GEOTECHNICAL ENGINEERING STUDY

Full-Sized legible color report must be on site and made available by the Permittee for all inspections.

PRRNSF20251517

For

DS CUSTOM CONSTRUCTION LLC SITE

SINGLE FAMILY RESIDENCE

921, 9TH ST. SW

PUYALLUP, PIERCE COUNTY, WA 98371

Prepared For

DS CUSTOM CONSTRUCTION

13609, 86TH AVENUE E

Prepared By

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

P.O. BOX 1419, ISSAQUAH, WASHINGTON 98027

PGE PROJECT NUMBER 25-819

May 23, 2025

June 2, 2025

Client: DS Custom Construction LLC
755, Oakhurst Drive
Pacific, WA 98047

Attn.: JMDesigns
Joseph Mathews

Re: DS Custom Construction LLC Site
Hout Single Family Residence
Geotechnical Engineering Study
921, 9th St. SW
Puyallup, Pierce County, WA 98371
PGE Project No. 25-819

Dear Mr. Joe:

As per the request, Pacific Geo Engineering, LLC (PGE) completed the geotechnical engineering study for the subject property in Puyallup, Washington, which is shown on Vicinity Map, Figure 1. The study included soil investigation and development of geotechnical engineering recommendations pertinent to the geotechnical aspect of the proposed two-storey single family residence. This geotechnical engineering study report summarizes the results of our evaluation and the recommendation.

This study is completed in accordance with the scope of services described in our final executed proposal no. 25-04-886, dated April 7, 2025. The scope of services was developed based on the preliminary understanding of the proposed new residence from the DS Custom Construction and JMDesigns. Our scope of services is planned to obtain as much subsurface information as possible within the time and budgetary constraints of the project.

The primary purposes of our geotechnical study were to perform site reconnaissance, explore and characterize subsurface soil and groundwater conditions in the site, perform laboratory testing of native soil, and review of available local geologic maps and geotechnical literatures, and to use the data and the information obtained from the above as a basis for formation of our geotechnical recommendations for the proposed new residence.

Our recommendations are provided for the design and construction of the proposed residence, allowable bearing capacity value, slab-on-grade floor, foundation types, subgrade preparation, site preparation, grading and earthwork operation, overexcavation, and fill placement and compaction. Also, recommendations are developed for site drainage and erosion control measures and evaluated site's susceptibility to liquefaction under seismic conditions liquefaction mitigation.

1.0 Proposed Development

The proposed development plan is shown in the Site Plan, Figure 2, provided by JMDesigns. The proposed development plan calls for constructing a new, double-story, single-family residence.

At the time of this study, the final grades of the proposed new residence and the final site grade were not available to PGE. However, we assume that the new residence will be built almost at the current grades without any significant grade changes within the proposed new development area. Based on the presence of loose soils, and to accomplish liquefaction mitigation measurements, there will be some overexcavation below grades will be required and the void areas will be backfilled with new 'fill pad' up to the final bottom subgrades of the foundation and the floor slab. The overexcavation depths will be in the order of 2 to 3 feet below the grades. The actual depths will be decided on-site during the actual construction of the residence.

Based on our experience with similar projects, we anticipate that wall loads will be in the range of 3 to 4 kips per lineal foot, isolated column loads in the range of 40 kips, and slab-on-grade floor loads of 150 pounds per square foot (psf).

The conclusions and recommendations contained in this report are based upon our understanding of the above design features of the development. We recommend that PGE should be allowed to review the final grades and the actual features after the final construction plans are prepared so that the conclusions and recommendations contained in this report may be re-evaluated and modified, if necessary.

2.0 Scope of Services

The purpose of this study was to evaluate the geotechnical aspects of the proposed development, and to identify and address the geotechnical issues that may impact the proposed site development.

The scope of our work did not include any wetland study, or any environmental analysis or evaluation to find the presence of any hazardous or toxic materials in the soil, surface water, groundwater, or air in or around this site.

2.1 Engineering Evaluation

The results from the field and laboratory tests were evaluated and engineering analyses were performed to develop the design information and the geotechnical engineering recommendations for the following items of the proposed development:

Subsurface Conditions

- Descriptions of the soil and the groundwater conditions;
- Soil Test Pit Logs;
- Depth to water table and any sign of high water table, if encountered;
- Laboratory soil index property test results.
- Native soil Classification as per USCS system;
- USGS Soil unit;

General Site Development & Earthwork & Grading

- Descriptions of the soil and groundwater conditions encountered.
- Grading and earthwork, including site preparation, temporary excavation, and fill placement and its compaction.
- Use of on-site soils as structural fill.
- Imported structural fills requirement guidelines.
- Underground utility structure trench backfilling and pipe bedding.
- Temporary and permanent excavation slopes.
- Temporary construction dewatering.
- Site drainage including permanent subsurface drainage systems and temporary groundwater control measures, if necessary.
- Dry and wet weather construction.
- Erosion control measurements.
- Potential geologic hazards: landslide, erosion, and seismic.
- Liquefaction evaluation.
- Geotechnical special inspection requirements.

Structures

- Foundation type and allowable bearing capacity value for designing the foundation.
- Estimated settlement of the foundation for the recommended bearing capacity value and observed soil conditions.
- Frictional and passive design parameters for designing the foundation to resist lateral earth pressures on foundation.
- Slab-on-grade floor for the proposed building.
- Final subgrade preparations for footings and slab-on-grade floor.
- Seismic design considerations, including the site coefficient per ASCE7-16.

Geologic Hazards & Mitigations

- Geologic hazards evaluation: erosion, seismic, and landslide;
- Liquefaction potential evaluation of native soil;
- Liquefaction mitigation;
- Erosion control measures.

3.0 Surface and Subsurface Features

3.1 Site Location

The proposed development is located at the northeast corner of 10th Avenue SW and 9th St SW in Puyallup, Pierce County, Washington as shown in Figure 1. The project site is bounded by single-family residences in the north and the east. The site can be accessed from 9th St SW.

3.2 Site Description

The project site is located within a region dominated by single and multifamily residences. The site is currently an open, vacant land covered with tall grasses. The site is almost a level flat ground.

4.0 Field Investigation

Our field explorations were performed on April 22, 2025. Three (3) test pits were excavated in the subject property to approximate depths of 10 feet below the existing grades. The test pit locations are shown on Site & Exploration Plan, Figure 2, attached with this report. The test pit locations were selected by PGE's engineer during the sub-surface exploration. The test pits were completed using a backhoe provided by the client, and the test pits were backfilled with loosely compacted excavated soils.

A geotechnical engineer from PGE observed the field exploration works including the test pit excavation, sampling, continually logging the subsurface conditions in the test pits, collecting representative bulk samples from different soil layers at different depths of the test pit, and visually-manually classifying the soil samples in the field as per the methods described in the ASTM D-2488-93 (based on soil samples' density/consistency, moisture condition, grain size, and plasticity estimations), and observing the pertinent site features. The soil samples were designated according to the test pit number and sampling depth, stored in watertight plastic containers, and later on transported to our laboratory for further visual examination and testing.

Results of the findings from our field investigation are presented later in Section 6, Soil and Groundwater Conditions section of this report. The descriptions of the soils encountered in the test pits are shown in the Soil Test Pit Log, Figure A-1, A-2, and A-3, Appendix A.

5.0 Laboratory Soil Testing

Laboratory tests were conducted on several selected representative soil samples to evaluate the general physical properties and the engineering characteristics of the soils encountered. The bulk samples were visually-manually classified in the laboratory following the procedure described in ASTM D-2488-17 (based on the soil samples' density/consistency, moisture condition, grain size, and plasticity estimations), and later on the soil samples' classifications were supplemented by laboratory tests data in accordance with the procedure described in ASTM D-2487-17.

Moisture content tests were conducted on selected samples in accordance with ASTM D-2216-10 procedures. One (1) Sieve Analysis test (Grain size distributions) were performed on selected samples in accordance with ASTM D-6913 procedure.

The results of the moisture content tests and the amount of percentages of minus #200 sieve passed are provided in the Soil Test Pit Log, Figure A-1, A-2, and A-3, Appendix A. The grain-size distributions of the soil obtained from the Sieve Analysis tests are shown in the laboratory test report, Figure B-1, of Appendix B.

6.0 Soil and Groundwater Conditions

A topsoil layer of approximately 6 inches thickness consisted of black silt with organics and roots is encountered at the test pit locations. The topsoil was in wet and soft conditions.

The topsoil was underlain by brown, sandy silt (USCS classification: ML), extending up to approximately 5.5 feet below the current grades. This soil layer was in moist and loose condition up to approximately 3 feet below the grade and medium dense below this depth.

The silt layer was then underlain by bluish-gray, silt with sand (USCS classification: SM) up to the bottom of the test pits, approximately 10 feet below the grades. The soil was in wet and medium dense condition. The soil deposits encountered in the test pits can be interpreted as fine grained alluvial soils.

Severe cave-in was encountered below the water depth.

Groundwater Condition

Groundwater was noticed in the test pits at approximately 5.5 feet below the grades. Scattered mottling signs (iron oxide) were noticed in the silt deposit below the water table.

Our exploration was performed in the spring weather. It is to be noted that groundwater conditions may be found different in the site due to the seasonal variations in the amount of rainfall, surface runoff, and other factors not apparent at the time of our exploration. Typically, the water seepage levels rise higher and the flow rates increase during the wet winter months. The possibility of the fluctuations and the presence of seepage and the signs of mottling should be considered when the foundation system is to be designed and constructed for the proposed townhomes.

The preceding discussion on the subsurface conditions of the site is intended as a general review to highlight the major subsurface stratification features and material characteristics. For more complete and specific information at individual test pit location, please review the Soil Test Pit Log in Appendix A. The logs include soil descriptions, stratification, and location of the samples, and the laboratory test results. It should be noted that the stratification lines shown on the log represent the approximate boundaries between various soil strata; actual transitions may be more gradual or more severe. The subsurface conditions depicted in the soil test pit log are for the test pit locations only, and it should not necessarily be expected that these conditions are representative at other locations of the site.

The subsurface conditions of the adjacent site that have been assumed similar in our site may vary in our site. The nature and extent of the variations may not become evident until the construction takes place. If variations do appear, it should be brought to the attention of PGE to reevaluate the recommendations of this report and to modify as necessary prior to proceeding with the construction.

7.0 Geologic Literature & Map Review

7.1 Regional Geology

The site lies within the Puget Sound Lowland, which is part of a regional north-south trending structural and topographic trough or depression lying between Olympic Mountains on the west and Cascade Mountains on the east and extends from southwestern British Columbia to near Eugene, Oregon. The lowland depression experienced successive glaciation and nonglaciation activities over the time of Pleistocene period. During the most recent Fraser glaciation, which advanced from and retreated to British Columbia between 13,000 and 20,000 years ago, the lowland depression was buried under about 3,000 feet of continental glacial ice. During the successive glacial and nonglacial intervals, the lowland depression, which is underlain by Tertiary volcanic and sedimentary bedrock, was filled up above the bedrocks to the present-day land surface with Quaternary sediments, which consisted of Pleistocene glacial and nonglacial sediments. The glacial deposits include concrete-like lodgement till, lacustrine silt, fine sand and clay, advance and recessional outwash composed of sand or sand and gravel, and some glaciomarine materials. The nonglacial deposits include largely fluvial sand and gravel, overback silt and clay deposits, and peat attesting to the sluggish stream environments that were apparently widespread during nonglacial times.

7.2 USGS Soil Unit

As per Figure 3, excerpt from the WA State DNR Map, the site as being underlain by Quaternary Alluvium (geologic Map Unit - Qa). The unit is described as unconsolidated or semiconsolidated alluvial clay, silt, sand, gravel, and (or) cobble deposits; locally includes peat, muck, and diatomite; locally includes beach, dune, lacustrine, estuaries, marsh, landslide, lahar, glacial, or colluvial deposits; locally includes volcanic or tephra deposits; locally includes modified land and artificial fill. Alluvial deposits in the area are expected to exhibit low to moderate strength and moderate compressibility in its undisturbed states. Generally, saturated alluvial deposits consisting of silty sand and sand in the upper 60 feet have a moderate to high potential for liquefaction during a strong earthquake.

In general, our test pit explorations encountered alluvial deposits as described above in the form of sandy silt and silt with sand (USCS classifications: ML and SM, respectively).

8.0 Geologic Hazards

8.1 Landslide Hazards

The site is a level flat ground therefore landslide hazard potential in the site is nil.

8.2 Seismic Hazard & Seismic Design Considerations

The subject site is located in Pacific Northwest, which is considered as a seismically active region. Based on our research on the available literature, it is found that the peak ground acceleration (PGA) of the site (0.5g, as per Table 1 of Section 8.2.2 of this report) during the design earthquake is much higher than 0.3g. Consequently, seismic activity and the resultant ground shaking under a moderate to major earthquake event should be anticipated during the design life of the proposed structure. Therefore, the proposed structure should be designed to resist earthquake loading using code-based seismic design as described in the following sections. We have described the seismic setting at the project site, provided recommendations to develop the code-based design response spectrum to design the proposed structure, and discussed the potential seismic hazards in this regard.

8.2.1 Regional Seismic Setting

The seismicity of western Washington is dominated by the existence of the Cascadia Subduction Zone in which the offshore Juan de Fuca plate is subducting beneath the continental North American plate. Three main types of earthquakes are typically associated with subduction zone environment: crustal, intraplate, and inerplate earthquakes. Seismic records in the Puget Sound Area clearly indicate the existence of a distinct shallow zone of crustal seismicity (the Seattle Fault) that may have surficial expressions and can extend to depths of up to 25 to 30 km. A deeper zone is associated with the subducting Juan de Fuca plate and produces intraslab earthquakes at depths of 40 to 70 km beneath the Puget Sound region (e.g., the 1949, 1964, and 2001 earthquakes) and interface earthquakes at shallow depths near the Washington coast (e.g., the 1700 earthquake with an approximate magnitude of 9.0).

8.2.2 Code-based Ground Motions, Seismic Design Parameters, & Design Response Spectrum

We understand that the seismic design of the proposed structure will be performed in accordance with the 2021 IBC and ASCE 7-16 Standard, which requires the computation of forces to be used for seismic design is based on seismological input and site soil response factors.

Based on the subsurface conditions of the subject site, the subsurface condition can be best described as 'Site Class Type E' and the liquefiable susceptibility is 'high', as per Figure 3, excerpt from the WA State DNR Map. Based on the above soil type, the seismic design parameters are derived from Table 20.3-1, Site Classification of ASCE 7-16 Standard, for site latitude and longitude, that are presented below in Table 1, Seismic Design Parameters.

As per the Section 11.4.2 of ASCE7-16 Standard, the seismological inputs are short-period spectral response acceleration, S_s, and long-term spectral accelerations, S_1, which are corresponding to 'Site Class E'. The S_s and S_1 are for a mapped Maximum Considered Earthquake (MCE_R), which corresponds to ground motions with a 2 percent probability of occurrence in 50 years (return interval of 2,475-years). The S_s and S_1 values are based on regional probabilistic ground motion studies obtained from 2008 USGS seismic hazard maps for the site latitude and longitude. For our analysis, a maximum earthquake magnitude of 7.0 and peak ground surface acceleration of 0.5g are used. The S_s and S_1 are scaled by site soil response factors (site coefficients) F_a (Table 11.4-1, Supplement 1) and F_v (Table 11.4-2, Supplements 1), respectively to account for site specific amplification/damping effects. These tables provide the F_a and F_v values based on the 'Site Class E'.

The Section 11.4.4 of ASCE-7-16 Standard ensures that the seismicity of the region be considered in building design by requiring that structure be designed for the mapped Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters S_{MS} and S_{M1} for short-period (S_S) and 1-second (S_1) period, respectively.

The design spectral acceleration parameters for short-period (S_{DS}) and 1-second (S_{D1}) corresponding to mapped parameters of S_{MS} and S_{M1} are determined based on Section 11.4.5 of ASCE7-16 Standard. As per this section, two-thirds (2/3) of the MCE_R spectral response acceleration parameters are taken as the design spectral acceleration parameters S_{DS} and S_{D1}.

The seismic design parameters provided in the table below

Table 1 - Seismic Design Parameter – Site Class E	
Mapped MCE_R Spectral Response Acceleration Parameter at Short Periods, S_S	1.274
Short-Period Site Coefficient, F_a	1.2
Design Spectral Response Acceleration Parameter at Short Periods, S_{DS}	1.019
Mapped MCE_R Spectral Response Acceleration Parameter at a Period of 1 Second, S_1	0.439
Long-Period Site Coefficient, F_v	Null
Design Spectral Response Acceleration Parameter at 1 Second Period, S_{D1}	Null
MCE_G Peak Ground Acceleration, PGA	0.5
Site Coefficient, F_{PGA}	1.2
Site Modified MCE_G Peak Ground Acceleration, PGA_M	0.6

Notes:

Unless the exceptions stated in ASCE 7-16 11.4.8 apply, for a site classified as Site Class E with an S_S greater than 1.0 and S_1 greater than or equal to 0.2, a site-specific ground motion hazard analysis shall be performed. We are assuming that base isolation will not be a part of the foundation design.

