Geotechnical Report

44th Street Marysville Feasibility

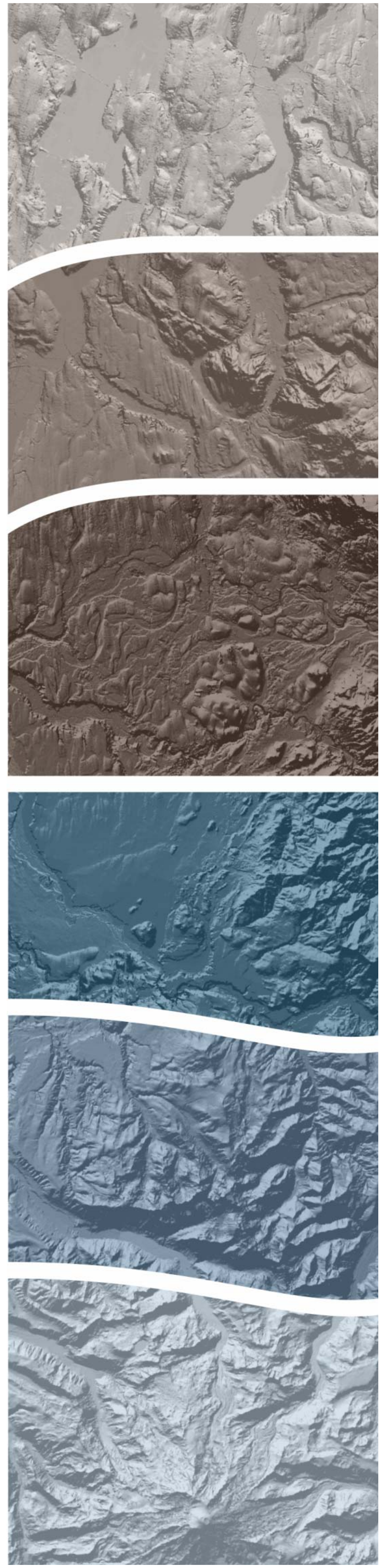
7315 and 7417 44th Street NE

Marysville, Washington

Prepared for:

Mr. David Morse
Toll Brothers, Inc.

RN File No. 2906-010A ● June 16, 2022



ROBINSON NOBLE

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Prepared for:

Mr. David Morse
Toll Brothers, Inc.
8815 122nd Avenue Northeast, Suite 200
Kirkland, Washington 98033

Robinson Noble, Inc.

Rick B Powell

Digitally signed by Rick B Powell
Date: 2022.06.16 13:50:44 -07'00'

Rick B. Powell, PE
Principal Engineer

SRB:RBP:am

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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 residential project in Marysville, Snohomish County, Washington. The site is located at 7315 and 7417 on 44th Street NE, Marysville, as shown on the Vicinity Map in Figure 1. We understand you wish to redevelop the site with 22 two-story single-family residences. For our use in preparing this report, we have been provided with an electronic document showing the preliminary planned conditions of the site, provided by D.R. Strong Consulting Engineers, dated August 6, 2021.

1.1 Scope

Our scope of services included:

Task 1: Geotechnical Evaluation - Completed

- Complete a site surficial evaluation.
- Review available geologic maps for the site.
- Explore the subsurface soil and groundwater conditions on site with test pits completed with a subcontracted excavator and operator.
- Evaluate pertinent physical and engineering characteristics of the soils encountered in the explorations.

Task 2: Geotechnical Report

- Prepare a **Geotechnical Report** containing the results of our subsurface explorations, and our conclusions and recommendations for geotechnical design elements of the project. Our report will include:
    - Description of the geologic materials encountered.
    - Description of depth to groundwater, if encountered.
    - Discussion of seismicity at the site along with seismic design parameters including Site Class based on current IBC criteria.
    - Discussion of geologic hazards.
    - Excavation considerations.
    - Recommendations for shallow foundations including allowable soil bearing values, minimum footing sizes, soil parameters for lateral load resistance, and footing drains.
    - Estimate the total and differential settlements of spread footings and floor slabs for variable loading within the building.
    - Geotechnical recommendations and considerations for support of concrete slab-on-grade floors.
    - Recommendations for earthwork and site preparation. An evaluation of the effects of weather and/or construction equipment on site soils and mitigation of any unsuitable soil conditions at the site will be included.
    - Provide geotechnical recommendations for stormwater drainage.

We completed these services in general accordance with our service agreement, dated October 8th, 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. 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 Marysville Quadrangle, Washington, by James P. Minard (U.S. Geological Survey, 1985). The site is mapped as being underlain by glacial till, with some small zones of recessional outwash nearby. 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 11.3 miles to the 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 Critical Areas Designation

Snohomish County PDS Map Portal indicates steep slopes (greater than 33 percent) exist on the eastern edge of Parcel 7417 and on the southwest portion of Parcel 7315. Also, according to the City of Marysville GIS critical areas map, which uses LiDAR imagery, the same areas contain slopes that are 15 to 25 percent, with two small areas containing slopes in excess of 25 percent along the boundary between the two parcels. Snohomish County maps the western portion of the site is as a Category III Wetland.

