Geotechnical Report

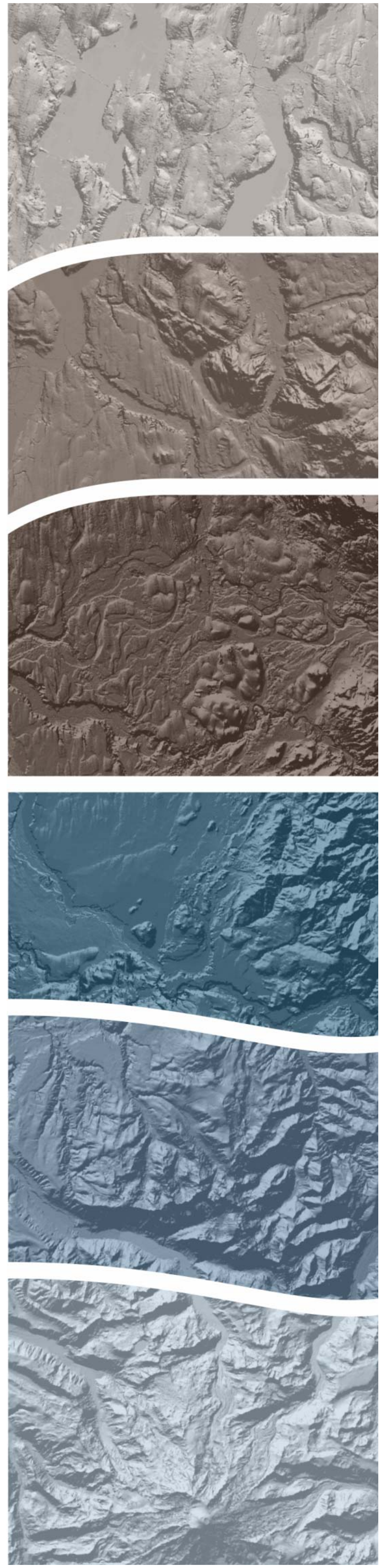
Kostenick Property

Marysville, Washington

Prepared for:

Joey Ferrick
KW Commercial

RN File No. 3395-004A ● May 5, 2021



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Prepared for:

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Cover sheet graphic shows western Washington geomorphology as a hillshade from Mount Rainier to the Seattle metropolitan area. Image is derived from a compilation of Washington State DNR LIDAR surveys obtained from the Washington Lidar Portal: http://lidarportal.dnr.wa.gov/

1 INTRODUCTION

This report presents the results of our geotechnical engineering investigation at your subdivision in Marysville, Snohomish County, Washington. The site is located at 5110 83rd Avenue NE, as shown on the Vicinity Map in **Figure 1**.

1.1 Project Description

We understand you plan to develop the site as a subdivision with multiple residential lots with associated access roads and improvements. We have not been provided a conceptual site plan at this time and understand the final configuration of the subdivision is subject to modification.

You have requested we produce this geotechnical report to evaluate the subsurface conditions at the site. To prepare this report, you have provided us with a survey of the parcel by Pacific Coast Surveys, Inc. dated March 9, 2021.

1.2 Scope

Our scope of services as outlined in our Services Agreement, dated February 23, 2021, includes the following:

- Review available geologic maps for the site.
- Explore the subsurface soil and groundwater conditions at the site with test pits using a subcontracted excavator.
- Evaluate pertinent physical and engineering characteristics of the soils encountered in the test pits.
- Prepare a geotechnical report containing the results of our subsurface explorations, and our conclusions and recommendations for geotechnical design elements of the project.

We completed these services in general accordance with our service agreement dated February 23, 2021. We received notice to proceed on February 23, 2021.

2 SITE CONDITIONS

2.1 Geologic Setting

Most of the Puget Sound Region was affected by past intrusion of continental glaciation. The last period of glaciation, the Vashon Stade of the Fraser Glaciation, ended approximately 14,000 years ago. Many of the geomorphic features seen today are a result of scouring and overriding by glacial ice and sediment deposition related to glacial advance and retreat. Many of the geomorphic features seen today are a result of scouring and overriding by glacial ice. During the Vashon Stade, areas of the Puget Sound region were overridden by over 3,000 feet of ice. Soil layers overridden by the ice sheet were compacted to a much greater extent than those that were not. Part of a typical glacial sequence within the area of the site includes the following soil deposits from newest to oldest:

> Artificial Fill (af) – Fill material is often locally placed by human activities, consistency will depend on the source of the fill. The thickness and expanse of this material will be dependent on the extent of fill required to grade land to the desired elevations. Density of the fill will depend on earthwork activities and compaction efforts made during the placement of the material.
>
> Recessional Outwash (Qvr) – These deposits were derived from the stagnating and receding Vashon glacier and consist mostly of stratified sand and gravel, but include unstratified ablation and melt-out deposits. Recessional deposits were not compacted by the glacier and are typically not as dense as those that were.
>
> Vashon Till (Qvt) – The till is a non-sorted mixture of clay, sand, pebbles, cobbles and boulders, all in variable amounts. The till was deposited directly by the ice as it advanced over and eroded irregular surfaces of previously deposited formations and sediments. The till was well compacted by the advancing glacier and exhibits high strength and stability. Drainage is considered very poor in the till.
>
> Advance Outwash (Qva) – The advance outwash typically is a thick section of mostly clean, pebbly sand with increasing amounts of gravel higher in the section. The advance outwash was placed by the advancing glaciers and was overridden and well compacted by the glacier.

The geologic units for this area are mapped on the Geologic Map of the Lake Stevens Quadrangle, Snohomish County, Washington, by James P. Minard (U.S. Geological Survey, 1985). The site is mapped as being underlain by glacial till. Our site explorations encountered glacial till consistent with the mapped geology.

2.2 Seismic Setting

The Pacific Northwest is very seismically active. Off the coast, the Juan de Fuca Oceanic Plate collides into and descends (subducts) under the North American Continental Plate. The contact between these plates forms an approximately 600 mile long fault known as the Cascadia Subduction Zone (CSZ). The resulting stresses generate three unique types of earthquakes that contribute to seismic risk in the region (Cascadia Region Earthquake Workgroup, 2013):

Subduction (or Megathrust) Earthquakes: Megathrust earthquakes are formed by a rupture of the contact between the plates along the CSZ. These events are capable of generating a magnitude 9 or larger earthquake. These earthquakes are relatively far from the Puget Sound, but still pose great risk due to their extreme intensity and duration. Along the CSZ, megathrust earthquakes are understood to have a recurrence interval of roughly every 500 years. The last such event along the CSZ happened in 1700 AD, lowering the coastline several feet and generating a large tsunami across the Pacific Ocean.