1. Per Exception 1 of ASCE 7-16 11.4.8, F_a was assumed to be 1.2, taken as equal to that of Site Class C, since S_s is greater than or equal to 1.0.
2. Exception 3 of ASCE 7-16 11.4.8 states that if the S_1 is greater than or equal to 0.2 and the period of the structure, T, is less than T_S, the equivalent static force shall be used for design. T_s was calculated to be 0.66 based on the F_v calculated per ASCE 7-16 Supplement 1. The F_v calculated from Supplement 1 is solely to be used to calculate T_s.
3. $S_{DS} = (2/3) * S_{MS}$ where $S_{MS} = F_a * S_S$
4. PGA_M = PGA * FPGA
5. MCE_R is the risk-targeted maximum considered earthquake
6. MCE_G is the geometric mean maximum considered earthquake

8.2.3 Seismically Induced Geotechnical Hazard (Liquefaction) and Mitigation

Based on the existing soil conditions explored during this study, our regional experience, and our knowledge of local seismicity, the potentials for the seismic hazards such as the liquefaction potential in this site is considered as 'high' during a moderate to major earthquake event.

Liquefaction typically occurs in sandy, water-saturated soils such as flood-plain deposits, delta deposits, alluvial sediments, which are the soils conditions found in our project site. These type of soils are highly susceptible to transient increase in pore-water pressure and corresponding loss of effective confining stress and soil strength, results in soils behaving like in a fluid like manner, with the potential for large strain deformation (disintegrate flow failure) during the liquefaction. Liquefaction of these types of deposits during earthquakes of moderate to large magnitude can have severely adverse affects on structures and lifelines. Liquefaction damage often leads to separation of pipeline conduits, settlement of foundations, bearing-capacity failure, lateral movement of ground and bridge piers, and uplift of storage tanks and other positively buoyant structures.

In this study, we have used an empirical method developed by Seed and Idriss, et, al to estimate the approximate post-liquefaction ground subsidence or total ground settlement (dynamic settlement) that could result during a design earthquake magnitude. Based on the analysis, it is probable that soil liquefaction could extend to a depth from below the groundwater table to at least 60 feet below the grade following an earthquake as strong as the MCE. Maximum Considered Earthquake (MCE) was selected in accordance with the U. S. Geological Survey (USGS) Earthquake Hazards Program website. A maximum earthquake magnitude of 7.0 and a peak horizontal ground surface acceleration of 0.5g were used. The analyses assumed groundwater at 5.5 feet below the grades. Our estimation indicated that the ground settlement (dynamic) could occur exceeding the allowable limit during the MCE.

Based on our engineering analysis and evaluation, an excessive dynamic differential settlement could be mitigated by using two foundation options; a combination of foundation system comprised of a rigid, reinforced, mat foundation with thickened edge bearing on 'fill pad' (Figure 4), or a rigid framed foundation comprised of isolated interior shallow footings and perimeter strip footings interconnected with grade beams bearing on 'fill pad' (Figure 5). The latter foundation system should be designed and built with structural floor slab spanning in between the grade beams and structurally connected to the grade beams.

A geogrid reinforced layer like a Mirafi 500X or of equivalent strength is to be needed below the bearing 'fill pad' above the final native subgrade for both foundation options. The above figures are not designed drawings and not for construction purposes. The actual foundation systems should be designed and the drawings should be prepared by a profession structural engineer to be retained for the project.

This combined system of expected to limit the static settlement in the order of 1 inch or less with a differential settlement across the structure in the order of ½ inch or less, respectively. The dynamic differential settlement is anticipated to be limited in the order of 2 to 3-inches across the mat foundation.

It should be noted that the above foundation options with underlying compacted bearing ‘fill pad’ are supposed to prevent catastrophic foundation collapse during a large seismic event, thereby providing the safety of the residents.

It is found that it is a typical practice in the Puget Sound area to build the similar type of structure over liquefiable soil deposit with potentials for excessive settlement using settlement tolerant construction method, e.g., wood and metal framing and siding, and avoiding use of settlement-sensitive materials like masonry, stucco, tiles, and any other similar type of material. Wood frame structures typically have performed satisfactorily in the liquefiable prone soil deposit, when founded on a combination of reinforced mat and a ‘fill pad’ foundation system.

However, it should not be expected that with the above prescribed recommendations the damage of the structures is completely prevented and the continued function of the structure is ensured.

8.3 Erosion Hazard

Uncontrolled surface water with runoff over unprotected site surfaces during construction activities is considered the single most important factor that impacts the erosion potential of a site. The erosion process may be accelerated significantly when factors such as soils with high fines, sloped surface, and wet weather combines together.

The erosion hazard can be mitigated if the following recommendations are implemented.

All erosion sediment control measures must conform to the City of Puyallup or Pierce County requirements, whichever is applicable. As a minimum, we recommend implementing the following erosion and sediment control Department of Ecology (DOE) best Management Practices (BMPs) prior to, during, and immediately after clearing and grading activities at the site.

Excavation and construction of the project can readily be accomplished without adversely impacting the site and surrounding properties by exercising care and being proactive with the maintenance and potential upgrading of the erosion control system through the entire construction process.

Mitigations

- Mass grading activities and the earthwork should be completed within the dry summer period.
- Measurements such as the control of surface water must be maintained during construction.
- Any cut slopes and soil stockpiles should be covered with plastic during wet weather.
- Soil stockpiles should be minimized.
- Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface.
- Vegetation clearing must be kept very limited within the proposed constriction area to reduce the exposed surface areas. It is recommended that following the clearing of the vegetations, grading the open exposed areas should be covered with mulch or hydro seed.
- No disturbance or removal of the existing vegetations, tress, and undergrowths should be made beyond the proposed construction area and the vegetation clearing limit.
- Limit disturbance to areas where construction is imminent. If possible, site clearing and grading should be performed in stages, with successive stages not being cleared until erosion control measures for the previous stages are in place.
- Determine staging areas for temporary stockpiles of excavated soils as part of the excavation planning.
- One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil areas to prevent accumulated water or runoff from the work area from becoming silty in the first place.
- Provide temporary cover for denuded areas including cut slopes and soil stockpiles during periods of inactivity. From October 1 to April 30, no soil shall remain un-stabilized for more than 48 hours. From May 1 to September 13, no soil shall remain un-stabilized for more than seven days. Temporary cover may consist of straw mulch or plastic sheeting that is securely anchored to the ground surface. Plastic covering should be placed and anchored, as specified in BMP C123 provided in Chapter 4.1 of the Stormwater Management Manual for Western Washington. Mulching should be conform to the guide lines outlined in the BMP C121 provided in Chapter 4.1 of the Stormwater Management Manual for Western Washington

- Establish permanent covers for exposed areas that will not be worked for period of 30 days or more by seeding in conjunction with a mulch cover or appropriate hydroseeding. Seeding should conform to the specifications outlined in BMP C120 provided in Chapter 4.1 of the Stormwater Management Manual for Western Washington.
- The temporary erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered during the site work.

- Temporary erosion and sedimentary control (TESC) plan, as a part of the Best Management Practices (BMP) must be developed and implemented as well. The TESC plan should include the use of geotextile barriers (silt fences) along any down-slope, straw bales to de-energize downward flow, controlled surface grading, limited work areas, equipment washing, storm drain inlet protection, and sediment traps. The TESC plan may need to be reviewed and modified periodically to address the changing site conditions during ongoing progress of the construction and the weather.
- A permanent erosion control plan is to be implemented following the completion of the construction. Permanent erosion control measurements such as establishment of landscaping, replantation of trees and groundcover vegetations as soon as feasible in areas that are necessarily disturbed by earthwork activities, control of downspouts and surface drains, control of sheet flow over the excavation slope, prevention of discharging water over the excavation slope and at the toe of the slope are to be implemented following the completion of the construction.
- Install temporary or permanent tightline pipes, where necessary and practical, to convey stormwater from above slope to appropriate downslope facilities on flatter terrain.
- Install permanent stormwater runoff diversion systems, such as swales, curbs, berms, or pipes, to prevent flow directly over any final slope grades.
- We recommend that completed graded-areas be restricted from traffic or protected prior to wet weather conditions. The graded areas may be protected by paving, placing asphalt-treated base, a layer of free-graining material such as pit run sand and gravel or clean crushed rock material containing less than 5 percent fines, or some combination of the above.

Containment

- Install a silt fence along the downhill side of the construction area that will be disturbed. The silt fence should be placed before cleaning and grading is initiated and should conform to the specifications outlined in BMP C233 provided in Chapter 4.2 of the Stormwater Management Manual for Western Washington.
- Construct interceptor dikes and shallow drainage swales to intercept surface water flow and route the flow away from the construction tare to be stabilized and approved point of controlled discharge. Some small detention ponds with pipe slope drains may be incorporated with the swales in order to collect and transport the runoff to the discharge point.
- Provide on-site sediment retention for collected runoff. Runoff should not flow freely over the top of the slope or off the site.
- The on-site contractor should perform daily review and maintenance of all erosion and sedimentation control measures at the site to ensure their proper working order.

Provided the recommended erosion and sedimentation control BMP's are properly implemented and maintained, it is our opinion that the planned development will not increase the potential for erosion at the site or on adjacent properties.

9.0 Executive Summary

Based on the review of the geological literatures and hazard maps, results of our subsurface explorations, and our engineering evaluations it is our opinion that the proposed residence is feasible in the subject site, provided that the geotechnical recommendations provided in this report are properly understood and interpreted, and strictly implemented during the design and construction phases of the proposed residence.

If the subsurface conditions are found to be different in the unexplored areas of the site than what it is found in the current explored areas then the recommendations provided in this report may need to be revisited and altered, to incorporate the changes if to be found on the subsurface conditions. This may calls for possible changes in the final design of the project as well. A contingency plan should be in place by the owner considering the above scenario.

The site is underlain by unconsolidated, soft to medium dense, saturated alluvial deposits that are primarily consisted of sand and silt deposited since the last recession of the glaciers almost 10,000 years ago. According to the WA State DNR map, provided in Figure 3, the site soils are mapped as 'highly' liquefiable. Soil liquefaction is a condition where saturated cohesionless soils undergo a substantial loss of strength due to the build-up of excess pore water pressures resulting from cyclic stress applications induced by earthquakes. Liquefaction of the above soils below the groundwater level may cause a significant amount settlement of the ground known as dynamic settlement. Based on our experience in the project area and our knowledge of the site soils and groundwater conditions, it is our opinion that the risk of soil liquefaction for the zones extending below groundwater table (i.e., about 5.5 feet) to 60 feet depth is moderate to high in this site during an earthquake as strong as the MCE level 7.0 design earthquake.

The alluvial deposits undergo significant amount of settlements comprised of post-construction static settlement and liquefaction induced dynamic settlement. Most of the anticipated static settlement is likely to occur during construction as dead loads are applied, which gradually reduces over the time due to the post-construction consolidation of the unconsolidated alluvial deposits. Based on our evaluation, footing settlement under static loading condition is estimated to be exceeding the allowable limit. The liquefaction induced dynamic settlement is estimated based on a procedure developed by Seed, Idriss, et al., which provides the approximate dynamic settlement as to be in the range of 6 to 8 inches exceeding the allowable limit for seismic shaking during an MCE level 7.0 magnitude design earthquake.

Based on the recent history and the research carried out on the seismic hazard in the Puget Sound area it is learnt that the amount of settlement becomes excessive for a structure that is older, smaller, relatively lightly loaded, and built on top of the above type of alluvial deposit with a lightly-reinforced conventional shallow spread footing. However, using recent construction practices, the similar type of new structure is found to be undergone less settlement when the structure is built on top of the above type of alluvial deposit with a heavily-reinforced shallow foundation system as recommended earlier and described below.

A reinforced, rigid mat foundation with thickened edge (Figure 4) or a rigid framed foundation system comprised of isolated interior shallow footings and perimeter strip footings with interconnected grade beams and structural floor slab (Figure 5) are found to be performing better to reduce the excessive post-construction static settlement as well as to reduce the liquefaction induced dynamic settlement, preventing potential collapse of the structure due to the liquefaction. This is because of the structural nature of the above foundation systems, which have the increased stiffness, structural rigidity, and the flexural strength to overcome the poor conditions of the underlying soils e.g., soils that are prone to excessive differential settlement, noticeable static settlement, and liquefaction induced dynamic settlement. Because of the structural continuity of the above foundation systems, they bridge over the pockets of excessive soft soils and localized soil liquefaction (sand boils).

A rigid mat foundation is essentially a very large spread footing that usually encompasses the entire footprint of the structure. Due to its larger size the mat foundation is able to transfer the building load into the underlying ground in a much reduced amount requiring a lower allowable bearing capacity value than a conventional spread footing would otherwise require to support the structure. The mat foundation is able to distribute the building load into the ground in a more uniform way to keep the settlement across the foundation in a more uniform way, hence lowering the amount of differential settlement.

In addition to the above foundation options, we recommend that a bearing 'fill pad' is to be placed below the footprint area of the proposed residence to reduce the settlement further and enhance the allowable bearing capacity to support the footings. We recommend that a minimum of 24 inches thick compacted 'fill pad' consisted of clean crushed rocks, such as 2- to 4-inch rock spalls or 2-inch ballast rocks be placed below the mat foundation. The rock layer must be placed in individually compacted one-foot thick layer. Each layer must be compacted with a roller using static mode only to minimize the ground-borne vibration. The rolling will help rearranging the rocks and bringing each rock layer in non-yielding condition. The rock layer should be wrapped up with Mirafi 140N fabric in all sides of the rock layer to prevent migration of fines from the surrounding soils to the void areas in the rock layer and resulting clogging of the void spaces. The Mirafi fabric should be overlapped on one another for 12 inches width when the layers will be laid side by side. The top of the rock layer can be capped with a 4 inches thick, compacted, clean 5/8-inch crushed rock layer with less than 2 to 3 percent fines by weight of the material passing #200 sieve, which can be served as a capillary break layer as well as an even level surface above the quarry spalls layer. A walk-behind, hand-held regular plate compactor should be used for compaction of this layer. The layer must be compacted to minimum of 95 % of the fills' maximum dry density value to be determined based on the laboratory Modified Proctor test method ASTM D1557, as described in Section 10.1.7, 'Fill Placement and Compaction Requirements'. The foundation subgrade for the proposed structure be over-excavated by a minimum of 24 inches and the over-excavated depth should be backfilled with the 'fill pad'. The foundation bearing 'fill pad' needs to be extended beyond the actual outer edges of the mat foundation a distance equal to the bearing pad thickness of 24 inches. This is because foundation stresses are transferred outward as well as downward into the bearing pad soils in an imaginary line at 1H:1V inclination.

The final native subgrade at the overexcavation depth should be compacted to a firm and unyielding condition prior to placing the 'fill pad' as per the Section 10.1.3, 'Final Subgrade Preparation'. The final native subgrade should be proofrolled and compacted with a roller using static mode only to minimize the ground-borne vibration. A geo-grid layer such as Mirafi 500X or equivalent must be placed on the exposed final naïve subgrades prior to placing the 'fill pad'.

We predict that the combination of the foundation system, bearing 'fill pad', and geogrid layer will help to reduce the static settlement to approximately 1 inch or less and the differential settlement to ½ inch or less, respectively. The combined system is predicted to reduce the dynamic differential settlement across the rigid mat foundation and the rigid framed foundation.

It is our opinion that it is cost-prohibitive to mitigate soil liquefaction for this project. It is to be noted that, even with the recommended combined system, it is considered to be still inadequate to mitigate the potential adverse effects of the potential liquefaction induce settlement on the site, as the improvement measure do not directly improve the liquefiable soil zones underlying the site at depth.

If liquefaction occurs it would likely result in differential settlement of the foundation that could result in some architectural and structural damage. The damages could be in the form of cracking of interior or exterior walls/slabs, settlement of sidewalks and driveways, and may require re-leveling of doors, windows, columns etc. The owner should be aware of this risk and must be willing to accept such risk. In our opinion, the potential damage would not significantly impede entrance or egress from these buildings following an MCE level design earthquake, provided the geotechnical recommendations of this report are incorporated into the design and construction of this project.