2.4 Surface Conditions

Both rectangular parcels 7417 and 7315 are sloping to the southwest, with a series of steep slopes (greater than 33 percent) existing on the western edge of Parcel 7417 and on the eastern portion of Parcel 7315. Steep slopes which are also sloping to the southwest are present on the southeast portion of Parcel 7315.

2.5 Field Explorations

We explored subsurface conditions at the site on October 19, 2021, by excavating six test pits with a mini excavator. The test pits were excavated to depths of 6.0 to 9.0 feet below the ground surface. The explorations were located in the field by a geologist 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 as shown in **Figure 3**. The logs of the test pits are presented in **Figures 4** through **9.**

2.6 Subsurface Conditions

The subsurface conditions at the site are briefly described below, based upon our completed field explorations of soils, laboratory testing, and review of geologic maps available for the site. For a more detailed description of the soils encountered, review the Test Pit Logs in **Figures 4** through **9.**

2.6.1 Stratigraphy/Soil Conditions

Based on our completed test pits, we interpret that the subsurface stratigraphy on site can be grouped into one soil unit: silty sand interpreted as glacial till (**Qvt**). The subsurface is mapped as being underlain by glacial till. Our explorations are in agreement with the mapped geology.

Glacial Till (Qvt): Silty sand characteristic of glacial till was encountered in all test pits. Beneath the topsoil (0.5 – 1.0 feet thick), loose to dense, moist, brown to reddish-brown silty sand with gravel was encountered to depths of 2.0 feet in Test Pits 1 and 6; 3.0 feet in Test Pits 2, 3, and 4; and 4.0 feet in Test Pit 5. Beneath these soils we observed very dense to dense, moist, gray silty fine sand with gravel, cobbles, and small boulders with trace to occasional rust mottling and various states of cementation. Soils in Test Pit 5 were observed to be overall texturally coarser and included more cobbles and small boulders. Soils were also observed to contain higher moisture contents and were noted to be moist-to-wet at refusal depth.

We interpret the reddish-brown to brown silty sand as weathered glacial till and the underlying gray silty sand as unweathered glacial till.

2.6.2 Hydrologic Conditions

In Test Pit 5, we observed the soils to be more coarse grained and containing a higher moisture content than the other test pits. After reviewing contour and LiDAR maps we observed Test Pit 5 was located within an erosional trough feature spanning the eastern parcel to the mapped wetland area in the western parcel. We suspect perched groundwater is present within this trough throughout the year. The amount of perched groundwater is assumed to vary throughout the year based upon upslope recharge conditions.

3 CONCLUSIONS AND RECOMMENDATIONS

3.1 Summary of Geotechnical Considerations

The glacial till subsurface soils will provide excellent foundation and roadway support based upon our understanding of the proposed project. We do not expect infiltration will be feasible on this site but we recommend that dispersion trenches be implemented in the project design to allow water to flow towards the existing wetland. The dispersion trenches are designed to be shallow and filled with washed rock. This allows the water to infiltrate if possible and then during large storm events to flow up out of the trench and disperse on the existing ground and then downward toward the wetland. This design helps mimic the existing and hydrologic rainfall cycles.

The on-site silty sand likely to be exposed during construction will disturb easily during the wetter times of the year. We expect these soils would be difficult, if not impossible, to compact to structural fill specifications in wet weather conditions. If the onsite soils are unable to be compacted to structural fill specifications we recommend the import of well-drained structural fill for locations where structural support is necessary.

3.2 Seismic Engineering

3.2.1 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). The Seismic Design Category shall be determined by the design in accordance ASCE 7 and IBC 2018.

Seismic design for geologic hazards including slope stability, liquefaction, seismic settlement, lateral spreading, and other seismic risks follow ASCE 7. The seismic design procedures in this standard are based on MCE_R peak ground acceleration (PGA) multiplied by a correction factor for site-specific amplification (F_{PGA}). This results in a site-modified peak ground acceleration (PGA_M).

We obtained seismic design parameters for this site from the ASCE 7 Seismic Hazard Tool. Input values based on our understanding of the proposed project and our interpretations of subsurface conditions (described in **Section 2.6**) are shown in **Table 1**, below. The output summary report from the ASCE 7 Hazards Tool is included in this report as **Appendix A**, and the seismic design parameters are shown in **Table 2**, below.

Table 1: Seismic Design Inputs

Seismic Design Maps Tool Inputs	Value
Site Latitude	48.036192
Site Longitude	-122.13172
Site Class	C

Table 2: Seismic Design Parameters

2018 IBC Design Parameter	Recommended Value
Seismic Design Category	D
PGA_M (2% in 50 years – 2,475 year event)	0.565
Design Kh (1/2 * PGA_M)	0.283

3.2.2 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 2.2**) 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.