Shallow (or Crustal) Earthquakes: Stress from the subduction zone fractures and deforms the continental crust across the Pacific Northwest. When these near-surface crustal faults break, they generate earthquakes that affect smaller areas, but can locally be more intense than the subduction events off the coast. Such faults happen to pass under some of the most populous areas in Washington State, including the greater Seattle and Tacoma areas. Because of their proximity and local intensity, these fault zones are often the greatest contributing factor to seismic risk in the Puget Sound.

Deep (or Intraslab) Earthquakes: Intraslab earthquakes are associated with fractures within the subducting Juan de Fuca plate. Because they occur at depths over 18 to 30 miles beneath the surface, the energy of these earthquakes is dissipated over large areas of ground surface, increasing their zone of influence but limiting their severity. However, these earthquakes are still capable of causing significant damage to structures and are the most frequent seismic events in the Puget Sound region. A magnitude 6.5 or larger earthquake affecting the region can be expected, on average, every 30 years. The 2001 Nisqually earthquake was an intraslab earthquake with over $4 billion in damages, 400 injuries, and one death. (Cascadia Region Earthquake Workgroup, 2008).

The site is mapped on the U.S. Quaternary Faults and Folds Database web application by the U.S. Geological Survey as located approximately 12 miles northeast of the South Whidbey Island Fault Zone (SWIFZ). The SWIFZ is a series of shallow, crustal thrust fault strands that trend from northwest to southeast. This is a class A fault, meaning there is sufficient evidence of fault displacement during the Quaternary Period for the fault to be considered active. Research from the area has shown at least 4 earthquakes since ice retreat approximately 16,000 years ago, with the potential to generate magnitude 7.0 to 7.5 earthquakes (Washington Department of Natural Resources, 2012-2013).

2.3 Surface Conditions

The site is bordered by 83th Avenue NE to the east and a transmission line right-of-way to the west. The surrounding area consists mostly of existing rural residential acreage, wetlands, patchy woodlands, and newly completed/under construction subdivisions of single family residences. A layout of the site is shown on the Site Plan in **Figure 2**.

The rectangular parcel is about 4.64 acres in size and has maximum dimensions of approximately 621 feet in the east-west direction and approximately 325 feet in the north-south direction. Access to the site is provided by 83th Avenue NE.

The site is developed with a single family residence located in the north-central portion of the lot and a barn/outbuilding in the east/central portion of the lot. The buildings are surrounded by grass lawn landscaping. The western and southern perimeter of the lot is vegetated with small to large deciduous trees, conifers, and dense underbrush. The lot slopes gently down to the east to a pond in the eastern third of the site. We observed ponded water extending through the trees to the southeast corner of the lot.

2.4 Field Explorations

We explored subsurface conditions at the site on February 23 and February 24, 2021, by excavating seven test pits with a trackhoe. The test pits were excavated to depths of 3.0 to 7.0 feet below the ground surface. The explorations were located in the field by a representative from this firm who also examined the soils and geologic conditions encountered, and maintained logs of the test pits. The approximate locations of the test pits are shown on the Site Plan in Figure 2. The soils were visually classified in general accordance with the Unified Soil Classification System, a copy of which is presented as Figure 3. The logs of the test pits are presented in Figures 4 through 10.

2.5 Subsurface Conditions

The subsurface conditions at the site are briefly described below, based upon our completed field explorations of soils and review of geologic maps for the site. For a more detailed description of the soils encountered, review the Test Pit Logs in Figures 4 through 10.

2.5.1 Stratigraphy/Soil Conditions

Based on our completed explorations, we interpret that the subsurface stratigraphy on site can be grouped into four soil units: intermittent disturbed soil grouped into artificial fill, loose surficial soils interpreted as topsoil and forest duff, medium dense to dense silty sands interpreted as weathered glacial till (Qvt), and dense to very dense silty sands interpreted as glacial till (Qvt).

Artificial Fill: Artificial fill was encountered in Test Pit 3, and is likely to be encountered near previously developed areas at the site. In Test Pit 3, the material generally consisted of grayish brown silty sand with trace gravel.

Topsoil/Forest Duff: Topsoil/forest duff was encountered at all other explorations at the ground surface. It averaged approximately 0.5 to 1.5 feet in thickness. Duff in forested areas tended to be a little thinner. This material generally consisted of loose dark brown sandy silt with roots and organics.

Weathered Glacial Till: Underlying the surficial topsoil/duff, we generally encountered soils interpreted as weathered glacial till. The soils were generally loose to medium dense, brown sandy silt with gravel and trace cobbles/boulders. This material was generally 0.5 to 1.7 feet thick and extended to depths up to 2.5 feet below the ground surface.

Glacial Till: This material was encountered underlying the surficial topsoil and/or weathered glacial till in all explorations except possibly Test Pit 6 (discussed below). The till generally consisted of gray dense to very dense silty sand to sandy silt with gravel and varying amounts of cobbles and boulders. We observed that the uppermost 1.0 to 3.0 feet of the glacial till was generally brownish gray, medium dense to dense and exhibited rust mottling. We interpret this till as weathered from the seasonally perched water above it. The transition between the rust mottled till and the unweathered till was generally encountered between depths of 3.0 and 4.0 feet. The glacial till extended to the depths explored in all test pits.

At Test Pit 6 and in stratified layers of Test Pit 4, the encountered soil appears sandier in texture. We have previously interpreted sandy soil zones in nearby sites to likely represent small, discontinuous meltout or similar till deposits. Based on the presence of the pond and the topographic low near these explorations, it is also possible that these soils represent an exposure of advance outwash sands underlying the till cap.

2.5.2 Hydrologic Conditions

We observed shallow groundwater seepage in every test pit at the site. We also observed standing water at the ground surface across the eastern side of the site extending from the southeast corner of the site, through the mapped pond, and then north from the pond along the topographic low to the northern property line. We generally observed seepage in the test pits from depths of 0.5 feet to 3.0 feet, with multiple zones of seepage in some test pits. A 1.0 to 3.0 foot thick layer of rust mottling from perched water over the till deposit was also observed.