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Other hard surfaces such as paved areas, patios, or walkways that are supported on the underlying compressible soils have some risk of future settlement, cracking, and the need for maintenance. To reduce this risk, we recommend that over-excavation a minimum of 12 inches to 24 inches of the upper loose soils from these areas and replacing this material with adequately compacted structural fills consisted of either 1.25-inch clean crushed rock or pit run. The fills must be compacted with a roller using static mode only to minimize the ground borne vibration. The fills must be compacted o minimum of 95 % of the fills' maximum dry density value to be determined based on the laboratory Modified Proctor test method ASTM D1557, as described in Section 10.1.7, 'Fill Placement and Compaction Requirements'. The final native subgrade at the overexcavation depth should be proofrolled and compacted to a firm and unyielding condition prior to placing the structural fills following the recommendations provided in Section 10.1.3, 'Final Subgrade Preparation'. We recommend that a roller with static mode only will be used to minimize the ground-borne vibration. If the final native subgrade cannot be proofrolled adequately to achieve the firm and unyielding conditions due to the softness and wet condition of the subgrade soil at the over-excavation level, it might be necessary to place a layer of geo-grid such as Mirafi 500X or equivalent on the exposed surface prior to placing the fills. The final decision of using the geogrid layer should be decided on-site by the geotechnical engineer. The above recommendation is only for exterior hard surface to be supported on grade and does not apply for the interior of the building.

In addition to the above recommendations, we recommend that the hard surface section should be thickened and reinforced with rebar to further reduce the effects of settlement due to the underlying compressible soils. However, the potential of long-term cracking of the hard surfaces would still exist and require repairs.

It is essential that an experienced geotechnical engineer should verify the final native subgrade, overexcavation, placing and compaction of 'fill pad', and allowable bearing pressure.

The remainder of this section (10.0) presents specific engineering recommendations on the pertinent geotechnical aspects that are anticipated for the design and construction of the proposed development. These recommendations should be incorporated into the final design and drawings, and construction specifications.

10.0 Conclusion & Recommendations

10.1 Site Preparation

Preparation of the site should involve clearing, stripping, subgrade preparation and proofrolling, cutting, filling, excavations, and drainage installations. The following paragraphs provide specific recommendations on these issues.

10.1.1 Clearing and Grubbing

Initial site preparation for construction of the proposed new residence and the concrete patio, driveway, parking area, and any other load-bearing structure, and placing new fills on the final 'competent' native subgrades should include stripping of vegetation and topsoil from the construction area. Based on the topsoil thickness encountered at our test pit location, we anticipate topsoil stripping depths of about 6 inches, however, thicker layers of topsoil may be present in unexplored portions of the site. It should be realized that if the stripping operation takes place during wet winter months, it is typical a greater stripping depth might be necessary to remove the near-surface moisture-sensitive silty soils disturbed during the stripping; therefore, stripping is best performed during dry weather period. Stripped vegetation debris should be removed from the site. Stripped organic topsoil will not be suitable for use as structural fill but may be used for future landscaping purposes.

10.1.2 Overexcavations of Loose Soils

Once the clearing of the vegetations and the topsoils from the proposed development area will be completed, the loose native soils encountered in this site must be completely removed upto the dense soil deposit, which must be verified by PGE's on-site geotechnical engineer. The overexcavation should be performed using smooth-edged bucket to limit the disturbances of the potential final native subgrades. Following the removals of the loose soils, the exposed native subgrade should be prepared as described in the following Section 10.1.3, Final Subgrade Preparation. After the subgrade preparation is completed and approved by the geotechnical engineer the overexcavated areas should be backfilled with newly imported structural fills upto the desired final subgrade. The new fills must be placed and compacted as described later on in Section 10.1.7, Fill Placement & Compaction of this report.

10.1.3 Final Subgrade Preparation

<u>Redensification</u>

After the clearing of the vegetations and topsoils, and following the completion of the overerxcavations upto the final native subgrades, as a part of the subgrade preparation, we recommend that all final native subgrades that are supposed to be supporting the load-bearing structure should be redensified to enhance the in-situ density of the final native subgrades, improving their bearing capacity hence reducing their potentials of undergoing excessive settlement. Typically, the redensification is effective for the upper one to two feet of soil below the final native subgrades. The depth of the in-situ density increase depends on the compaction equipment to be used. The redensification of the final native subgrades can be accomplished either using a big, heavy-weight, single- or double-drum, vibratory roller or a walk-behind, heavy-duty, vibratory plate compactor (similar to TMG-PC330K Reversible Plate Compactor with 14HP Kohler Engine), whichever equipment will be found suitable by the contractor for the observed native subgrade conditions. The redensification is achieved by having the compaction equipment make several passes as to be found necessary by the on-site geotechnical engineer. One pass is considered to consist of a passage of the compactor in each direction, forwards and backwards, over the same strip of subgrade. The redensification process should be carried out over the whole of the excavated “at grade” footing subgrade and slab-on-grade areas, and any other load-bearing structures such as the new fill pad and sidewalk.

Proofrolling

Any exposed subgrades that are intended to provide direct support for new construction and/or require new fills should be adequately proofrolled to evaluate their conditions and to identify the presence of any isolated soft and yielding areas and to verify that stable subgrades are achieved to support the proposed structures, and any new fills. The proofrolling of the final native subgrades can be accomplished either using a big, heavy-weight, single- or double-drum, vibratory roller or a walk-behind, heavy-duty, vibratory plate compactor (similar to TMG-PC330K Reversible Plate Compactor with 14HP Kohler Engine), whichever equipment will be found suitable by the contractor for the observed native subgrade conditions. The proofrolling should be done under the supervision of PGE’s on-site geotechnical engineer. If it is found by the on-site geotechnical engineer that the soil is too wet near the subgrade to be proofrolled or it not feasible to proofroll the subgrade, then an alternative method (i.e., visual evaluation and probing with a 1/2-inch diameter steel T-probe) can be used by the geotechnical engineer to identify the presence of any isolated soft and yielding areas and to verify that stable subgrades are achieved to support the proposed structures and any new fills.

If any subgrade area is found in soft and moist conditions, ruts and pumps excessively, and cannot be stabilized in place by compaction the affected soils should be over-excavated completely to firm and unyielding suitable bearing materials, and to be replaced with new structural fills to desired final native subgrade levels. If the depth of overexcavation to remove unstable soils becomes excessive, a geotextile fabric, such as Mirafi 500X or equivalent in conjunction with structural fills may be considered to achieve a firm bearing final subgrades to support the proposed structures and any new fills.

Any final native subgrades and foundation bearing surfaces should not be exposed to standing water. If water is present in the final native subgrades or in the base of the footing excavation, it must be removed completely to bring the subgrades into dry condition before placing any new fills and formwork and rebars. Protection of exposed soil, such as placing a 6-inch thick layer of crushed rock or a 3- to 4-inch layer of lean-mix concrete, could be used to limit disturbance to bearing surfaces.

If the base of an overerxcavated area is excessively soft and wet and needs stabilization then we recommend considering a 6 to 12-inch layer of ballast rock or quarry spalls should be placed to form a base on which the structural fill needs to be placed and compacted to achieve the final grade. Ballast rock should meet the requirements for Class B Foundation Material in Section 9-03.17 and quarry spalls should meet the requirements in Section 9-13.1(5) of the 2024 WSDOT Standard Specifications. The ballast rock or quarry spalls should be pushed into the subgrade with the back of a backhoe bucket or with the use of a large-vibratory steel drummed roller without the use of vibration. Such decision should be made the on-site geotechnical engineer during the actual construction of the project.

10.1.4 Backfilling of Test Pit Area

The loosely backfilled soils in the area of exploratory test pits should be overexcavated completely to the firm native soils and backfilled with adequately compacted new, imported structural fills to the final grades, following the procedures described later on in Section 10.1.7, 'Fill Placement and Compaction Requirements' of this report. The new, imported structural fills should be granular materials like sand and gravel meeting the requirements provided in Section 10.1.6, ‘Structural Fills’ of this report. Prior to placing the new fills the final native subgrades at the bottom of the overexcavated areas must be proofrolled adequately to firm and unyielding conditions as recommended earlier in Section 10.1.3, ‘Subgrade Preparation’ of this report and accepted by PGE’s on-site geotechnical engineer prior to placing new fills.

10.1.5 Reuse of Native Soils as Structural Fills

The ability to use native soils as structural fills, to be obtained during the mass grading activities, will depend on the factors such as the quality of the native soils, i.e., the presence of excessive roots and organics, fines content, larger-size particles, moisture content, soil types and their gradation, and the prevailing weather conditions during the time of the construction i.e., dry or wet weather. The weather plays a significant role in determining if the native soils can be compacted adequately during the wet weather period, especially when the native soils content higher percentages of fines.

Typically, native soils containing unsuitable materials such as the excessive roots and organics are not considered suitable for use as structural fills.

No existing fills of uncontrolled and undocumented nature and containing any type of debris can be used as new structural fills.

If the native soils contain percentage of fines equal to or less than the allowable percentage of fines (typically 5% or less) recommended for ‘imported structural fills’ then the native soils are to be considered as moisture insensitive soils and can be considered for reuse as structural fills.

However, if the native soils contain higher percentages of fines compared to the typical ‘imported structural fills’ that contains 5% or lesser fines, then the native soils should be considered as moisture sensitive soils, which can only be reused based on the weather conditions and following a careful evaluation of the native soils by a geotechnical engineer. Typically, when the fines content (that portion passing the U.S. No. 200 sieve) of soil increases, the soil becomes increasingly sensitive to small changes in moisture content, which makes the soils' compaction more difficult or impossible. Soils containing more than about 5 percent fines by weight cannot be consistently compacted to the recommend degree when the moisture content is more than about 2 percent above or below the optimum. Especially, if the soils with higher fines content are used during the wet weather period, typically between October and May, significant reduction in the soils strength and support capabilities occur. Also, when these soils become wet they may be slow to dry and thus significantly retard the progress of grading and compaction activities. Therefore, the native soil containing higher percentage of fines cannot be used as structural fills during the wet weather period. However, this type of native soil can be used as borrow materials for general filling purposes during the dry season, provided the optimum moisture content of the soils can be maintained during the compaction.

In addition to the higher percentage of fines, if the native soils are found in excessively over the optimum moisture content, then the soils would pose problems during their compaction. This may require moisture conditioning of the native soils prior to their placement and compaction.

Other criteria that is to be considered prior to use native soils as structural fills is the presence of significant amount of larger-size particles such cobbles and boulders. Typically, this type of soil is not considered suitable to use as structural fills, since the cobbles and the boulders pose problems during the compaction of the fills. Therefore, the native soils if considered to be used as borrow materials then the cobbles and the boulders must be removed from the native soils. This can be accomplished either by screening the native soils on-site or by selectively handpicking the larger-size particles, whichever methodology is feasible and economical. The PGE's on-site geotechnical engineer should inspect the final fill product to verify that the fills do not contain larger size particles. The final fills should contain a maximum of 2 to 3-inch particle diameter for being able to be adequately compacted.

The suitability of using the native soils should be verified and approved by the on-site geotechnical engineer prior to their use. If the native soils cannot be used after the inspection and asked by PGE's on-site geotechnical engineer to discard then imported new structural fills are to be brought in to the site for backfilling purposes. In the event that whether the fill materials are to be imported to the site, we recommend that the imported fill materials be verified and approved by the on-site geotechnical engineer prior to their use. We recommend that a contingency plan should be in place in the project budget if the native soils are to be exported out and new structural fills need to be imported into the site.

10.1.6 Structural Fill

General Requirements

Typically, excavated native soils containing topsoil, unsuitable materials such as excessive roots and organics, wood debris and pieces, trash, left over construction debris are not considered suitable for use as structural fills, and should be properly disposed offsite.

If the native soils are found unsuitable for using as structural fills then we recommend that imported structural fill should be used for backfilling purposes. The workability of material for use as structural fill will depend on the gradation and moisture content of the soil. Structural fill is defined as non-organic soil, free of any debris and deleterious materials, and well-graded and free-draining granular material, such as sand and gravel or crushed rock with a maximum particle size of 3 inches for any individual particle and less that 5 percent fines by weight based on the minus ¾-inch fraction. We recommend that washed crushed rock or select granular fill, as described below, be used for structural fill during wet weather. If prolonged dry weather prevails during the earthwork phase of construction, materials with somewhat higher fines content may be acceptable. Weather and site conditions should be considered when determining the type of import fill materials purchased and brought to the site for use as structural fill. Frozen material should not be used as structural fills. All materials should be approved by the project geotechnical engineer prior to use. A sample of each fill material type should be submitted to the project geotechnical engineer for evaluation and approval prior to use.

A typical gradation for structural fill is presented in the following table.

Table 2 - Structural Fill

U.S. Standard Sieve Size	Percent Passing by Dry Weight
3 inch	100
¾ inch	50 –100
No. 4	25 – 65
No. 10	10 – 50
No. 40	0 – 20
No. 200	5 Maximum*

* Based on the ¾ inch fraction.

WSDOT Structural Fills

For reference purpose, the following table provides the specifications for various types of structural fills that can be considered in this site for use as new, imported structural fills.

Table 3 - WSDOT 2024 Structural Fills Specifications	
Fill Type	Recommended Materials
Structural Fill	9-03.9(1) Ballast 9-03.9(3) Crushed Surfacing Base Course 9-03-12(1)A Gravel Back fill for Foundation Class A 9-03.14(1) Gravel Borrow
Common Fill	Section 9-03.14(3) Common Borrow
Free-draining Granular Fill	9-03.9(2) Permeable Ballast 9-03.12(2) Gravel Backfill for Walls 9-03.12(4) Gravel Backfill for Drains

For most applications, we recommend that structural fill consist of material similar to ‘Gravel Borrow’ or ‘Select Borrow’ as described in Section 9-03.14(1) or Section 9-03.14(2), respectively, of the WSDOT 2024 Standard Specifications.

Select Granular Fill

Imported materials with gradation characteristics similar to WSDOT 2024 Standard Specification 9-03.9 (Aggregates for Ballast and Crushed Surfacing), or 9-03.14 (Gravel Borrow) is suitable for use as select granular fill, provided that the fines content is less than 5 percent (based on the minus ¾-inch fraction) and the maximum particle size is 6 inches.

Other Fill Materials

Other materials may also be considered suitable for use as structural fill provided they are approved by the project geotechnical engineer. Such materials typically used include clean, well-graded sand and gravel (pit-run); clean sand; various mixtures of gravel; crushed rock; controlled-density-fill (CDF, it should meet the requirements in Section 2-09.3(1)E of the WSDOT 2024 Standard Specifications); and lean-mix concrete (LMC). Recycled asphalt, concrete, and glass, which are derived from pulverizing the parent materials also potentially useful as structural fill in certain applications. These materials must be thoroughly crushed to a size deemed appropriate by the geotechnical engineer (usually less than 2 inches). The structural fills should have a maximum 2 to 3-inch particle diameter.

Stabilization Material

Stabilization rock should consist of pit or quarry run rock that is well-graded, angular, crushed rock consisting of 4- or 6-inch-minus material with less than 5 percent passing the US Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material. WSDOT SS 9-13.(15) - Quarry Spalls can be used as a general specification for this material with the stipulation of limiting the maximum size to 6 inches.

10.1.7 Fill Placement and Compaction Requirements

Generally, quarry spalls, controlled density fills (CDF), lean mix concrete (LMC) do not require special placement and compaction procedures. In contrast, clean sand, crushed rock, soil mixtures and recycled concrete should be placed under special placement and compaction procedures and specifications described here.

The structural fills under structural elements should be placed in uniform loose lifts not exceeding 12 inches in thickness for a big, heavy-weight, single- or double-drum, vibratory roller or a big, heavy-duty, hand-guided, walk-behind, vibratory plate compactor (similar to TMG-PC330K Reversible Plate Compactor with 14HP Kohler Engine). A regular, walk-behind vibratory plate compactor can be used when the loose fill thickness will be kept within 4 to 6 inches.

No heavy compaction equipment such as hoe pack or big vibratory roller should be used to compact the backfills to be placed behind the footing stem walls, within the horizontal distance equal to the heights of the walls. Use of the heavy compaction equipment will impose excess surcharge load on the walls, which may cause permanent lateral instability to the walls. We recommend that the fills behind the footing stem walls should be placed in 4 inches lifts and to be compacted with a hand held smaller and lighter compaction equipment.

Each lift of fills whether 12 inches or 4 inches or 6 inches should be compacted to a minimum of 95 percent of the fill's maximum dry density as to be determined in the laboratory by ASTM Test Designation D-1557 (Modified Proctor) method, or to the applicable minimum City or County standard, whichever is the more conservative.

The fill should be moisture conditioned such that its final moisture content at the time of compaction should be at or near (typically within about 2 percent) of its optimum moisture content, as determined by the ASTM Test Designation D-1557 (Modified Proctor) method. This should help enhance the compatibility of the materials and avoid the risks involved with wet, moisture sensitive soils. Fills should not be placed on frozen subgrades.

