, or concentrated stormwater discharge onto a slope. Slides that occur within the upper several feet of weathered soils typically do not extensively impact the underlying soils.

We observed evidence of minor, recent, or incipient surficial slide activity on the Site steep slopes. The slopes are densely vegetated with mature evergreens and deciduous trees, with an established and moderately dense understory. In general, the mature evergreens trees on the steep slope were relatively straight and only little noticeable trunk curvature was observed. The steep slope has an ongoing risk of surficial landslide activity as is typical of steep slopes in this region. Any surficial failure would likely be limited to the outer weathered soils and would not affect the overall slope stability. Following the steep slope management recommendations included in this report will help the overall stability of the Site slopes.

3.3.1.3 Erosion Hazard

Due to the relatively flat topography of the uplands area, this portion of the Site has a low erosion potential. Soils on the steep slope portion of the Site have a moderate erosion potential during normal (undisturbed) conditions, but will increase to high erosion potential where groundcover is sparse or disturbed. Areas outside of the proposed construction area with dense groundcover have low erosion potential. Care should be taken during construction to prevent any erosion on the face of the steep slope.

3.3.2 Slope Stability Analysis

Based on our review of the existing Site topography, Site reconnaissance, and geotechnical explorations, we conducted a stability analysis of a profile (A–A') of the critical section transecting the east-facing steep slope on the Site (Figure 2 and Appendix B). These analyses helped us to determine minimum setbacks from the steep slope to the proposed structures and were completed using the computer program Slide (Rocscience, 2017). The engineering soil parameters used in our analysis were based on the completed subsurface explorations and our experience with local geology. These parameters are presented in Table 1.

Table 1 – Summary of Soil Engineering Properties Used in Slope Stability Analyses

Geologic Unit	Unit Weight (pcf)	Cohesion (psf)	Phi (deg)
Colluvium	120	0	30
Vashon Till	130	0	40

Notes:
Pcf = pound per cubic foot psf = pounds per square foot; and deg = degrees

We developed a generalized cross-section of the east-facing slope using publicly available topographic data derived from City LiDAR data (COBI, 2017). The strength values assumed for the subsurface units were roughly correlated with the blow count data collected in the borings and are within the bounds of the suggested parameters in Chapter 5 of the Washington State Department of Transportation (WSDOT) Geotechnical Design Manual (GDM; WSDOT, 2015). Back calculations of existing slope configurations indicate the cohesion values assumed are reasonable and conservative. Groundwater was modeled based on our field explorations, observations, and experience with local geology.

The Slide program performs slope stability computations based on the modeled slope conditions and calculates a factor of safety against slope failure, which is defined as the ratio of resisting forces to driving forces. A factor of safety of 1.0 indicates a "just-stable" condition, and a factor of safety less than one would indicate unstable conditions.

We conservatively assumed a 400 pound per square foot (psf) aerial load for an average surcharge pressure from a typical residence. We utilized Spencer's method in our Slide analyses. For seismic calculations, we set the horizontal seismic coefficient for pseudostatic analysis (K_h) equal to ½ x PGA = 0.33g, where g is the acceleration of gravity. The results of our stability analyses for the proposed conditions are summarized in Table 2 below.

Table 2 – Summary of Profile A-A' Slope Stability Analysis Results

Distance from Top of Slope	Static Factor of Safety	Seismic* Factor of Safety
35 feet	2.0	1.0

Notes:

* Pseudostatic seismic analysis based on peak ground acceleration (PGA) of 0.66g in accordance with the methods presented in ASCE Standard 7-10.

The analyses identified that the most critical (minimum factor of safety) surfaces are on the face of the steep slope under seismic conditions. For potential failure surfaces daylighting 35 feet or more west of the top of the steep slope, we calculated minimum factors of safety of 2.0 for static conditions and 1.0 for seismic conditions. These factors of safety satisfy the City's minimum requirements of 1.5 and 1.0, respectively (COBI, 2016).

The results of our slope stability analyses indicate the steep slope is relatively stable with regards to rotational landslide mechanisms. Given the geologic units that comprise the Site and steep slope, it is our opinion that the likelihood of a rotational failure of the steep slope is low. The more pertinent hazard to the proposed structure is the accumulation of incremental erosion and small surficial slides on the steep slope (slope retreat).

3.3.3 Allowable Slope Setback

The east-facing steep slope at the Site is approximately 64 feet tall. The standard buffer and building setback associated with this slope would be 64 feet plus a 15-foot building setback, for a total standard setback of 79 feet from the top of the slope. However, based on the results of our explorations, reconnaissance, and slope stability evaluations, the steep slope at the Site is relatively stable and meets the criteria for the limited setback exemption. Based on our slope stability analysis, we recommend a reduced minimum 20-foot buffer plus the 15-foot building setback for a total of 35 feet from the top of the steep slope modeled in cross section A-A' to the planned residence. This minimum setback recommendation applies to all structures, including the stormwater detention vault. The excavation for the vault will reduce the loads on the nearby steep slope because a concrete-lined vault full of water weighs less than the soil excavated for the vault. Therefore, the existing slope stability model for the Project is more conservative than the final configuration of Site in the location of the planned vault.

Our analyses assume no leakage from the stormwater detention vault. Given the location of the proposed vault at the crest of a steep slope critical area, we recommend the civil engineer include adequate redundancy in the design to minimize the risk of leakage.

3.3.4 Slope Management

The most likely impact from slope instability to the Site would be surficial landslides triggered by saturation of the near-surface soils. These failure types are typically limited to the upper 2 to 3 feet of soil on the slope. Table 3 includes factors that can affect stability of the near-surface soil layer:

Table 3 – Stabilizing and Destabilizing Factors for Slope Management[1]

Stabilizing Factors	Destabilizing Factors
Root Reinforcement Roots mechanically reinforce a soil by transfer of shear stresses in the soil to tensile resistance in the roots.	**Surcharge** Weight of vegetation on a slope exerts both a downslope (destabilizing) stress and a stress component perpendicular to the slope, which tends to increase resistance to sliding.
Soil Moisture Modification Evapotranspiration and interception in the foliage lower soil moisture content.	**Root Wedging** Alleged tendency of roots to invade cracks, fissures, and channels in a soil and thereby cause local instability by a wedging or prying action.
Buttressing and Arching Anchored and embedded stems can act as buttress piles or arch abutments in a slope, counteracting shear stresses.	**Windthrowing** Destabilizing influences from an overturning moment exerted on a slope as a result of strong winds blowing downslope through trees.

Notes:
1) from Gray and Leiser (1982)

Other causes of surficial slope instability include improperly-managed storm and surface water runoff flowing near or over the top of the slope. Uncontrolled runoff or surface water should never be allowed to flow across the Site slopes.

Yard debris should never be placed on the surface of the steep slope, as this adds unnecessary, extra weight and limits the growth potential of the surficial plants.

We recommend maintaining dense vegetative groundcover on the Site slopes outside of the building area. If soils on or near the slopes become exposed through erosion and/or shallow failures, we recommend immediately covering and aggressively revegetating the exposed area. This may require the placement of plastic sheeting replaced by a woven jute-mat to provide temporary ground cover while vegetation takes root.

For specific vegetation recommendations, the Washington State Department of Ecology (Ecology) has several good publications on the subject including:

- Vegetation Management: A Guide for Puget Sound Bluff Property Owners, Ecology Publication 93-31, at http://www.ecy.wa.gov/programs/sea/pubs/93-31/intro.html.
- Slope Stabilization and Erosion Control Using Vegetation: A Manual of Practice for Coastal Property Owners, Ecology Publication 93-30, at http://www.ecy.wa.gov/programs/sea/pubs/93-30/index.html.

3.4 Foundation, Slabs, and Wall Design Considerations

3.4.1 Shallow Foundations

Although current Site plans are preliminary, the following general foundation criteria are assumed. Shallow foundations or spread footings may be used for building supports on the Vashon till as encountered in our explorations. Bearing surfaces for the footings should be prepared as described in this report. Based on both borings, depth to the bearing strata is anticipated 5 to 5.5 feet bgs.

For shallow foundations gaining support from the bearing strata described above, we recommend an allowable foundation bearing pressure of 2,000 psf be utilized for design purposes, including both dead and live loads for the proposed structure. Recommendations within the Site Preparation and Structural Fill sections of this report must be followed in order for this allowable bearing pressure to be used. An increase in the above-mentioned bearing pressure of one-third may be used for short-term wind or seismic loading. Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection and be a minimum of 14 inches wide; interior footings require only 12 inches burial below outside grade. The minimum footing width may control the design. No footing should be founded in or above yielding/loose or organic soils.

We estimate the total settlement of the foundation designed in accordance with our recommendations will be less than 1 inch. Differential settlements can be expected to be less than half of the total settlement. Our experience indicates the majority of these settlements will occur during construction.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structures to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structures.