Landslides: The core of the site is inferred to be composed of glacially overridden soils. We consider these soils to be of high strength and considered to be stable with regard to deep-seated seismic slope failures. Potential landslide hazard is discussed further in **Section 3.3**.

3.3 Slope Stability

3.3.1 Landslide Hazard

The core of the site is inferred to be composed of glacially overridden soils. We consider these soils to be of high strength and considered to be stable with regard to deep-seated slope failures. We did not observe indications of surficial seepage on the site, nor did we observe indications of shallow or deep-seated slope failures. However, since the soils on both parcels are glacially consolidated and the entire site will be developed and thus reinforced, it is our opinion that deep-seated slope failures are unlikely to occur.

In general accordance with the City of Marysville code section 22E.010.310, it is our opinion that the existing slope conditions can be graded in manner that will create a more stable condition. Additional landslide hazard classifications are not necessary.

3.4 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%) and Tokul gravelly medial loam (8 to 15%). The corresponding geologic unit for these soils is till, which is in agreement with the soils encountered in our site explorations. In our opinion the erosion hazard for these soils listed are slight for the gently sloping conditions at the site. We recommend that typical BMP be utilized control the site erosion potential during construction.

3.5 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 3** 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 3: Recommendations for Shallow Foundation Design

Parameter	Structural Fill or glacial till
Allowable Bearing Pressure	2,000 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.

3.6 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 4** 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 design parameters discussed in **Section 3.2**, above, to compute the additional seismic lateral loads for the site.

Table 4: Lateral Earth Pressure Parameters

Earth Pressure Condition	Backslope Angle	Equivalent Fluid Density (pcf)	Seismic Earth Pressure Kicker (psf)
Active (K_a)	Level	35	6H
At-Rest (K_o)	Level	56	10H
Active (K_a)	2H:1V	50	20H
At-Rest (K_o)	2H:1V	79	32H

*Kicker is to be applied as a uniform horizontal load

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 5** 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 5: Passive Resistance to Lateral Earth Pressure Parameters

Soil Type	Coefficient of Friction	Equivalent Fluid Density (pcf)
Glacial till / Structural Fill	0.45	250

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.

3.6.2 Recommended Retaining Wall Design Options

Retaining wall systems should be designed systems and could include rockeries, mechanically stabilized earth (MSE) walls with or without segmental block facing, ecology block walls, or concrete cast-in-place (CIP) walls. Each option is briefly discussed below. We can provide design of rockeries, MSE walls, and ecology block walls. A concrete CIP wall would need to be designed by a structural engineer.

Based on the encountered subsurface soil conditions, slope stability analysis, site access constraints, and an anticipated maximum wall height of approximately 5 feet, we anticipate that an MSE wall or rockery will likely provide the best combination of value, performance, and installation options.

MSE Walls: MSE walls consist of soil placed with layers of artificial reinforcing that typically consist of geogrid. The geogrid can be connected to a segmental block facing and extends back behind the facing for a designed length. MSE walls facing cuts require additional area in front of the cut face to fit the necessary geogrid lengths.

To build a 5-foot MSE wall with segmental block facing along a cut bordering the property line, we anticipate that the modular block facing would be built approximately 3 to 5 feet in front of the cut face. Construction of MSE walls can be accomplished using mainly hand construction methods in tandem with smaller construction equipment. Due to the easier construction techniques the cost of MSE walls is typically lower than other retaining wall systems.

Rockeries: Rockeries consist of large rocks stacked on top of one another to create a protective facing for an exposed stable cut in native soil or reinforced fill soil face. The size of the rock is dependent on the retained soil properties and the height of the rock wall being constructed. A drainage zone typically consisting of quarry spalls with a perforated PVC pipe at the base of the wall is established behind the rocks. Rockery construction requires specialized equipment capable of lifting and placing the necessary rock sizes. The quality of a rockery is also very dependent on the skill and experience of the contractor.

Ecology Block Walls: Ecology block walls consist of large (2'x2'x6' or 2.5'x2.5'x5') concrete blocks stacked to create a gravity stabilization system. Construction requires larger equipment capable of lifting and placing the approximately 1-ton blocks.

CIP Walls: Cast in place walls consist of structurally designed systems using concrete poured in place with reinforcing steel. The cost of these systems is typically higher due to the need for specialty contractors to construct the formwork and reinforcing steel and to place the concrete.

3.6.3 Retaining Wall Drainage

Adequate drainage is essential for any retaining wall to prevent the buildup of hydrostatic pressures. Retaining wall drains should consist of 4-inch-diameter, perforated PVC pipe at the base of the wall that is surrounded by free-draining material, such as pea gravel. Retaining wall drains should discharge into tightlines leading to an appropriate collection and discharge point.

