GEOTECHNICAL ENGINEERING STUDY

For

Hidden Ridge Estates

City of Camas, Washington

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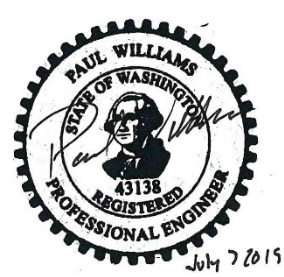

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1.0 Introduction

This report presents the results of the geotechnical engineering study completed by Engineering Northwest for the proposed Hidden Ridge Estates subdivision project in the City of Camas, Washington. The general location of the site is shown on the Vicinity Map, figure 1. The site includes one parcels, which total approximately 7.61 acres. At the time our study was performed, the site and our exploratory locations were approximately as shown on the site plan map, figure 1.

The purpose of this study was to explore subsurface conditions at the site, and based on the conditions encountered, provide geotechnical recommendations for the proposed construction. This report is subject to the limitations expressed in Section 6.0, Conclusion and Limitations.

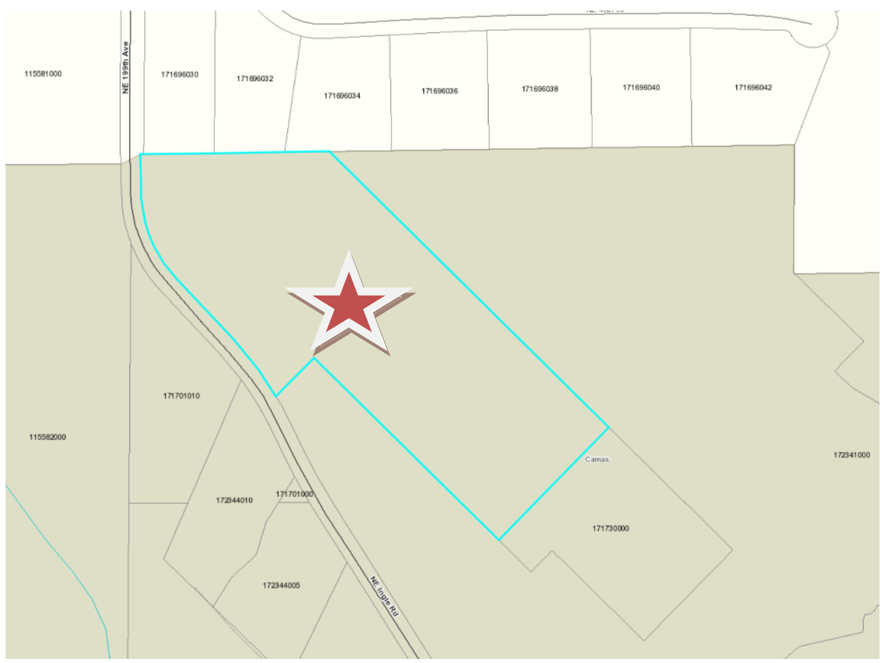


FIGURE 1

1.1 General Site Information

As indicated on Figures 1 the 7.61-acre subject site is located eastside of NE Ingle Road in the City of Camas, Washington. The approximate latitude and longitude are 45°39'19.72"N and 122°27'56.85"W and the legal description is a portion of the SW ¼ Section 17 Township 2N, Range 3E, Willamette Meridian. The regulatory jurisdictional agency is the City of Camas, Washington.

1.2 Proposed Development

The proposed Hidden Ridge Estates subdivision project is located on eastside of NE Ingle Road. The subdivision project proposes 14 lots served by proposed private road. The project includes the future construction of utility, public road, a storm water facility and other related infrastructural improvements.

2.0 Regional Geology and Soil Conditions

According to the Geologic Map of the Vancouver Quadrangle, Washington and Oregon (Washington division of Geology and Earth Resources, Open File Report 87-10, Revised November 1987), near-surface geology is:

Geologic Unit Age: Pleistocene
Geologic Unit Name: Basaltic andesite of Green Mountain
Unit Description: Olivine phyric basaltic andesite erupted from cinder cone at west end of Green Mountain. Light-gray, microvesicular, generally platy lava flow, extending about 1 km to northwest of Green Mountain, consists of olivine phenocrysts (2-4 percent; 0.5 to 3 mm across; contains inclusions of chromian spinel; rims variably replaced by iddingsite) in a fine-grained trachytic groundmass of plagioclase, clinopyroxene, orthopyroxene, and Fe-Ti oxide; locally contains quartzite pebbles and small, dark, fine-grained clots that may be sedimentary xenoliths, both presumably derived from underlying gravels (units QTc and Ttfc). Conical hill at west end of Green Mountain consists largely of deeply weathered basaltic ash; platy basaltic andesite lava crops out at summit and presumably fills vent. Lava flow has normal magnetic polarity (J.T. Hagstrum, written commun., 1999) and yielded an 40Ar/39Ar age of 575+/-7 ka (table 2)
Age-Lithology: Quaternary volcanic rocks

The Soil Survey of Clark County, Washington (United States Department of Agriculture, Soil Conservation Service [USDA SCS], November 1972) identifies three separate surface soils with the subject property. Although actual on-site soils may vary from the broad USDA descriptions, soil types and associated descriptions are presented below.