For use in design, an ultimate coefficient of friction of 0.55 may be assumed along the interface between the base of the footing and subgrade soils. An ultimate passive earth pressure of 325 pcf may be assumed for native soils adjacent to below-grade elements. The upper 1 foot of passive resistance should be neglected in design. The recommended coefficient of friction and passive pressure values are ultimate values that do <u>not</u> include a safety factor. We recommend applying a factor of safety of at least 1.5 in design for determining allowable values for coefficient of friction and passive pressure.

3.4.2 Floor Slabs

Concrete slabs-on-grade, such as garage floors or basement slabs, should be designed in accordance with the American Concrete Institute (ACI) Committee 360 Guide to Design of Slabs-on-Ground (ACI, 2010). We recommend over-excavation of any loose soil or deleterious matter and replacement with structural fill beneath all structural slabs. To provide uniform support for the floor slab and to provide a capillary break, we recommend a 6-inch-thick capillary break layer. The capillary break material should be freely-draining sand and gravel with a maximum particle size of 3/4 inch, with no more than 80 percent passing the No. 4 sieve and less than 5 percent fines (material passing the U.S. Standard No. 200 sieve). In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structures, a 10-mil polyethylene vapor barrier should be placed directly over the capillary break. The vapor barrier should be installed in accordance with the manufacturer's recommendations.

Slab-on-grade floors prepared as described above and designed as beam on elastic foundation can utilize a modulus of subgrade reaction of 150 pounds per cubic inch (pci).

3.5 Drainage and Moisture Considerations

The Vashon glacial till that underlies the Site is relatively impermeable and not suitable for concentrated stormwater infiltration. Current Project plans include a stormwater detention vault constructed under the planned driveway setback 35 feet from the top of the east-facing steep slope. The vault is about 20 feet wide by 8 feet tall by 75 feet long, with the bottom of the vault at approximately Elevation 136 feet. This vault will have an overflow system to route excess stormwater through a high-density polyethylene (HDPE) pipe to outfall at the toe of the steep slope.

Final grades around the proposed structures should be sloped such that water drains away from the structures. At no time should stormwater or runoff be allowed to collect or discharge onto the slope. Water from hard surfaces should be collected and diverted to the proposed stormwater detention vault. To reduce the potential for flooding foundation drains and clogging, downspouts and roof drains should not be connected to the foundation drains and under-slab drains. Drains should include clean-outs to allow periodic maintenance and inspection.

The outside edge of all perimeter footings and the upslope side of all walls should be provided with a drainage system consisting of 4-inch-diameter, perforated, rigid plastic pipe embedded in a clean, free-draining sand and gravel meeting the requirements of Section 9-03.12(4) of the WSDOT Standard Specifications for Gravel Backfill for Drains (WSDOT, 2016). The drainpipe and surrounding drain rock could be wrapped in filter fabric to minimize the potential for clogging and/or ground loss due to piping. A washed rock drain curtain at least 1-foot-thick should extend from the footing continuously upward to within 1 foot of the ground surface. A layer of low permeability soils should be used on the upper foot to reduce potential for surface water to enter the drain curtain.

We recommend that the stormwater overflow pipe from the stormwater detention vault to the base of the ravine be designed with an appropriately sized, fuse-welded, HDPE piping suspended from the vault. An HDPE in-line wall anchor should be utilized to secure the outfall pipe to vault. A diffuser should be installed at the outlet of the tightline near the base of the ravine and buried in quarry spalls around the outlet. Backfill for the tightline trench between the catch basin and the top of the slope should consist of relatively impermeable material such as bentonite-amended soil or controlled density fill (CDF)/lean concrete to prevent water migration along the tightline trench.

A properly designed and constructed steep slope tightline can accommodate small and moderate shallow flow landslide events without significant damage to the outfall. Larger landslide events could require repairs or rebuilding of the system. The steep slope tightline should be inspected and tested for tightness before each winter and after any landslide event.

A landscape pond is located about 20 feet west of the top of the steep slope. It is our understanding that this pond will not remain when the Site is redeveloped.

4 Construction Considerations

4.1 Earthwork

Earthwork is typically most economical when performed under dry weather conditions. Appropriate erosion control measures should be implemented prior to beginning earthwork activities in accordance with the local regulations. In our opinion, excavation can generally be accomplished using standard excavation equipment. While not directly observed in our subsurface explorations, the presence of potential obstructions, such as small boulders or other debris, in any of the materials encountered should be anticipated.

4.1.1 Wet Weather Conditions

If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is above optimum and difficult to control, the following recommendations apply:

- Earthwork should be performed in small areas to minimize exposure;
- Structural fill placed during wet weather should consist of material meeting the criteria for Gravel Borrow as specified in Section 9-03.14(1) of the WSDOT Standard Specifications (WSDOT, 2016);
- Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of the specified structural fill;
- The size, type, and access of construction equipment used may have to be limited to prevent soil disturbance;
- The ground surface within the construction area should be graded to promote runoff of surface water away from the slopes and to prevent water ponding;
- The ground surface within the construction area should be properly covered and under no circumstances should be left uncompacted and/or exposed to moisture. Soils that become too wet for compaction should be removed and replaced with specified structural fill;
- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed prior to placement, compaction requirement are met, and site drainage is appropriate; and
- Erosion and sedimentation control are implemented in accordance with City best management practices (BMPs).

4.2 Site Preparation

Site preparation within the proposed construction footprint should include removal of fill and soils containing roots, organics, debris, and any other deleterious materials. The contractor must use care during site preparation and excavation operations so that any bearing surfaces are not disturbed. If disturbance does occur, the disturbed material should be removed to expose undisturbed material or be compacted in-place to acceptable criteria as determined by the geotechnical engineer. Over-excavated soils in footing subgrade areas should be replaced with compacted crushed surfacing base course (CSBC) specified in Section 9-03.9(3) of the WSDOT Standard Specifications (WSDOT, 2016) as described in the Structural Fill section of this report.

All footing excavations should be trimmed neat and the bottom of the excavation should be carefully prepared. All loose or softened soil should be removed from the footing excavation or compacted in-place prior to placing reinforcing steel bars. We recommend that footing excavations be observed by the geotechnical engineer prior to placing steel and concrete to verify the recommendations in this report have been followed.

If footing excavations are exposed during the winter or periods of wet weather, we recommend providing a layer of crushed rock or gravel to help preserve the subgrade until the concrete placement. Gravel used to protect the bearing surfaces should meet the gradation requirements for Class A Gravel Backfill for Foundations, as described in Section 9-03.12(1)A of the WSDOT Standard Specifications (WSDOT, 2016).

Construction of the proposed driveway and residence will involve limited clearing and grubbing of existing vegetation. The subgrade under the AC pavement section areas should be prepared by scarifying, moisture conditioning, and recompacting a minimum of 12 inches below the bottom of the base course. Materials generated during earthwork should be transported off site or stockpiled in areas designated by the owner's representative.

4.3 Temporary and Permanent Slopes

Maintenance of safe working conditions, including temporary excavation stability, is the sole responsibility of the contractor. All temporary cuts in excess of 4 feet in height that are not protected by trench boxes, or otherwise shored, should be sloped in accordance with Part N of Washington Administrative Code (WAC) 296-155 (WAC, 2009).

With time and the presence of seepage and/or precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. We recommend planning the construction schedule to have excavation occur during the summer months and to minimize the amount of time that the temporary slopes will be unsupported during construction. The contractor should monitor the stability of the temporary cut slopes and adjust the construction schedule and slope inclination accordingly. Vibrations created by traffic and construction equipment may cause caving and raveling of the face of the temporary slopes. At no time should soil stockpiles, equipment, and other loads be placed immediately adjacent to an excavation.

In general, shallow surface soils, such as topsoil, fill, colluvium, and weathered soils that will be subject to excavation and sloping on the Site classify as Occupational Safety and Health Administration (OSHA) Soil Classification Type C. Temporary excavation side slopes (cut slopes) are anticipated to stand as steep as 1½ H:1V within the topsoil and weathered soils. The underlying Vashon till can be classified as OSHA Type A Soil; temporary cuts in the till may be inclined as steep as ¾H:1V. The cut slope inclinations estimated above are for planning purposes only and are applicable to excavations without inflowing perched groundwater or runoff.

Permanent slopes for the Project should have a maximum inclination of 2H:1V. Access roads and pavements should be located at least five feet from the top of temporary slopes. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face.

4.4 Structural Fill

Excavated soil will generally be fill, colluvium, and Vashon till deposits, which contain a significant fraction of fine-grained (silt and clay) materials. These soils will only be suitable for re-use as structural fill if properly moisture conditioned (dried back). Material excavated that is not suitable for use as structural fill may be used as fill in landscape areas or other areas not sensitive to settlement.