If the fill materials are on the wet side of optimum, they can be dried by relatively inexpensively periodic windrowing and aeration or by intermixing lime or cement powder to absorb excess moisture. An ordinary Portland cement powder can be used in this regard. In using concrete we have found that the hydration of the cement not only results in water absorption, but also develops some “concrete-like” strength within the soil and cement matrix. In our experience the soil cement matrix can sometimes generates a compressive strength in excess of two thousand (2,000) psi. If this option is selected, we recommend that for a preliminary estimation purpose, the cement powder may be intermixed at a rate of about 3% by weight of the soil. The actual cement content should be decided during the mass grading activity depending on the wet weather, soils’ natural moisture content, and the soil types. This form of soil treatment is not suitable for any type soils that are considered as free-daring backfills.

The compacted structural fill pad should extend outside all foundations and other load bearing structures elements for a minimum distance equal to the thickness of the fill pad.

Because of the sensitivity of this project we recommend that any and all structural fills and /or load bearing backfills be tested for determining the in-place density and the water content of the fills as per the Nuclear Density Gauge method (ASTM D6938). This test results will help to verify that the backfills have the achieved the appropriate degree of compaction and the moisture content. We recommend that compaction of the fills be tested periodically throughout the fill placement. A field compaction testing program should be prepared by the contractor with the assistance from the project geotechnical engineer. If field density tests indicate that the last lift of compacted fills has not been achieved the required percent of compaction or the surface is pumping and weaving under loading, then the fill should be scarified, moisture-conditioned to near optimum moisture content, re-compacted, and re-tested prior to placing additional lifts.

We recommend that a minimum of one test be performed for one hundred (100) square feet of compacted or backfill surface area or for every one hundred (100) cubic feet of fill or backfill, whichever generates the greater number of compaction tests.

We also recommend that to verify the compaction of the fill pad in both horizontal and vertical directions, when the fill thickness will be more than one foot, the compaction test locations and the elevations should be spaced in both directions. In this manner it should be possible to show with a reasonable degree of accuracy the "density profile" through the backfill. This is an important element of the QA/QC program of the project in the event there is a problem with the fill or backfill performance and subsequent litigation.

10.1.8 Dry Weather Construction

We prefer the proposed construction should be completed during the dry season to mitigate any erosion related issues that may otherwise arise during the construction activities in the wet season. Erosion particularly happens, when uncontrolled surface runoff is allowed to flow over unprotected excavation areas of the site during the wet winter months.

10.1.9 Wet Weather Construction

If the construction takes place during the wet weather, the near surface soils, which is anticipated as to be moisture sensitive, will be found susceptible to degradation and disturbed when get wet. Therefore, it may be necessary to adopt some remedial measures to enhance the subgrade conditions in this site if the construction takes place in the winter. The contractor should include a contingency in the earthwork budget for this possibility. The appropriate remedial measure is best determined by the geotechnical engineer during the actual construction of the project. The following remedial measures may be considered in this regard:

- The earth contractor must use reasonable care during site preparation and excavation so that the subgrade soils are remained firm, unyielding, and stable.
- Removal of the affected soil that is already wet exposing suitable bearing subgrades and replacing with imported free-draining materials as structural fills that can be compacted.
- Aeration of the surficial materials during favorable dry weather by methods such as scarifying or windrowing repeatedly and expose to sunlight to dry near optimum moisture content prior to placement and compaction
- Chemical modification of the subgrades with admixtures like hydrated lime or Portland cement, depending on the soil type.
- Limiting the size of areas that are stripped of topsoil and left exposed.
- Limiting construction traffic over unprotected soils.
- Sloping excavated surfaces to promote runoff.
- Limiting the size and type of construction equipment used.
- Providing gravel or quarry spalls "working mat" over areas of protected subgrade.
- Removing wet surficial soil prior to commencing fill placement each day.
- Sealing the exposed ground surface by rolling with a smooth drum compactor or rubber-tired roller at the end of each day.

- Providing upgradient perimeter ditches or low earthen berms and using temporary sumps to collect runoff and prevent water from ponding and damaging exposed subgrades.
- Mechanical stabilization with a coarse crushed aggregate (such as sand and gravel, crushed rock, or quarry spalls) compacted into the subgrade, possibly in conjunction with a geotextile fabric, such as Mirafi 500X.
- In the event earthwork takes place during the wet season, we recommend that special precautionary measurements should be adopted to minimize the impact of water and construction activities on the moisture sensitive soils.
- It is recommended that earthwork be progressed part by part in small sections to minimize the soil's exposure to wet weather. Traversing of construction equipment can cause considerable disturbance to the exposed subgrades, therefore, should be restricted within the specific drive areas. This will also prevent excessive widespread disturbance of the subgrades. Construction of a new working surface from an advancing working surface could be used to avoid trafficking the exposed subgrade soils.
- Any excavations or removal of unsuitable soils should be immediately followed by the placement of backfill or concrete in footings.
- At the end of each day, no loose on-site soils and exposed subgrades be left uncompacted or properly tamped, which will help seal the subgrade and thereby to minimize the potential for moisture infiltration into the underlying layers of fills or subgrades.
- In case site filling must proceed during wet weather the contractor should include a contingency in the earthwork budget for the possibility of using imported clean, granular fill. For general structural fill purposes, we recommend that using well-graded sand and gravel, such as 'Ballast' or 'Gravel Borrow' per 2024 WSDOT Standard Specifications 9-03.9(1) and 9-03.14(1), respectively. Alternatively, 'free-draining' soil similar to the one described earlier in the Structure Fill Table may also be considered suitable as filling material for the wet weather construction. This type of fill refers to soils that have a fines content of 5 percent or less (by weight) based on the minus ¾-inch soil fraction.

10.1.10 Subgrade Degradation Prevention

The near surface subgrades may become susceptible to degradation during the wet weather conditions. To protect against subgrade degradation due to construction traffic we recommend a 'working mat' be placed over final prepared subgrades. We recommend this 'working mat' consists of 12 inches thick free draining materials consist of crushed rocks or quarry spalls, possibly in conjunction with a geotextile fabric, such as Mirafi 500X placed underneath the crushed rocks or quarry spalls layer. Construction traffic should be limited to these 'working mat' areas. The stabilization materials can be as per the requirements recommended later on in Section 10.1.6, 'Stabilization Materials'.

10.2 Site Drainage

Surface Drainage

The final site grades of the finished development must be such that surface runoff will flow by gravity away from the building and other structure, such as the pavement and sidewalks, using sloped and drainage gradients towards the local stormwater collection system. We recommend providing a minimum drainage gradient of about 2% for a minimum distance of about 10 feet from the building perimeter. Surface water should not be allowed to pond and soak into the ground surface near buildings or paved areas during or after constructions. A combination of using controlled surface drainage and capping of the building surroundings by concrete, asphalt, or low permeability silty soils will help minimize or preclude surface water infiltration around the perimeter of the building and beneath the garage basement floor slab. Paved areas should be graded to direct runoff to catch basins and or other collection facilities. Collected water should be directed to the on-site drainage facilities by means of properly sized smooth walled PVC pipe. Interceptor ditches or trenches or low earthen berms should be installed along the upgrade perimeters of the site to prevent surface water runoff from precipitation or other sources entering in to the lower area of the lot. It should be noted that surface water runoff from precipitation flows as a sheet flow over slope is considered to be the primary cause of surficial sloughing and triggering slope failure. Therefore, the surface drainage system should be designed in such a way that stormwater runoff over the finished lot must not create any sheet flow over the sloped areas of the site, instead, the stormwater runoff must be collected in drain pipes to discharge in approved discharge points at the toe of the slope. Surface drainage system and the water collection facilities should be designed by a professional civil engineer.

Footing Excavation Drain

Water must not be allowed to pond in the foundation excavations or on prepared subgrades either during or after construction. If due to the rainfall, runoff, seasonal fluctuations, groundwater seepage is encountered within footing depths, we recommend that the bottom of excavation should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff, and then direct the water to ditches, and to collect it in prepared sump pits from which the water can be pumped and discharged into an approved storm drainage system. Water handling needs will typically be lower during the summer and early fall months.

Footing Drain

Footing drains should be used where (1) crawl spaces or basements will be below a structure, (2) a slab below the outside grade, and (3) the outside grade does not slope downward from a building. To reduce the potential for groundwater and surface water to seep into the interior spaces of the building we recommend that an exterior footing drain system be constructed around the perimeter of the building footings as shown in Figure 4 and 5 of this report. The drains must be laid with a gradient sufficient to promote positive flow by gravity to a controlled point of approved discharge. The foundation drains should be tightlined separately from the roof drains to this discharge point. Footing drains should consist of at least 6-inch diameter, heavy-walled, perforated PVC pipe or equivalent. The pipe should be surrounded by at least 6 inches of free-draining gravel over the pipe and 3 inches of free-draining gravel below the pipe. The free-draining material may consist of open-graded drain rocks consisted of ¾" minus washed gravels should be wrapped up by a non-woven geotextile filter fabric (Mirafi 140N) to limit the ingress of fines into the gravel and the pipe. The free-draining material should contain less than 2 percent by weight passing the U.S. Standard No. 200 sieve (based on a wet sieve analysis of that portion passing the U.S. Standard No. 4 sieve). The drains should be located along the outside perimeter of the spread footings or the footing stem walls. Also, the invert of the footing pipe should be placed at approximately the same elevation as the bottom of the footing or 12 inches below the adjacent floor slab grade, whichever is deeper, so that water will not seep through walls or floor slabs. The footing drains should discharge to an approved drain system and include cleanouts to allow periodic maintenance and inspection.

Downspout or Roof Drain

These should be installed once the building roof in place. They should discharge directly in tightlines to a positive, permanent stormwater collection system. Under no circumstances connect these tightlines to the perimeter footing drains. The drain is shown in Figure 4 and 5 of this report.

10.3 Construction Dewatering

As discussed earlier in Section 6.0, very wet soils were noticed at the time of explorations within the exploration depths. Based on this observation, groundwater may be expected at the bottom or within the overexcavation depth, especially if the construction takes place during the wet winter months. If such condition arises then we recommend that temporary measures such as typical sump excavations and sump pumps will be used to de-water the areas for short term work. The construction areas must be maintained in a complete dry condition throughout the excavation and the construction period, and to prevent the excavation face and the foundation subgrades against degradation. In our opinion, dewatering techniques involving some positive elements such as interceptor trenches, collection ditches, and directing water flow to sumps where water can be removed by conventional filtered sump pumps, can be adopted in this regard. Also, surface water from seepage must not be allowed to flow over slope area or pond near the top of the slope, instead the water should be directed away from the slope via ditches or trenches or drain pipes to approved discharge points at the toe of the slope. If the situation seems severe, then more specialized dewatering techniques, such as vacuum wells, well points, etc., may be needed. The contractor and its civil engineer should be responsible for the design and adoption of appropriate dewatering system in this regard. We recommend that to minimize the possibility of any water conditions during the excavation and the construction period, the construction should take place during the dry summer months.

10.4 Temporary Excavations

As we understand from the project plan that the proposed site development is likely to involve some overexcavations for removing the loose soils and for installing the underground utility lines. The overexcavations depths may be approximately 3 feet below the current grades. The inclination of the overexcavation embankment should be made as per the recommendations provided below.

As a general rule, all temporary soil excavations in excess of 4 feet in height and less than 20 feet in depth, the side slopes should be adequately sloped back or properly shored in accordance with Safety Standards for Construction Work Part N, WAS 296-155-657 to prevent sloughing and collapse. As for the current estimation purposes, in our opinion, the side slopes in the native soils (OSHA soil Type C) should be laid back at a minimum slope inclination of 1.5:1 (Horizontal:Vertical), and the side slopes in the native soils (OSHA soil Type B) should be laid back at a minimum slope inclination of 1:1 (Horizontal:Vertical), for up to almost 6 feet depth below the grades from the crest to the toe of the slope. However, estimation of the proper inclination of excavation side slopes should be made on-site after inspecting the soil and groundwater conditions, which will be revealed during the actual construction in the site.

It should be recognized that slopes of the above gradients do ravel and require occasional maintenance. All temporary exposed slopes and excavations should be protected as soon as possible using appropriate methods to prevent erosion to occur during periods of wet weather. This can be achieved by installing a durable reinforced plastic membrane, jute matting, or other erosion control mats with proper anchorage to the ground. In addition, we recommend that experienced personnel of the contractor should regularly check the slope condition to notice if any signs of raveling or sloughing off is underway to prevent any catastrophic slope failure.

All temporary soil cuts greater than 4 feet in height, if cannot be sloped back because of the limited horizontal distance to be available between the top of the excavation line and the property line, a properly shoring system is to be considered to prevent sloughing and collapse of the slope.

Any excavation side inclinations will assume that the ground surface behind the cut slopes is level, that surface loads from equipment and materials are kept a sufficient distance away from the top of the slope. If these assumptions are not valid, we should be contacted for additional recommendations. Flatter slopes may be required if soils are loose or caving and/or water, are encountered along the slope faces. If such conditions occur and the excavation cannot stand by itself, or the excavation slope cannot be flattened because of the space limitations between the excavation line and the boundary of the property, temporary shoring may be considered. The shoring will assist in preventing slopes from failure and provide protection to field personnel during excavation. Because of the diversity available of shoring stems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor engaged to complete the installation. We can assist in designing the shoring system by providing with detailed shoring design parameters including earth-retaining parameters, if required.

Where sloped embankments are used, the top of the slopes should be barricaded to prevent vehicles and storage loads within 10 feet of the top of the slopes. Greater setbacks may be necessary when considering heavy vehicles, such as concrete trucks and cranes. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the top of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. All temporary slopes should be protected from surface water runoff.

The owner and the contractor should be aware that in no case should the excavation slopes be greater than the limits specified in local, state, and federal safety regulations, particularly, the Occupational Safety and Health Administration (OSHA) regulations in the "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P, dated October 31, 1989" of the Federal Register, Volume 54, the United States Department of Labor. As mentioned above, we also recommend that the owner and the contractor should follow the local and state regulations such as WSDOT Section 2-09.3(3) B, Washington Industrial Safety and Health Act (WISHA), Chapter 49.17RCW, and Washington Administrative Code (WAC) Chapter 296-115, Part N. These documents are to better insure the safety of construction worker entering trenches or excavation. It is mandated by these regulations that excavations, whether they are for utility trenches or footings, be constructed in accordance with the guidelines provided in the above documents. We understand that these regulations are being strictly enforced and, if they are not closely followed, both the owner and the contractor could be liable for substantial penalties.

Stability of temporary excavations is a function of many factors including the presence of, and abundance of groundwater and seepage, the type and density of the various soil strata, the depth of excavation, surcharge loadings adjacent to the excavation, and the length of time and weather conditions while the excavation remains open. It is exceedingly difficult under these unknown and variable circumstances to pre-establish a safe and maintenance-free temporary excavation slope angle at this time of the study. We therefore, strongly recommend that all temporary, as well as permanent, cuts and excavations in excess of 4 feet be examined by a representative of PGE during the actual construction to verify that the recommended slope inclinations are appropriate for the actual soil and groundwater seepage conditions exposed in the cuts. If the conditions observed during the actual construction are different than anticipated during this study then, the proper inclination of the excavation and cut slopes or requirements of temporary shoring should be determined depending on the condition of the excavations and the slopes.

The above information is provided solely for the benefit of the owner and other design consultants, and under no circumstances should be construed to imply that PGE assumes responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred. Therefore, the contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures.

We expect that the excavation can be completed using conventional equipments such as bulldozers or backhoes.

10.5 Utility Support and Backfill

Based on the soils encountered at the site within the exploration depths, the majority of the soils appear to be adequate for supporting utility lines; however, softer soils may be encountered at isolated locations, where, it should be removed to a depth that will provide adequate support for the utility. A major concern with utility lines is generally related to the settlement of the trench backfill along utility alignments and pavements. The trench backfill settlement causes misalignment of the utility lines and breaking apart of the joints. Therefore, it is important that each section of utility be adequately supported on proper bedding material and properly backfilled. We recommend that the on-site geotechnical engineer should evaluate the final subgrades of the bottom of the utility trench to verify if the subgrade is competent to support the utility lines and the backfills, or the subgrades need some proofrolling and recompaction, or require overexcavation of unsuitable loose fills and replacement with suitable structural fills.

We recommend that if needed the bottom grades of the utility trench must be adequately proofrolled and compacted to firm and unyielding conditions. A layer of geo-grid such as Mirafi 500X or equivalent should be placed on the proofrolled subgrades prior to placing the bedding materials and laying the utility lines. This should be decided on-site by the geotechnical engineer on-site based on the observed subgrade conditions at the bottom of the trench.