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

3.7 Slabs-On-Grade

Slab-on-grade areas should be prepared as recommended in **Section 3.11.1**. 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.

3.8 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 3.11.1. 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 3.11.3.

3.9 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.

3.10 Stormwater Management

3.10.1 Dispersion Trenches

We recommend that dispersion trenches be use on the upgradient slopes of the wetlands. Where feasible the water collected form the roof drains, which is consider clean water, should be directed to the dispersion trenches. These trenches will allow as much water to infiltrate into the near-surface soils as possible and will help maintain the hydrologic balance of the site. Excessive water will surface-flow through the vegetation to the wetland areas similar to the existing conditions.

3.10.2 Detention Pond

If a storm water detention pond is planned to collect water from other surface such as roadways, driveways, sidewalks, etc., it should be excavated into the underlying 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 3.11.3. 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.

3.10.3 Detention Vault

If a stormwater detention vault is planned, the concrete walls of the stormwater detention vault may be supported on footing foundations bearing on the underlying dense soils. We recommend a soil bearing pressure of 4,000 pounds per square foot (psf) for the design of the wall footings poured on undisturbed dense glacial till.

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. Equivalent fluid densities for active and at rest earth pressure of 35 pcf and 56 pcf, respectively, may be used for design for a level backslope. 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**. For undrained soil conditions, the active and at-rest pressures should be increased to 83 pcf and 92 pcf, respectively. 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.

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.

We recommend that an equivalent fluid density of 300 pcf be used to calculate the allowable lateral passive resistance for the case of a level ground surface adjacent to the footing. A coefficient of friction between footings and soil of 0.55 may be used, and should be applied to the vertical dead load only. A factor of safety of 1.5 has been applied to the passive pressure to account for required movements to generate these pressures. The friction coefficient does not include a factor of safety.

3.11 Earthwork and Construction Considerations

3.11.1 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.

3.11.2 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).). Cuts in the dense to very dense till may stand at a 0.75H:1V inclination or possibly steeper. 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.

3.11.3 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.

3.11.4 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 some groundwater seepage may develop during and following the wetter times of the year. We expect this seepage to mostly occur in pockets. We do not expect significant volumes of water in these excavations.

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.

3.11.5 Dewatering

We also expect that some groundwater seepage may develop during and following the wetter times of the year. We expect this seepage to mostly occur in pockets. 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. Groundwater seepage behind the proposed retaining wall should be collected in a drainage system as discussed in Section 3.6.3.

3.11.6 Wet Weather Considerations

The on-site silty sand likely to be exposed during construction will disturb easily during the wetter times of the year. We expect these soils would be difficult, if not impossible, to compact to structural fill specifications in wet weather conditions. If the onsite soils are unable to be compacted to structural fill specifications we recommend the import of well-drained structural fill for locations where structural support is necessary.

4 FUTURE WORK

4.1 Engineering and Design

The intent of this geotechnical report is to provide Toll Brothers 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.

Once Toll Brothers has determined how to proceed with the project, we may be retained to provide additional services including engineering, design work, and project management specific to their chosen design.