The encountered groundwater seepage is considered to be perched. Perched water does not represent a regional groundwater "table" within the upper soil horizons. The underlying partially cemented till is considered to be nearly impermeable. Volumes of perched groundwater vary depending upon the time of year and the upslope recharge conditions. Based on the frequent areas of standing water and extensive seepage, as well as rust mottling observed in test pits, we expect that perched water conditions in the shallow soil horizons could occur widely across the entire project site during the wetter times of the year. In some topographic lows and drainages, this perched water can be expected to reach the ground surface in the wet season, and shallow seepage could remain all year round.

We noted the soils adjacent to the pond appeared to be sandier and may represent advance outwash soils. These soils were found to be wet at depths near pond-level in Test Pit 4 and Test Pit 6. If these soils are advance outwash and extend laterally underneath the till encountered elsewhere on site, then it is possible the pond level is associated with a regional water table in the permeable advance outwash. Further explorations would be necessary to determine whether the sandy soils are in fact advance outwash.

3 CONCLUSIONS AND RECOMMENDATIONS

3.1 Summary of Geotechnical Considerations

It is our opinion that the site is compatible with the planned development. The underlying medium dense to very dense glacial till deposits are capable of supporting the planned structures. We recommend that the foundations for the structures extend through any fill, topsoil, loose, or disturbed soils, and bear on the underlying medium dense or firmer native glacial till, or on structural fill extending to these soils. Based on our site explorations, we anticipate these soils will generally be encountered at typical footing depths after the topsoil is stripped.

The near-surface soils likely to be exposed during site stripping and construction contain significant quantities of perched water. Volumes of water seepage likely vary seasonally, but may be present year round in places. We anticipate that the on-site soils will be very sensitive during grading and nearly impossible to compact when wet or during wet conditions. We recommend that construction take place during the drier summer months, if possible. If construction takes place during the wet season, additional expenses and delays should be expected due to the wet conditions. Additional expenses could include costs for cement-treating the on-site soils and/or an increased depth of site stripping (up to roughly 4 feet depth to reach unaffected soils below the perched water), export of on-site soil, and the import of clean granular soil for fill.

3.1 Seismic Engineering

3.1.1 Seismic Design

Seismic design for the 2018 International Building Code (IBC) is based on the mapped values for the risk-targeted maximum considered earthquake (MCE_R). Ground motion values in these maps include a probability of exceedance equal to 2% in 50 years, which corresponds to a 2,475-year return period. These mapped values have been prepared by the USGS in collaboration with the FEMA-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE).

The mapped MCE_R spectral response accelerations are referred to as S_s for short periods (0.2 seconds) and S_1 for a 1 second period. IBC 2018 directs that correction factors be applied to these response spectra based on an evaluation of site specific subsurface conditions, referred to as the soil site class (defined in ASCE 7 Section 20.3), as well as additional project specific factors as determined by the structural engineer. The Seismic Design Category shall be determined by the design in accordance ASCE 7 and IBC 2018.

Table 1: Seismic Design Inputs

Seismic Design Maps Tool Inputs	Value
Site Latitude	48.0425309254953
Site Longitude	-122.12031551017272
Site Class	C

3.1.2 Seismic Hazards.

Aside from the direct impact of ground shaking on structures, additional seismic hazards to be considered in a seismic event include ground surface displacement from fault rupture, liquefaction and amplification of ground motion, and landslides.

Surface Displacement: Due to the distance from the site to the nearest known strand (discussed in Section 3.1.1) and the lack of evidence of past fault displacement onsite, we expect the site to have a low risk for surface displacement.

Liquefaction: The liquefaction potential is highest for loose sand with a high groundwater table. The underlying dense till is considered to have a very low potential for liquefaction and amplification of ground motion and seismically induced lateral spread.

3.2 Erosion Hazard

The erosion hazard criteria used for determination of affected areas includes soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types (group classification), which are related to the underlying geologic soil units. We reviewed the Web Soil Survey by the Natural Resources Conservation Service (NRCS) to determine the erosion hazard of the on-site soils. The site surface soils were classified using the SCS classification system as Tokul gravelly medial loam, 0 to 8 percent slopes. The corresponding geologic unit for these soils is volcanic loess over till, which is in general agreement with the soils encountered in our site explorations. The erosion hazard for the soil is listed as being slight for the gently sloping conditions at the site.

3.3 Foundation Design

Conventional shallow spread foundations should be founded on undisturbed, medium dense or firmer soil. If the soil at the planned bottom of footing elevation is not suitable, it should be overexcavated to expose suitable bearing soil. Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection. Minimum foundation widths should conform to IBC requirements. IBC guidelines should be followed when considering short-term transitory wind or seismic loads. Standing water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

We recommend the allowable design bearing pressure value in **Table 2** for foundations constructed as outlined above. Higher soil bearing values may be appropriate with wider footings. These higher values can be determined after a review of a specific design.

Table 2: Recommendations for Shallow Foundation Design

Parameter	Value for Weathered Glacial Till
Allowable Bearing Pressure	3,000 psf
Approximate total settlement[1]	1 inch
Approximate differential settlement[2]	½ inch

Notes:

[1] Assumes foundation built upon firm, medium dense or denser native soil.

[2] Differential settlement between footings or across a distance of about 30 feet.

3.4 Slabs-On-Grade

Slab-on-grade areas should be prepared as recommended in the Site Preparation and Grading subsection. Slabs should be supported on medium dense or firmer native soils, or on structural fill extending to these soils. Where moisture control is a concern, we recommend that slabs be underlain by 6 inches of pea gravel for use as a capillary break. A suitable vapor barrier, such as heavy plastic sheeting, should be placed over the capillary break. An additional 2-inch-thick damp sand blanket can be used to cover the vapor barrier to protect the membrane and to aid in curing the concrete. This will also help prevent cement paste bleeding down into the capillary break through joints or tears in the vapor barrier. The capillary break material should be connected to the footing drains to provide positive drainage.