It is recommend that utility trenching, installation, and backfilling conform to all applicable Federal, State, and local regulations such as WISHA and OSHA for open excavations.

Pipe Bedding & Pipe Zone

Trench backfill to be placed beneath, adjacent to, and for at least 2 feet above utilities line should consist of well-graded granular material with a maximum particle size of 1 inch and less than 10 percent by dry weight passing the US Standard No. 200 Sieve, and should meet the standards of ‘Gravel Backfill for Pipe Zone Bedding’ described in Section 9-03.12(3) of the 2024 WSDOT Standard Specifications. Trench backfill must be free of debris, organic material and rock fragments larger than 1 inch. The bedding materials should be hand tamped to ensure support is provided around the pipe haunches. Trench backfill should be carefully placed and hand tamped to about 12 inches above the crown of the pipe before any heavy compaction equipment is brought into use. In order to reduce the potential for damaging the utilities, heavy compaction equipment should not be permitted to operate directly over utilities until a minimum of two (2) feet of backfill will be placed. In general, pipe bedding should be placed in loose lifts not exceeding 6 inches in thickness and compacted to at least 90 percent of the fills’ maximum dry density value as to be determined by the laboratory Modified Proctor (ASTM D1557) test method. The fill materials within the pipe bedding and pipe zone, their thicknesses and compactions should be suitable for the utility system and materials installed, and in accordance with any applicable manufacturers' recommendations or local building department. Pipe bedding materials should be placed on relatively undisturbed native soil. Based on our field explorations, we anticipate relatively coarse-grained soils comprised of poorly graded gravel with cobbles. Some overexcavation and removal of cobbles should be anticipated at the pipe invert elevation to maintain a uniform grade for the utility installation. Where overexcavation is needed, additional pipe bedding materials should be placed to restore the grade.

Trench Backfills

We recommend that the backfills to be placed 2 feet above the pipe and upto the final pavement subgrade level should be consisted of materials similar to ‘Gravel Borrow’ described in Section 9-03.14(1) or ‘Select Borrow’ as described in Section 9-03.14(2), of the 2024 WSDOT Standard Specifications. Where excavations occur in the wet, alternative such as ‘Select Granular Fill’ described earlier in Section 10.1.6, Structural Fills’ should be considered. Trench backfill must be free of roots, debris, organic matter and rock fragments larger than 3 inches. Other materials may be appropriate depending on manufacturer specifications and/or local jurisdiction requirements. For site utilities located within the City of Puyallup right-of-ways, bedding and backfill should be completed in accordance with the city specifications. As a minimum, 5/8 inch pea gravel or clean sand may be used for bedding and backfill materials. The trench backfills to be placed 2 feet above the pipe and upto the final pavement subgrade level should be compacted to 95 percent of the fills' maximum dry density value as to be determined by the laboratory Modified Proctor (ASTM D-1557) test method. The backfill should be placed in lifts not exceeding 4 inches if compacted with hand-operated equipment or 8 inches if compacted with heavy equipment. Catch basins, utility vaults, and other structures installed flush with the pavement should be designed and constructed to transfer wheel loads to the base of the structure.

The utility trenches should not be left open for extended periods to prevent water entry, accumulation, and softening of the subgrade. Should soft soils be encountered at the bottom of the trench, it should be overexcavated and replaced with select fills. As an alternative to undercutting, a Geotextile fabric or crushed rock may be used to stabilize the trench subgrade. Where water is encountered in the trench excavations, it should be removed prior to fill placement. Alternatively, quarry spalls or pea gravel could be used below the water level if allowed by the local authority or the project specifications.

10.6 Foundations Recommendations

Mat Foundation

Mat foundation is essentially a slab-on-grade foundation typically designed using the approximate flexible method known as Winkler Foundation. Foundations designed using this method is also known as Winkler Foundation. For such type of analysis, a modulus of vertical subgrade reaction can be used to design the mat foundation. We recommend a subgrade modulus value of (ks1) of 90 pound per cubic inch (pci). This subgrade modulus is for a 1-ft by 1-ft square plate and is not the overall modulus of the larger footing. The actual modulus for the mat varies based on the footing size according to the following equation:

$$ks = ks1\left[\frac{(B+1)}{2B}\right]^2$$

Where 'ks' is the actual footing modulus; ks1 is the modulus for a 1-ft by 1-ft square plate; and B is the width or lateral dimension of the actual footing.

Deflections will depend on the stiffness of the slab, but we anticipate total deflections under static conditions over the time to be on the order of 1 inch or less with a differential settlement across the structure on the order of ½ inch or less, respectively. Most of these settlements are expected to occur immediately following the building loads are to be applied. The predicted settlement values may be expected larger if soft, loose, organic soil is encountered, or if the foundation subgrade is disturbed and becomes soft during construction. The settlement evaluation was done without the aid of any laboratory consolidation test data, and on the basis of our experience with similar types of structure and subsoil conditions. The liquefaction induced dynamic settlement is anticipated to be in the order of 2 to 3 inches of differential settlement across the mat foundation.

The mat foundation should be thickened at least 18 inches below the adjacent finish grade around the perimeter of the mat, and this thickened edge should have a minimum width of 12, 15, and 18 inches for 1-, 2-, and 3-strory residential structures as presented in the Table 1805.4.2 of 2018 International Building Code (IBC).

Rigid Framed Foundation

The individual interior columns may be supported by isolated shallow conventional spread footings and perimeter walls can be supported by strip footings. These footing elements should be connected or tied up with structural grade beams spanning in between the footing elements to make the different footing elements interlocked and act as a rigid frame structure. The floor slab should be a structural floor slab to be designed for spanning in between the grade beams. The rigid foundation system has the structural integrity that would act as a single frame structure to reduce the potential of total settlement and the differential settlement.

Bearing 'Fill Pad'

As it was discussed earlier in Section 9.0 it is our opinion that the proposed residence can be supported either on a mat foundation or on a rigid frame foundation, which should be supported on at least a 2 feet (24 inches) thick adequately compacted bearing 'fill pad'. The 'fill pad' would reduce the settlement and enhance the allowable bearing capacity to support the structure. As it was mentioned earlier in Section 10.1.3, 'Final Subgrade Preparation', a geo-grid layer such as Mirafi 500X or equivalent should be placed on the exposed final native subgrades prior to placing the 'fill pad'. The combination of the 'fill pad' and the geo-grid layer would assist in reducing the total settlement and the differential settlement of the footing.

The 'fill pad' must be placed on the final native subgrades that are adequately proofrolled and compacted to firm and unyielding conditions prior to placing the 'fill pad' (see Section 10.1.3, 'Final Subgrade Preparation'). The 'fill pad' must be consisted of compacted clean crushed rocks, such as 2- to 4-inch rock spalls or 2-inch ballast rocks be placed below the mat foundation. Standing water should not be allowed to accumulate in the final footing subgrades. All loose or disturbed soils should be removed from the foundation excavation prior to placing the forms and rebar.

The foundation bearing ‘fill pad’ and the geogrid layer need to be extended beyond the actual outer edges of the mat foundation in all directions for a distance equal to the bearing pad thickness of 48 inches.

The final footing subgrade must be at least 3 feet above the static groundwater table encountered at approximately 5.5 feet depth below the grades.

Allowable Bearing Capacity

For the design of the foundation systems to be supported on the bearing ‘fill pad’ we recommend using a maximum net allowable bearing capacity of 1500 psf for the foundation design. The “net allowable bearing pressure” refers to the pressure that can be imposed on the soil at foundation level resulting from the total of all dead loads plus the long-term live loads, exclusive of the weight of the footing or any backfill placed above the footing, i.e., these loadings can be ignored in calculating footing sizes. The recommended bearing pressure should not be increased by 1/3rd when design for seismic conditions.

We recommend that continuous footings have a minimum width of 18 inches and individual column footings a minimum width of 24 inches. All exterior footings should bear at least 18 inches below the final adjacent finish grade to provide adequate confinement of the bearing materials and frost protection.

Lateral Resistance

Lateral loads due to wind and seismic forces transferred to the mat foundation or the rigid framed foundation may be resisted by friction between the foundation base and the bearing soil, and by passive earth pressure acting on the vertical face of the foundation system to be embedded below the adjacent final grades. We recommend using a coefficient of friction of 0.35 to calculate friction between the concrete, and the ‘fill pad’ soils. For passive earth pressure, the available resistance may be determined using an equivalent fluid pressure of 250 pcf, which includes a factor of safety of 1.5. This value assumes the foundations are cast "neat" against the undisturbed native soils or structural fills placed and compacted as recommended in Section 10.1.7, ‘Fill Placement & Compaction’ of this report. We recommend to disregard the upper 12 inches of soil while computing the passive resistance value because this depth can be affected by weather or disturbed by future grading activity. To achieve the adequate passive resistance from the embedded soils as well as for frost and erosion protection, we recommend that all exterior footings must be embedded at least 18 inches below the final adjacent outside grades consisted of either the undisturbed native soils or structural fills placed and compacted as recommended in Section 10.1.7, ‘Fill Placement and Compaction Requirements’ of this report.

Settlement

Based on our settlement potential evaluation of the shallow foundation options, we anticipate that properly designed and constructed foundations supported on the recommended bearing materials should experience total settlement of less than 1 inch for the allowable bearing pressures presented above. Differential settlement could be on the order of ¼ to ½ inch between similarly loaded foundations over a distance of 50 feet of continuous footings. This estimation was done without the aid of any laboratory consolidation test data, but on the basis of our experience with similar types of structures and subsoil conditions. The soil response to applied stresses caused by building and other loads is expected to be predominantly elastic in nature with most of the settlements occurring during construction as loads are applied; however, due the fines content of the site soils, the estimated settlements could occur over a longer time, and disturbance of the foundation subgrades during construction could result in larger settlements than predicted.

Footing Subgrade Inspection

We recommend that PGE representative examine the bearing materials prior to placing forms or rebar. Variations in the quality and strength of the potential bearing soils can occur with depth and distance away from the test pits. Therefore, a careful evaluation of the bearing material and the design bearing capacity value as recommended in this report must be verified at the proposed footing locations at the time of footing construction. We recommend that a PGE representative examine the bearing materials prior to placing forms or rebar.

10.7 Structural Floor Slab

The floor slab should be a structural floor slab for the rigid framed foundation option, spanning in between the grade beams. The slab can be placed directly on a capillary break layer to retard the upward wicking of ground moisture beneath the floor slab to prevent the migration of the water vapor in the upward direction through the fill pad and the concrete slab to the space above the slab. In absence of a capillary break layer the water vapor transmitted to the slab and space above the slab may damage and deteriorate the slab, and moisturized the room above the slab, respectively. A suitable vapor barrier or moisture barrier is to be placed over the capillary break layer. This layer is needed especially when the vapor transmission through the floor slab is undesirable for the slab to be covered with tile, wood, carpet, impermeable floor coverings, or to be supporting any moisture-sensitive equipment or products.

The capillary break layer would consist of a minimum of 6-inch thick clean, 5/8-inch crushed rock with less than 2 to 3 percent fines by weight of the material passing #200 sieve. Alternatively, 'Gravel Backfill for Drains' per WSDOT Standard Specifications 9-03.12(4) can be used as capillary break materials. The vapor barrier must be a durable vapor barrier or retarder such as a layer of 6-mil (0.006 inch) plastic membrane or sheeting (such as Crossstuff, Moistop, or Visqueen) should be laid over the capillary break layer to prevent the upward migration of ground moisture vapors through the slab. The vapor barrier should be placed with seams being taped and overlapped by a minimum of one foot. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when tested in accordance with ASTM E-36. During the casting of the slab, care should be taken to avoid puncturing the vapor barrier. At owner's or architecture's discretion, the membrane may be covered with 2 inches of clean, moist sand as a 'curing course' to guard the vapor barrier against damage during construction and to facilitate uniform curing of the overlying concrete slab. The addition of 2 inches of sand over the vapor barrier is a non-structural recommendation. For heated areas, an additional layer of Styrofoam may be placed between the slab and the capillary break layer for provision of better insulation. A typical sectional view of the slab with the above features is provided in Figure 5 of this report. All intrusions through the slab should be adequately sealed.

10.8 Construction Monitoring

Problems associated with earthwork and construction can be avoided or corrected during the progress of the construction if proper inspection and testing services are provided. Since this project involves so many aspects of geotechnical engineering related construction activities such as stripping of vegetations, removals of topsoils, the identification of loose soils and their removals, overexcavations, final native subgrades preparation and proofrolling, fill placement and compaction of fills, slab-on-grade-floor installation, footing embedment depth, and verification of the allowable bearing capacity value under the mat foundation, mat foundation construction, site drainage, we recommend that PGE's on-site geotechnical engineer should inspect all the above activities. A list of inspection items are provided in the following section, 'Geotechnical Special Inspection' of this report. It is recommended that the above construction activities be monitored by a representative from our firm since we have the prior knowledge, familiarity, and better understanding with our recommendations.

10.9 Geotechnical Special Inspection

The construction of the proposed development in this site involves several aspects of the geotechnical engineering that are considered to be critical for the successful completion of the project and continue that throughout the project life. Therefore, PGE recommends that the following geotechnical special inspection services to be performed during the construction of the proposed development. According to PGE, the following items should be considered as a minimum but not limited to.

- A professional geotechnical engineer should be retained to provide geotechnical consultation, material testing, and construction monitoring services during the construction of the project.

- A pre-construction meeting should be held on-site to discuss the geotechnical aspects of the development and the special inspection services to be performed during the construction.
- The site preparation activities including but not limited to stripping, cut and filling, final subgrade preparation for foundation, floor slab, paved driveway, and retaining wall be monitored by a geotechnical engineer or his representative under the engineer's supervision.
- A list of the possible items that require special geotechnical inspection and approval by the geotechnical engineer is as follows:

➢ Stripping of topsoils.
➢ Removal of loose, native soils, and the uncontrolled, existing fills.
➢ Compaction and proofrolling of any exposed native subgrades that are intended to provide direct support for any load bearing structure such as new fill pad, slab-on-grade floor, footing, retaining wall, and paved driveway.
➢ Any structural fills to be used in this site, and structural fills placement and its compaction.
➢ Temporary or permanent excavation inclinations, and excavation stability.
➢ Backfilling and its compaction, and drainage behind retaining walls.
➢ The footing bearing materials, bearing capacity value, and the embedment depth of the footings prior to placing forms and rebars.
➢ Subgrade preparation for soil supported slab-on-grade floors.
➢ Subgrade preparation for paved driveways.
➢ Compaction of CSBC, CSTC, and laying of concrete pavement in driveway.
➢ Site drainage.
➢ Installation of drainage system such as footing excavation drain and footing drain, and daylighting of such drains and downspout or roof drains.
➢ Bedding and the backfilling materials, and backfilling of utility lines.
➢ Performing field verification percolation test at the proposed drywell location.
➢ Observing the construction of drywell system.
➢ Any other items specified in the approved project plans to be prepared by other consultants relevant to the geotechnical aspect of the project.

11.0 Infiltration Potential Evaluation

As a part of the scope of this geotechnical study the permeability characteristic of the native soil was evaluated to assess the feasibility of using a below grade infiltration system in this site for managing the stormwater runoff from the proposed new residence.

To achieve this, the surface and subsurface conditions in the site was observed as a basis for assessing a site-specific infiltration system in the subject site. Specifically the scope of services includes the following:

- Considering performing a small-scale pilot infiltration test (PIT) in test pit, if applicable, based on the presence of the following geological features such as the glacial till, silt or clay deposit (considered as 'restrictive layer'), heavy mottling signs, groundwater table, perched water seepage;
- Exploring surface and subsurface conditions by reconnoitering the site and monitoring the excavation of one test pit at the site;
- Describing surface and subsurface conditions, including soil type, and depths to any 'restrictive layer' (like glacial till) and groundwater, if encountered;
- Providing our opinion about the feasibility of on-site infiltration including a design infiltration rate based on the measured infiltration rate from our in-situ infiltration testing;

As was found in the test pits, the presence of bluish-gray silt deposit in cohesive chunks at shallow depth (5.5 feet below grade) containing higher percentage of fines (~ 79.8%) was considered a hydraulically 'restrictive layer' for a below-grade infiltration system in this site. In addition, the following conditions such as the presence of groundwater at the above depth and mottling signs in the silt deposit were also considered in this regard. Considering the above geological conditions PGE's on-site geotechnical engineer found it unrealistic to consider a below grade infiltration system in this site hence decided not to perform PILOT test in the soil. Based on the combination of the above geological factors we assume that there may be problems for proper functioning of a below-grade infiltration system. In our opinion, the presence of silt deposit and groundwater at shallow depth may pose difficulty to adopt an adequate vertical separation distance below the bottom of an infiltration system and the above restrictive deposit. Therefore, in our opinion, the subject site is not considered suitable for a below grade infiltration system.