- Olympic (OmE): The Olympic series consists of well drained, gently sloping to very steep soils underlain by basalt bedrock at a depth of 40 inches or more. Hydrologic soil group "B"

3.0 Regional Seismology

Recent research and subsurface mapping investigations within the Pacific Northwest appear to suggest the historic potential risk for a large earthquake event with strong localized ground movement may be underestimated. Past earthquakes in the Pacific Northwest appear to have caused landslides and ground subsidence, in addition to severe flooding near coastal areas. Earthquakes may also induce soil liquefaction, which occurs when elevated horizontal ground acceleration and velocity cause soil particles to interact as a fluid as opposed to a solid. Liquefaction of soil can result in lateral spreading and temporary loss of bearing capacity and shear strength.

There are at least four major known fault zones in the vicinity of the site that may be capable of generating potentially destructive horizontal accelerations. These fault zones are described briefly in the following text.

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of server northwest-trending faults located along the eastern boundary of the Portland Hills. The fault zone is approximately 12 miles in length and is located approximately 8.5 southwest of the site. According to Seismic Design Mapping, State of Oregon (Geomatrix Consultants, 1995), there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested, including various strike-slip and dipping thrust fault theories. Evidence exists to suggest that fault movement has impacted shallow Holocene deposits and deeper Pleistocene sediments. Seismologists recorded a M3.9 earthquake thought to be associated with the fault zone near Kelly Point Park in April 2003, and a M3.5 earthquake possibly associated with the fault zone occurred approximately 1.3 miles east of the fault in 1991. Therefore, the Portland Hills Fault Zone is generally thought to be potentially active and capable of producing possible damaging earthquakes.

Gales Creek-Newberg-Mt. Angel Fault Zone

Located approximately 30 miles south of the site, the 50-mile long Gales Creek-Newberg-Mt. Angel Structural Zone consists of a series of discontinuous northwest-trending faults. Possible late-Quaternary geomorphic surface deformation may exist along the structural zone (Geomatrix Consultants, 1995). Although no definitive evidence of impacts to Holocene sediments has reportedly been observed, a M5.6

earthquake occurred in March 1993 near Scotts Mills, approximately four mile south of the mapped extent of the Mt. Angel fault. It is unclear if the earthquake occurred along the fault zone or a parallel structure. Therefore, the Gales Creek-Newberg-Mt Angel Structural Zone is considered potentially active.

<u>Lacamas Creek-Sandy River Fault Zone</u>

The northwest-trending Lacamas Creek Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 3.5 southeast of the site. According to Geologty and Groundwater Conditions of Clark County Washington (USGS Water Supply Paper 1600, Mundorff, 1964) and the Geologic Map of the Lake Owego Quandrangle (Oregon DOGAMI Series GMS-59, 1989), the Lacamas Creek fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site. Recorded mild seismic activity during the recent past indicates this area may be potentially seismogenic.

<u>Cascadia Subduction Zone</u>

The Cascadia Subduction Zone has recently been recognized as a potential source of strong earthquake activity in the Portland/Vancouver Basin. This phenomenon is the result of the earth's large tectonic plate movement. Geologic evidence indicates that volcanic ocean floor activity along the Juan de Fuca ridge in the Pacific Ocean causes the Juan de Fuca Plate to perpetually move east and subduct under the North American Continental Plate. The subduction zone results in historic volcanic and potential earthquake activity in proximity to the plate interface, believed to lie approximately 20 to 60 miles west of the general location of the Oregon and Washington coast (Geomatrix Consultants, 1995).

4.0 Geotechnical Field Investigation

A geotechnical field investigation consisting of visual reconnaissance and one test pit explorations (TP-1) was conducted at the site. Test pit exploration was performed with a hand auger. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Disturbed soil samples were collected from relevant soil horizons and analyzed. Test pit locations are indicated on Figure 2.

4.1 Subsurface Exploration and Investigation

We explored subsurface conditions for one test pits (TP-1) were excavated at the site to a maximum depth of 6 feet on June 1, 2019. The approximate locations of the test pit are shown in Figure 2.

Select soil samples from the test pit were tested to determine the natural moisture content, dry density, organic content.