3.5 Drainage

We recommend that runoff from impervious surfaces, such as roofs, driveway and access roadways, be collected and routed to an appropriate storm water discharge system. The finished ground surface should be sloped at a gradient of 5 percent minimum for a distance of at least 10 feet away from the buildings, or to an approved method of diverting water from the foundation, per IBC Section 1804.4. Surface water should be collected by permanent catch basins and drain lines, and be discharged into a storm drain system.

We recommend that footing drains be used around all of the structures where moisture control is important. The underlying till may pond water that could accumulate in crawlspaces. It is good practice to use footing drains installed at least 1 foot below the planned finished floor slab or crawlspace elevation to provide drainage for the crawlspace. At a minimum, crawlspaces should be sloped to drain to an outlet tied to the drainage system. If drains are omitted around slab-on-grade floors where moisture control is important, the slab should be a minimum of 1 foot above surrounding grades.

Where used, footing drains should consist of 4-inch-diameter, perforated PVC pipe that is surrounded by free-draining material, such as pea gravel. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point. Crawlspaces should be sloped to drain, and a positive connection should be made into the foundation drainage system. For slabs-on-grade, a drainage path should be provided from the capillary break material to the footing drain system. Roof drains should not be connected to wall or footing drains.

Our experience with gently-sloping till sites is that the volume of water collected by residence foundation drains and routed to the stormwater detention system is insignificant when considered in the storm drainage design. We do not expect that the foundation drain water will impact the design of the stormwater detention system.

3.6 Retaining Wall Design

3.6.1 Lateral Loads

The lateral earth pressure acting on retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement, which can occur as backfill is placed, and the inclination of the backfill. Walls that are free to yield at least one-thousandth of the height of the wall are in an "active" condition. Walls restrained from movement by stiffness or bracing are in an "at-rest" condition.

We recommend design earth pressure values as given in **Table 3** below. H represents the wall height. These values assume that the on-site soils or imported granular fill are used for backfill, and that the wall backfill is drained. The given values do not include the effects of surcharges, such as due to foundation loads or other surface loads. Surcharge effects should be considered where appropriate. Seismic lateral loads are a function of the site location, soil strength parameters and the peak horizontal ground acceleration (PGA) for a given return period. We used the seismic input parameters discussed in **Section 3.1.1** above, to obtain PGA parameters for the site from the SEAOC Seismic Design Maps Tool web application. We used the output parameters to compute the additional seismic lateral loads for the site.

Table 3: Lateral Earth Pressure Parameters

Earth Pressure Condition	Backslope Angle	Equivalent Fluid Density (pcf)	Seismic Earth Pressure Kicker* (psf)
Active (K_a)	Level	35	5H
At-Rest (K_o)	Level	55	5H

*Kicker is to be applied at 60% of the wall height

The above lateral pressures may be resisted by friction at the base of the wall and passive resistance against the foundation. We recommend resistance values as given in **Table 4** below. To achieve these values of passive resistance pressure, the foundations should be poured “neat” against the native dense soils, or compacted fill should be used as backfill against the front of the footing, and the soil in front of the wall should extend a horizontal distance at least equal to three times the foundation depth. A resistance factor of 0.67 has been applied to the passive pressure to account for required movements to generate these pressures.

Table 4: Passive Resistance Parameters

Soil Type	Coefficient of Friction	Equivalent Fluid Density (pcf)
Weathered Glacial Till	0.5	320

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

3.7 Stormwater Management

3.7.1 Detention Pond

If a stormwater detention pond is planned, it should be excavated into the underlying dense native soils. We recommend that any fill berms be constructed of soils having a maximum permeability of 1 x 10^{-5} centimeters per second (4 x 10^{-6} inches/second). The on-site till encountered in our test pit explorations meets this criterion. We should evaluate any proposed berm fill material prior to construction of the berm.

If a pond is to be constructed, the cut slopes of the pond should be no steeper than 3H:1V on the inside of the detention pond and no steeper than 2H:1V above the water table or on the outside portions of the pond berms. Inside slopes as steep as 2H:1V are possible but may require maintenance until vegetation is established. Areas with seepage may require a blanket of rock spalls or other measures to limit sloughing.

Where any berms for the pond are to be constructed, the topsoil and loose soils should be removed down to the medium dense to very dense till. Areas to receive new fill should be stripped of unsuitable surface soils and compacted to a firm, non-yielding state prior to placement of the new fill. The excavation should be kept dry to allow the proper placement of structural fill. Structural fill should be placed and compacted as discussed in **Section 3.8.4**. We recommend that the fill in any pond berms be compacted to a minimum of 92 percent of its maximum dry density as determined by the ASTM D1557 compaction test procedure. After each lift of the fill in a berm is compacted to specification, the surface should be scarified to a depth of 2 inches prior to placement of the next lift. The purpose of the scarification is to reduce the risk of creating preferential seepage paths through the pond or berms.

It will be important to compact the face of any pond fill embankments. This should be made explicit to the contractor performing the on-site work. Uncompacted soils on a berm face will be more susceptible to erosion and sloughing. If groundwater seepage is encountered within a cut slope face, a layer of rock spalls may be necessary to minimize erosion of the slope face. The spall layer can be placed at the time of construction, or in the future if sloughing of the slope is observed.

3.7.2 Detention Vault

If a stormwater detention vault is planned, the concrete walls of the vault may be supported on footing foundations bearing on the underlying dense native glacial till soils. We recommend a soil bearing pressure as described in **Table 5** below for the design of the wall footings poured on undisturbed dense glacial till.

Table 5: Detention Vault Foundation Design

Parameter	Value for Glacial Till
Allowable Bearing Pressure	4,500 psf
Approximate total settlement	1 inch
Approximate differential settlement	½ inch

We recommend that footing drains be installed on the outside of perimeter footings. The footing drains should be at least 4 inches in diameter and should consist of perforated or slotted, rigid, smooth-walled PVC pipe, laid at the bottom of the footings. The drain line should be surrounded with free-draining pea gravel or coarse sand and wrapped with a layer of non-woven filter fabric. A vertical drainage blanket at least 12 inches thick, consisting of compacted pea gravel or other free-draining granular soils, should be placed against the walls. A vertical drain mat, such as G100N by Mirafi Inc., may be placed against the walls in lieu of the vertical drainage blanket. Structural fill is then placed behind the vertical drainage blanket or drain mat to backfill the walls. The vertical drainage blanket or drain mat should be hydraulically connected to the drain line at the base of the walls. Sufficient number of cleanouts at strategic locations should be installed for periodic cleaning of the wall drain line to prevent clogging.

