2024 GEOTECHNICAL REPORT

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GEOTECHNICAL REPORT

PROPOSED BUILDERS FIRST SUPPLY CENTER
GREEN MOUNTAIN ROAD
WOODLAND, WASHINGTON

PREPARED FOR:

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REVIEW COPY

February 2, 2024

STRATA Project #23-1022

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TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 General

Strata Design (STRATA) completed a geotechnical investigation for the proposed Builders First Supply Center located off of Green Mountain Road in Woodland, Washington. The purpose of our investigation was to evaluate subsurface conditions at the site and develop geotechnical conclusions and recommendations for the design and construction of foundations for the proposed structures, pavements, and related development improvements. The general site location is shown on the Vicinity Map (Figure 1). The approximate locations of STRATA's explorations in relation to existing and proposed site features are shown on the Site Plan, Figure 2. This report describes the work accomplished and provides our geotechnical-related conclusions and recommendations for design and construction of the site improvements. On January 21, 2024, we logged two additional test pit explorations, to accompany our earlier subsurface explorations completed in May 2020. Subsurface logs were reviewed as part of our analysis for development of the recommendations and conclusions in this report.

1.2 Project Understanding

Based on review of available preliminary site plans provided to us, and our discussions with the project team, we understand the project includes construction of a 29,600 square foot multi-use building and a pre-fabricated metal building as shown in Figure 2. We understand that the maximum loadings will be on the order of 100 kip column loads, and 5,000 pounds per foot lineal loads. Additionally, we understand paved access roads (and parking) will be constructed for heavier loads, such as 18-wheel trucks and forklifts.

2 SITE CONDITIONS

2.1 Topography and Surface Description

Based on our ground-based reconnaissance and review of available topographic information, we understand the ground surface to be relatively flat with elevations ranging from about 30 feet to 35 feet. The Burris Creek channel bottom has an elevation of about 34.5 feet where located north and adjacent to the site improvements. The elevation contours noted in Figure 2 are from 2017 LiDAR and reference the North American Datum of 1983 (NAD 83). In mid-2023, the existing site grades within the areas proposed for the improvements described above, were raised by about 5 to 6 feet by the addition of fill soil that was predominantly borrow (spoils) from the adjoining parcel to the north. Crushed rock was imported from excavation screenings from a earthwork-development project in Kalama, Washington.

2.2 Geologic Setting and Landslides

The site lies in the Western Cascades geologic province near the northern margin of the Portland Basin, which forms the southern portion of the Puget Lowlands. Following mild folding, faulting and erosion, the bedrock units in the Western Cascade Range volcanic arc formed a low-relief terrain within which the Portland Basin began to develop. Basaltic lavas of the Miocene-age Columbia River Basalt Group and fluvial deposits of the ancestral Columbia River were deposited on the older Paleogene bedrock within the subsiding or 'pull-apart' Portland basin. Erosion during the geologically recent (late Pleistocene-age, +/- 14,000 ya) Missoula Catastrophic Floods, caused by periodic failure of the ice dam that impounded water in glacial Lake Missoula, is interpreted to have created a flow-through channel or terrace that is present below an elevation of about 300 feet. In the area around the town of Kalama, Washington, this flood-terrace feature is approximately ½ to ¾ miles wide and extends to the south for a distance of approximately four (4) miles. The stripped flood terraces can be identified by the wide, level and gently sloping ground surfaces with the occasional basalt bedrock ridges or buttes protruding above the flood plain surface. Basaltic and andesitic rock outcrops and flat-topped depositional surfaces with thin deposits of micaceous and pumiceous sands along their bases, indicate stripping by the rising and peak floodwaters and sedimentation by slack and receding floodwaters. Throughout the hillside regions of the Columbia River corridor region of Cowlitz County, larger ancient landslides occurred hundreds or even thousands of years ago as evolving geologic equilibrium activity during repeated cycles of heavy, sustained rainfall events and seismic activity. The majority of landslide activity of recent times stems from development impacts to land such as deforestation and earthwork.

According to the Washington Department of Natural Resources (DNR) published report on landslides in the area, which produced a geologic hazard study of the Cowlitz County Urban Corridor (Wegmann, 2006), there are no landslides mapped or cited in the DNR publication.

According to the geologic quadrangle map (R.C. Evarts, 2004), the site area is interpreted as Holocene alluvial fan deposits (map unit Qaf). The deposit is described as moderately to poorly sorted sand and gravel, composed of well rounded pebble and cobble. The far west extents of the site may also be comprised of a volcanic tuff unit (Tt) according to the geologic map.

2.3 Subsurface Conditions

The site was explored on January 15, 2024 by excavating two test pits using a trackhoe. Previously (March 2021), we had drilled a boring (B-1), and excavated four other test pits (TP-7, TP-8, TP-9, and TP-10). The current and previous exploration locations are shown in Figure 2. All tests pits were excavated to depths ranging from 8 feet to 12 feet bgs.

STRATA has summarized the subsurface units as follows:

Structural FILL	Approximately 5 to 6 feet of borrow (clayey-silt w/gravel) was placed in lifts and compacted in Summer 2023. Borrow was excavated from the adjoining parcel to the north using large mass 'scrapers'.
Clayey SILT:	Clayey silt was encountered below the FILL extending to depths explored. This unit has a relative consistency of medium stiff, with seepage.

Sandy Silty CLAY	Sandy silty clay was encountered below the clayey silt in Boring B-1, extending to a depth of 13 feet bgs. The clay is typically grey and contains fine- to coarse-grained sand. This unit was not encountered within the Test Pit depths of excavation to the west and to the south of B-1.
Sandy SILT with Gravel (Decomposed Basalt):	Sandy silt containing occasional gravel (native volcanic) was encountered in Boring B-1 to the depth explored (19 feet bgs). This unit is indicative of the majority of the 5 to 6 feet of borrow material placed over the subject site. The volcanic deposit is typically red-brown and contains fine- to coarse-grained sand and subangular to subrounded gravel. Based on SPT N-values, the relative consistency of the decomposed basalt is very stiff to hard.

2.4 Groundwater

Groundwater was observed at the time of our explorations. Groundwater was observed at depths of between 4 feet bgs in most of the test pits. We anticipate groundwater closely reflects water levels in the nearby Burris Creek, and shallow perched-groundwater conditions may approach the ground surface in response to extended wet periods or heavy/flooding creek flows. Prior to placement of Fill in 2023, the original native surficial soils were characteristic of hydric soil (wetland, floodplain, etc).

3 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

Based on our completed analysis and findings of the subsurface explorations, the proposed building construction at the site will be conducive to conventional spread footing design, as detailed further below. In the production of this document, we have made reasonable assumptions for building loads of up to about 100 kips (columns), and 5 kips per foot (continuous footings).