12.0 Additional Services

Additional services described below can be performed by PGE in the event the project requires such services. These services will be performed upon written authorization of the client or the civil engineer, and with additional cost to perform such services, under a separate contract between PGE and the client.

12.1 Design Phase Engineering Services

As the geotechnical engineer of record for the proposed development, at owner's option, PGE can provide additional geotechnical recommendations if it is to be needed to provide aide to the project design team.

12.2 Final Plan Review Service

As the geotechnical engineer of record for the proposed development, at owner's option, PGE can perform a review of the geotechnical aspect of the final project plans and specifications to verify that the geotechnical recommendations provided in this report have been properly interpreted and incorporated into the project final design and specifications. PGE's review of the final plan would allow re-evaluating the geotechnical recommendations provided in this report, and if necessary, modifying the recommendations before the construction begins. We believe this would be helpful for the project's speedy completion and success.

12.3 Construction-time Testing and Inspection

As the geotechnical engineer of record for the proposed development, at owner's option, PGE can provide geotechnical consultation, material testing, and construction monitoring services during the construction of the project recommended earlier in Section 10.8, 'Construction Monitoring' of this report. These services are important for the project to confirm that the earthwork and the general site development are in compliance with the general intent of design concepts, specifications, and the geotechnical recommendations presented in this report. Also, participation of PGE during the construction will help PGE engineers to make on-site engineering decisions in the event that any variations in subsurface conditions are encountered or any revisions in design and plan are made.

PGE can assist the owner before construction begins to develop an appropriate monitoring and testing plan to aid in accomplishing a fast and cost-effective construction process.

13.0 Report Limitations

The conclusions and recommendations presented in this report are based on our soil investigation, the laboratory test results, and our engineering evaluation. The study was performed using a mutually agreed-upon scope of work between PGE and the client.

It should be noted that PGE cannot take the responsibility regarding the accuracy of the information provided in the project plan prepared by other consultants. If any of the information considered during this study is not correct or if there are any revisions to the plans for this project, PGE should be notified immediately of such information and the revisions so that necessary amendment of our geotechnical recommendations can be made. If such information and revisions are not notified to PGE, no responsibility should be implied on PGE for the impact of such information and the revisions on the project. Such revision work and amendment of the geotechnical recommendations and conclusions would be additional work beyond the current scope of work for this study.

Variations in subsurface (soil and groundwater) conditions may reveal during the construction of the proposed below grade infiltration system. The nature and the extent of the subsurface variations may not be evident until construction occurs. If any subsurface conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations if there are any changes in the project scope. Such revision work and necessary amendment of the geotechnical recommendations and conclusions would be additional work beyond the current scope of work for this study.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or others factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PGE should be notified if the project is delayed by more than 24 months from the date of this report so that we may review to determine that the conclusions and recommendations of this report remain applicable to the changed conditions.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' method, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances.

This report including its evaluation, conclusions, specifications, recommendations, or professional advice has been prepared for planning and design purposes for specific application to the proposed project in accordance with the generally accepted standards of local practice at the time this report was written. No warranty, express or implied, is made.

This report is the property of our client DS Custom Construction and has been prepared for the exclusive use of our client and its authorized representatives for the specific application to the proposed development at the subject site in Puyallup, Washington.

It is the client's responsibility to see that all parties to this project, including the civil engineer, designer, contractor, subcontractor, future homeowner, etc., are made aware of this report in its entirety. We recommend that the information contained in this report, in its entirety, should be included in the bidding documents and project contract documents at the owner’s or contractor's option. Any party other than the client who wishes to use this report shall notify PGE of such intended use and for permission to copy this report. Based on the intended use of the report, PGE may require that additional work be performed and that and updated report be reissued. Noncompliance with any of these requirements will release PGE from any liability resulting from the use this report.

Closure

We trust the information presented in this report is sufficient for your current needs. We appreciate the opportunity to provide the geotechnical services at this phase of the project and look forward to continued participation during the design and construction phase of this project. Should you have any questions or concerns, which have not been addressed, or if we may be of additional assistance, please do not hesitate to call us at 425-218-9316 or 425-643-2616.

Respectfully submitted,

Santanu Mowar, MSCE, P.E.

PGE Pacific Geo Engineering
Geotechnical Engineering, Consulting & Inspection

D:\Geotechnical\2025-proj\25-819

Attachments:	Figure 1	Vicinity Map
	Figure 2	Site Plan & Exploration Plan
	Figure 3	WA State DNR Map - USGS Geologic Unit
	Figure 4	Mat Foundation & Fill Pad
	Figure 5	Rigid Framed Foundation & Fill Pad
	Appendix A	Soil Test Pit Log, Figure A-1, A-2, and A-3
	Appendix B	Laboratory Test Report, Figure B-1

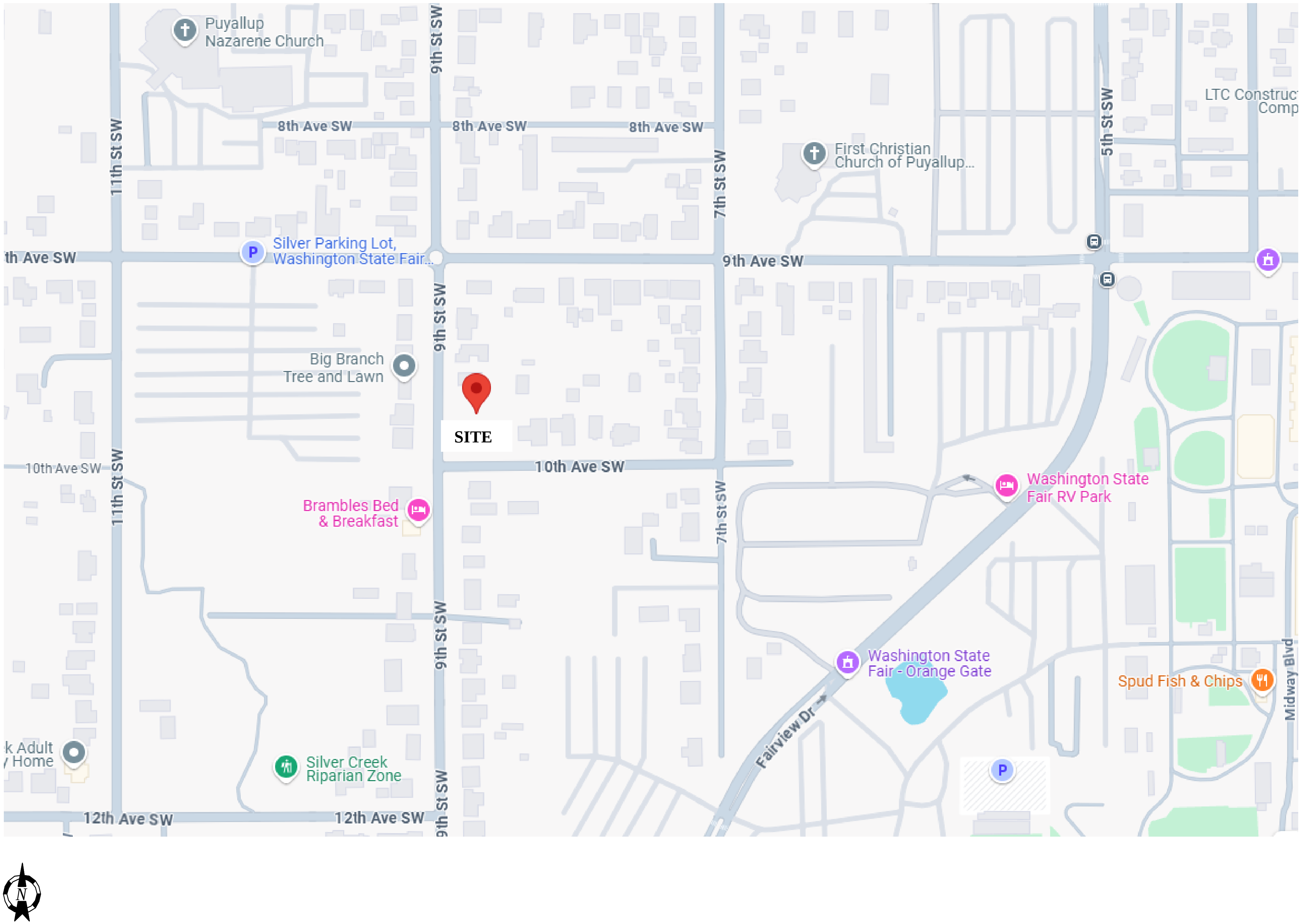


Figure 1 – Vicinity Map

921 9TH ST SW PUYALLUP

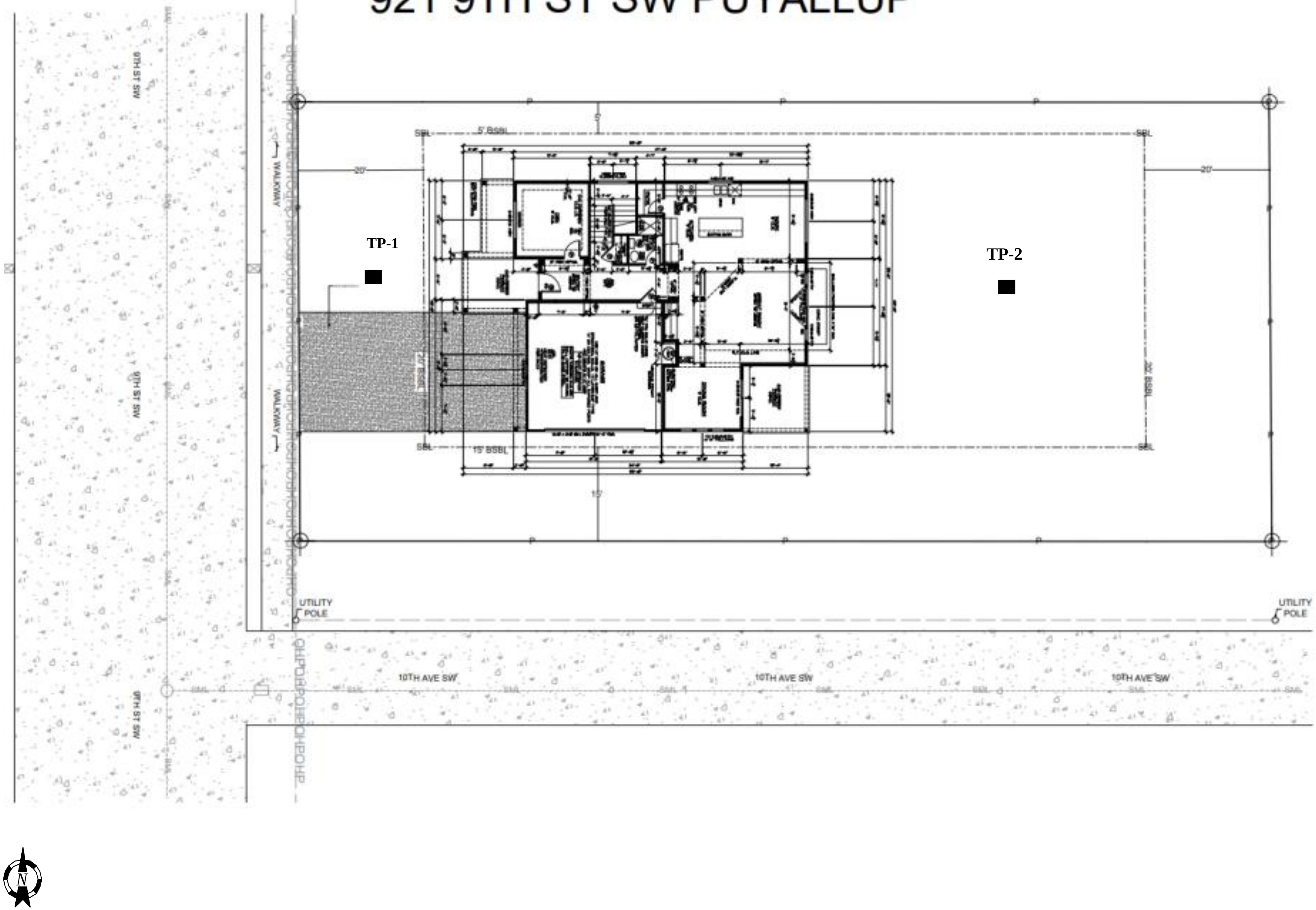


Figure 2 – Site Plan

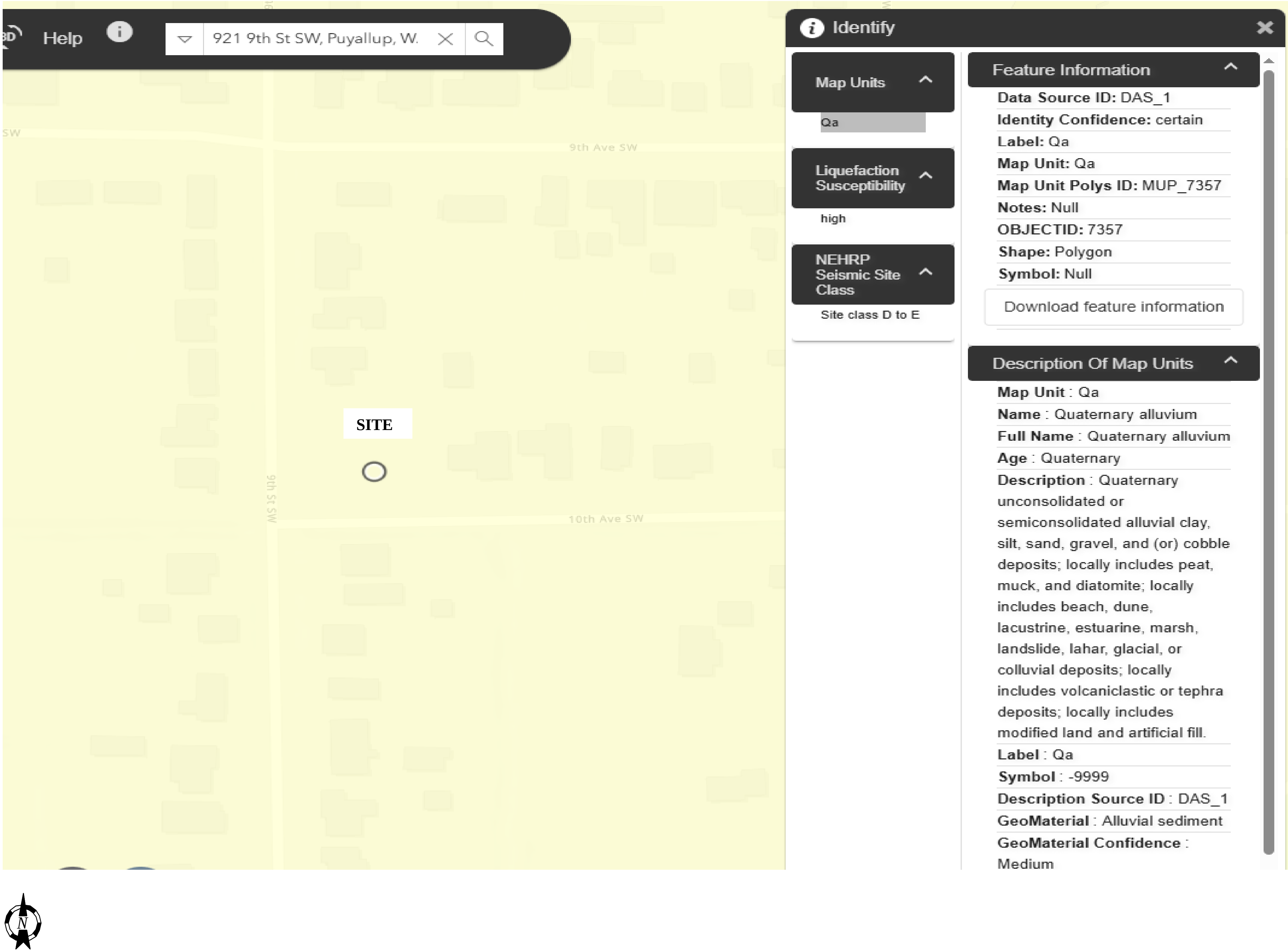
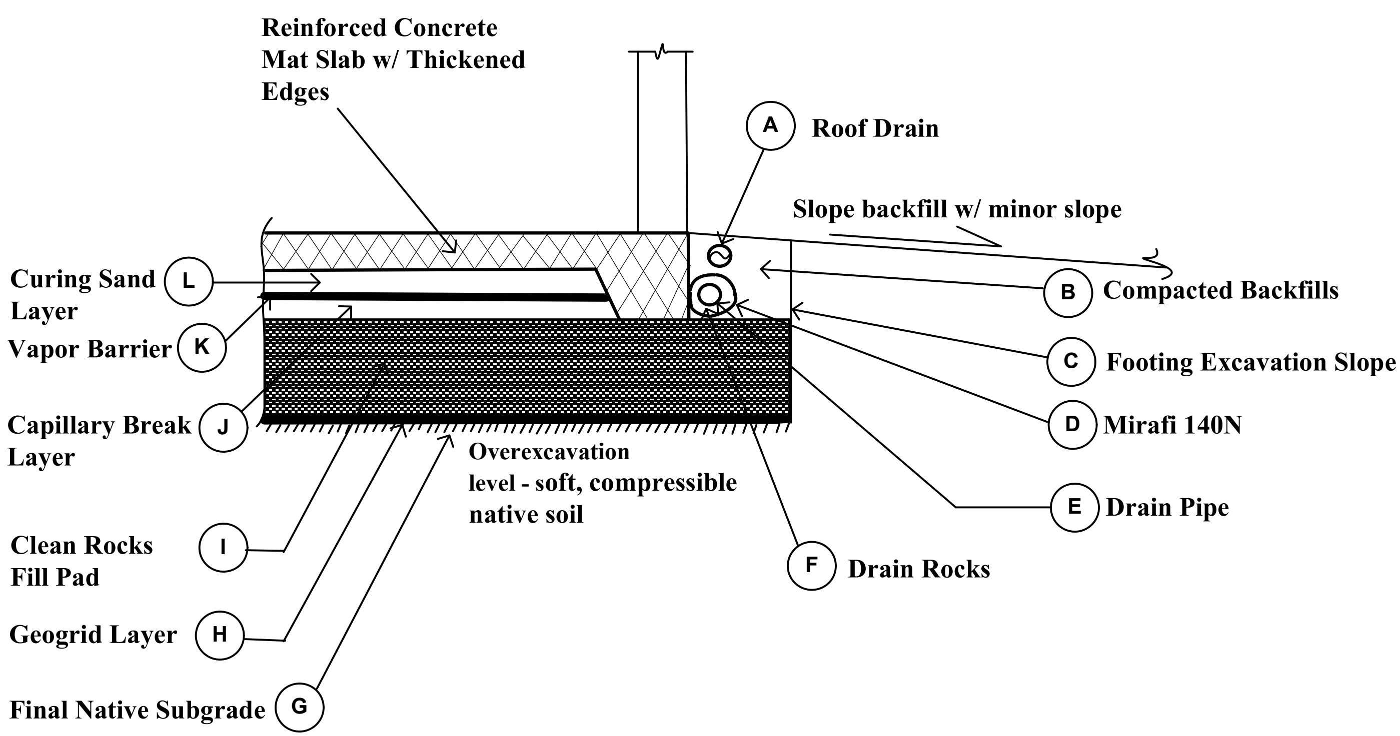