The perimeter walls of the concrete vault with a lid would be restrained at their top from horizontal movement and should be designed for at-rest lateral soil pressure, while the perimeter walls of a vault without a lid would be unrestrained at the top and may be designed for active lateral soil pressure. Active earth pressure and at rest earth pressure can be calculated based on equivalent fluid density. We recommend design earth pressures for the vault as given in Table 6 below. These values assume that the on-site soils are used for backfill, and that the wall backfill is drained. The preceding values do not include the effects of surcharges due to foundation loads, traffic or other surface loads. Surcharge effects should be considered where appropriate. Recommended seismic lateral loading is provided in Section 3.6.1. Undrained conditions may occur in the lower portion of the vault if there is not suitable fall to place a wall drain at the footing elevation.

Table 6: Detention Vault Lateral Earth Pressure Parameters

Earth Pressure Condition	Backslope Angle	Equivalent Fluid Density (pcf)	Undrained Equivalent Fluid Density (pcf)
Active (K_a)	Level	35	80
At-Rest (K_o)	Level	55	90

The above lateral pressures may be resisted by friction at the base of the wall and passive resistance against the foundation. We recommend resistance values as given in Table 7 below. To achieve these values of passive resistance pressure, the foundations should be poured “neat” against the native dense soils, or compacted fill should be used as backfill against the front of the footing, and the soil in front of the wall should extend a horizontal distance at least equal to three times the foundation depth. A resistance factor of 0.67 has been applied to the passive pressure to account for required movements to generate these pressures.

Table 7: Detention Vault Passive Resistance Parameters

Soil Type	Coefficient of Friction	Equivalent Fluid Density (pcf)	Undrained Equivalent Fluid Density (pcf)
Glacial Till	0.6	360	190

3.7.3 Infiltration

We understand that the City of Marysville has adopted the 2012 Department of Ecology Stormwater Management Manual for Western Washington as Amended in December 2014 (SWMMWW). This manual provides guidelines for evaluating the feasibility of infiltration facilities in Volume III, Section 3.3. The baseline soil conditions that must be available at the site for infiltration to be feasible is a vertical separation of at least 5 feet from the base of an infiltration facility to bedrock, a seasonal high groundwater table, or impermeable layer. The glacial till underlying the site is considered to be an impermeable layer due to the highly compact and cemented nature of the deposit. The existing hydrologic conditions of the site consist of perched groundwater that sits on top of this till layer and is unable to percolate through the deposit. We observed late winter-early spring high groundwater levels to range from the ground surface (0.0 feet) to 2.0 feet below the ground surface. It is our opinion that the use of infiltration best management practices are not feasible at the site due to the lack of sufficient vertical separation to the perched groundwater and the glacial till layer.

3.8 Earthwork and Construction Considerations

3.8.1 Site Preparation and Grading

The first step of site preparation should be to strip the vegetation, topsoil, or loose soils to expose medium dense or firmer native soils in pavement and building areas. The excavated material should be removed from the site, or stockpiled for later use as landscaping fill. The resulting subgrade should be compacted to a firm, non-yielding condition. Areas observed to pump or yield should be repaired prior to placing hard surfaces. Special care should be taken to overexcavate disturbed soils and backfill with structural fill as described in **Section 3.8.4** in the artificial ponds or demolished basement areas.

3.8.2 Pavement Subgrade

The performance of roadway pavement is critically related to the conditions of the underlying subgrade. We recommend that the subgrade soils within the roadways be prepared as described in **Section 3.8.1**. Prior to placing base material, the subgrade soils should be compacted to a non-yielding state with a vibratory roller compactor and then proof-rolled with a piece of heavy construction equipment, such as a fully-loaded dump truck. Any areas with excessive weaving or flexing should be overexcavated and recompacted or replaced with a structural fill or crushed rock placed and compacted in accordance with recommendations provided in **Section 3.8.4**.

3.8.3 Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, such as the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable temporary cut slope geometry. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations, since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered.

For planning purposes, we recommend that temporary cuts in the near-surface weathered soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). Temporary cuts in the dense to very dense glacial till should be no steeper than 0.75H:1V. If groundwater seepage is encountered, we expect that flatter inclinations would be necessary.

We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than 4 feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to local and WISHA/OSHA standards.

Final slope inclinations for granular structural fill and the native soils should be no steeper than 2H:1V. Lightly compacted fills, common fills, or structural fill predominately consisting of fine grained soils should be no steeper than 3H:1V. Common fills are defined as fill material with some organics that are "trackrolled" into place. They would not meet the compaction specification of structural fill. Final slopes should be vegetated and covered with straw or jute netting. The vegetation should be maintained until it is established.

3.8.4 Structural Fill

All fill placed beneath buildings, pavements or other settlement sensitive features should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is observed by an experienced geotechnical professional or soils technician. Field observation procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction.

Materials: Imported structural fill should consist of a good quality, free-draining granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about 3 inches. Imported, all-weather structural fill should contain no more than 5 percent fines (soil finer than a Standard U.S. No. 200 sieve), based on that fraction passing the U.S. 3/4-inch sieve.

The use of on-site soil as structural fill will be dependent on moisture content control. Some drying of the native soils may be necessary in order to achieve compaction. During warm, sunny days this could be accomplished by spreading the material in thin lifts and compacting. Some aeration and/or addition of moisture may also be necessary. We expect that compaction of the native soils to structural fill specifications would be difficult, if not impossible, during wet weather.

Fill Placement: Following subgrade preparation, placement of the structural fill may proceed. Fill should be placed in 8- to 10-inch-thick uniform lifts, and each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas, and within a depth of 2 feet below pavement and sidewalk subgrade, should be compacted to at least 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D1557 compaction test procedure. Fill more than 2 feet beneath sidewalks and pavement subgrades should be compacted to at least 90 percent of the maximum dry density. The moisture content of the soil to be compacted should be within about 2 percent of optimum so that a readily compactable condition exists. It may be necessary to overexcavate and remove wet surficial soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

3.8.5 Utilities

Our explorations indicate that deep dewatering will not be needed to install standard depth utilities. Anticipated groundwater is expected to be handled with pumps in the trenches. We also expect that groundwater seepage may develop during and following the wetter times of the year. Based on our test pit explorations, we expect that undrained or unpumped utility trenches may fill with water if left open during the wet season, especially along topographically low areas.