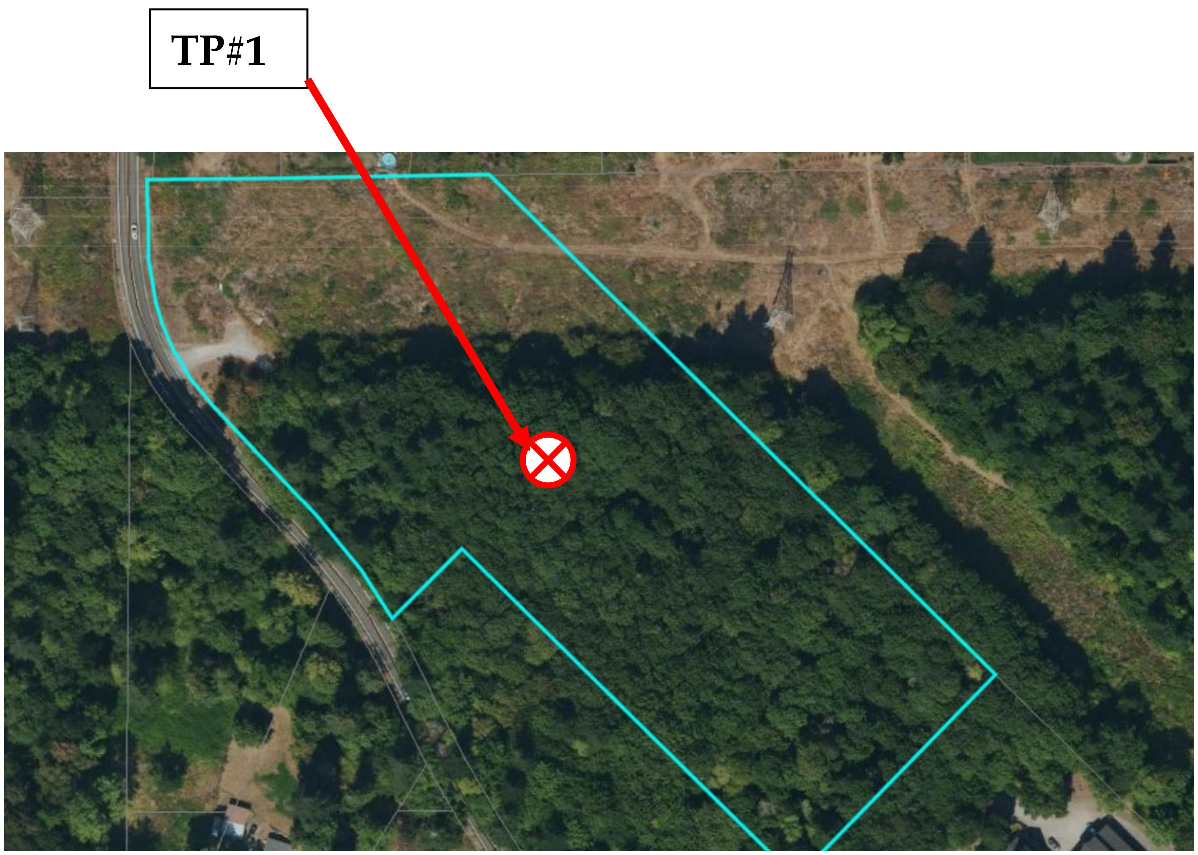


Figure 2

4.2 Surface Investigation and Site Description

The surface topography of the subject property typical slopes from east to west. The majority of the site is covered with, with several large and small trees.

In general, the surface soil over the majority of the project site varies from dark reddish brown to dark brown which is typical Olympic soil type. The near- surface soil conditions has relatively high organic content extended to an approximate depth of 10 inches.

Laboratory testing of selected samples resulted in soil moisture content varying between 7 and 10 percent.

4.2.1 Soil Type Description

The field sample soil results are listed below.

Soil	USDA Texture	Unified	AASHO	Hydrologic Group
OmE	Clay Loam	ML	A-7	B

5.0 Design Recommendations

The geotechnical site investigation suggests that proposes development is generally compatible with surface and subsurface soils, provided the recommendations presented in this report are used and incorporated into the design and construction process.

5.1 Site Preparation and Grading

Trees, grasses, and other grubbing items should be removed from all building, slab, structural fill, and pavement and sidewalk areas. Root balls should be grubbed out to the depth of the roots, which could exceed 2.0 to 3.0 feet depth. Depending on the methods used to remove the root balls, considerable disturbance and lessening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

The existing topsoil/root zone should be stripped and removed from all proposed building, slab, structural fill, pavement and sidewalk areas, and for a 5-foot margins around such areas. Based on our explorations, the average depth of stripping in will be approximately 10 inches. Greater stripping depths will be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field site for disposal or used in landscaped areas.

The near-surface soil conditions included till zone material to a depth of approximately 6 to 12 inches over the northern portion parcel (BPA power lines). The till zone material consists of the on-site silty material, generally includes variable organic content, and typically exhibits signs of softening/ disturbance through years of cultivation. After site stripping has been completed, we recommend scarifying the remainder of the tripped ground surface to the depth of the till zone within all building and paved fill areas prior to placing structural fill. As discussed in the "Structural Fill" section of this report, the on-site silt can be sensitive to small changes in moisture content and will be difficult, if not impossible, to compact adequately during wet weather. Accordingly, scarification and compaction of the subgrade will likely only be possible during extended dry periods and following moisture conditioning of the soil. As an alternative, Portland cement treatment, or a combination of cement and lime treatment, may be possible for this zone.