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 foundation and wall drainage installation
- Observe shoring installation and testing
- 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 Toll Brothers 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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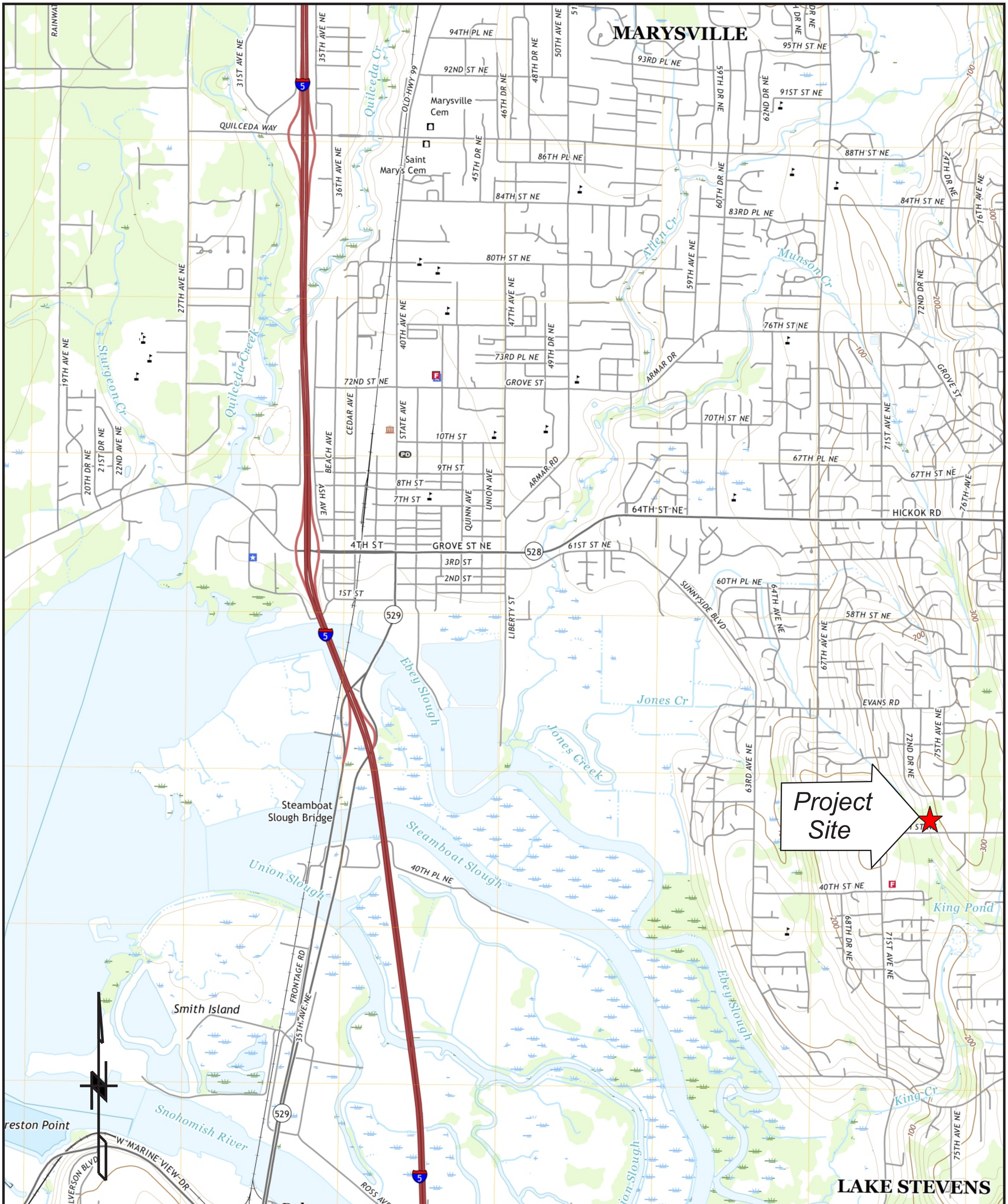

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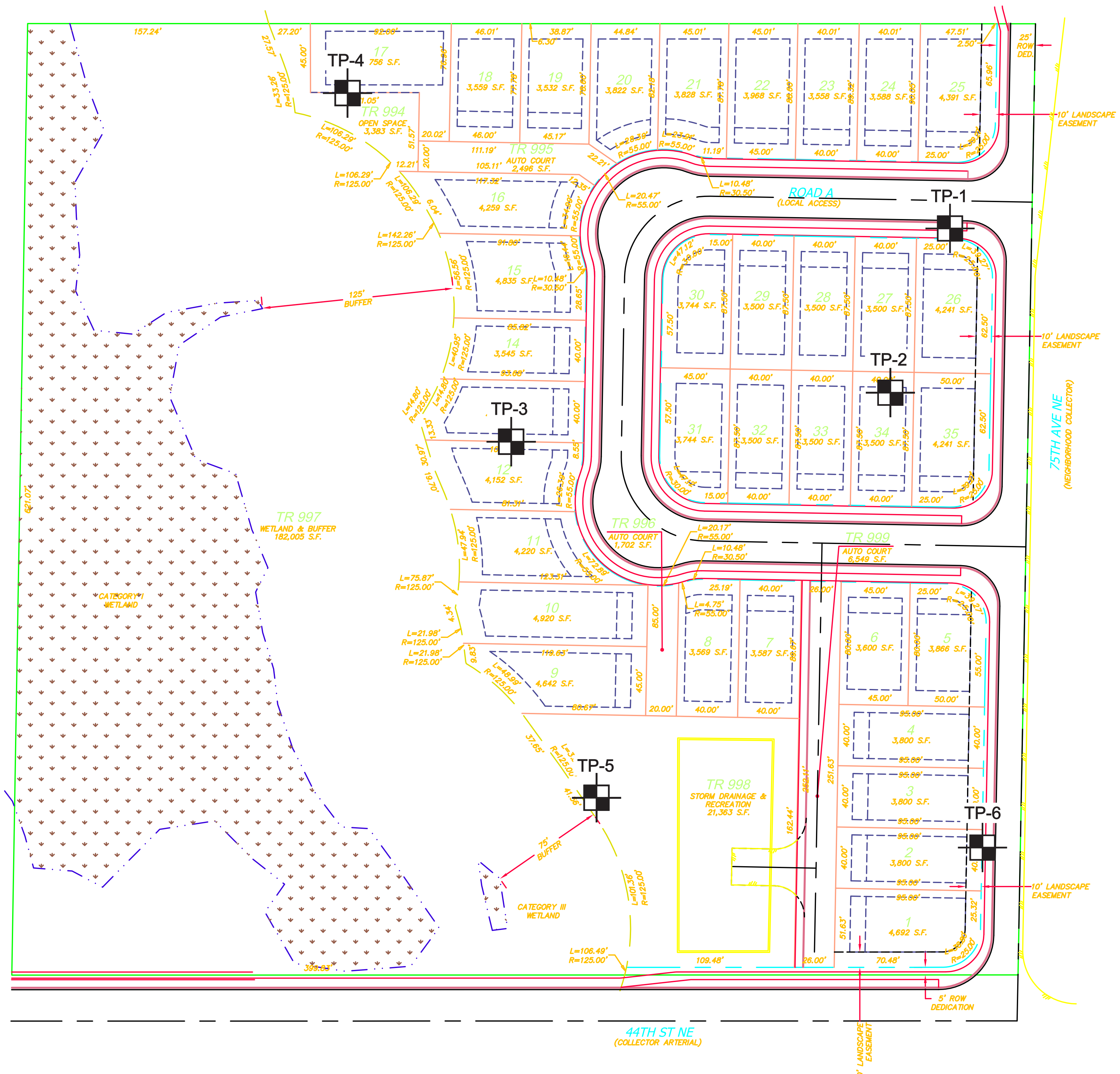
Note:
Basemap taken from Marysville 7.5-minute series. USGS 2020.