The soils likely to be exposed in utility trenches after site stripping are considered highly moisture sensitive. We recommend that they be considered for trench backfill during the drier portions of the year. Provided these soils are within 2 percent of their optimum moisture content, they should be suitable to meet compaction specifications. During the wet season, it may be difficult to achieve compaction specifications; therefore, soil amendment with kiln dust or cement may be needed to achieve proper compaction with the on-site materials.

3.8.6 Dewatering

We expect that shallow groundwater seepage will be encountered during and following the wetter times of the year as water impounds over the impermeable glacial till. We do not expect significant volumes of water in these excavations. Encountered groundwater seepage is expected to be handled with pumps in the excavated area. Temporary ponds may be needed to collect seepage and pumped water to avoid sediment-laden runoff. Groundwater seepage behind any proposed retaining walls should be collected in a drainage system as discussed in Section 3.5.

3.8.7 Wet Weather Considerations

The on-site glacial till soils likely to be exposed during construction will disturb easily when wet. We expect these soils would be difficult, if not impossible, to compact to structural fill specifications in wet weather. We recommend that earthwork be conducted during the drier months. Additional expenses of wet weather or winter construction could include extra excavation and use of imported fill or rock spalls. During wet weather, alternative site preparation methods may be necessary. These methods may include utilizing a smooth-bucket trackhoe to complete site stripping and diverting construction traffic around prepared subgrades. Soil amendment with kiln dust or cement may be needed to achieve proper compaction with the on-site materials. Disturbance to the prepared subgrade may be minimized by placing a blanket of rock spalls or imported sand and gravel in traffic and roadway areas. Cutoff drains or ditches can also be helpful in reducing grading costs during the wet season. These methods can be evaluated at the time of construction.

4 FUTURE WORK

4.1 Engineering and Design

The intent of this geotechnical report is to provide KW Commercial with a professional evaluation of existing subsurface and slope conditions at the site and to provide recommendations for geotechnical design elements of the proposed project.

As KW Commercial proceeds with the project, we may be retained to provide additional services including engineering, design work, and project management specific to their needs.

4.2 Construction Observation

We should be retained to provide observation and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, and to provide recommendations for design changes, should the conditions revealed during the work differ from those anticipated. As part of our services, we would also evaluate whether or not installation activities comply with contract plans and specifications.

We recommend that Robinson Noble perform the following tasks:

- Review contractor submittals
- Observe foundation installation
- Observe wall foundation and drainage installation
- Perform compaction tests
- Perform laboratory tests as needed
- Attend meetings as needed
- Provide geotechnical consultation

5 USE OF THIS REPORT

We have prepared this report for KW Commercial and their agents, for use in planning and design of this project. The data and report should be provided to prospective contractors for their bidding and estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report, for consideration in design. There are possible variations in subsurface conditions. We recommend that project planning include contingencies in budget and schedule, should areas be found with conditions that vary from those described in this report.

Within the limitations of scope, schedule and budget for our services, we have strived to take care that our services have been completed in accordance with generally accepted practices followed in this area at the time this report was prepared. No other conditions, expressed or implied, should be understood.

We appreciate the opportunity to be of service to you. If there are any questions concerning this report or if we can provide additional services, please call.