Management and control of the encountered shallow groundwater levels will be required during utility trenching. The fine-grained soils encountered in the borings overlying the decomposed or weathered basalt are compressible. We anticipate that the placement of new fills (Summer 2023) has induced some consolidation, thus likely improving the shear strength of the native soils by allowing some dissipation of pore water pressure, draining to the edge faces of the fill. Long term, we advise that the completed grading for the site include perimeter drainage ditches and/or backfilled trench drains to help drawdown the water table. During utility trenching, some form of trench dewatering (and possibly shoring) will be necessary to allow placement of pipes/bedding/backfill.

Should the grading and preliminary development plans for the site change substantially, STRATA should be engaged to review the project plans and update our recommendations for earthwork, temporary excavation support and dewatering, foundation support, and additional geotechnical concerns, as necessary.

The following sections of this report provide our recommendations for planning and preliminary design of the proposed bridge foundations and associated earthwork.

3.2 Site Preparation and Grading

The ground surface in the locations of the proposed improvements was stripped of existing vegetation, surface organics, and loose surface soils during rough grading activity in mid-2023. We intermittently observed that the 2023 stripping was completed to a depth of about 12 inches. Following the placement of fill (described above), a variable thickness (average of about 12-inches) granular working surface was placed to protect silty subgrade soils from disturbance by repetitive heavy construction loads.

To minimize disturbance of the near-surface, silt and clay subgrade soils, we recommend site stripping, and all excavations be completed with excavators equipped with smooth-edged buckets. Upon completion of demolition, site stripping, and excavation to subgrade level, all notably soft areas or areas of unsuitable material should be overexcavated to undisturbed soil and backfilled with structural fill.

3.3 Temporary Excavations

Construction of temporary cut slopes in the adjacent soft soils may be problematic and should be shored or carefully carried out. Cuts on the north abutment may be into non-excavatable rock. Permanent cut and fill slopes should be constructed at 2H:1V or flatter. The near-surface soils on the south abutment of the site can be excavated with conventional earthwork equipment. Rock excavation may be encountered on the north abutment depending on the ultimate bridge abutment location and grading plans. Sloughing and caving should be anticipated. Excavation below the water table will be difficult without dewatering or shoring installed. Excavation adjacent to the active creek channel will likely be problematic. The creek flows should be captured and channelized or piped through the construction zone to prevent a potential washout.

All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The contractor is solely responsible for adherence to the OSHA requirements. Trench cuts should stand relatively vertical to a depth of approximately 4 feet bgs, provided no groundwater seepage is present in the trench walls. Open excavation techniques may be used provided the excavation is configured in accordance with the OSHA requirements, groundwater seepage is not present, and with the understanding that some sloughing may occur. Trenches/excavations should be flattened if sloughing occurs or seepage is present. Use of a trench shield or other approved temporary shoring is recommended if vertical walls are desired for cuts deeper than 4 feet bgs. The method of excavation and design of excavation support are the responsibilities of the contractor and should conform to applicable local, state, and federal regulations. If dewatering is used, we recommend that the type and design of the dewatering system be the responsibility of the contractor, who is in the best position to choose systems that fit the overall plan of operation.

3.4 Structural Fill

3.4.1 On-Site Soils

For this project, we anticipate no need for general fill placement. Should it take place in minor forms, we recommend it occur during moderate, dry weather when moisture content can be maintained by air drying and/or addition of water. The fine-grained fraction of the site soils are moisture sensitive, and during wet weather, may become unworkable because of excess moisture content. In order to reduce moisture content, some aerating and drying of fine-grained soils may be required. The material should be placed in lifts with a maximum uncompacted thickness of approximately 6 inches and compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor).

3.4.2 Select Granular Fill

Imported granular material used during periods of wet weather or for haul roads and staging areas should be pit or quarry run rock, crushed rock, or crushed gravel and sand, and should meet the specifications provided in WSDOT SS 9-03.14(2) – Select Borrow. In addition, the imported granular material should be well graded between coarse and fine, and of the fraction passing the US Standard No. 4 Sieve, less than 5 percent by dry weight should pass the US Standard No. 200 Sieve.

During wet conditions, where imported granular material is placed over potentially soft-soil subgrades, we recommend a geotextile be placed between the subgrade and imported granular material. Depending on site conditions, the geotextile should meet WSDOT SS 9-33.2 – Geosynthetic Properties for soil separation or stabilization. The geotextile should be installed in conformance with WSDOT SS 2-12.3 – Construction Geosynthetic (Construction Requirements) and, as applicable, WSDOT SS 2-12.3(2) – Separation or WSDOT SS 2-12.3(3) – Stabilization.

3.5 Foundation Support

Based on our experience and the site conditions as we understand it, it is our opinion that shallow footings may be utilized provided that an undisturbed, competent native subgrade is established for a subbase. To achieve an allowable bearing capacity of 2,500 psf for design, we recommend placing and compacting a 12-inch thick base course of compacted crushed rock placed as structural fill. This is a net bearing pressure and apply to the total of dead and long-term live loads. The allowable pressure used for design may be increased by a factor of 1.5 when considering seismic or wind loads. Excavations near footings for buried utilities should not extend within a 1H:1V plane projected out and down from the outside, bottom edge of the footings.

It is necessary that subgrades for structural fill placement occur only in competent native soils which have been field-verified by the geotechnical engineer during construction. Where soft, loose, or otherwise unsuitable soils are encountered, additional over-excavation may be recommended to mitigate those soils. The resulting over-excavation should be brought back to grade with structural fill. Structural fill for footings should be constructed a minimum of 12 inches wider on each side of the footing for every vertical foot of over-excavation below the concrete footing base. Excavations near footings should not extend within a 1H:1V plane projected downward from the outside, bottom edge of the footings.

Placement of foundation drains is recommended at the base elevations of footings on the outside of the footings. Foundation drains should consist of a minimum 4-inch- diameter, perforated, PVC drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile fabric in order to provide separation from the surrounding soils. Foundation drains should be positively sloped and should outlet to an appropriate discharge point, or stormwater collection system.

3.5.1 Settlement

Where utilizing shallow spread footings and/or a thickened edge, reinforced slab for the building foundations, we estimate that the structure will experience no more no more than 1.5 inches of total settlement and 1/2-inch of differential settlement per 100 feet under the assumed loading.

This stated settlement values assume that subgrade soils are verified in the field by the geotechnical professional, with the 18-inches of granular structural fill placed over non-disturbed native subgrade. Code tolerances for a multi-story wood framed structure are outlined in Table 12.13-3 of ASCE 7-16. The stated differential settlement value are within the tolerances provided by the code.