After grubbing, stripping, and required site cutting have been completed, we recommend proof rolling the subgrade with a fully-loaded dump truck or similar size, rubber tire construction equipment to identify areas of excessive yielding. The proof rolling should be observed by a geotechnical engineering who will evaluate the subgrade. If areas of excessive yielding are identified, the material should be excavated and replaced with compacted materials recommended for structural fill. Areas that appear to be too wet and soft to support proof rolling equipment should be prepared in accordance with the recommendations for wet weather construction presented in the following section of this report

The test pit excavations were backfilled using the relatively minimal compactive effort of the hoe bucket; therefore, soft spots can be expected at these locations. We recommend that these relatively uncompacted soils be removed from the test pits to a depth of 3.0 feet below finished subgrade. The resulting excavation should be brought back to grade with structural fill.

5.2 Engineered Structural Fill

The native silt can be used as structural fill provided it is adequately moisture conditioned. Silty soils are generally sensitive to small changes in moisture content and are difficult, if not impossible, to compact adequately during wet weather or when their moisture content is more than a few percentage points above the optimum moisture content. Some moisture conditioning will likely be necessary even during the dry weather construction season. We recommend using clean. Angular imported granular material for structural fill if site soils cannot be properly moisture conditioned. As an alternative, use of the native silt soil as structural fill may be acceptable if it is properly amended with Portland cement or lime.

When used as structural fill, the silt soils should be placed in lifts with a maximum uncompact thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density, as determined by American Society for Testing and Materials (ASTM) D 1557.

Engineered structural fill should be placed in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. A field density at least equal to 90 percent of the maximum dry density, obtained from the modified Proctor moisture-density relationship test (ASTM D1557), is recommended for structural fill placement. For engineered structural fill placed on sloped grades, the area should be benched to provide a horizontal surface for compaction. Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938-08. Field compaction testing should be performed for each vertical foot of engineered fill placed. Engineered fill placement should be observed by an experienced geotechnical engineer or designated representative. Engineered structural fill placement occurs during dry weather conditions, clean non-organic achieve recommended compaction specifications. If adequate compaction is not achievable with clean native soils, import structural fill consisting of well-graded granular material with a maximum particle size of 3 inches and no more than 5 percent passing the No. 200 sieve is recommended. Representative samples of proposed engineered structural fill should be submitted for laboratory analysis and approval by the geotechnical engineer prior to placement. Laboratory analyses should include particle-size gradation and modified Proctor moisture-density analysis.

Trench backfill for the utility pipe base and pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 8 percent by weight passing the U.S. Standard No. 200 Sieve. The material should be free of roots, organic matter, and other unsuitable materials. Backfill for the pipe base and pipe zone should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as recommended by the pipe manufacturer. Within building and pavement areas, trench backfill placed above the pipe zone should be compacted to at least 92 percent of ASTM D 1557 at depths greater than 2.0 feet below the finished subgrade and as recommended for structural fill within 2 feet of finished subgrade. In all other areas, trench backfill above the pipe zone should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557.

As an alternative to the use of imported granular material, an experienced contractor may be able to amend the on-site with Portland cement to obtain suitable support properties. Depending on the size of the area, it is generally less costly to amend on-site soils than to remove and replace soft soils with granular material. Based on the moisture contents, soil types, and processing speed, cement amendment would be more suitable at this site than lime amendment.

The amount of cement used to amend the soils generally varies with moisture content. It is difficult to predict field performance of soils to cement amendment due to variability in soil response and we recommend laboratory testing to confirm expectations. However, for preliminary design purposes, we expect acceptable soil strength will be obtained using an amendment rate of 6 pounds Portland cement tilled to a depth of 12 inches. This translates to approximately 6 percent cement by weight. The amount of cement added to the soil may need to be adjusted based on field observations and performance.

5.2.1 Reuse of Undocumented Fill Material

As discussed in Section 5.1 Site Preparation and Grading, undocumented fill was encountered at the site. In general, the fill encountered consisted of sand and gravel. If minor debris is encountered the contractor shall notify the geotechnical engineer or designated representative. Cobbles and boulders larger than 6 inches that cannot be broken into smaller fragments should be removed. Crushing and mixing processes should be observed and approved by an experienced geotechnical engineer.

5.3 Cut and Fill Slopes

Fill placed on grades steeper than 5H:1V should be horizontally benched at least 10 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. Drainage implementations, including subdrains or perforated drain pipe trenches, may also be required in proximity to cut and fill slopes if seeps, springs, or soft mottled soils are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by the geotechnical engineer during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion. Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required.