Figure 3 – WA State DNR USGS Map

MAT FOUNDATION

Schematic Only
(Not a construction drawing)



NOTES:-

A **<u>Stormwater roof drain,</u> must be tightlined and must not be connected to footing drain. Pipe should be sloped towards approved discharge point so that no backflow should occur into the pipe.**

B **<u>Backfills</u> in the void areas between the excavation face & the footing must be placed and compacted to 95% of fills' Mod. Proc. Max. dry density value as per ASTM D 1557. Fills must contain no organic & other deleterious materials. Fills must be placed in 6 inch thk individually compacted lifts w/a walk-behind regular plate compactor. No hoe-pack should be used because of the proximity of the footing wall**

C **<u>Excavation face</u> slope to be determined based on actual soil and groundwater conditions to be revealed during the construction**

D **<u>Non-woven Geotextile Filter Fabric</u> - Mirafi 140 N must wrap around the drain rocks**

E **<u>6" dia. rigid PVC pipe</u> w/perforations (1/4" max. dia.) to be in the lower half of pipe, & lower quadrant segment un-perforated to facilitate flow of water. The pipe must be placed as low as possible (at least 6 inches below slab or crawl space. Pipe should be sloped towards approved discharge point so that no backflow should occur into the pipe.**

F **The pipe must be enveloped by <u>drain rocks</u> consisted of ¾" minus washed gravel (free draining). 6" min. gravel on top & sides & 3" min. gravel at bottom of pipe**

G **<u>Final subgrade</u> must be adequately proofrolled to firm & unyielding conditions, must be verified on-site by geotechnical engineer prior to the placing rock layer**

H **A layer of <u>geo-grid</u> such as Mirafi 500X or equivalent on the final native subgrade prior to placing the rock 'fill pad'**

I **<u>Fill Pad</u> of 24" thk. minimum, consisted of clean rocks, such as 2- to 4-inch rock spalls or 2-inch ballast rocks, which must be extended 24" beyond all sides of the mat foundation. Fill pad should be wrapped up with Mirafi 140N fabric and the fabrics should be overlapped on each other for 12 inch width when to be laid side by side**

J **<u>Capillary Break layer</u> – min. 4" thk, of free-draining 5/8-inch crushed rocks containing no more than 2% fines. Slab-on-grade floor should be placed directly on a capillary break layer in unheated areas e.g., garage, storage rooms**

K **<u>Vapor Barrier</u> – a durable 6-mil. (0.006 inch) plastic membrane be placed over capillary break layer as a vapor retarder**

L **<u>Curing Sand Layer</u> - as an additional layer can be placed above the vapor barrier or plastic membrane to guard the membrane against damage during construction and to facilitate uniform curing of the overlying concrete slab.**

Figure 4	**Not to Scale**

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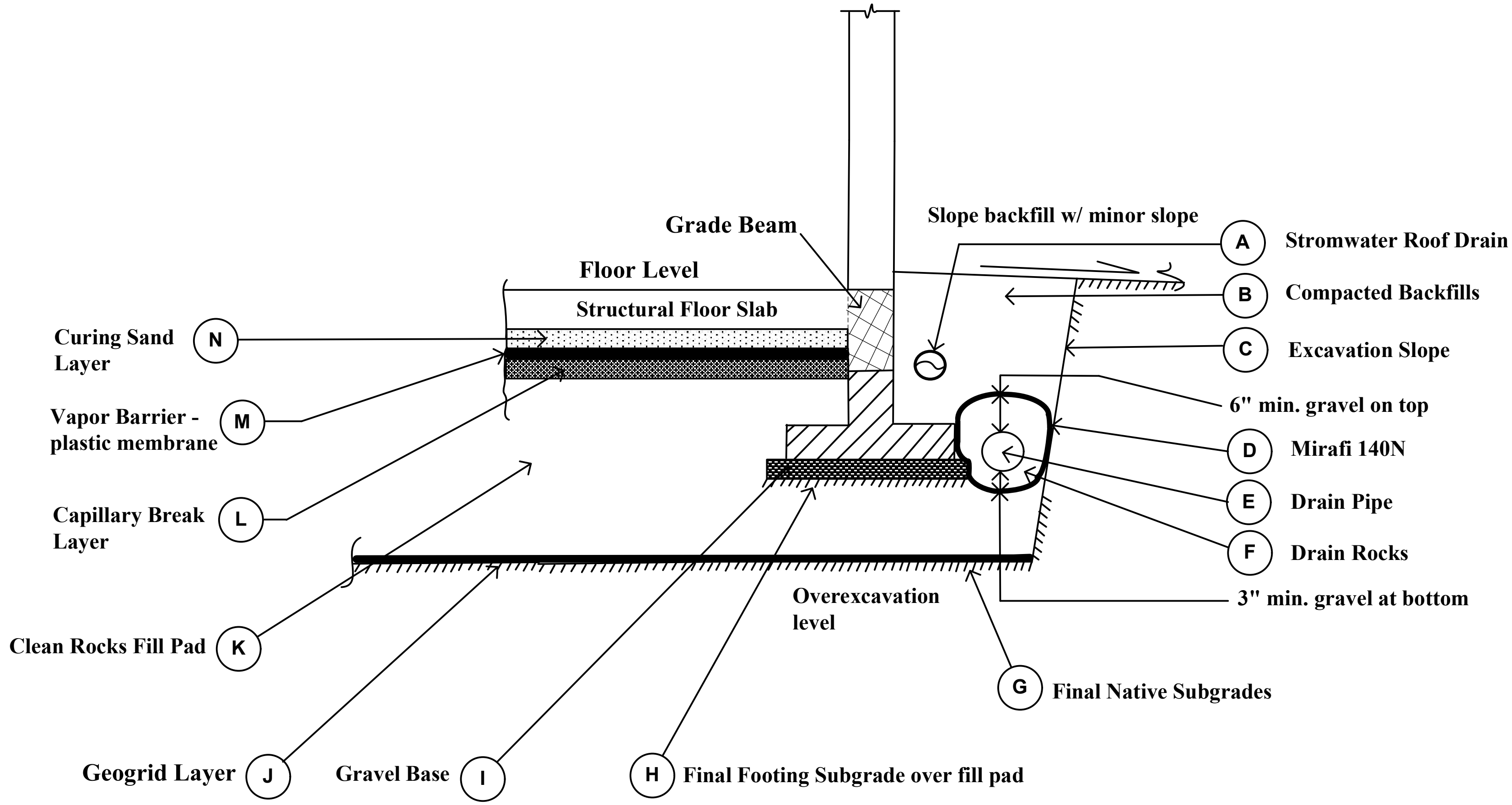
RIGID FRAMED FOUNDATION

(Conceptual drawing, not a construction drawing)



NOTES:- Refer Figure 6 for recommendations provided for different features circled above.

Figure 5	Not to Scale

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RIGID FRAMED FOUNDATION

NOTES:-

A **Stormwater roof drain,** must be tightlined and must not be connected to footing drain. Pipe should be sloped towards approved discharge point so that no backflow should occur into the pipe.

B **Backfill compaction** - void areas on both sides of the footing wall to be created due to the excavation for the footing wall & footing constructions, and also below the floor slab must be backfilled with approved new structural fills, which must be compacted to 95% of fills' max. dry density value (to be determined as per the laboratory Mod. Proctor Test ASTM D1557). The fills to be placed should be compacted with care within the horizontal distance equal to the height of the wall to avoid over compaction and overstressing the footing wall & the footing. No heavy compaction equipment such as vibratory roller or hoe-pac be used to compact the fills because of these equipment will impose excess surcharge loading on the wall, which may cause a lateral instability to the wall. Fills must be placed in 6 inch thick loose lifts close to the footing & the footing wall with a walk-behind regular plate compactor. Fills should be placed in 12 inch thick loose lifts if a walk-behind big heavy-duty vibratory plate compactor is used. The new structural fills should be as per the geotech report.

C **Excavation face** slope to be determined based on actual soil and groundwater conditions to be revealed during the construction.

D **Non-woven Geotextile Filter Fabric** - Mirafi 140 N must wrap around the drain rocks to prevent migration of fines into the drain rocks.

E 6" dia. minimum diameter, perforated or slotted rigid concrete, metal, or plastic pipe with tight plastic joints, with a positive gradient (~2%) sufficient to generate gravity flow w/out backflow to occur into the pipe and provided with accessible cleanouts at regular intervals. The pipe must be taken to final discharge point (approved). The pipe must be placed as low as possible, at least 6 inches below footing or crawl space. Perforations (¼" diameter) to be in lower half of pipe, with lower quadrant segment un-perforated to facilitate water flow. Slotted pipe to have 1/8" maximum width slots. Must NOT be tied to roof downspout or perimeter footing grain lines.

F **Drain rocks** - the drain pipe must be enveloped ¾" minus washed gravel (free draining).

G **Final native subgrades** underneath the new 'fill pad' must be thoroughly redensified and then adequately proofrolled to firm & unyielding conditions prior to placing the new fills.

H **Final footing subgrades** to be consisted of new adequately compacted structural fills must be adequately proofrolled to firm & unyielding conditions. the allowable bearing capacity of 1500 psf to be verified on-site by the geotechnical engineer @ final footing subgrades level over the new fills, prior to placing rebars and forms.

I **Gravel base** - Min. 6" thk compacted (95% or more), which must be extended 6" beyond both sides of the footing.

J A layer of **geo-grid** such as Mirafi 500X or equivalent on the final native subgrade prior to placing the clean rock 'fill pad' .

K **Fill Pad** of 24" thk. minimum, consisted of clean rocks, such as 2- to 4-inch rock spalls or 2-inch ballast rocks, which must be extended 24" beyond all sides of the perimeter strip footings or the footprint area of the building. Fill pad should be wrapped up with Mirafi 140N fabric and the fabrics should be overlapped on each other for 12 inch width when to be laid side by side.

L **Capillary Break layer** – min. 4" thk, of free-draining 5/8-inch crushed rocks containing no more than 2% fines or pea-gravel. Slab-on-grade floor should be placed directly on a capillary break layer in unheated areas e.g., garage, storage rooms.

M **Vapor Barrier** – a durable 10 to 15-mil. plastic membrane be placed over capillary break layer as a vapor retarder.

N **Curing Sand Layer** - as an additional layer can be placed above the vapor barrier or plastic membrane to guard the membrane against damage during construction and to facilitate uniform curing of the overlying concrete slab.

Figure 6 **Not to Scale**

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Appendix A

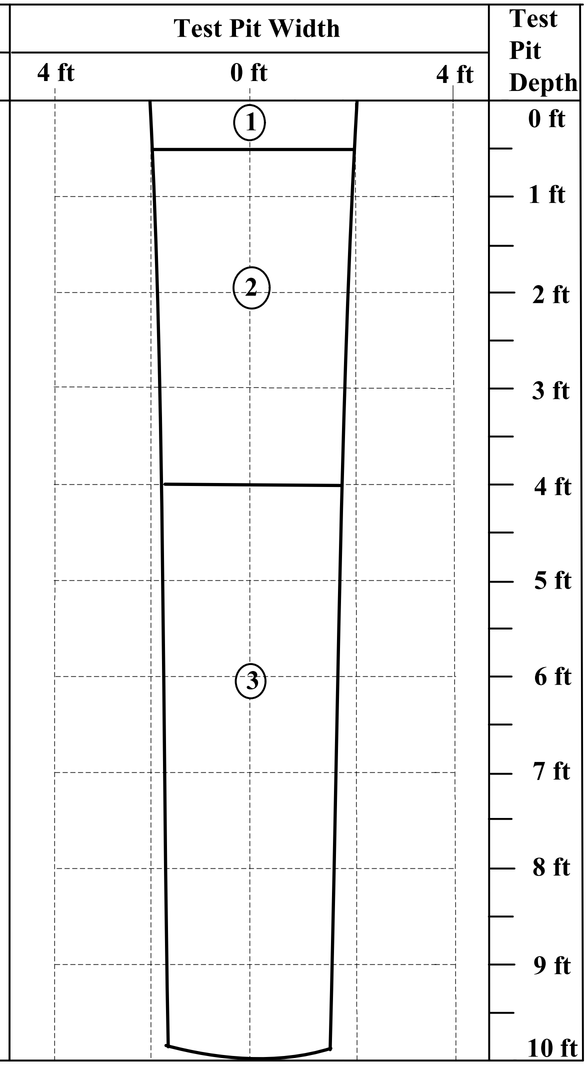
Soil Test Pit Log

TEST PIT – 1

Date of Excavation	04/22/2025

Surface Elev. Ft.	