3.5.2 Lateral Resistance

Lateral loads can be resisted by passive earth pressure on the sides of footings and grade beams, and by friction at the base of the footings. A passive earth pressure of 250 pounds per cubic foot (pcf) may be used for footings confined by native soils and new structural fills. The allowable passive pressure has been reduced by a factor of two to account for the large amount of deformation required to mobilize full passive resistance. Adjacent floor slabs or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. For footings supported on native soils or new structural fills, use a coefficient of friction equal to 0.35 when calculating resistance to sliding. These values do not include a factor of safety.

3.5.3 Proofrolling/Subgrade Verification

Following site preparation and prior to placing aggregate base over shallow foundation, floor slab, and pavement subgrades, the exposed subgrade should be evaluated either by proofrolling or another method of subgrade verification. The subgrade should be proofrolled with a fully loaded dump truck or similar heavy, rubber-tire construction equipment to identify unsuitable areas. If the evaluation of the subgrades occurs during wet conditions, or if proofrolling the subgrades will result in disturbance, they should be evaluated by STRATA using a steel foundation probe. We recommend that STRATA be retained to observe the proofrolling and perform the subgrade verifications. Unsuitable areas identified during the field evaluation should be compacted to a firm condition or be excavated and replaced with structural fill.

3.6 Floor Slabs

Support of floor slabs can be obtained from the undisturbed, competent native soil or on structural fill that extends to this material. Where undocumented fill soils or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with structural fill.

Base rock material placed directly below the slab should have a maximum particle size of ¾-inch or less and should be a minimum 12-inches thick. For floor slabs constructed as recommended, a static modulus of subgrade reaction of 150 kips per cubic foot (kcf), or 240 kcf for dynamic modulus, is recommended for design. Floor slabs constructed as recommended will likely encounter total settlement of no greater than 1-inch. For general floor slab construction, slabs should be jointed around columns and walls to permit slabs and foundations to settle differentially.

The crushed rock base recommended may also serve as a capillary break in providing some protection against moisture intrusion. Ultimately, use of a vapor retarding membrane or other additional vapor barrier will be determined by the architect and/or owner. If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction.

3.7 *Capillary and Radon Gas Break*

The recommended crushed rock base may also serve as a capillary break in providing some protection against moisture intrusion. Ultimately, use of a vapor-retarding membrane or other vapor barrier will be determined by the architect and/or owner. If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction.

In locations designated in the plans (typically living spaces), concrete floor slabs should be supported on a minimum 6-inch-thick layer of open-graded, gas-permeable base rock. Where specified in the plan, the gas-permeable base rock will facilitate collection of radon gas from under the floor slab.

Gas-permeable base rock should consist of crushed rock containing no organic matter or debris, with all material passing through a 1-inch sieve and no more than 10 percent passing a ½-inch sieve, with a free void space ratio of approximately 50 percent. Wherever specified in the architectural plans, a minimum 10-mil polyethylene sheeting (or equivalent material with equal or greater resistance to puncture) with a maximum perm rating of 0.3 should be placed on top of the gas-permeable base rock and, to act as a soil-gas-retarder.

3.8 Pavement Design

The provided pavement recommendations were developed using the American Association of State Highway and Transportation Officials (AASHTO) design methods and associated references to Washington Department of Transportation (WaDOT) specifications for construction. Our evaluation considered a maximum of two trucks per day for a 20-year design life.

The minimum recommended pavement section thicknesses are provided in Table 1, below. Depending on weather conditions at the time of construction, a thicker aggregate base course section could be required to support construction traffic during the preparation and placement of the pavement section.

Table 1. Minimum AC Pavement Sections

Traffic Loading	AC (inches)	Base Course (inches)	Subgrade
Pull-in Car Parking	2.5	12	To be verified in construction. Subgrade shall pass ASTM proofroll test
Drive Lanes	3	12	

The asphalt cement binder should be selected following ODOT SS 00744.11 – Asphalt Cement and Additives. The AC should consist of ½-inch hot mix asphalt concrete (HMAC) with a maximum lift thickness of 3 inches. The AC should conform to ODOT SS 00744.13 and 00744.14 and be compacted to 91 percent of the maximum theoretical density (Rice value) of the mix, as determined in accordance with ASTM D2041.

Heavy construction traffic on new pavements or partial pavement sections (such as base course over the prepared subgrade) will likely exceed the design loads and could potentially damage or shorten the pavement life; therefore, we recommend construction traffic not be allowed on new pavements or that the contractor take appropriate precautions to protect the subgrade and pavement during construction.

If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section.

3.9 Temporary Excavation and Shoring

In accordance with OR-OSHA, temporary exposed cut excavations of more than 4 feet should be sloped or shored. The contractor shall be responsible for maintaining safe excavation of the slopes and/or shoring. From our site characterization, soils to the depths explored are consistent with a OSHA Type B soil designation. OSHA specifies that temporary slope laybacks for this soil may be planned as steep as 1H:1V. STRATA will avail themselves to be consulted to review the contractor’s shoring plan prior to construction.

If significant seepage, running-soil conditions, or slope instability become evident during excavation, flatter slopes may be necessary. Some minor amounts of sloughing, slumping, or running of temporary slopes should be anticipated during and shortly after excavation. Open-cut excavations should be completed and backfilled in the shortest practical sequence. In our opinion, the short-term global stability of temporary slopes will be adequate if surcharge loads due to construction traffic, vehicle parking, material laydown, foundations for existing nearby structures, etc., are maintained a horizontal distance equal to the height of the slope away from the top of the excavation. However, smaller horizontal offsets may be appropriate for surcharge loads that act over smaller areas, such as point loads and foundation loads of limited areal extent. In this regard, any planned temporary excavation slopes in close proximity to the existing structures at the site should be reviewed by a qualified geotechnical engineer prior to construction.

Other measures that should be implemented to reduce the risk of localized sloughing or failures of temporary slopes include 1) using geotextile fabric to protect the exposed slopes from surface erosion; 2) providing positive drainage away from the top and bottom of the excavation slopes; 3) constructing and backfilling embedded structures or overexcavated areas as soon as practical after completing the excavation; and 4) periodically monitoring the slopes and the areas around the top of the excavation for evidence of distress or ground cracking.

It is recommended that heavy construction equipment, building materials, excavated soil, and vehicular traffic not be allowed within a distance equal to 1/3 of the slope height from the top of any excavation. Temporary excavations and cut slopes should be monitored by the contractor during construction based on actual observed soil conditions. Cut slopes may require more layback in the extreme wet seasons and/or be covered with clear plastic sheets. During the time of our investigation, groundwater (or seasonal perched water) was observed ranging from between 4 and 8 feet below current grades. When groundwater or seepage is encountered during construction, the stability of the excavation or trench may be undermined. If this occurs, the sidewalls should be flattened or shored.