Final cut or fill slopes should not exceed 2H:1V or 20 feet in vertical height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 5.2, Engineered Structural Fill and horizontally benching where appropriate. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face. Proper cut and fill slope construction is critical to overall project stability and should be observed by and experienced geotechnical engineer.

5.4 Foundations

Shallow foundations for the proposed structure may consist of continuous perimeter footing or column spread footing. Footing design for structures should be performed by a licensed engineer. Foundations should not be permitted to bear upon undocumented fill or disturbed soil.

Allowable bearing capacity is a function of footing dimension, embedment, and subsurface soil properties, including settlement and shear resistance. The time this report was written specific structural design loads were not available. Based on the encountered subsurface soil conditions and assuming compliance with the preceding Site Preparation and Grading section, the proposed apartment units may be supported on conventional shallow spread footing bearing on compacted structural fill.

Footings should be at least (24) inches and (18) inches in width and should extend to a depth of at least eighteen (18) inches below the lowest adjacent finished sub grade. Individual spread footings or continuous wall footings providing support for the proposed buildings may be designed for a maximum allowable bearing value of two thousand five hundred (2500) pounds per square foot (psf). ASTM soil classification for the subbase soil is CL and CH Silt and Clay.

These basic allowable bearing value are for dead plus live loads and may be increased one-third for combined dead, live, wind, and seismic forces. It is estimated that total and differential footing settlements for the relatively light building will be approximately three-quarters and one-quarter inch, respectively.

Lateral loads can be resisted by friction between the foundation and the supporting sub grade or by passive earth pressure acting on the buried portions of the foundation. For the latter, the foundations must be poured "neat" against the existing soil or back filled with a compacted fill meeting the requirements of structural fill.

Passive Pressure	= 300 pcf equivalent fluid weight
Coefficient of friction	=0.40
Lateral Bearing	= 200 psf

We recommend that all footing excavations be observed by a representative of CNE services prior to placing forms or rebar, to verify that sub grade support conditions are as anticipated in this report, and/or provide modifications in the design as required.

Floor Slabs

Satisfactory subgrade support for slabs-on-grade for building supporting up to 500 psf areal loading can be obtained on the existing undisturbed native soils or on structural fill. A minimum 6-inch-thich layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. Imported granular material should be crushed rock or crushed gravel and sand that is fairly well-graded between course and fine, contains no deleterious materials, has a maximum particle size of 1 ½ inches, and has less than 5 percent by weight passing the U.S. Standard No. 200

Sieve. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density as determined by ASTM D 1557.

A typical moisture barrier consists of 6-mil visqueen plastic overlain with 2 inches of fine sand constructed over the crushed aggregate subgrade. Slabs should be appropriately waterproofed in accordance with the desired type of finished flooring. Slab thickness and reinforcement should be designed by an experienced engineer in accordance with anticipated loads.

5.5 Temporary Excavations

The following information is provided solely as a service to our client. Under no circumstances should this information be interpreted to mean that CNE is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

In no case should excavation slopes be greater than the limits specified in local, state and federal safety regulations. Based on the information obtained from our field exploration and laboratory testing, the site soils expected to be encountered in excavations, firm to stiff silt (ML) and silty sand (SM) would be classified as a Type "B" soil by OSHA guidelines.

Therefore, temporary excavations and cuts greater than four feet in height, should be sloped at an inclination no steeper than 1H:1V (horizontal:vertical) for type "B" soils. If slopes of this inclination, or flatter, cannot be constructed or if excavations greater than ten feet in depth are required, temporary shoring may be necessary.

The shoring would help protect against slope or excavation collapse, and would provide protection to workmen in the excavation. If temporary shoring is required, we will be available to provide shoring design criteria, if requested.

5.6 Lateral Earth Pressure

Lateral earth pressure should be carefully considered for design of retaining walls or below-grade structures. Hydrostatic pressure and additional surcharge loading should also be considered. Retained material may include engineered structural backfill or relatively undisturbed native soil. Structural wall backfill may consist of recompacted fine-textured soils or imported granular material. Backfill should be prepared and compacted to at least 90 percent of maximum dry density as determined by the modified Proctor test (ASTM D1557). If seismic design is required, seismic forces for unrestrained wall may be calculated by superimposing a uniform lateral force of $10H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. The resultant force should be applied at 0.6H from the base of the wall.

A continuous one-foot-thick zone of free-draining, washed, open-graded 1-inch by 2-inch drain rock and a 3-inch perforated gravity drain pipe is assumed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and fine-textured backfill soil. Specifications for drainpipe design are presented in Section 5.10, Drainage. If walls cannot be gravity drained, saturated base conditions and /or applicable hydrostatic pressures should be assumed.