Imported structural fill should be granular material with less than 10 percent fines, such as Select Borrow as specified in Section 9-03.14(2) of the WSDOT Standard Specifications (WSDOT, 2016). In wet weather conditions or situations requiring free-draining backfill, material meeting the criteria for Gravel Borrow as specified in Section 9-03.14(1) of the WSDOT Standard Specifications (WSDOT, 2016) should be used. Crushed Surfacing Base Course as specified in Section 9-03.9(3) of the WSDOT Standard Specifications. If desired, lean concrete or controlled density fill (CDF) can also be used as structural fill under foundations.

In general, suitable structural fill material for the Project is fill placed within 3 percent of its optimum moisture content per ASTM D1557 (modified Proctor test) and does not contain deleterious materials or particles larger than 3 inches in diameter. Structural fill material should be compacted to a minimum of 95 percent of the maximum dry density (MDD) based on ASTM D1557. Structural fill adjacent to a wall should be compacted to a minimum of 90 percent of the MDD based on ASTM D1557.

The extent of site grading is currently unknown; however, we estimate cuts and fills will be limited in depth/thickness to less than 2 feet. Structural fill, including base rock, should be placed over subgrades that have been prepared in conformance with the Site Preparation section of this report.

If fill and excavated material will be placed on slopes steeper than 5H:1V, these must be keyed/benched into the existing slopes and installed in horizontal lifts. Vertical steps between benches should be approximately two feet.

4.5 Additional Project Design and Construction Monitoring

At the time of this report, site plans, site grading, structural plans, and construction methods have not been finalized, and the recommendations presented herein are based on preliminary Project information. If Project developments result in changes to the assumptions made herein, we should be contacted to determine if our recommendations should be revised. We recommend that, once design plans are fully developed, Aspect is consulted in order to verify that our recommendations were properly interpreted and applied.

We recommend a pre-construction meeting be organized at the start of construction with you, your contractor, and Aspect. During this meeting, we will understand the goals and schedule to be upheld during construction. We will also discuss effective lines of communication. The integrity of the foundation and overall Site stability depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent.

Upon completion of construction, the City will require Aspect to complete a Step 3 Form prior to issuing the Certificate of Occupancy. For Aspect to complete this form, we must be involved during certain construction activities, including but not limited to, foundation subgrade preparation, installation of the stormwater outfall system, and installation of the site and foundation drainage elements.

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Limitations

Work for this project was performed for Mr. and Ms. Rein (Client), and this report was prepared consistent with recognized standards of professionals in the same locality and involving similar conditions, at the time the work was performed. No other warranty, expressed or implied, is made by Aspect Consulting, LLC (Aspect).

Recommendations presented herein are based on our interpretation of site conditions, geotechnical engineering calculations, and judgment in accordance with our mutually agreed-upon scope of work. Our recommendations are unique and specific to the project, site, and Client. Application of this report for any purpose other than the project should be done only after consultation with Aspect.

Variations may exist between the soil and groundwater conditions reported and those actually underlying the site. The nature and extent of such soil variations may change over time and may not be evident before construction begins. If any soil conditions are encountered at the site that are different from those described in this report, Aspect should be notified immediately to review the applicability of our recommendations.

Risks are inherent with any site involving slopes and no recommendations, geologic analysis, or engineering design can assure slope stability. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the Client.

It is the Client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, and agents, are made aware of this report in its entirety. At the time of this report, site plans, site grading, structural plans, and construction methods have not been finalized, and the recommendations presented herein are based on preliminary project information. If project developments result in changes from the preliminary project information, Aspect should be contacted to determine if our recommendations contained in this report should be revised and/or expanded upon. We recommend that, once design plans are fully developed, Aspect is consulted in order to verify that our recommendations were properly interpreted and applied.

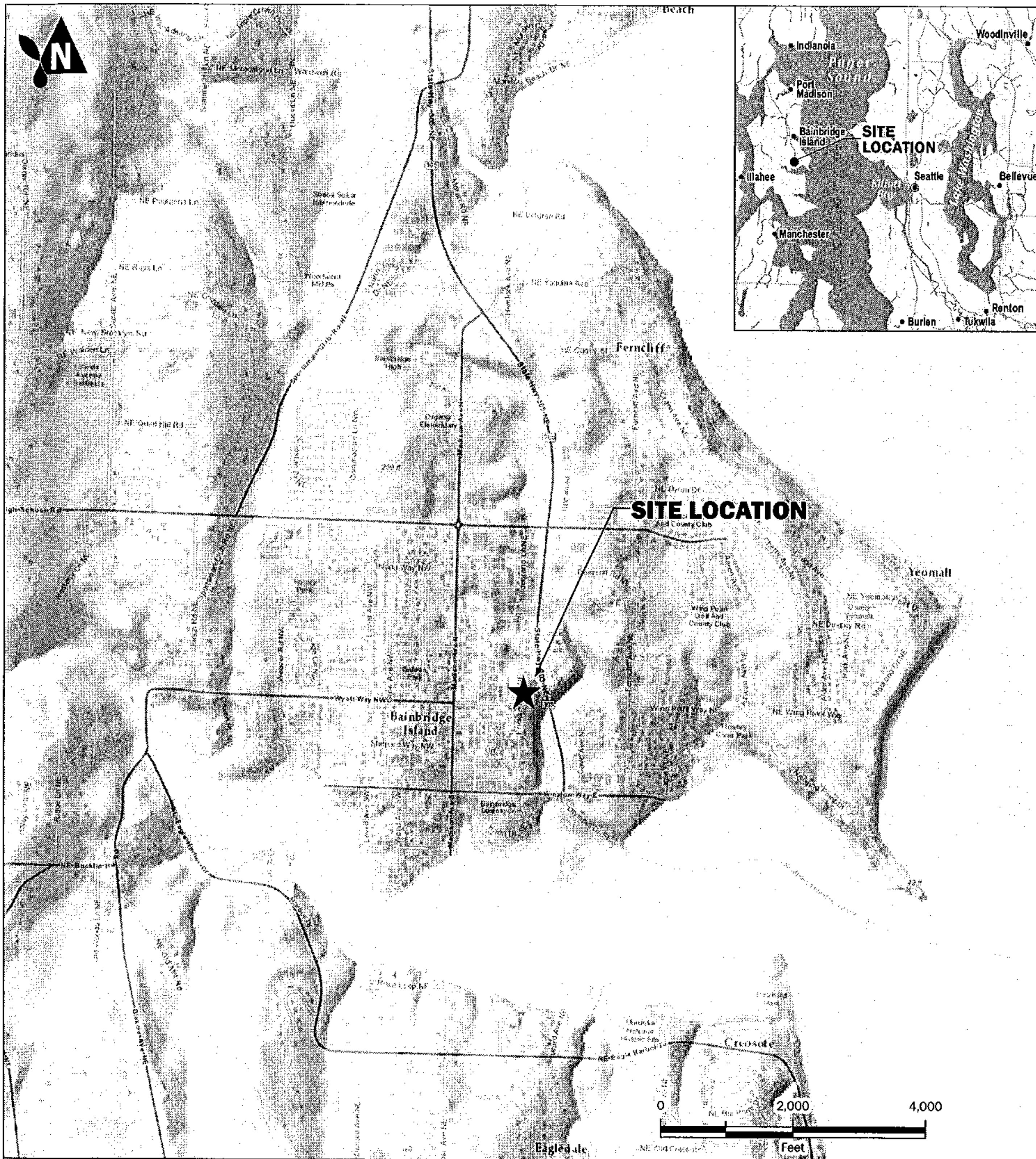
The scope of work does not include services related to construction safety precautions. Site safety is typically the responsibility of the contractor, and our recommendations are not intended to direct the contractor's site safety methods, techniques, sequences, or procedures. The scope of our work also does not include the assessment of environmental characteristics, particularly those involving potentially hazardous substances in soil or groundwater.

All reports prepared by Aspect for the Client apply only to the services described in the Agreement(s) with the Client. Any use or reuse by any party other than the Client is at the sole risk of that party, and without liability to Aspect. Aspect's original files/reports shall govern in the event of any dispute regarding the content of electronic documents furnished to others.

Please refer to Appendix C titled "Report Limitations and Guidelines for Use" for additional information governing the use of this report.

We appreciate the opportunity to perform these services. If you have any questions please call Alison Dennison, LEG, Senior Project Geologist, at 206-780-7717.