It is the contractor's responsibility to select the excavation and dewatering methods, to monitor excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. All excavations should be in accordance with applicable OSHA and State regulations.

3.10 Utility Trenches

Trench backfill for the utility pipe base and pipe zone should consist of import, well-graded crushed rock with a maximum particle size of 1.5-inches and less than 7 percent fines content. Backfill should be placed in accordance with the pipe manufacturer's recommendations. In the absence of manufacturer guidelines, backfill in the pipe zone and trench backfill throughout should be placed in maximum 10-inch-thick loose lifts and compacted to not less than 95 percent of the material's maximum dry density, as determined in general accordance with ASTM D698.

3.11 Wet Weather Considerations

We recommend stabilizing the areas of the site experiencing heavily loaded construction traffic with a support layer of crushed rock with 3-inch or greater particle size. Silt fences, inlet protection, soil stockpile covers, etc., are required to reduce sediment transport during construction to acceptable levels. Measures to reduce erosion should be implemented in general accordance with project civil site plan and state, county, and city regulations.

3.12 Stormwater Runoff Disposal

Based on our findings of a shallow water table, or seasonally high perched water, at the site, we would not recommend that stormwater be infiltrated below surface. In addition, the infiltration rates within the fine-grained soils is low. Under these conditions, infiltration into the subsurface would likely undermine the integrity of the subgrades below the structure foundations and pavement areas. Therefore, we recommend overflow from the planters be piped over to a public street collection system, or publicly owned stormwater utility line (if available).

4 SEISMIC DESIGN

Based on 2022 Washington Structural Specialty Code, we referenced the American Society of Civil Engineers (ASCE) document ASCE/SEI 7-16, titled "Minimum Design Loads and Associated Criteria for Buildings and Other Structures.". The seismic hazard levels are based on a Risk-Targeted Maximum Considered Earthquake (MCER). Based on our review of the soils disclosed by our subsurface explorations, we recommend using Site Class D code-based ground-surface MCER response spectrum to perform seismic design of the structure. The maximum horizontal-direction spectral response accelerations SS and S1 were obtained from the USGS Seismic Design Maps for the project coordinates. The design-level response spectrum is calculated as two-thirds of the ground-surface MCER spectrum. Based on our investigation, the parameters in the following Table should be used to compute seismic base shear forces if the site improvements are designed using the applicable provisions of the current editions of the IBC.

ASCE 7-16 BASED RESPONSE SPECTRUM MCER GROUND MOTION - 5% DAMPING 1% IN 50 YEARS PROBABILITY OF COLLAPSE	
S_S	0.822 g
S_1	0.392 g
MAPPED MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL RESPONSE ACCELERATION PARAMETER (SITE CLASS D)	
S_{MS}	0.987 g
S_{M1}	NULL – SEE 11.4.8
DESIGN SPECTRAL RESPONSE ACCELERATION PARAMETER	
S_{DS}	0.658 g
S_{D1}	NULL – SEE 11.4.8

4.1 *Liquefaction Hazards*

In general, liquefaction occurs when deposits of loose/soft, saturated, cohesionless soils, generally sands and sandy silts, are subjected to strong earthquake shaking. If these deposits cannot drain quickly enough, then the pore water pressures can increase and approach the value of the overburden pressure. When the pore water pressure increases to this value, the shear strength of the soil trends toward zero, causing a liquefiable condition. Subsurface soils consist of moist to wet, soft to medium stiff clay and silt soils with varying amounts of sand and isolated layers of stiff to very dense clayey sand and medium dense fine sand. Due to the low to medium plasticity of these soils and consistency of the sandy layers, we consider the risk of liquefaction at the site to be low.

4.2 *Lateral Spread / Ground Deformation*

In our opinion, the risk of liquefaction-induced lateral spreading and ground deformation at the site is low. Table 12.13-2 of ASCE 7-16 states a tolerance of up to 18-inches, which we believe will not be exceeded for this site.

4.3 *Other Seismic Design Considerations*

Based on our review of the Quaternary Fault and Fold Database for the United States published by the U.S. Geological Survey (USGS 2006), the nearest fault zone is the Portland Hills fault zone, which is located about 13 miles from the project site. Therefore, due to the absence of mapped crustal faults near the site, it is our opinion the potential for earthquake induced liquefaction, lateral spreading, or surface rupture is low.

5 ADDITIONAL SERVICES AND CONSTRUCTION OBSERVATIONS

In most cases, other services beyond completion of the geotechnical engineering report are necessary or desirable to complete the project. Occasionally, conditions or circumstances arise that require additional work that was not anticipated when the geotechnical report was written. STRATA should be retained to review the plans and specifications for this project before they are finalized. Such a review allows us to verify that our recommendations and concerns have been adequately addressed in the design.

Satisfactory earthwork performance depends on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that STRATA be retained to observe general excavation, stripping, fill placement, footing subgrades, and/or pile installation. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

6 LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers, for aiding in the design and construction of the proposed development and is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without express written consent of the client and STRATA. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, field explorations, laboratory testing, and engineering analyses.

It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that STRATA is notified immediately so that we may reevaluate the recommendations of this report.

Unanticipated fill, soil and rock conditions, and seasonal soil moisture and groundwater variations are commonly encountered and cannot be fully determined by merely taking soil samples or completing explorations such as soil borings or test pits. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly constructed project; therefore, we recommend a contingency fund to accommodate such potential extra costs.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations presented herein. Land use, site conditions (both on and off site), or other factors may change over time and could materially affect our findings; therefore, this report should not be relied upon after three years from its issue, or if the site conditions change.

7 REFERENCES

Allen, Burns, & Burns, "Cataclysms on the Columbia", Ooligan Press-Portland State University, 2009.

Evarts, R.C., 2004, Geologic map of the Woodland quadrangle, Clark and Cowlitz Counties, Washington, U.S. Geological Survey, Scientific Investigations Map SIM-2827, 1:24,000.

Petersen, M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y., Rezaeian, S., Harmsen, S. C., Boyd, O. S., Field, N., Chen, R., Rukstales, K. S., Nico, L., Wheeler, R. L., Williams, R. A., and Olsen, A. H., 2014, Documentation for the 2014 update of the United States national seismic hazard maps, U.S. Geological Survey, Open-File Report 2014-1091, 243 pages, http://dx.doi.org/10.3133/ofr20141091.

U.S. Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed January 13, 2021, from USGS website: https://www.usgs.gov/naturalhazards/earthquake-hazards/faults.