5.7 Seismic Design Considerations

Based upon Site Class Map of Clark County, Washington (Washington State Department of Natural Resources, 2004), shallow site soils may be represented by Site Class C as defined in 2009 IBC Table 161.5.2. This assessment is preliminary, pertains to near-surface soils, and is based upon limited field exploration and research of existing published literature. Additional exploration would be necessary to provide soil site class information at greater depths. Amplification of seismic energy due to depth to competent bedrock, compression and shear wave velocity of bedrock, presence and thickness of loose, unconsolidated alluvial deposits, soil plasticity, grain size, and other factors is possible at the site

According to Clark County Maps Online, the site is mapped as very low to low potential for liquefaction.

Identification of specific seismic response spectra, probabilistic ground motions, and liquefaction analysis for the site are beyond the scope of this investigation. If site structures are designed in accordance with recommendations specified in the 2009 IBC, the potential for peak ground accelerations in excess of adjusted and amplified values should be understood.

5.9 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed Stormwater management structures and facilities. Drainage design in general should conform to the Clark County regulations. Finished site grading should be conducted with positive drainage away from structures. Depressions or shallow areas that may retain ponding water should be avoided. Roof drains and perimeter foundation drains are recommended for proposed structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from foundations into the storm water system or approved discharge location.

5.10 Bituminous Asphalt And Portland Cement Concrete

According to the preliminary short plat plan the subject site is not anticipated to include asphalt concrete for the new public street. Based upon analytical laboratory test results and field exploration. Engineering Northwest PLLC recommends the general pavement design consist of 12 inches of compacted crushed aggregate base overlain with a minimum of 3.0 inches of asphalt concrete pavement for truck loading and traffic areas.

For dry weather construction, pavement surface sections should bear upon competent subgrade consisting of compacted native soil or engineered structural fill. Wet weather pavement construction is discussed later in Section 5.44, Wet Weather Construction Methods and Techniques. Subgrade conditions should be evaluated and tested by a licensed geotechnical engineer or designated representative prior to placement of crushed aggregate base. Subgrade evaluation should include nuclear gauge density testing and wheel proof-roll observations conducted with a 12-cubic yard, double-axle dump truck or equivalent. Nuclear gauge density testing should be conducted at 250-foot intervals or as determined by the onsite geotechnical engineer. Subgrade soil should be compacted to at least 95 percent of the modified Proctor dry density as determined by ASTM D1557. Areas of observed deflection or rutting during proof-roll evaluation should be excavated to a firm surface and replaced with compacted crushed aggregate.

Crushed aggregate base should be compacted and tested in accordance with the specifications outlined above. Asphalt concrete pavement should be compacted to at least 91 percent of maximum Rice Density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with Washington Department of Transportation and the Clark County specifications.

Portland cement concrete curbs and sidewalks should be installed in accordance with the Clark County specifications. Curb and sidewalk aggregate base should be observed and proof-rolled in the presence of an experienced geotechnical engineer or designated representative. Soft areas that deflect or rut should be stabilized prior to pouring concrete. Concrete should be tested during installation in accordance with ASTM C171, C138, C231, C143, C1064, and C31. This includes casting of cylinder specimen at a frequency of four cylinders per 100 cubic yards of poured concrete. Recommended filed and analytical laboratory concrete testing includes slump. Air entrainment. Temperature, and unit weight.

5.11 Wet Weather Construction Methods and Techniques

Wet weather construction often results in significant shear strength reduction and soft areas that my rut or deflect. Installation of granular working layers may be necessary to provide a firm support base and sustain construction equipment. Granular layers should consist of all-weather gravel, 4-inch by 6inch gabion, or other similar material (6-inch maximum size with less than 5 percent passing the No. 200 sieve).

Construction equipment traffic across exposed native soil should be minimized. Equipment traffic induces dynamic loading, which may result in weak areas and significant reduction in shear strength for soils above plastic limit. Wet weather construction may generate significant excess quantities of soft wet soil, which should be removed from the site or stockpiled in a designated area.

5.12 Soil Erosion Potential

Based upon review of the Soil Survey of Clark County, Washington and field observations, the erosion hazard for the site soil is considered low to moderate. For flat to shallow- gradient portions of the property the erosion hazard is likely to be low. Erosion potential generally increases in sloped areas. Therefore, disturbance to vegetation in sloped areas should be minimized during construction activities. Soil is also prone to erosion if unprotected and unvegetated during periods of increased precipitation.

Erosion can be minimized by performing construction activities during dry summer months.

Site-specific erosion control measures should be implemented to address the maintenance of exposed areas. This may include silt fence, biofilter bags, straw wattles, or other suitable methods. During construction activities, exposed areas should be well-compacted and protected from erosion with visqueen, surface tactifier, or other means, as appropriate. Temporary slopes or exposed areas may be covered with straw, crushed aggregate, or riprap in localized areas to minimize erosion. Erosion and water runoff during wet weather conditions may be controlled by application of strategically placed channels and small detention depressions with overflow pipes. After grading the surface should be vegetated as soon as possible with erosion-resistant native grasses and forbs. Jute mesh or straw may be applied to enhance vegetation. Once established, vegetation should be properly maintained. It is also recommended that disturbance to existing native vegetation and surrounding organic soil be minimized during construction activities.