FIGURES

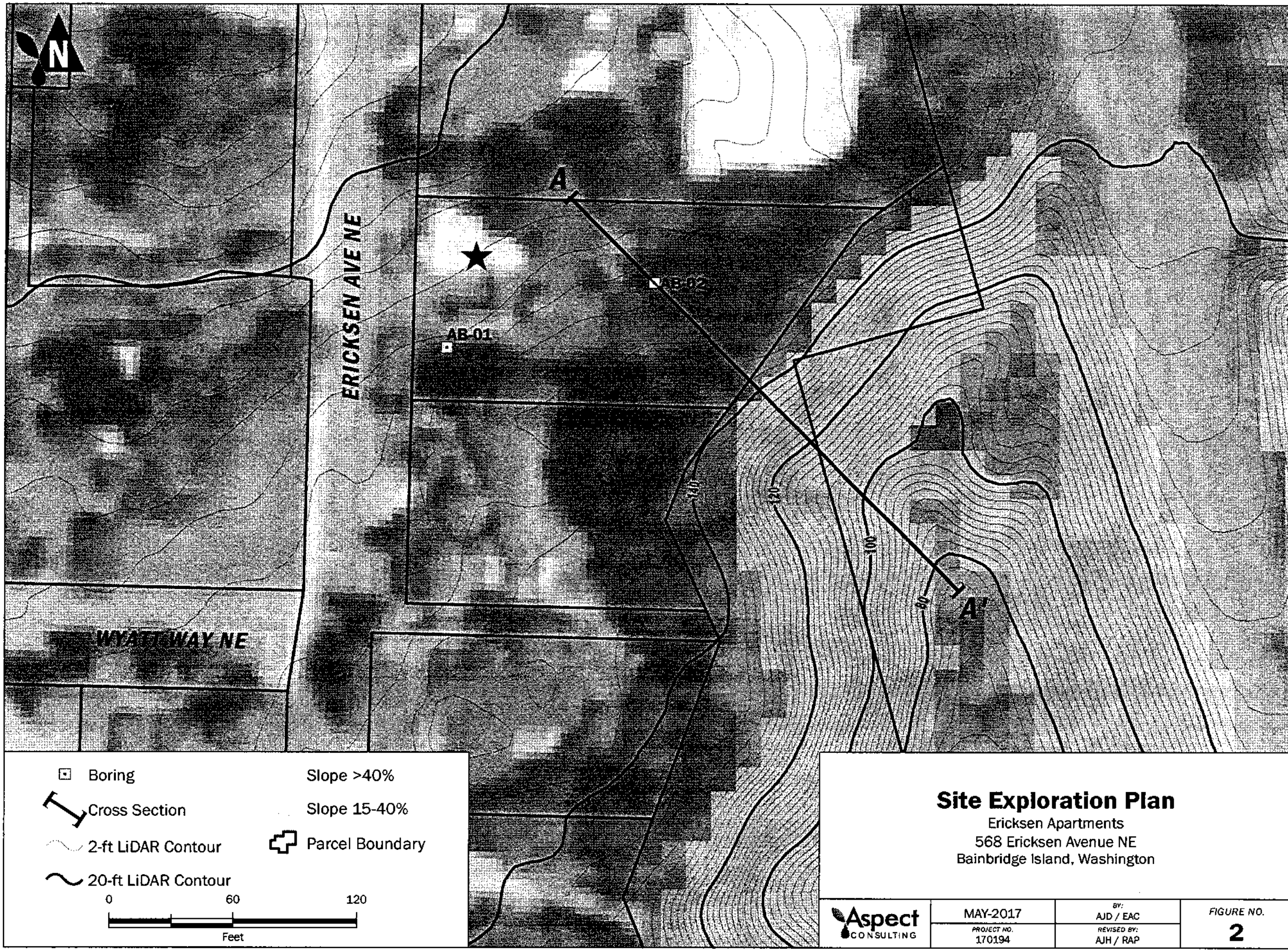


Site Location Map

Ericksen Apartments
568 Ericksen Avenue NE
Bainbridge Island, Washington

Aspect CONSULTING	MAY-2017	BY: AJD / EAC	FIGURE NO.
	PROJECT NO. 170194	REVISED BY: - - -	1

Basemap Layer Credits || Sources: Esri, HERE, DeLorme, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community


GIS Path: T:\projects_8\568EricksenApartments_170194\Delivered\02 Site Exploration Plan.mxd || Coordinate System: NAD 1983 StatePlane Washington North FIPS 4601 Feet || Date Saved: 5/22/2017 || User: ecrumbaker || Print Date: 5/22/2017
N
A
A'
AB-01
AB-02
ERICKSEN AVE NE
WYATT WAY NE
140
120
100
80
Boring
Cross Section
2-ft LiDAR Contour
20-ft LiDAR Contour
Slope >40%
Slope 15-40%
Parcel Boundary
0
60
120
Feet
Site Exploration Plan
Ericksen Apartments
568 Ericksen Avenue NE
Bainbridge Island, Washington
Aspect CONSULTING
MAY-2017
PROJECT NO.
170194
BY:
AJD / EAC
REVISED BY:
AJH / RAP
FIGURE NO.
2

APPENDIX A

Soil Boring Logs

A. Field Exploration Program

A.1. Soil Borings

Two machine drilled borings, AB-01 and AB-02 were advanced on the Site on April 26, 2017. The machine drilled borings were advanced using hollow-stem auger methods by Boretec, Inc. under subcontract to Aspect using a track-mounted drill rig that was equipped with a 140-pound autohammer. Samples were obtained every 2.5 feet to 15 feet bgs and then every 5 feet to the depths explored using the Standard Penetration Test (SPT) in general accordance with ASTM Method D1586.

The SPT method involves driving a 2-inch-outside-diameter split-barrel sampler with a 140-pound hammer free-falling from a distance of 30 inches. The number of blows for each 6-inch interval is recorded and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. If a total of 50 blows are recorded for a single 6-inch interval, the test is terminated and the blow count is recorded as 50 blows for the total inches of penetration. Samples were placed in labeled plastic jars and taken to a laboratory for further classification.

The locations of explorations are shown on Figure 2 and were collected in the field using a hand-held global positioning system (GPS) receiver. The borings were backfilled with bentonite chips and capped with about 1 foot of excavated soils, in accordance with Washington State Department of Ecology regulations.

Major Divisions				Group Symbol	Description
Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]		GW	Well-graded gravel and gravel with sand, little to no fines
				GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% Fines[5]		GM	Silty gravel and silty gravel with sand
				GC	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]		SW	Well-graded sand and sand with gravel, little to no fines
				SP	Poorly-graded sand and sand with gravel, little to no fines
		≥15% Fines[5]		SM	Silty sand and silty sand with gravel
				SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50			ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
				CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
				OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More			MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
				CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
				OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils				PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT[2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50

	Consistency	SPT[2] blows/foot
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

FC = Fines Content
GS = Grain Size
MC = Moisture Content
AL = Atterberg Limits
C = Consolidation
DD = Dry Density
K = Permeability
Str = Shear Strength
Env = Environmental
PiD = Photoionization Detector

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

[3] Estimated Percentage

Percentage by Weight	Modifier
<5	Trace
5 to 15	Slightly (sandy, silty, clayey, gravelly)
15 to 30	Sandy, silty, clayey, gravelly)
30 to 49	Very (sandy, silty, clayey, gravelly)

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch

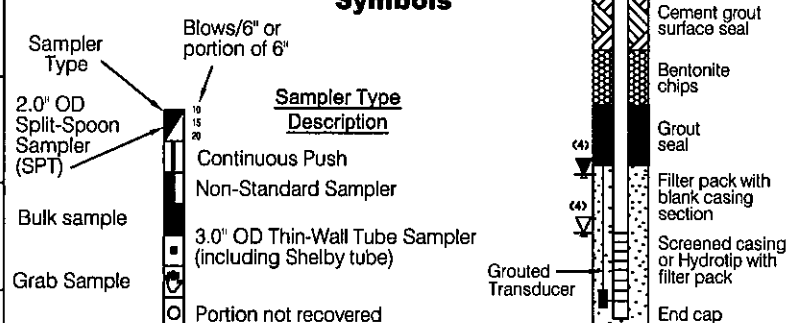
Slightly Moist - Perceptible moisture

Moist - Damp but no visible water

Very Moist - Water visible but not free draining

Wet - Visible free water, usually from below water table

Symbols



(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of groundwater — ATD = At time of drilling; Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 15% as estimated in General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)

BGS = below ground surface

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

Aspect consulting earth+water
www.aspectconsulting.com
a limited liability company

Exploration Log Key

DATE:
DESIGNED BY:
DRAWN BY:
REVISED BY:
PROJECT NO.
FIGURE NO.
A-1

Q:_ACAD Standards\Standard Details\Exploration Log Key A1.dwg

Aspect CONSULTING

Ericksen Apartments - 170194

Geotechnical Exploration Log

Project Address & Site Specific Location	Coordinates (Lat,Lon WGS84)	Exploration Number
568 Ericksen Ave NE, next to driveway	47.629, -122.517 (est)	**AB-01**

Contractor	Equipment	Sampling Method	Ground Surface (GS) Elev. (NAVD88)
Boretec1, Inc.	EC95 Track Drill	Autohammer; 140 lb hammer; 30" drop	151'(est)

Operator	Exploration Method(s)	Work Start/Completion Dates	Top of Casing Elev. (NAVD88)	Depth to Water (Below GS)
Juan Carlos	Hollow-stem auger	4/26/2017	NA	No Water Encountered

Exploration Completion and Notes: Borehole backfilled with bentonite chips and capped with sod.