Washington State Department of Transportation (WDOT GDM), 2020, Geotechnical Design Manual, M 46-03.13

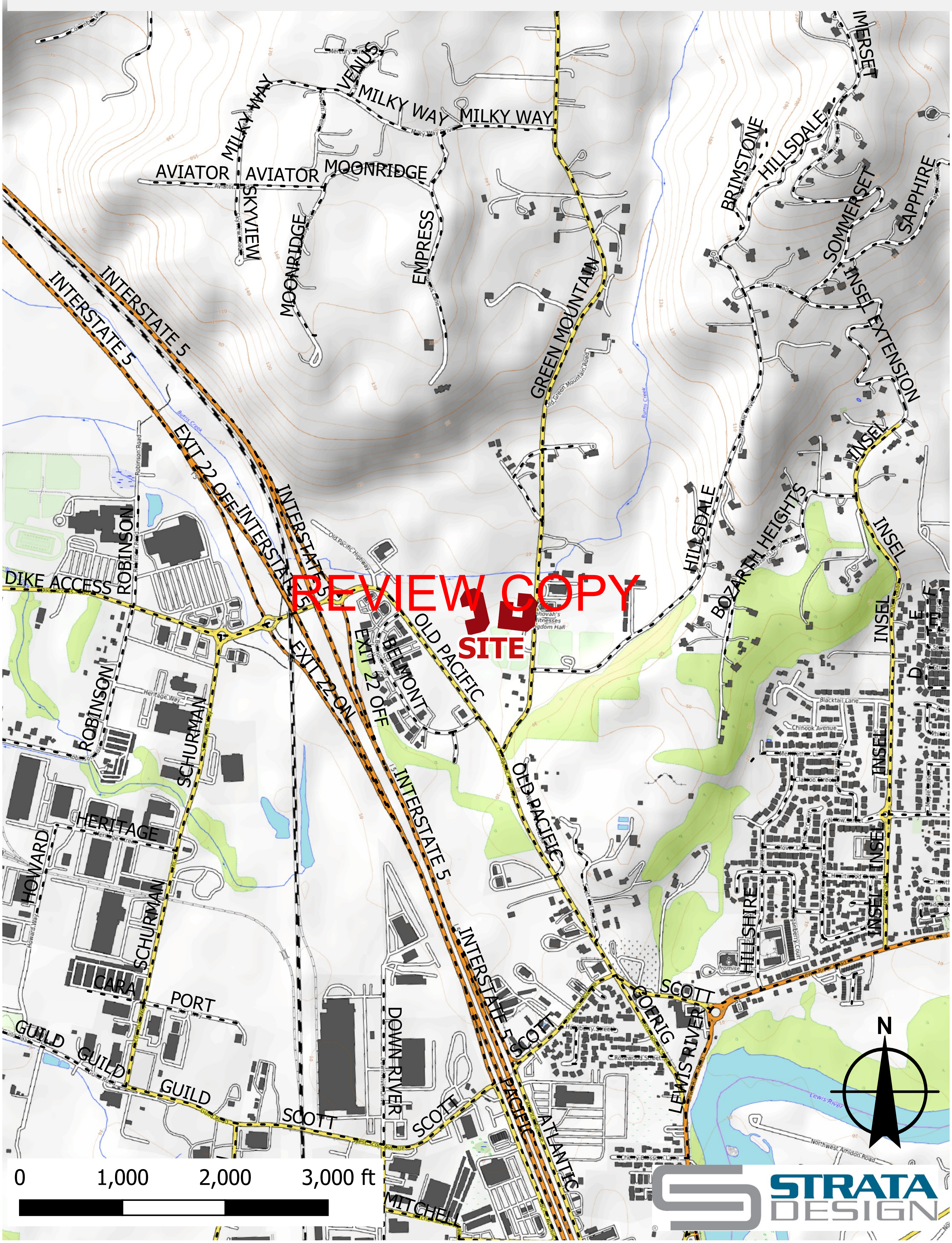
Wegmann, Karl W., 2006, Report of Investigations 35 Digital Landslide Inventory for the Cowlitz County Urban Corridor, Washington version 1.0. Washington State Department of Natural Resources (DNR) Division of Geology and Earth Resources.