5.13 Soil Shrink/Swell Potential

The Soil Survey of Clark County, Washington indicates moderate potential for shrinking and swelling of the native site soils or imported subbase material.

5.14 Utility Installation

Utility installation at the site may require subsurface excavation and trenching. Excavation, trenching and shoring should conform to federal Occupational Safety and Health Administration (OSHA)(29 CFR, PART 1926) and WISHA (WAC, Chapter 296-155) regulations. Site soils may slough when cut vertically and sudden precipitation events or perched ground water may result in accumulation of water within excavation zones and trenches. These areas should be dewatered in accordance with appropriate discharge regulations.

Utilities should be installed in general accordance with manufacturer's recommendations. Utility trench backfill should consist of crushed aggregate or other coarse-textured, free-draining material acceptable to the Clark County and the site geotechnical engineer. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). The remaining backfill should be compacted to at least 90 percent of maximum dry density as determined by the modified Proctor moisture-density test (ASTM D1557). With exception of the pipe zone, backfill should be placed in loose lifts not exceeding 12 inches in thickness.

Compaction of utility trench backfill material should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938-08. Field compaction testing should be performed at 250-foot intervals along the utility trench centerline at the surface and midpoint depth of the trench. Compaction frequency and specifications may be modified for non-structural areas in accordance with recommendations of the site geotechnical engineer.

5.15 Groundwater

No groundwater was encounter in any of the test pits to the maximum exploration depth of 6 feet below the existing ground surface.

It is important to note that groundwater conditions are not static; fluctuation may be expected in the level and seepage flow depending on the season, amount of rainfall, surface water runoff, and other factors. Generally, the groundwater level is higher and seepage rate is greater in the wetter winter months (typically November through May).

5.17 Lab Soil Test Results

Laboratory test were conducted on representative soil samples to verify or modify the field soil classification of the units encountered, and to evaluate the general physical properties as well as the engineering characteristics of the soils encountered. The following provides information about the testing procedures performed on representative soil samples and the general condition of subsurface soil conditions encountered:

- Moisture Content (ASTM-D2216-92) test were performed on representative samples. In the upper layer of poorly graded gravel-sand mixes, the moisture content ranges from seven to ten percent.

- Grain Size Analyses (ASTM-D1140-97 and D422-90) were performed on samples collected from the proposed subbase. These tests indicate that soil consists predominantly of silt loam. Passing the #200 sieve result below, sample taken at 3.5- feet below the existing ground.

Test Pit	Percent Passing #200 sieve
1	85

- The result of laboratory tests performed on specific samples are provided at the appropriate sample depth on the individual test pit logs. However, it is important to note that some variation of subsurface conditions may exist. Our geotechnical recommendations are based on our interpretation of these test results.

5.18 Infiltration Testing

No infiltration test was performed.