Sample Type/ID	Depth (ft, approx.)	Blows/6"	Tests
S1	2.5–4	1, 1, 0	
S2	5–6.5	2, 6, 15	
S3	7.5–9	9, 11, 11	
S4	10–11.5	6, 9, 9	
S5	12.5–14	4, 5, 8	
S6	15–16.5	17, 30, 37	FC FC=33%
S7	20–21	14, 50/6	

Depth (ft)	Elev. (ft)	Description
0–5	151–146	**FILL** Very loose, moist, brown to dark brown, gravelly, silty SAND (SM); rounded fine gravel.
5–9	146–142	**WEATHERED VASHON TILL** Medium dense, moist, brown to light gray and brown, slightly gravelly, slightly silty SAND (SP-SM) interbedded with moist, light gray and brown, slightly gravlley, slightly sandy SILT (ML); rounded to subrounded fine gravel, trace organic fragments, moderate iron-oxide staining.
9–12	142–139	Medium dense, very moist, light grayish brown, very sandy SILT (ML); low plasticity, fine to medium sand, silty sand pockets.
12–14	139–137	Medium dense, very moist, brown, very silty SAND (SM); trace rounded fine gravel, trace iron-oxide staining, silty sand pockets.
14–21	137–130	**VASHON TILL** Very dense, moist, brown to light gray, very silty SAND (SM); trace subrounded fine gravel, subrounded fine to coarse sand.
20	131	Becomes gravelly and silty; fine to coarse gravel, stratified, trace iron-oxide staining.
21	130	Bottom of exploration at 21 ft. bgs.

Legend

Sample Method:
- No Soil Sample Recovery
- Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├──┤ Liquid Limit

Water Level: No Water Encountered

See Exploration Log Key for explanation of symbols

Logged by: NHC
Approved by: AJD 5/19/2017

Exploration Log AB-01

Aspect CONSULTING	Ericksen Apartments - 170194			Geotechnical Exploration Log	
	Project Address & Site Specific Location			*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	568 Ericksen Ave NE, backyard			47.629, -122.516 (est)	**AB-02**
Contractor	*Equipment*	*Sampling Method*		*Ground Surface (GS) Elev. (NAVD88)*	
Boretec1, Inc.	EC95 Track Drill	Autohammer; 140 lb hammer; 30" drop		149'(est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*		*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Juan Carlos	Hollow-stem auger	4/26/2017		NA	41.7' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/6"	Tests	Description	Depth (ft)
						COLLUVIUM Loose, moist, brown, gravelly, silty SAND (SM); subrounded fine to coarse gravel, predominantly fine sand, trace rootlets, moderate iron-oxide staining.	
1	148	Borehole backfilled with bentonite chips and capped with sod.					1
2	147						2
3	146		S1	3 2 6			3
4	145						4
5	144		S2	12 20 41		Becomes dense and very moist.	5
6	143					**VASHON TILL** Very dense, slightly moist, brown, sandy, silty GRAVEL (GM); rounded fine to coarse gravel.	6
7	142					Very dense, slightly moist, gravelly, silty SAND (SM); fine to coarse sand, subangular to subrounded fine to coarse gravel, trace iron-oxide staining, socketing.	7
8	141		S3	18 40 48			8
9	140						9
10	139		S4	40 50/6		Very dense, slightly moist, brown, very sandy, silty GRAVEL (GM); fine sand, coarse gravel.	10
11	138						11
12	137					Very dense, very moist, brown, slightly gravelly, silty SAND (SM); predominantly fine to medium sand, subrounded fine gravel, trace iron-oxide staining	12
13	136		S5	21 37 34			13
14	135					Very dense, very moist, brown, very sandy SILT (ML); fine sand.	14
15	134		S6	28 50/6	GS FC=5%	Very dense, moist to very moist, gray, gravelly SAND (SW); trace silt, fine to coarse sand, subangular to subrounded fine gravel.	15
16	133						16
17	132						17
18	131					Very dense, moist, light grayish brown, SAND (SP); trace silt, predominantly fine sand.	18
19	130						19
20	129		S7	20 27 40			20
21	128						21
22	127						22
23	126					Very dense, moist to very moist, brown, gravelly, silty SAND (SM); fine to coarse sand, subrouned to rouned fine to coarse gravel.	23
24	125						24

Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50

Legend

Sample Method: No Soil Sample Recovery; Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├──┤ Liquid Limit

Water Level: ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: NHC
Approved by: AJD 5/19/2017

Exploration Log AB-02

Aspect Consulting	Ericksen Apartments - 170194		Geotechnical Exploration Log	
	Project Address & Site Specific Location		*Coordinates (Lat,Lon WGS84)*	*Exploration Number*
	568 Ericksen Ave NE, backyard		47.629, -122.516 (est)	**AB-02**
Contractor	*Equipment*	*Sampling Method*	*Ground Surface (GS) Elev. (NAVD88)*	
Boretec1, Inc.	EC95 Track Drill	Autohammer; 140 lb hammer; 30" drop	149'(est)	
Operator	*Exploration Method(s)*	*Work Start/Completion Dates*	*Top of Casing Elev. (NAVD88)*	*Depth to Water (Below GS)*
Juan Carlos	Hollow-stem auger	4/26/2017	NA	41.7' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/6"	Description
25–27	124–122		S8	20, 50/5	Very dense, moist to very moist, brown, gravelly, silty SAND (SM); fine to coarse sand, subrouned to rouned fine to coarse gravel. (continued)
27.5–32.5	121.5–116.5		S9 (30')	29, 50/4	Very dense, very moist, brown, gravelly SAND (SW); trace silt, subrounded fine to coarse sand, subrounded fine to coarse gravel.
32.5–36	116.5–113		S10 (35')	17, 29, 44	Very dense, wet, brown to light gray, slightly silty SAND (SP-SM); rapid dilatancy, fine sand.
36–38	113–111				Very dense, moist, brown, very silty SAND (SM); predominantly fine sand, diamict texture, trace iron-oxide staining.
38–43	111–106	▽ 4/26/2017 (41.7')	S11 (40')	50/6	Very dense, moist, light grayish brown, very sandy GRAVEL (GW); trace silt, predominantly fine to medium sand, subangular fine to coarse gravel.
43–50	106–99		S12 (45')	29, 50/6	Very dense, wet, light grayish brown SAND (SP); trace silt, predominantly fine sand.

Depth scale: 26 (123), 27 (122), 28 (121), 29 (120), 30 (119), 31 (118), 32 (117), 33 (116), 34 (115), 35 (114), 36 (113), 37 (112), 38 (111), 39 (110), 40 (109), 41 (108), 42 (107), 43 (106), 44 (105), 45 (104), 46 (103), 47 (102), 48 (101), 49 (100)

Columns also shown: Blows/foot ▲, Water Content (%) ● (0 10 20 30 40 50), Tests, Material Type.

Legend

Sample Method: No Soil Sample Recovery; Split Barrel 2" X 1.375" (SPT)

Plastic Limit ├──┤ Liquid Limit

Water Level: ▽ Water Level ATD

See Exploration Log Key for explanation of symbols

Logged by: NHC
Approved by: AJD 5/19/2017

Exploration Log
AB-02
Sheet 2 of 3

Aspect CONSULTING	Ericksen Apartments - 170194		Geotechnical Exploration Log	
	Project Address & Site Specific Location: 568 Ericksen Ave NE, backyard		Coordinates (Lat,Lon WGS84): 47.629, -122.516 (est)	Exploration Number: AB-02
Contractor: Boretec1, Inc.	Equipment: EC95 Track Drill	Sampling Method: Autohammer; 140 lb hammer; 30" drop	Ground Surface (GS) Elev. (NAVD88): 149'(est)	
Operator: Juan Carlos	Exploration Method(s): Hollow-stem auger	Work Start/Completion Dates: 4/26/2017	Top of Casing Elev. (NAVD88): NA	Depth to Water (Below GS): 41.7' (ATD)

Depth (feet)	Elev. (feet)	Exploration Completion and Notes	Sample Type/ID	Blows/foot ▲ Water Content (%)● 0 10 20 30 40 50	Blows/6"	Tests	Material Type	Description	Depth (ft)
51	98		S13	▲	25 44 50/5			Very dense, wet, light grayish brown SAND (SP); trace silt, predominantly fine sand. (continued)	51
52	97							Bottom of exploration at 51.4 ft. bgs.	52
53	96								53
54	95								54
55	94								55
56	93								56
57	92								57
58	91								58
59	90								59
60	89								60
61	88								61
62	87								62
63	86								63
64	85								64
65	84								65
66	83								66
67	82								67
68	81								68
69	80								69
70	79								70
71	78								71
72	77								72
73	76								73
74	75								74