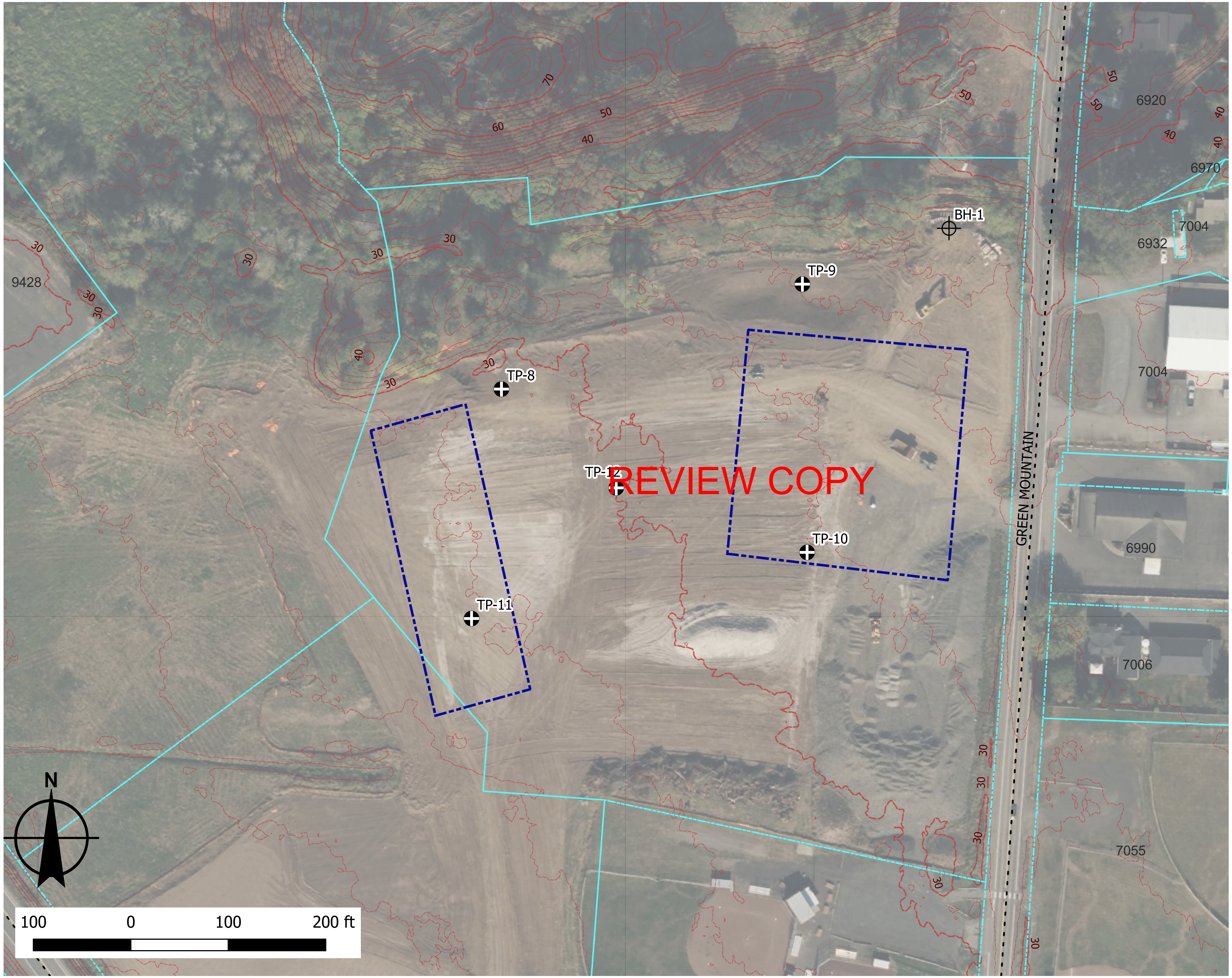
APPENDIX A

Field Explorations

FIGURE 1 - VICINITY MAP

BUILDERS FIRST SUPPLY; WOODLAND, WA





SITE PLAN TOPOGRAPHIC MAP

BUILDERS FIRST SUPPLY
WOODLAND, WA

2-Foot Contours from 2017 LiDAR

Job #23-1022

DECEMBER 2023

Lot lines shown are from GIS and should be considered approximated

LEGEND

- 2-ft
- Project Sites
- Boreholes
- Test Pits

STRATA DESIGN

Figure 2

APPENDIX B

Test Pit Logs

APPENDIX A: FIELD EXPLORATIONS

A1 GENERAL

The approximate locations of the explorations are shown on the Site Plan, Figure 2. The procedures used to advance the borings, collect samples, and other field techniques are described in detail in the following paragraphs. Unless otherwise noted, all soil sampling and classification procedures followed engineering practices in general accordance with relevant ASTM procedures. “General accordance” means that certain local drilling/excavation and descriptive practices and methodologies have been followed.

A2 BORINGS

A2.1 Drilling

Borings were advanced using a small, crawler style rubber-tired drill rig provided and operated by Geoservices Northwest . Boring B-1 was advanced using hollow-stem auger techniques. The borings were observed by a member of the STRATA geotechnical staff, who maintained a log of the subsurface conditions and materials encountered during the course of the work.

A2.2 Sampling

Disturbed soil samples were taken in the borings at selected depth intervals. The samples were obtained using a standard 2-inch outside diameter (OD), split-spoon sampler following procedures prescribed for the standard penetration test (SPT). Using the SPT, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance (N-value). The N-value provides a measure of the relative density of granular soils such as sands and gravels, and the consistency of cohesive soils such as clays and plastic silts. The disturbed soil samples were examined by a member of the STRATA geotechnical staff and then sealed in plastic bags for further examination and physical testing in our laboratory.

A2.3 Boring Logs

The boring logs show the various types of materials that were encountered in the borings and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during drilling, along with their sample identification number, are shown to the right of the classification of materials. The N-values are shown further to the right.

A.3 MATERIAL DESCRIPTION

Initially, samples were classified visually in the field. Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the soil samples were noted. Afterward, the samples were reexamined in the STRATA laboratory and the field classifications were modified where necessary. The terminology used in the soil classifications and other modifiers are defined in Table A-1, Guidelines for Classification of Soil.

Borehole: B-1 South Sitde of Creek

Project:	Borings for Bridge Design	Date start:	3/20/2021
Project ID:	Field Drilling	Date end:	3/20/2021
Location:	South Side of Creek		
Client	Jeffries	Easting:	0.00
Drilling Co.:	GSNW	Northing:	0.00
Method of drilling:	HSA	Ground Elevation:	36.00
Logged by:	rgt Checked by:	Altitude system:	user
Notes:		Scale:	one page

36.00

Depth [ft]	Samples and GWT	Recovery [%]	RQD [%]	Number of Blows [BL/12 in]	USCS Classification	Graphical log	Layers description	Environmental Data - Vapor
0.00				1	OL		loose dark brown TOPSOIL	
1.00	3"							
2.00								
3.00				4			Medium Stiff Grey-Brown Clayey Silt	
4.00								
5.00	12.5"							
6.00								
7.00					ML-CL			
8.00	8.50 during drilling							
9.00	13"			1			Soft Grey fine-coarse sandy SILT/CLAY	
10.00		100.0						
11.00								
12.00								
13.00	18"							
14.00								
15.00								
16.00				30	Basalt		Hard red-brown fine-coarse sandy/gravelly SILT	
17.00	12"							
18.00								
19.00	6"			50			Hard Weathered Basalt	
20.00								

REVIEW COPY

Key:

GWT bored disturbed

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ML	Stiff, brown SILT; moist. (8-inch thick heavily rooted zone at the ground surface)					
		ML	Stiff, mottled brown, fine to medium SANDY SILT; moist.	1				
5								
		ML	Soft, gray SILTY CLAY with fine to coarse sand; wet.	2		▼		
10			End at 9 feet in soft silty clay. Severe caving observed at 8 feet. Groundwater seepage at XX feet during site exploration. Standing groundwater to 8 feet.					
15								
20								

REVIEW COPY

Station:

Approximate Elevation: 30 +/-

Excavation Started: 8/24/20

LOGGED BY: Rick Thrall, PE

Excavator: TB-1140 Trackhoe

Excavation Completed: 8/24/20

20-0262 Jeffries

2117 NE Oregon Street #502
Portland OR 97232
Tel 503-819-4423

STRATA DESIGN

LOG OF TEST PIT

TP-8

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ML	Stiff, brown SILT (TOPSOIL) with organics (roots)l; moist. (_-inch thick heavily rooted zone at the ground surface)					
		ML	Stiff, brown, fine SANDY SILT with trace clay; moist.	1				
5		SC	Medium dense, mottled gray, fine to coarse CLAYEY SAND; wet.	2		▼		
10			End at 8 feet in medium dense clayey sand. No caving observed during exploration. Groundwater observed at 7 feet during site exploration. Standing water at 7 feet.					
15								
20								

Station:		LOGGED BY:	Rick Thrall, PE
Approximate Elevation:	36 +/-	Excavator: TB-1140 Trackhoe	
Excavation Started:	8/24/20	Excavation Completed:	8/24/20