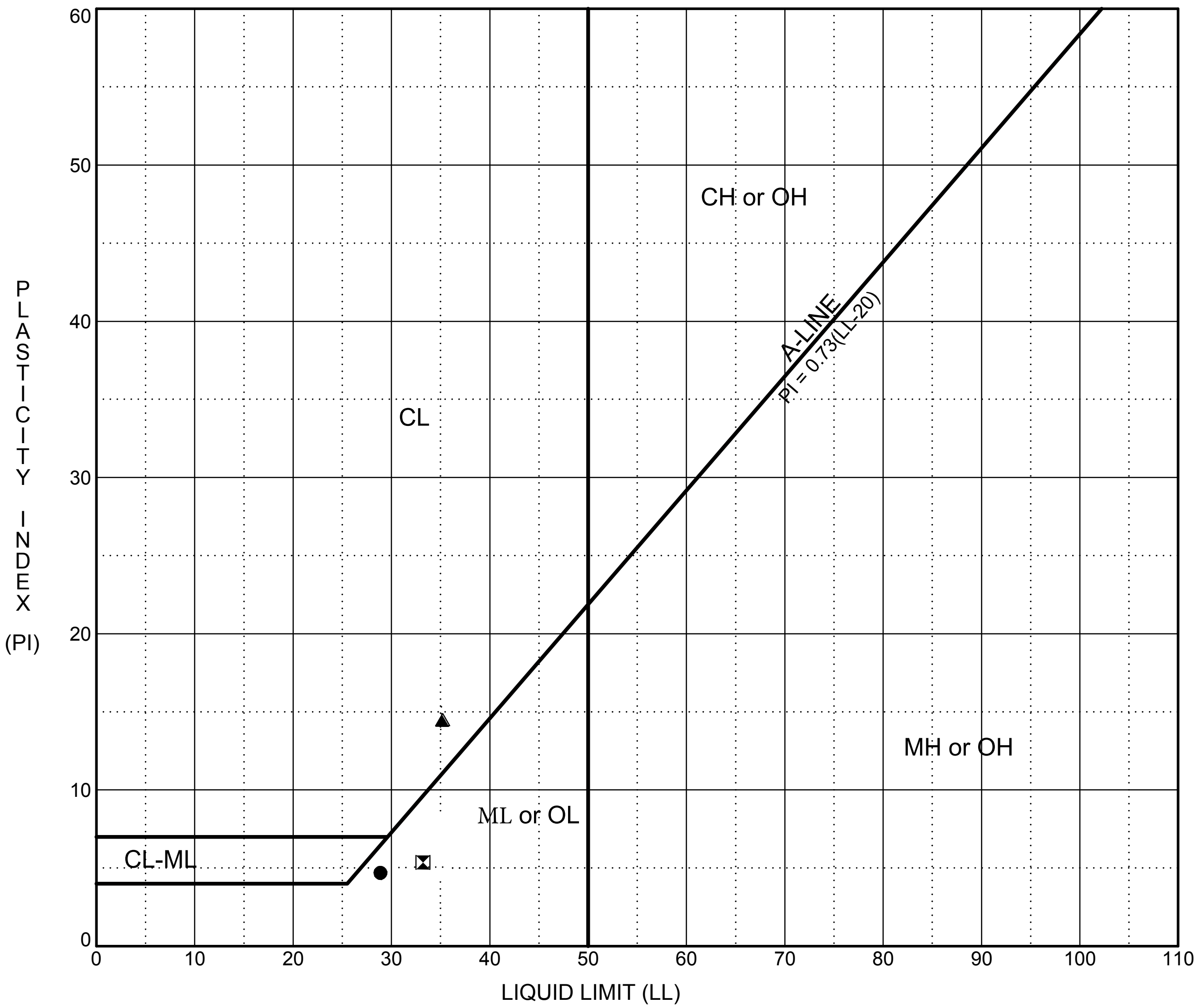
5.19 Conclusion

This geotechnical site investigation report was prepared in accordance with accepted standard conventional principles and practices of geotechnical engineering. This investigation pertains only to material tested and observed as of the date of this report, and is based upon proposed site development as described in the text herein.

5.20 Limitations

Our recommendations and conclusions are based on the site materials observed, selective laboratory testing, engineering analyses, the design information provided to Engineering Northwest PLLC and our experience as well as engineering judgment. The conclusions and recommendations are professional opinions derived in a manner consistent with that level of care and skill ordinarily exercised by other members of the profession currently practicing under similar conditions in this area. No warranty is expressed or implied.

The recommendations submitted in this report are based upon the data obtained from the test pits. If soil variations do appear Engineering Northwest PLLC should be requested to reevaluate the recommendations contained in this report and to modify or verify them in writing prior to proceeding with the proposed construction.



TP-/B-	SAMPLE	DEPTH FT.	USCS UNIT	LL	PL	PI	WC %	ORGANIC %	VISUAL-MANUAL CLASSIFICATION
● TP-1	S-1	2'	**ML**	**27**	**24**	**4**	**5.3**		Dark brown
⧗ TP-1	S-2	4'	**ML**	**33**	**28**	**5**	8.5		Dark reddish brown Clay (rocks)
▲ Tp-1	S-3	6'	C**L**	35	**21**	**14**	10.2		Dar reddish brown t Clay Loam (rocks)

NOTE: Atterberg Limits were performed on minus NO.40 sieve size material

GROUP	Unified Soil Classification System (USCS) Fine-Grained Soil Groups
OL	Organic Silts & Organic Silty Clays Low Plasticity
ML	Inorganic Clayey Silts to & Very Fine Sands Slight Plasticity
CL	Inorganic Clays Low to Medium Plasticity
OH	Organic Clays & Silts Medium to High Plasticity
MH	Inorganic Silts & Clayey Silts High Plasticity
CH	Inorganic Clays High Plasticity

LL = Liquid Limit

PL = Plastic Limit

PI = Plasticity Index

WC% = Water Content %

Hidden Ridge
Camas Wa

PLATICITY CHART

ATTERBERG

Figure **1**

LOG OF TEST PIT -1

Surface Elevation: 278
Boring Date: 6/15/19
Boring Location: 986043-773-000
Dilling Method: HAND SOIL AUGER

Depth	Remarks	COLOR	MOISTURE	MOISTURE CONTENT (%)	CONSISTENCY	Percent Pass		(ASHTO)	SOIL CLASSIFICATION
0		Dark Brown		7	Stiff to	85	X ■	A-7	High organic content Clay Loam
5		Dark Brown	Slighty Damp	10	Very Stiff	65	■	A-7	Clay Loam Large Rocks No groundwater
10									
15									
20									
25									
30								END	Boring completed at and depth of about 6 feet below the ground surface.
35									

LOG OF BORING

ENGINEERING NORTHWEST	Hidden Ridge Estate Project No. 082021 Camas WA	Plate 1