PM: RBP
June 2022
2906-010A

Figure 1

Vicinity Map

Toll Brothers, Inc: 44th Street Marysville



LEGEND

TP-1 Number and Approximate Location of Test Pit

0 80 160

Scale 1" = 80'

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Note: Basemap taken from AutoCAD file provided by D.R. Strong Consulting Engineers, Inc.

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2906-010A

Figure 2

Site Plan

Toll Brothers, Inc: 44th Street Marysville

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

Test Pit 1

Date:	10/19/2021
Logged By:	RWCM
Location:	44th Street NE Marysville, WA

Depth (ft.)	Soil Description	USC
0.0 - 0.5	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM
0.5 - 2.0	Reddish-brown silty fine sand with gravel and roots (loose, moist)	SM
2.0 - 8.5	Gray mottled silty fine sand with gravel and cobbles (dense to very dense, moist) **(Glacial Till)**	SM

View of Test Pit 1

Notes

- Test pit completed at 8.5 feet
- Groundwater was not observed
- Samples collected at 1.5, 4.0, and 8.5 feet.

Tacoma
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Tacoma, Washington 98402
253.475.7711

Woodinville
17625 - 130th Avenue NE, Suite 102
Woodinville, Washington 98072
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Figure 4

Test Pit 2		**Date:** 10/19/2021 **Logged By:** RWCM	**Location:** 44th Street NE Marysville, WA
Depth (ft.)	Soil Description	USC	View of Test Pit 2
0.0 - 1.0	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM	
1.0 - 2.7	Reddish-brown silty fine sand with gravel, cobbles, and roots (medium dense to dense, moist)	SM	
2.7 - 9.0	Gray silty fine sand with gravel, cobbles, and trace rust mottling (dense to very dense, moist) **(Glacial Till)**	SM	

Notes

- Test pit completed at 9.0 feet
- Groundwater was not observed
- Samples collected at 2.0, 3.5, and 9.0 feet.

Tacoma
2105 South C Street
Tacoma, Washington 98402
253.475.7711

Woodinville
17625 - 130th Avenue NE, Suite 102
Woodinville, Washington 98072
425.488.0599

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Figure 5

Test Pit 3

Date: 10/19/2021
Logged By: RWCM
Location: 44th Street NE, Marysville, WA

Depth (ft.)	Soil Description	USC
0.0 - 1.0	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM
1.0 - 3.0	Brown silty fine sand with gravel and roots (medium dense to dense, moist)	SM
3.0 - 8.0	Gray moderately cemented silty fine sand to sand with silt with gravel, cobbles, and trace rust mottling (dense to very dense, moist) **(Glacial Till)**	SM/ SP-SM

View of Test Pit 3

Notes

- Test pit completed at 8.0 feet
- Groundwater was not observed
- Samples collected at 2.5, 6.0, and 8.0 feet.

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Tacoma, Washington 98402
253.475.7711

Woodinville
17625 - 130th Avenue NE, Suite 102
Woodinville, Washington 98072
425.488.0599

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Figure 6

Test Pit 4		**Date:** 10/19/2021 **Logged By:** RWCM	**Location:** 44th Street NE Marysville, WA
Depth (ft.)	**Soil Description**	**USC**	**View of Test Pit 4**
0.0 - 1.0	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM	
1.0 - 3.0	Brown silty fine to medium sand with gravel and roots (loose to medium dense, moist)	SM	
3.0 - 8.0	Gray moderately cemented silty fine sand with gravel, cobbles, small boulders, and occasional rust mottling (dense to very dense, moist) **(Glacial Till)**	SM	

Notes

- Test pit completed at 8.0 feet
- Groundwater was not observed
- Samples collected at 2.0 and 5.0 feet.