6 REFERENCES

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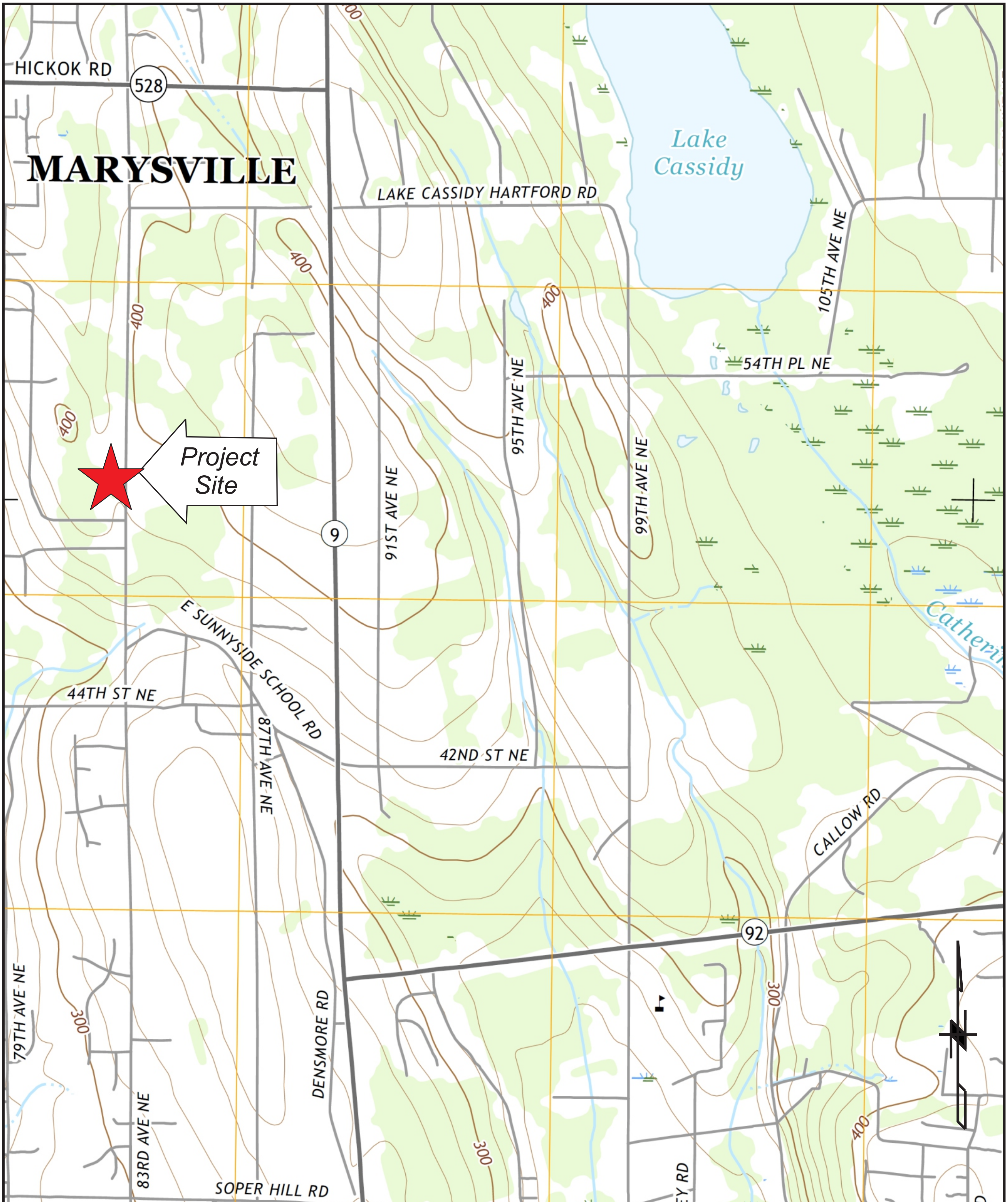
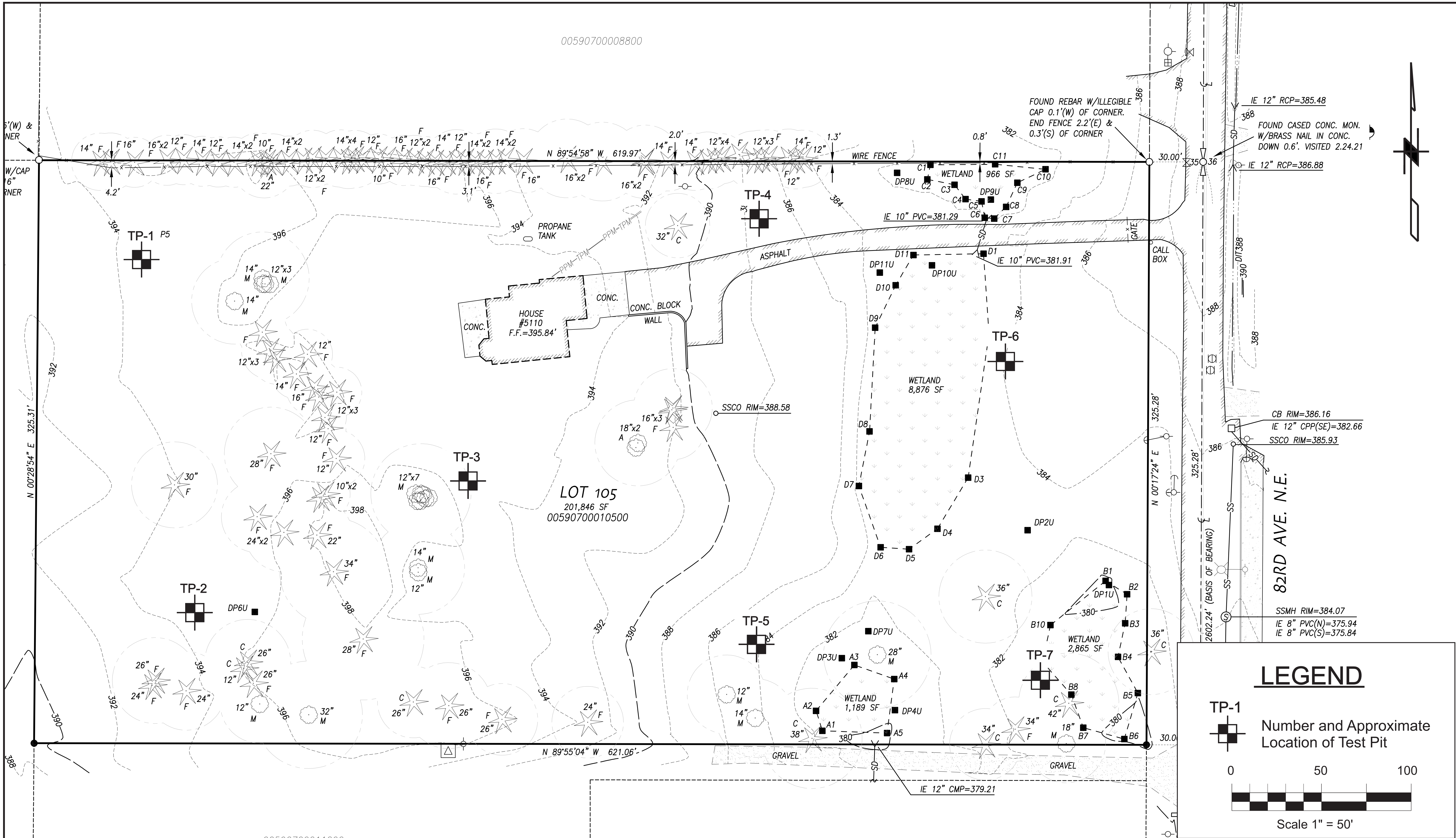


ROBINSON NOBLE	Note: Basemap taken from Lake Stevens 7.5-minute series. USGS 2017.	PM: BRP May 2021 3395-004A

Figure 1

Vicinity Map

Groundhog Land Development Company, LLC: Kostenick Property



LEGEND

TP-1 — Number and Approximate Location of Test Pit

0 50 100

Scale 1" = 50'

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Note: Basemap taken from Topographic Survey prepared by Pacific Coast Surveys, Inc. dated 3/9/2021.