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ML	Stiff, brown SILT (TOPSOIL) with organics (roots); moist. (_-inch thick heavily rooted zone at the ground surface)					
		ML	Stiff, brown, fine SANDY SILT; moist.	1				
5		SC	Soft, gray, fine to coarse CLAYEY SAND; wet.	2				
10			End at 8 feet in soft clayey sand. Caving at 5 feet during site exploration. Groundwater not observed during site exploration.					
15								
20								

Station:

LOGGED BY: Rick Thrall, PE

Approximate Elevation: 31 +/-

Excavator: TB-1140 Trackhoe

Excavation Started: 8/24/20

Excavation Completed: 8/24/20

2117 NE Oregon Street #502
Portland OR 97232
Tel 503-819-4423

STRATA DESIGN

LOG OF TEST PIT

TP-10

2021 GEOTECHNICAL ENGINEERING REPORT

STANDRIDGE
PLANNING | ENGINEERING | SURVEYING

Geotechnical Engineering Report

New Bridge Foundation – Adjacent to Green Mountain Road
Woodland, WA

Prepared for:

Mark and Patrick Jeffries, Owners
18518 NW 41st Avenue
Ridgefield, WA 98642

May 10, 2021
Strata Design Project No. 21-0379

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TABLE OF CONTENTS

FIGURES

APPENDICES

1 INTRODUCTION

1.1 General

Strata Design (STRATA) completed a geotechnical investigation for the proposed bridge across Burris Creek adjacent to Green Mountain Road in Woodland, Washington. The purpose of our investigation was to evaluate subsurface conditions at the site and develop geotechnical conclusions and recommendations for the design and construction of foundations for the proposed bridge. The general site location is shown on the Vicinity Map, Figure 1. The approximate locations of STRATA's explorations in relation to existing and proposed site features are shown on the Site Plan, Figure 2. This report describes the work accomplished and provides our geotechnical-related conclusions and recommendations for design and construction of the bridge foundations and associated earthwork.

We informally completed test pit explorations of the uplands and lower portions of the terrain surrounding the bridge site on August 24, 2020 (results transmitted to client verbally in consultation with Mark and Patrick Jeffries). Several of the logs are pertinent to the bridge explorations and were reviewed as part of our analysis for development of the recommendations and conclusions in this report.

1.2 Project Understanding

Based on review of available preliminary site plans provided by AKS and our discussions with the project team, we understand the project includes construction of a new approximately 32-foot-wide by 30-foot-long bridge over Burris Creek, located approximately 70 feet west of Green Mountain Road in Woodland, Washington (Figure 2). We understand that in the short term the bridge will need to carry loaded off-highway trucks for grading between the uplands and lowland site areas. We also understand that the bridge will be converted for residential use and that the maximum loadings (after construction) will likely be for fire access.

Additionally, we understand paved access roads will be constructed south and north of the proposed bridge to connect parcel 508620100 (south of the bridge) to parcel 508630100 (north of the bridge). We understand that geotechnical information for the connecting roads will be completed later. Our work is focused on the approach fills, abutments, and the bridge structure foundations.

Grading plans and anticipated structural loading for the proposed bridge and pavement features are not available at this time, and STRATA should be consulted to provide updated recommendations when grading plans and additional information are made available. We understand that you are contemplating a steel structure supporting the bridge deck. We are not aware of the clearance requirements based on Burris Creek rise analysis, but we assume that the bridge will be a single span with no intermediate vertical supports. Thus, our explorations were conducted at the north and south ends of the proposed bridge location as indicated on Figure 2. Due to the presence of relatively soft, compressible soils at the site, the new bridge will likely be supported by deep foundation elements that extend into the underlying weathered basalt at depth.

We understand the bridge will be designed in general accordance with the current American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor

Bridge Design Specifications (LRFD BDS) and Washington Department of Transportation (WSDOT) design requirements, except as superseded by Cowlitz County (County).

2 SITE CONDITIONS

2.1 Topography and Surface Description

Based on our ground-based reconnaissance and review of available topographic information, we understand the ground surface adjacent to Burris Creek in the location of the proposed bridge, as well as immediately south of the bridge, is relatively flat with elevations ranging from about 36 feet to 37 feet. The Burris Creek channel bottom has an elevation that ranges from about 34.5 feet on the downstream side of the bridge to about 35 feet on the upstream side of the bridge. The ground surface north of the bridge slopes gently upward to the north with a slope of about 5H:1V (Horizontal to Vertical).

All elevations noted in this report reference the North American Datum of 1983 (NAD 83) unless otherwise noted.

2.2 Geologic Setting and Landslides

The site lies in the Western Cascades geologic province near the northern margin of the Portland Basin, which forms the southern portion of the Puget Lowlands. Following mild folding, faulting and erosion, the bedrock units in the Western Cascade Range volcanic arc formed a low-relief terrain within which the Portland Basin began to develop. Basaltic lavas of the Miocene-age Columbia River Basalt Group and fluvial deposits of the ancestral Columbia River were deposited on the older Paleogene bedrock within the subsiding or ‘pull-apart’ Portland basin. Erosion during the geologically recent (late Pleistocene-age, +/- 14,000 ya) Missoula Catastrophic Floods, caused by periodic failure of the ice dam that impounded water in glacial Lake Missoula, is interpreted to have created a flow-through channel or terrace that is present below an elevation of about 300 feet. In the area around the town of Kalama, Washington, this flood-terrace feature is approximately ½ to ¾ miles wide and extends to the south for a distance of approximately four (4) miles. The stripped flood terraces can be identified by the wide, level and gently sloping ground surfaces with the occasional basalt bedrock ridges or buttes protruding above the flood plain surface. Basaltic and andesitic rock outcrops and flat-topped depositional surfaces with thin deposits of micaceous and pumiceous sands along their bases, indicate stripping by the rising and peak floodwaters and sedimentation by slack and receding floodwaters. Throughout the hillside regions of the Columbia River corridor region of Cowlitz County, larger ancient landslides occurred hundreds or even thousands of years ago as evolving geologic equilibrium activity during repeated cycles of heavy, sustained rainfall events and seismic activity. The majority of landslide activity of recent times stems from development impacts to land such as deforestation and earthwork.