Soil Layer Depth	Soil Layer Descriptions	USCS Soil Class	Sample Nos.	Sample Depth	Laboratory Test Results: Moist. Content	Laboratory Test Results: - #200 Sieve
0 – 0.5 ft	1 Top Soil – Blk., Silt w/ heavy organics & heavy root zone Wet, Soft					
0.5 ft – 5.5 ft	2 Brn. Sandy Silt Moist, Loose up to 3 ft then Med. Dense	SM	S-1	@ 2 ft	16.8 %	
5.5 ft – 10 ft	3 Bluish Gray, Silt w/ Sand Wet (visible water sheen), Med. Dense	ML	S-2	@ 8 ft	65.5 %	79.8 % (Sieve Test Graph B-1)



Visual-Manual Soil Identification by ASTM D2488-17	Field Logging by ASTM D5434-12	Soil Sampling by ASTM D-75-19

Notes -

Test Pit Location	See site plan	Mottling Depth	Scattered mottling signs @ approx. 5 ft below grade
Ground Cover	Grass	Water/Seepage Depth	Groundwater @ 5.5 ft below grade
Test Pit Depth	10 ft	Cave in Depth	Severe cave-in below water level
Permeability			

Figure A-1 Not to Scale

Project – DS Custom Construction SFR
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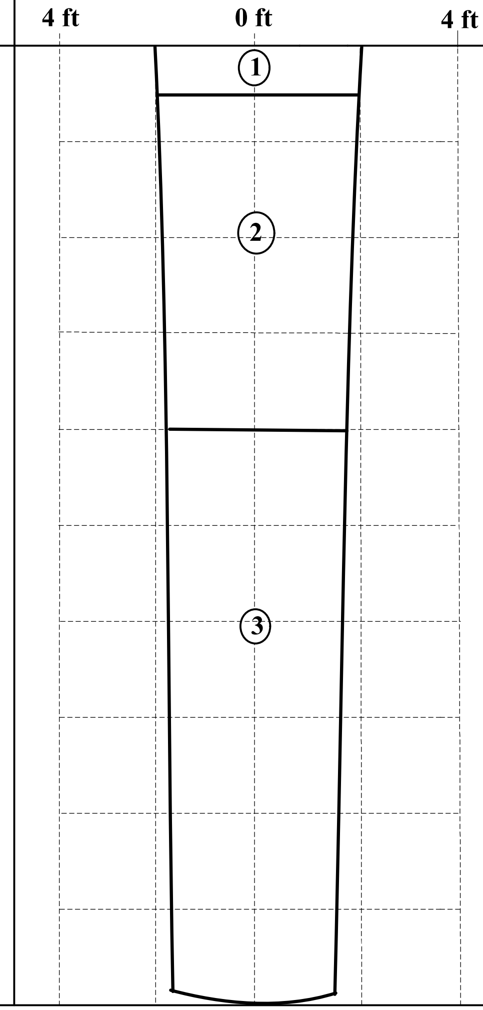
TEST PIT – 2

Date of Excavation	04/22/2025

Surface Elev. Ft.	

Soil Layer Depth	Soil Layer Descriptions	USCS Soil Class	Sample Nos.	Sample Depth	Laboratory Test Results: Moist. Content	Laboratory Test Results: - #200 Sieve
0 – 0.5 ft	(1) Top Soil – Blk., Silt w/ heavy organics & heavy root zone Wet, Soft					
0.5 ft – 5.5 ft	(2) Brn. Sandy Silt Moist, Loose up to 3 ft then Med. Dense	SM				
5.5 ft – 10 ft	(3) Bluish Gray, Silt w/ Sand Wet (visible water sheen), Med. Dense	ML				

Test Pit Width



Test Pit Depth: 0 ft, 1 ft, 2 ft, 3 ft, 4 ft, 5 ft, 6 ft, 7 ft, 8 ft, 9 ft, 10 ft

Visual-Manual Soil Identification by ASTM D2488-17	Field Logging by ASTM D5434-12	Soil Sampling by ASTM D-75-19

Notes -

Test Pit Location	See site plan	Mottling Depth	Scattered mottling signs @ approx. 5 ft below grade
Ground Cover	Grass	Water/Seepage Depth	Groundwater @ 5.5 ft below grade
Test Pit Depth	10 ft	Cave in Depth	Severe cave-in below water level
Permeability			

Figure A-2 Not to Scale

Project – DS Custom Construction SFR
921, 9th St. SW, Puyallup, WA 98371

Project No. – 25-819

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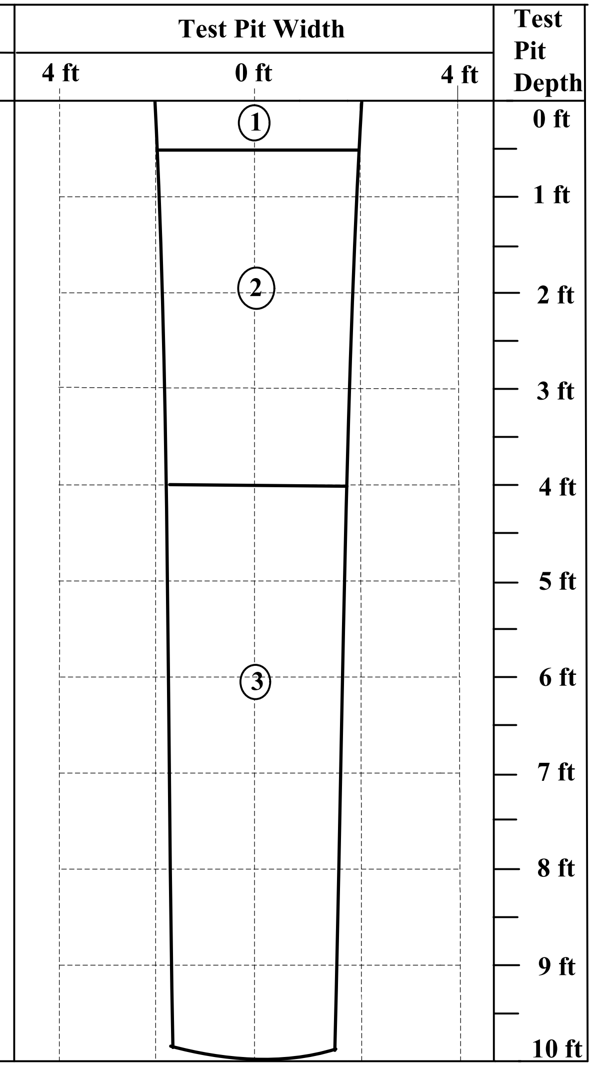
TEST PIT – 3

Date of Excavation	04/22/2025

Surface Elev. Ft.	

Soil Layer Depth	Soil Layer Descriptions	USCS Soil Class	Sample Nos.	Sample Depth	Laboratory Test Results: Moist. Content	Laboratory Test Results: - #200 Sieve
0 – 0.5 ft	1 Top Soil – Blk., Silt w/ heavy organics & heavy root zone Wet, Soft					
0.5 ft – 5.5 ft	2 Brn. Sandy Silt Moist, Loose up to 3 ft then Med. Dense	SM				
5.5 ft – 10 ft	3 Bluish Gray, Silt w/ Sand Wet (visible water sheen), Med. Dense	ML				



Visual-Manual Soil Identification by ASTM D2488-17	Field Logging by ASTM D5434-12	Soil Sampling by ASTM D-75-19

Notes -

Test Pit Location	See site plan	**Mottling Depth**	Scattered mottling signs @ approx. 5 ft below grade
Ground Cover	Grass	**Water/Seepage Depth**	Groundwater @ 5.5 ft below grade
Test Pit Depth	10 ft	**Cave in Depth**	Severe cave-in below water level
Permeability			

Figure A-3 Not to Scale

Project – DS Custom Construction SFR
921, 9th St. SW, Puyallup, WA 98371

Project No. – 25-819

PGE Pacific Geo Engineering

KEY TO EXPLORATION LOGS

Sample Descriptions:

Classification of soils in this report is based on visual field and laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates, and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual classification methods in accordance with ASTM D-2488-17 were used as an identification guide. Where laboratory data available, soil classifications are in general accordance with ASTM D2487-17. Soil density/consistency in borings is related primarily to the Standard Penetration Resistance values. Soil density/consistency in test pits is estimated based on visual observations of excavations. Undrained shear strength = ½ unconfined compression strength.

RELATIVE DENSITY OR CONSITENCY VS. SPT N-VALUE					
COARSE GRAINED SOILS: SAND OR GRAVEL			FINE GRAINED SOILS: SILT OR CLAY		
Density	N (Blows/ft.)	Approx. Relative Density (%)	Consistency	N (Blows/ft.)	Approx. Undrained Shear Strength (psf)
Very Loose	0 – 4	0- 15	Very Soft	0 – 2	<250
Loose	4 – 10	15 – 35	Soft	2 – 4	250 –500
Medium Dense	10 – 30	35 – 65	Medium Stiff	4 – 8	500 – 1000
Dense	30 – 50	65 – 85	Stiff	8 – 15	1000 – 2000
Very Dense	>50	85 – 100	Very Stiff Hard	15 – 30 > 50	2000 – 4000 > 4000

MOISTURE CONTENT DEFINITIONS	
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

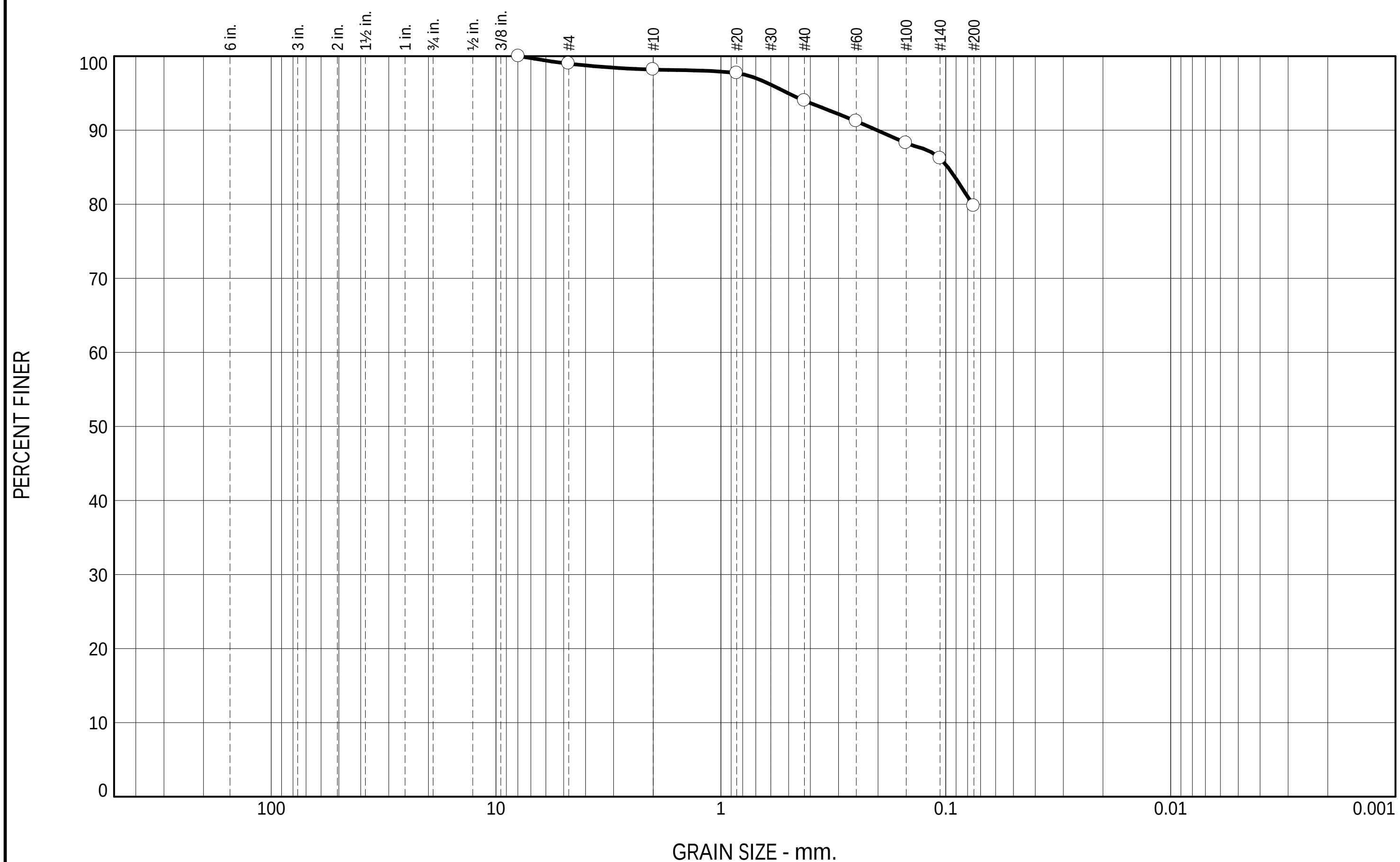
DESCRIPTIONS FOR SOIL STRATA AND STRUCTURE					
General Thickness or Spacing		**Structure**		**General Attitude**	
Parting	< 1/16 in	Pocket	Erratic, discontinuous deposit of limited extent	Near Horizontal	0 - 10 deg
Seam	1/16 - 1/2 in	Lens	Lenticular deposit	Low Angle	10 - 45 deg
Layer	½ - 12 in	Varved	Alternating seams of silt and clay	High Angle	45 - 80 deg
Stratum	> 12 in	Laminated	Alternating seams	Near Vertical	80 - 90 deg
Scattered	< 1 per ft	Interbedded	Alternating Layers		
Numerous	> 1 per ft	Fractured	Breaks easily along definite fractured planes		
		Slickensided	Polished, glossy, fractured planes		
		Blocky, Diced	Breaks easily into small angular lumps		
		Sheared	Disturbed texture, mix of strengths		
		Homogeneous	Same color and appearance throughout		

Appendix B

Laboratory Test Report

Particle Size Distribution Report

ASTM D6913



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	1.0	0.8	4.2	14.2	79.8	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	0.0977							

Material Description	Test Date	USCS	NM
○ Bluish Gray, Silt w/ Sand	04-25-2025	ML	

Project No. 25-819 Client: DS Custom Construction

Project: Hout Residence

921, 9th St SW, Puyallup, WA 98371

○ Location: TP-1 Depth: 8 ft below grade Sample Number: S-2

Remarks:

Pacific Geo Engineering, LLC

Geotechnical Engineering, Consultation, Testing, & Inspection

Figure B-1

Tested By: Sraboni Checked By: Santanu Mowar, PE

PGE Pacific Geo Engineering LLC

Geotechnical Engineering, Consulting & Inspection

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel[F, G, H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[E]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G, H, I]
			Fines classify as CL or CH	SC	Clayey sand[G, H, I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K, L, M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K, L, M]
		organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$	OL	Organic clay[K, L, M, N] / Organic silt[K, L, M, O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K, L, M]
			PI plots below "A" line	MH	Elastic silt[K, L, M]
		organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$	OH	Organic clay[K, L, M, P] / Organic silt[K, L, M, Q]
Highly organic soils		Primarily organic matter, dark in color, and organic odor		PT	Peat

[A]Based on the material passing the 3-in. (75-mm) sieve.

[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C]Gravels with 5 to 12% fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay

[D]Sands with 5 to 12% fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

[E]$Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F]If soil contains ≥ 15% sand, add "with sand" to group name.

[G]If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H]If fines are organic, add "with organic fines" to group name.

[I]If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J]If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K]If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel", whichever is predominant.

[L]If soil contains ≥ 30% plus. No. 200 predominantly sand, add "sandy" to group name.

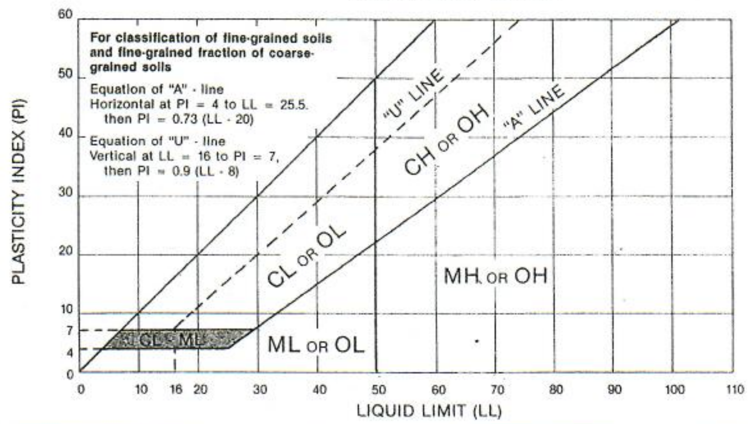
[M]If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

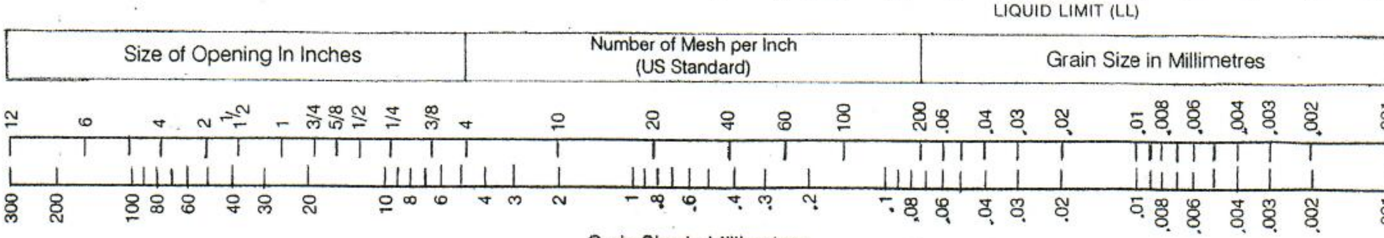
[N]PI ≥ 4 and plots on or above "A" line.

[O]PI < 4 or plots below "A" line.

[P]PI plots on or above "A" line.

[Q]PI plots below "A" line.





Cobble	Gravel		Sand			Silt and/or Clay
	Coarse	Fine	Coarse	Medium	Fine	