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Figure 2

Site Plan

Groundhog Land Development, LLC: Kostenick Property

Unified Soil Classification System

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50% RETAINED ON number 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50% PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50%	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50% OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-83.
2) Soil classification using laboratory tests is based on ASTM D 2487-83.
3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS

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

Moist- Damp, but no visible water

Wet- Visible free water or saturated, usually soil is obtained from below water table

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Figure 3

Unified Soil Classification System

Groundhog Land Development, LLC: Kostenick Property

Test Pit 1

Date: 2/23/2021
Logged By: BRP
Location: 5110 - 83rd Avenue NE, Marysville, WA

Depth (ft.)	Soil Description	USC
0.0 - 1.2	Dark brown organic silt with roots (loose, moist) **(Duff)**	OL
1.2 - 2.0	Brown sandy silt with gravel trace roots and cobbles (medium dense, moist)	ML
2.0 - 3.5	Rust stained brownish-gray sandy silt to silty fine to coarse sand with gravel and cobbles (medium dense, moist to wet)	ML/SM
3.5 - 7.0	Brownish-gray weakly cemented sandy silt to silty fine to coarse sand with gravel and cobbles (dense to very dense, moist)	ML/SM

View of Test Pit 1

Notes

- Test pit completed at 7.0 feet
- Groundwater observed at 2.5 feet
- Samples collected at 1.0 and 4.0 feet

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Figure 4

Test Pit 2		**Date:** 2/24/2021 **Logged By:** BRP	**Location:** 5110 - 83rd Avenue NE Marysville, WA
Depth (ft.)	Soil Description	USC	View of Test Pit 2
0.0 - 0.8	Dark brown sandy silt with organics, wood and roots (loose, moist) **(Duff)**	ML	
0.8 - 2.5	Brown sandy silt with gravel trace roots and cobbles (medium dense, moist to wet)	ML	
2.5 - 4.0	Rust stained brownish-gray sandy silt to silty fine to coarse sand with gravel and cobbles, banded rust mottling (medium dense to dense, moist to wet)	ML/SM	
4.0 - 5.5	Brownish-gray weakly cemented sandy silt to silty fine to coarse sand with gravel and cobbles (dense to very dense, moist)	ML/SM	

Notes

- Test pit completed at 5.5 feet
- Groundwater observed at 2.5 feet
- Samples collected at 2.0, 3.0 and 5.0 feet

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Figure 5

Test Pit 3		**Date:** 2/24/2021 **Logged By:** BRP	**Location:** 5110 - 83rd Avenue NE Marysville, WA
Depth (ft.)	Soil Description	USC	View of Test Pit 3
0.0 - 0.5	Grayish-brown silty sand with trace gravel and roots (loose, wet) **(Fill)**	SM	
0.5 - 1.0	Brown sandy silt with gravel trace roots and cobbles (loose, moist to wet)	ML	
1.0 - 4.0	Rust mottled brownish-gray sandy silt to fine to coarse sand with gravel (medium dense to dense, moist to wet)	ML/SM	
4.0 - 5.0	Brownish-gray weakly cemented sandy silt to silty fine to coarse sand with gravel and cobbles (dense to very dense, moist)	ML/SM	

Notes

- Test pit completed at 5.0 feet
- Groundwater observed at 0.5 and 3.0 feet
- Samples collected at 2.0 and 4.5 feet

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Figure 6

Test Pit 4		**Date:** 2/24/2021 **Logged By:** BRP	**Location:** 5110 - 83rd Avenue NE, Marysville, WA
Depth (ft.)	Soil Description	USC	View of Test Pit 4
0.0 - 0.7	Dark brown sandy silt with roots and gravel (loose, moist) **(Topsoil)**	ML	
0.7 - 2.0	Brown sandy silt trace roots, gravel and cobbles (loose, moist to wet)	ML	
2.0 - 3.5	Rust stained brownish-gray silty fine sand with gravel and cobbles (medium dense, moist)	SM	
3.5 - 7.0	Brownish-gray layered fine to medium sand with silt, silty sand, and sandy silt trace gravel (dense, moist to wet)	SM/ML/ SP-SM	

Notes

- Test pit completed at 7.0 feet
- Groundwater observed at 1.5 and 5.0 feet
- Sample collected at 3.5 feet

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Figure 7

Test Pit 5

Date: 2/24/2021
Logged By: BRP
Location: 5110 - 83rd Avenue NE, Marysville, WA

Depth (ft.)	Soil Description	USC
0.0 - 0.5	Dark brown sandy silt with organics, wood and roots (loose, moist) **(Duff)**	ML
0.5 - 2.0	Brown sandy silt with gravel trace roots and cobbles (medium dense, moist to wet)	ML
2.0 - 3.0	Lightly rust mottled brownish-gray sandy silt to silty fine to coarse sand with gravel and cobbles (medium dense to dense, moist)	ML/SM
3.0 - 5.5	Brownish-gray weakly cemented sandy silt to silty fine to coarse sand with gravel and and cobbles (dense to very dense, moist)	ML/SM

View of Test Pit 5

Notes

- Test pit completed at 5.5 feet
- Groundwater observed at 2.0 feet
- Sample collected at 5.0 feet

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Figure 8

Test Pit 6

Date: 2/24/2021
Logged By: BRP
Location: 5110 - 83rd Avenue NE, Marysville, WA

Depth (ft.)	Soil Description	USC
0.0 - 0.5	Dark brown sandy silt with organics, wood and roots (loose, moist) **(Duff)**	ML
0.5 - 1.0	Brown sandy silt with gravel trace roots and cobbles (medium dense, moist)	ML
1.0 - 3.0	Rust stained brownish-gray sandy silt to silty fine to coarse sand with gravel and cobbles (loose to medium dense, wet)	ML/SM
3.0 - 3.5	Rust mottled brownish-gray fine to medium sand trace to with silt trace gravel (dense, moist)	SP/SP-SM
3.5 - 7.0	Rust mottled brownish-gray fine to medium sand trace to with silt trace gravel (dense, wet)	SP/SP-SM

View of Test Pit 6

Notes

- Test pit completed at 7.0 feet
- Groundwater observed at 2.5 and 3.5 feet
- Samples collected at 2.5, 3.5 and 7.0 feet

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Figure 9

Test Pit 7		**Date:** 2/24/2021 **Logged By:** BRP	**Location:** 5110 - 83rd Avenue NE, Marysville, WA
Depth (ft.)	Soil Description	USC	View of Test Pit 7
0.0 - 0.5	Dark brown sandy silt with organics, wood and roots (loose, moist) **(Duff)**	ML	
0.5 - 1.5	Dark brown to black organic silt with roots and gravel (loose, wet)	OL	
1.5 - 2.0	Brown sandy silt with gravel trace roots and cobbles (loose, wet)	ML	
2.0 - 3.0	Rust stained brownish-gray sandy silt to silty fine to coarse sand with gravel and cobbles (medium dense, wet)	ML/SM	

Notes

- Test pit completed at 3.0 feet
- Groundwater observed at 0.5 feet
- Samples collected at 1.5 and 2.5 feet

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Figure 10