Legend

Plastic Limit ├──┤ Liquid Limit

Sample Method:
- No Soil Sample Recovery
- Split Barrel 2" X 1.375" (SPT)

Water Level:
- ∇ Water Level ATD

See Exploration Log Key for explanation of symbols

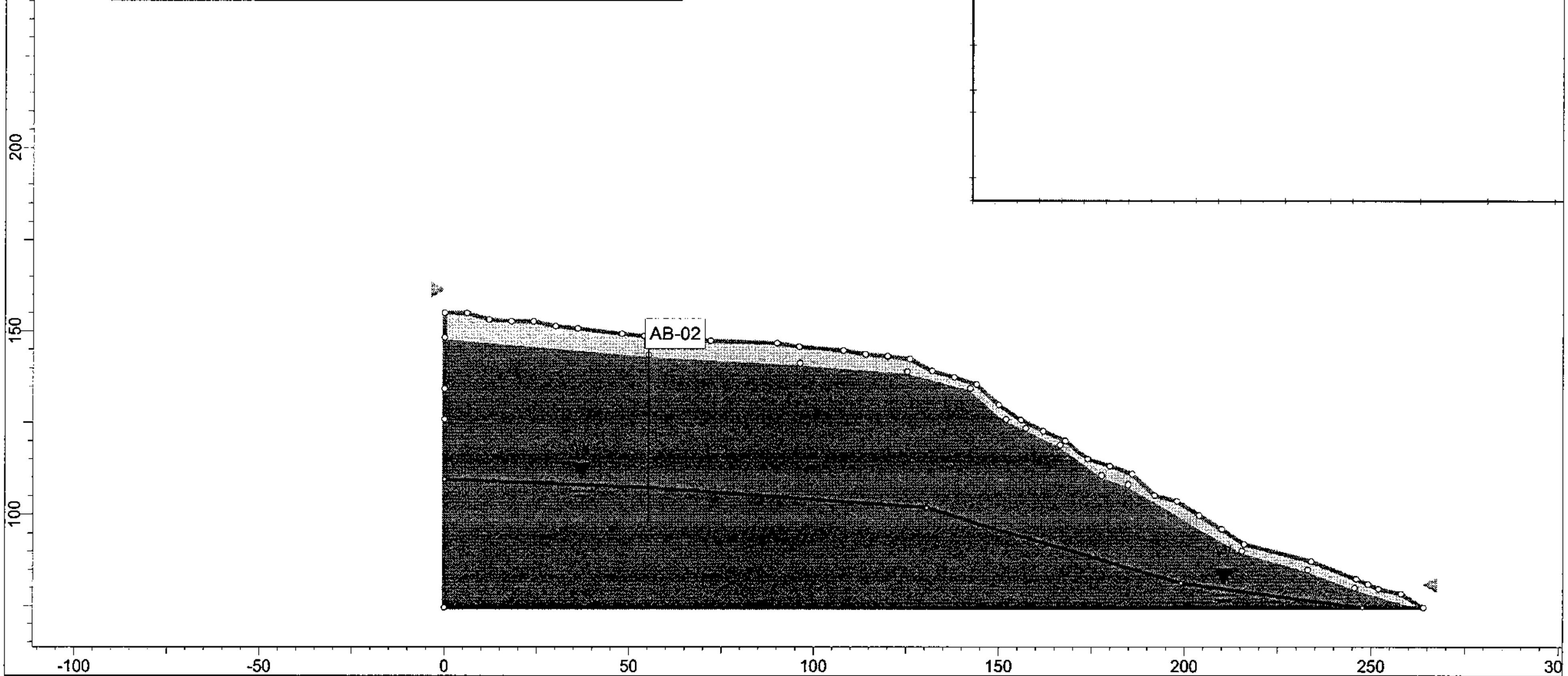
Logged by: NHC
Approved by: AJD 5/19/2017

Exploration Log AB-02

APPENDIX B

Slope Stability Analyses

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium		120	Mohr-Coulomb	0	30
Vashon Till		130	Mohr-Coulomb	0	40



Legend

- Search Grid
- Search Limits
- Modelled Groundwater Level
- Boring Location and Depth

Existing Conditions Model Setup

Slope Stability Analysis

Ericksen Apartments

568 Eriksen Apartments

SCALE: 1" = 40'

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6/18/2018

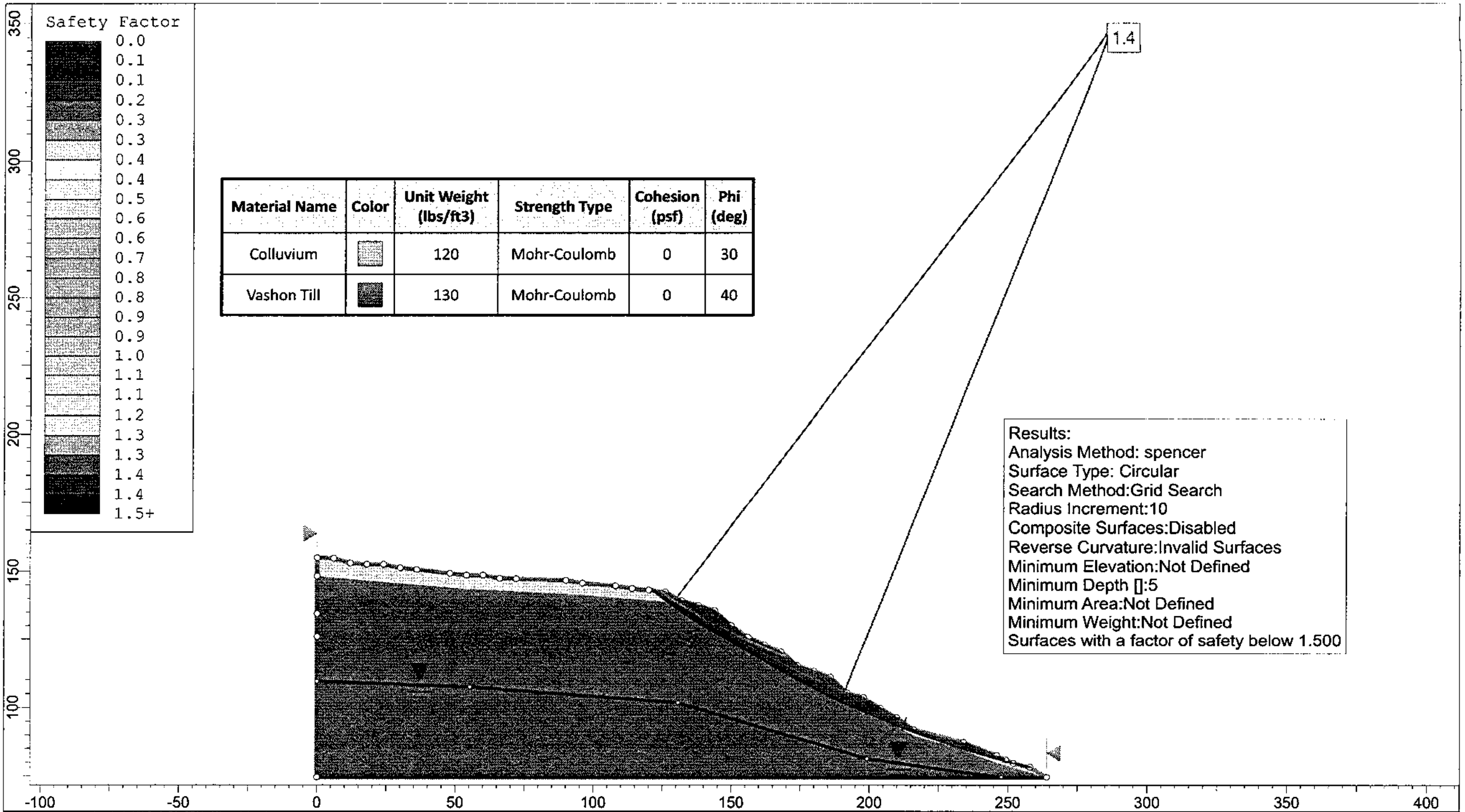
PROJECT NO. 170194

BY: NHC

REVISED BY: AJH

APPENDIX: B-1

SLIDE 7.024



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium		120	Mohr-Coulomb	0	30
Vashon Till		130	Mohr-Coulomb	0	40

Existing Conditions
Static

Slope Stability Analysis

Ericksen Apartments
568 Eriksen Apartments

SLIDEINTERPRET 7.024

SCALE: 1" = 50'