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Tacoma, Washington 98402
253.475.7711

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Woodinville, Washington 98072
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Figure 7

Test Pit 5		Date: 10/19/2021 Logged By: RWCM	Location: 44th Street NE Marysville, WA
Depth (ft.)	Soil Description	USC	View of Test Pit 5
0.0 - 0.5	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM	
0.5 - 4.0	Brown silty fine sand with gravel, small boulders, and roots (loose to medium dense, moist)	SM	
4.0 - 6.0	Tan to gray silty fine to coarse sand to sand with silt, fine to coarse gravel, cobbles and boulders (dense to very dense, moist to wet) **(Glacial Till?)**	SM/ SP-SM	

Notes

- Test pit completed at 6.0 feet, refusal on boulders
- Groundwater was not observed
- Samples collected at 1.5 and 4.5 feet.

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Tacoma, Washington 98402
253.475.7711

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Woodinville, Washington 98072
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Toll Brothers, Inc: 44th Street Marysville
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Figure 8

Test Pit 6

Date:	10/19/2021	**Location:**	44th Street NE
Logged By:	RWCM		Marysville, WA

Depth (ft.)	Soil Description	USC
0.0 - 1.0	Dark brown silty fine sand with organics (loose, moist) **(Topsoil)**	SM
1.0 - 2.0	Reddish-brown silty fine sand with gravel and roots (loose to medium dense, moist)	SM
2.0 - 8.0	Gray moderately cemented silty fine sand with gravel, cobbles, and occasional rust mottling (dense to very dense, moist) **(Glacial Till)**	SM

View of Test Pit 6

Notes

- Test pit completed at 8.0 feet
- Groundwater was not observed
- Samples collected at 2.0 and 6.0 feet.

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Woodinville
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Toll Brothers, Inc: 44th Street Marysville
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Figure 9

Appendix A

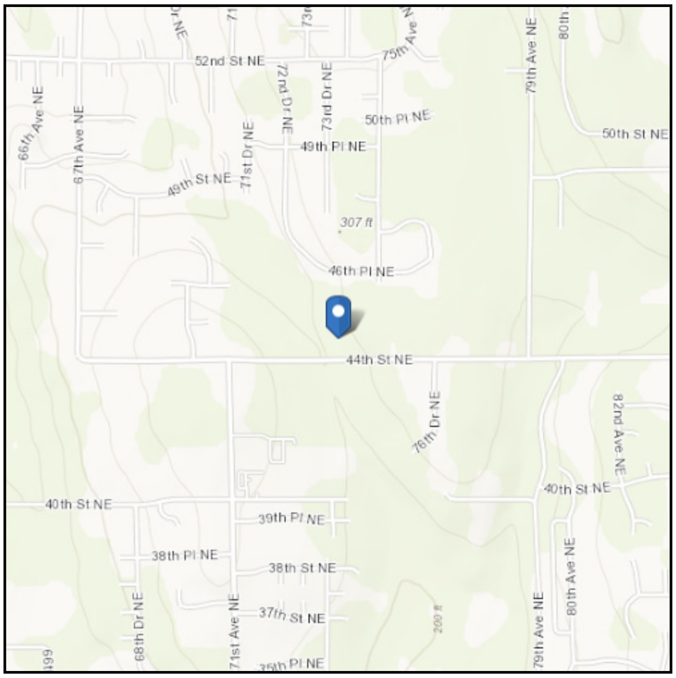
- ASCE 7 Hazards Report

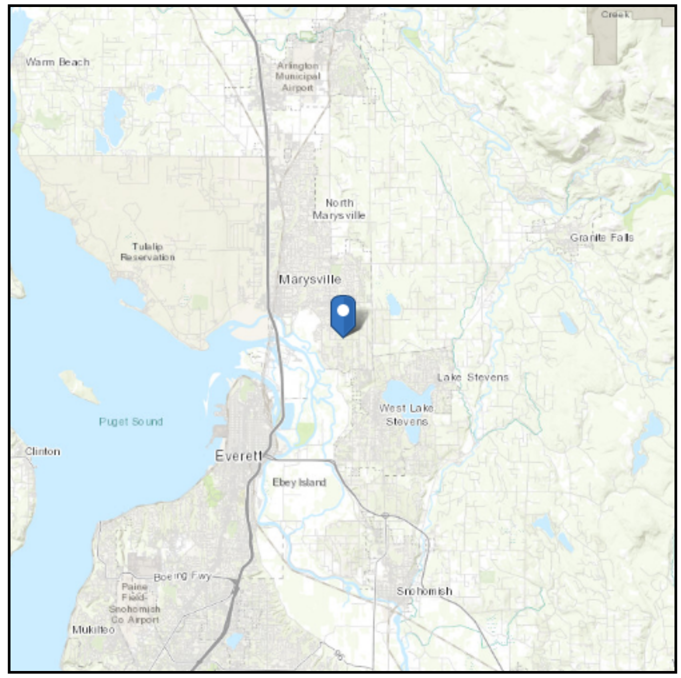
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: C - Very Dense Soil and Soft Rock