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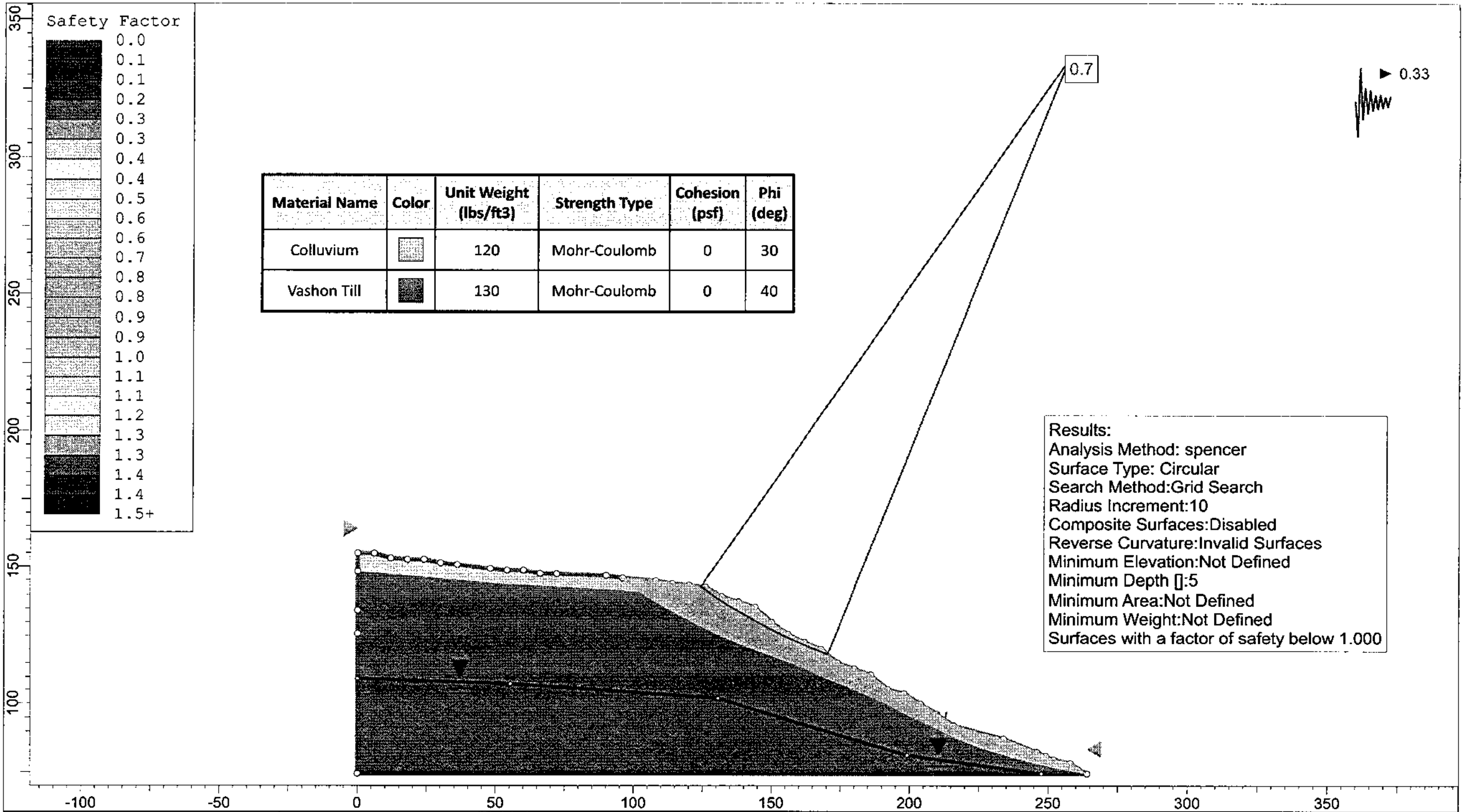
REVISED BY: AJH

APPENDIX: **B-2**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium		120	Mohr-Coulomb	0	30
Vashon Till		130	Mohr-Coulomb	0	40

Results:
Analysis Method: spencer
Surface Type: Circular
Search Method:Grid Search
Radius Increment:10
Composite Surfaces:Disabled
Reverse Curvature:Invalid Surfaces
Minimum Elevation:Not Defined
Minimum Depth []:5
Minimum Area:Not Defined
Minimum Weight:Not Defined
Surfaces with a factor of safety below 1.000

Existing Conditions
Seismic

Slope Stability Analysis

Ericksen Apartments
568 Eriksen Apartments

SLIDEINTERPRET 7.024

SCALE: 1" = 50'

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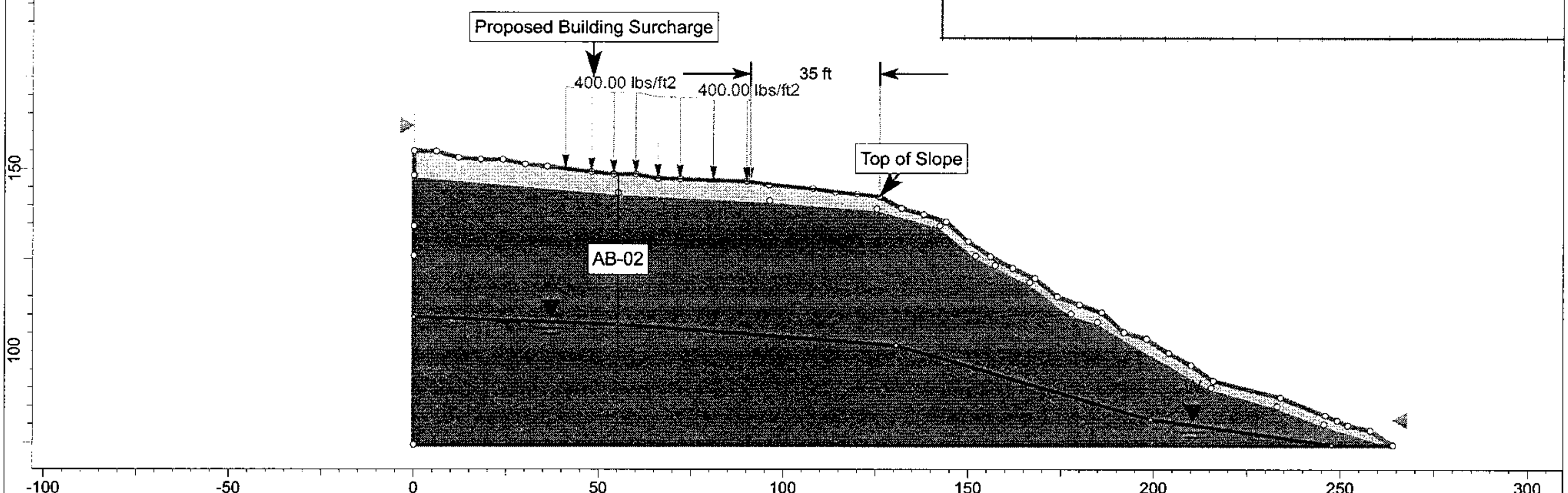
PROJECT NO. 170194

BY: NHC

REVISED BY: AJH

APPENDIX: B-3

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium		120	Mohr-Coulomb	0	30
Vashon Till		130	Mohr-Coulomb	0	40



Legend
- Search Grid
- Search Limits
- Modelled Groundwater Level
- Boring Location and Depth

Proposed Conditions Model Setup

Slope Stability Analysis

Ericksen Apartments
568 Eriksen Apartments

SCALE: 1" = 40'

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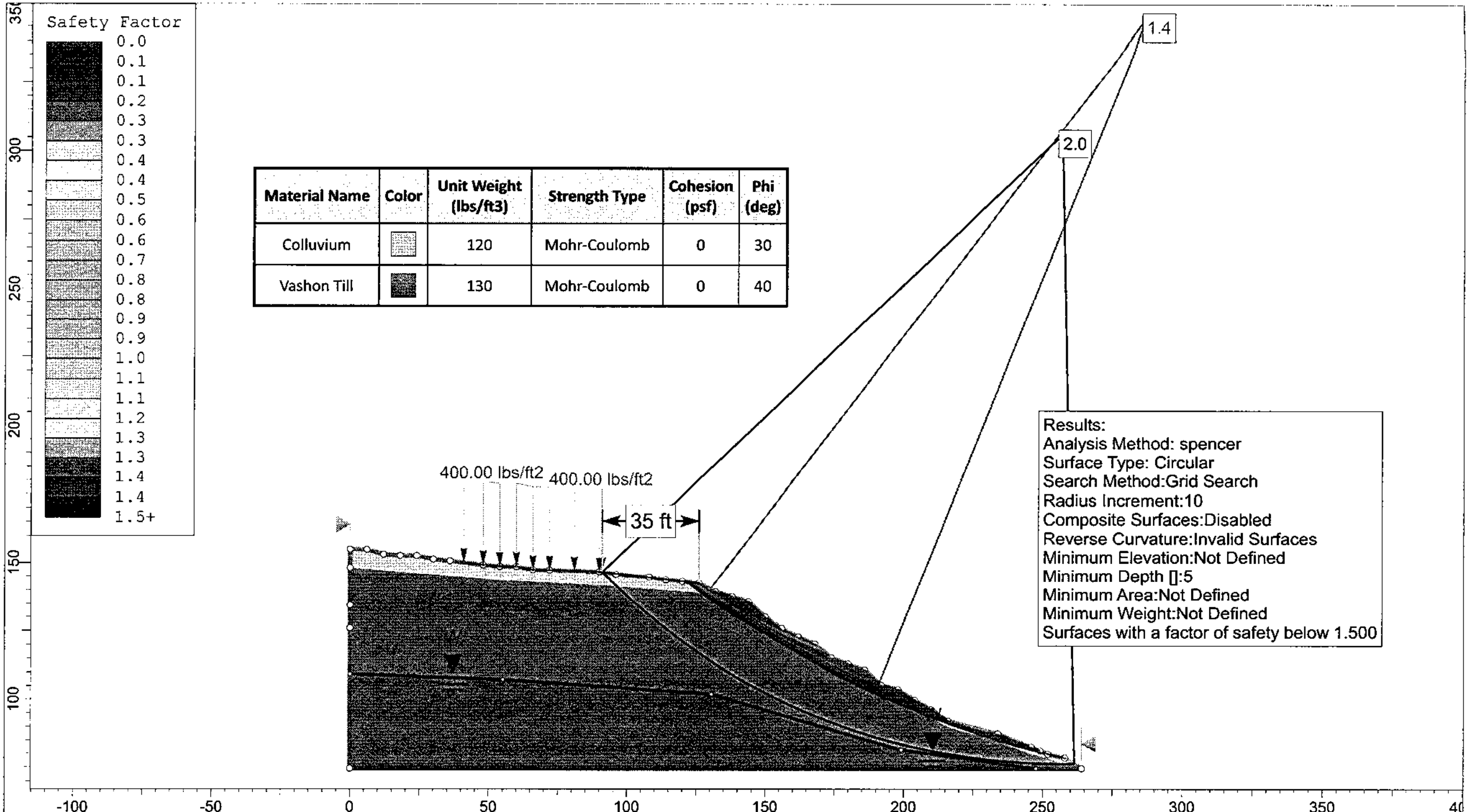
PROJECT NO. 170194

BY: NHC

REVISED BY: AJH

APPENDIX: B-4

SLIDE 7.024



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium		120	Mohr-Coulomb	0	30
Vashon Till		130	Mohr-Coulomb	0	40

Legend

- Search Grid
- Search Limits
- Modelled Groundwater Level
- Boring Location and Depth

Proposed Conditions
Static Conditions

Slope Stability Analysis

Ericksen Apartments
568 Eriksen Apartments

SLIDEINTERPRET 7.024

SCALE: 1" = 50'

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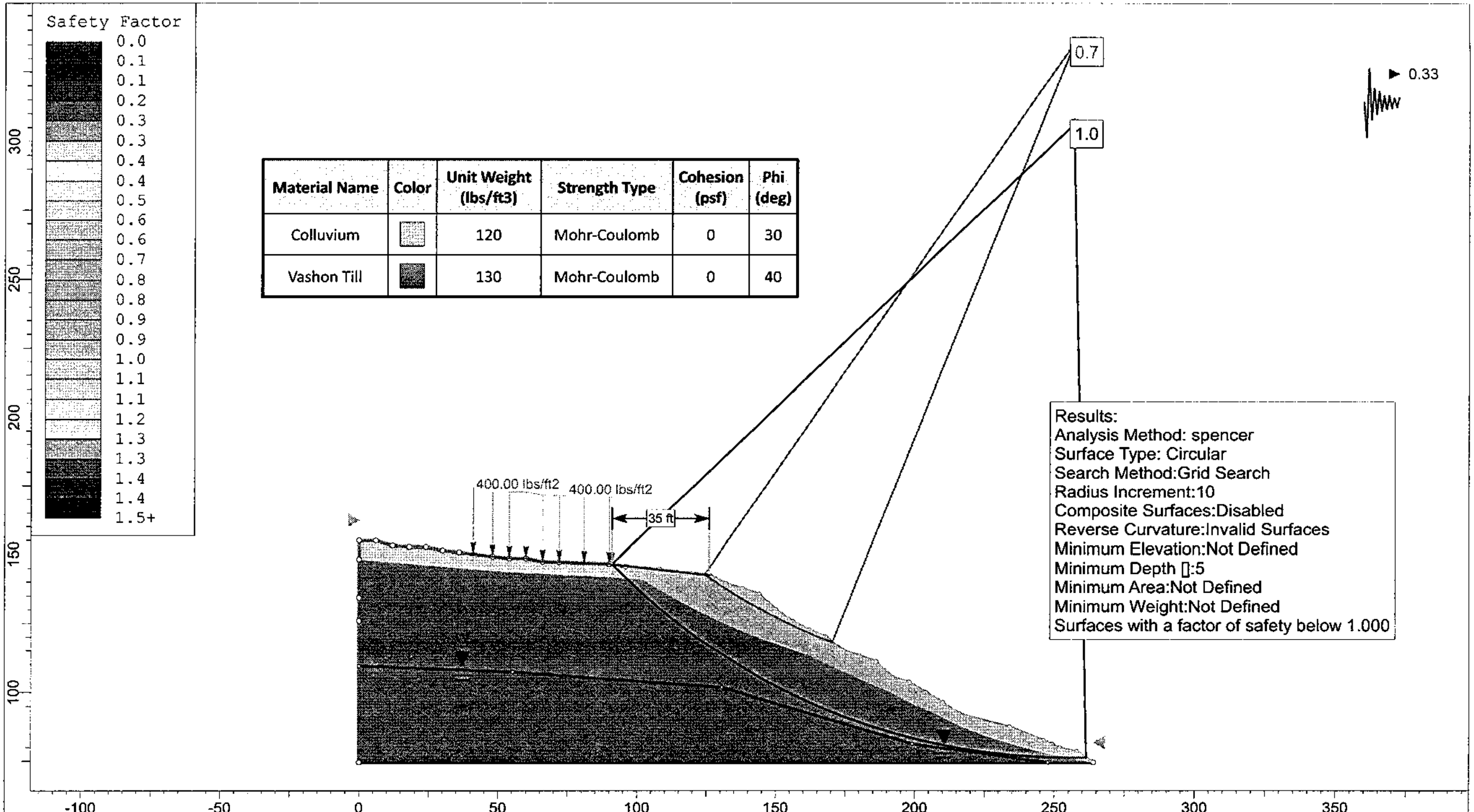
PROJECT NO. 170194

BY: NHC

REVISED BY: AJH

APPENDIX: **B-5**



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Colluvium		120	Mohr-Coulomb	0	30
Vashon Till		130	Mohr-Coulomb	0	40

Legend

- Search Grid
- Search Limits
- Modeled Groundwater Level
- Boring Location and Depth

Proposed Conditions
Seismic Conditions

Slope Stability Analysis

Ericksen Apartments
568 Eriksen Apartments

SLIDEINTERPRET 7.024

SCALE: 1" = 50'

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6/20/2018

PROJECT NO. 170194

BY: NHC

REVISED BY: AJH

APPENDIX: **B-6**

APPENDIX C

Report Limitations and Guidelines for Use

REPORT LIMITATIONS AND GUIDELINES FOR USE

This Report and Project-Specific Factors

Aspect Consulting, LLC (Aspect) considered a number of unique, project-specific factors when establishing the Scope of Work for this project and report. You should not rely on this report if it was:

- Not prepared for you
- Not prepared for the specific purpose identified in the Agreement
- Not prepared for the specific real property assessed
- Completed before important changes occurred concerning the subject property, project or governmental regulatory actions

Geoscience Interpretations

The geoscience practices (geotechnical engineering, geology, and environmental science) require interpretation of spatial information that can make them less exact than other engineering and natural science disciplines. It is important to recognize this limitation in evaluating the content of the report. If you are unclear how these "Report Limitations and Use Guidelines" apply to your project or site, you should contact Aspect.

Reliance Conditions for Third Parties

This report was prepared for the exclusive use of the Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against liability claims by third parties with whom there would otherwise be no contractual limitations. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with our Agreement with the Client and recognized geoscience practices in the same locality and involving similar conditions at the time this report was prepared

Property Conditions Change Over Time

This report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by events such as a change in property use or occupancy, or by natural events, such as floods, earthquakes, slope instability, or groundwater fluctuations. If any of the described events may have occurred following the issuance of the report, you should contact Aspect so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Discipline-Specific Reports Are Not Interchangeable

The equipment, techniques, and personnel used to perform a geotechnical or geologic study differ significantly from those used to perform an environmental study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually address any environmental findings, conclusions, or recommendations (e.g., about the likelihood of encountering underground storage tanks or regulated contaminants). Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding the subject property.

We appreciate the opportunity to perform these services. If you have any questions please contact the Aspect Project Manager for this project.