Elevation: 257.37 ft (NAVD 88)
Latitude: 48.036192
Longitude: -122.13172





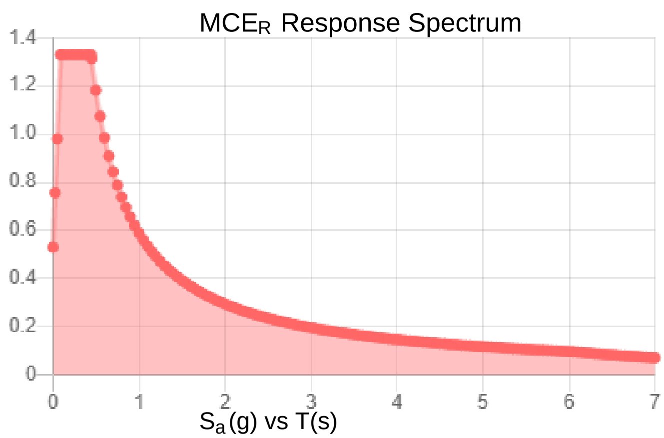
Seismic

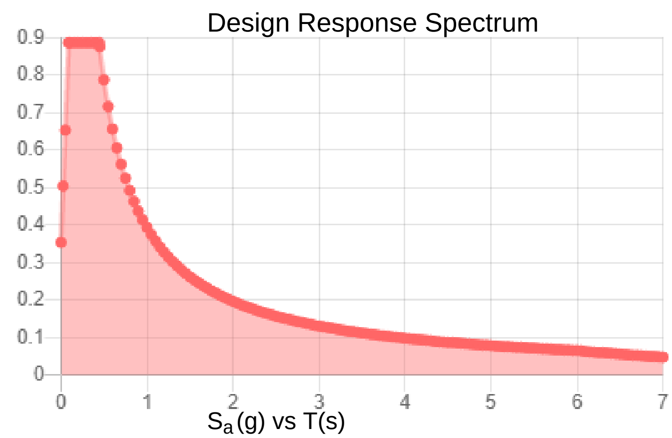
Site Soil Class: C - Very Dense Soil and Soft Rock

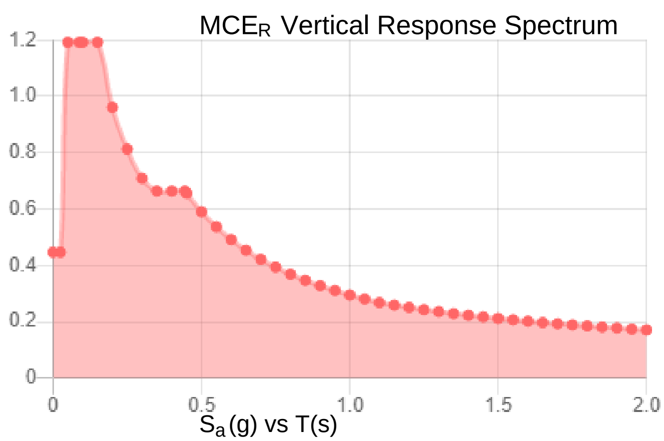
Results:

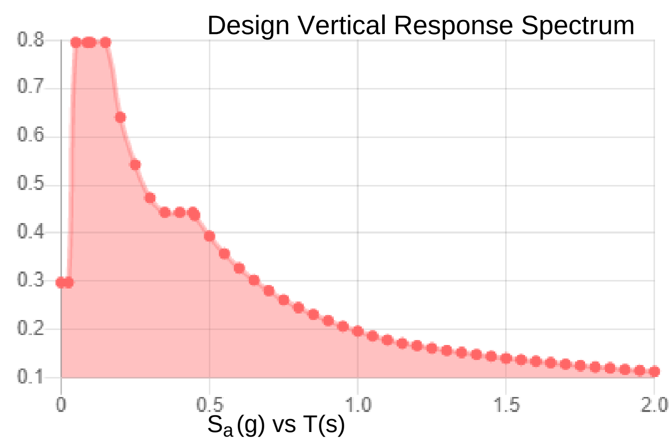
S_S :	1.107	S_{D1} :	0.394
S_1 :	0.394	T_L :	6
F_a :	1.2	PGA :	0.471
F_v :	1.5	PGA_M :	0.565
S_{MS} :	1.329	F_{PGA} :	1.2
S_{M1} :	0.59	I_e :	1
S_{DS} :	0.886	C_v :	1.121

Seismic Design Category D









Data Accessed: **Thu Jun 02 2022**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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