As shown in Figure 2, we understand slopes are present within or near the proposed locations of development which are mapped within an overlay zone of “ancient-inactive, deep seated landslide”. According to the Washington Department of Natural Resources (DNR)-published report on landslides in the area, which produced a geologic hazard study of the Cowlitz County Urban Corridor (Wegmann, 2006), there is an ancient deep-seated landslide that is depicted on GIS mapping abutting the northeastern edge of the property, but not extending into the 10 acre site north of the bridge more than about 5 to 10 feet. The DNR publication cites the ancient landslide located nearby to the west as 'dormant-relict', originating as a slide-rotational, and provides a general description of the landslide mass as fluvial sediments of the Troutdale Formation (QTtd). The publication narrative indicates this large scale ancient landslide (GIS Slide #78) as not exhibiting indications of recent movement, except at a distant upgradient location that is about 500 feet from the study site. According to the geologic quadrangle map (R.C. Evarts, 2004), the site area is interpreted as Holocene alluvial fan deposits (map unit Qaf). The deposit is described as moderately to poorly sorted sand and gravel, composed of well rounded pebble and cobble. The far west extents of the site will also be comprised of a volcanic tuff unit (Tt).

2.3 Subsurface Conditions

The site was explored on March 20, 2021 by drilling two borings, designated B-1 and B-2, to depths of 20 feet and to 12.5 feet below ground surface (bgs). The drilling was performed by Geoservices Northwest using a crawler style rubber-tired drill rig and hollow-stem auger drilling techniques. Encased falling head infiltration testing was also performed at a depth of 5 feet bgs in the location of infiltration test I-1, as shown on Figures 2 and 3.

STRATA has summarized the subsurface units as follows:

Topsoil	Topsoil consisting of clayey silt with variable percentages of sand was encountered at the ground surface and extends to a depth of about 1 foot in borings B-1 and B-2. The topsoil is typically dark brown and contains abundant fine roots and organics. Based on SPT N-values, the relative consistency of the topsoil is very soft.
Clayey SILT:	Clayey silt was encountered below the topsoil in borings B-1 and B-2 and extends to depths of about 5.5 and 11.5 feet bgs, respectively, in these borings. The clayey silt is typically gray brown. Based on SPT N-values, the relative consistency of the clayey silt is soft to medium stiff.
Sandy Silty CLAY	Sandy silty clay was encountered below the clayey silt in boring B-1 and extends to a depth of about 13 feet bgs. The clay is typically grey and contains fine- to coarse-grained sand. Based on SPT N-values, the relative consistency of the clay is very soft.
Sandy SILT with Gravel (Decomposed Basalt):	Sandy silt containing variable percentages of gravel identified as decomposed basalt was encountered below the clay in boring B-1 and extends to a depth of about 19 feet bgs in this boring. The decomposed basalt is typically red-brown and contains fine- to coarse-grained sand and subangular to subrounded gravel. Based on SPT N-values, the relative consistency of the decomposed basalt is very stiff to hard.

Basalt:	Weathered basalt was encountered below the decomposed basalt in boring B-1, and below the clayey silt in boring B-2 and extends to the maximum depths of exploration. The basalt is typically gray, slightly weathered, and soft (R2 by rock description). We consider the weathered basalt to be the foundation zone.

2.4 Groundwater

Groundwater was observed at the time of drilling using hollow-stem auger techniques. Groundwater was observed at depths of 8.5 feet and 5 feet bgs in borings B-1 and B-2, respectively. However, we anticipate groundwater closely reflects water levels in the nearby Burris Creek, and shallow perched-groundwater conditions may approach the ground surface in response to extended wet periods or heavy/flooding creek flows. There is also a "lake/pond" to the west of the site which may influence the groundwater levels.

3 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

The proposed bridge construction at this site is feasible. We understand that bridge loadings, span length, bridge deck elevation and scour potential has not yet been determined and are currently under consideration and design by others. This information is typically used by us to determine approach fill extent and thickness (for consolidation), type and foundation requirements for the abutments, and foundation requirements to support the bridge under static and seismic loadings. We have made reasonable assumptions for the above indicated items in the production of this document.

Subsurface explorations completed for this investigation indicate the site is mantled with clay and silt soils to depths ranging from about 11.5 to 20 feet bgs. The silt encountered below a depth of about 13 feet bgs in boring B-1 was identified as decomposed basalt and contains gravel. The clay and silt soils at the site are underlain by weathered basalt. We anticipate the basalt will becomes less weathered with depth and will likely become harder immediately beneath the weathered basalt unit.

We anticipate groundwater at the site is near the water level of Burris Creek and will fluctuate in response to water levels in the creek. Groundwater levels may approach the ground surface during periods of intense or prolonged precipitation or flooding. The fine-grained soils encountered in the borings overlying the decomposed or weathered basalt are compressible. As discussed, the character of the grading and approach fills, are not known at this time; however, we anticipate placement of new fills will cause settlement that will require time to consolidate. Some type of preloading of the approach fills (particularly on the south abutment) should be contemplated. An alternative would be to construct the bridge and keep a gravel pavement during heavy off road truck use during grading operations and refresh as it settles during construction. Repair and relevel before paving for long term residential use.

We also estimate a risk of seismically induced strain softening of the very soft to medium-stiff silt and clay soils following the code-based earthquake. Soil strain softening would result in reduced soil strength and significant seismic settlement potentially cutting off access to the proposed residential units. Due to static and seismic settlement, conventional spread footings are not considered appropriate for foundation support of the bridge. In our opinion, driven steel pin piles will likely be the most feasible foundation type for bridge support from a cost and construction standpoint and will reduce the risk of post-construction settlement of the bridge. Additionally, the presence of moisture-sensitive fine-grained soils near the ground surface will be a significant construction consideration. Recommendations for protection of the subgrade from construction traffic loading are provided in the Site Preparation and Grading section of this report.

As mentioned, the depth and extent of scour is unknown at this time. The scour depth will potentially affect the type, depth and support of the north and south abutment structures and the functioning of the deep foundation bridge supports.

As noted above, we understand the grading and final development plans for the project had not been completed when this report was prepared. Once completed, STRATA should be engaged to review the project plans and update our recommendations for earthwork, temporary excavation support and dewatering, foundation support, and additional geotechnical concerns, as necessary.

The following sections of this report provide our recommendations for planning and preliminary design of the proposed bridge foundations and associated earthwork.

3.2 Site Preparation and Grading

The ground surface in the locations of all bridge foundations, pavements, and areas to receive structural fill should be stripped of existing vegetation, surface organics, and loose surface soils. We estimate stripping will generally be necessary to a depth of about 12 inches. Deeper excavations may be necessary to remove roots larger than about 1 inch in diameter. Strippings should be removed from the site or stockpiled for use in landscaped areas.

Underground utility lines or other abandoned structural elements in the location of the planned improvements should also be removed. The voids resulting from removal of existing features should be backfilled with compacted structural fill in accordance with the structural fill recommendations in this report. The base of these excavations should be excavated to firm native subgrade before filling. Materials generated during demolition should be transported off site or stockpiled in areas designated by the owner's representative.

To minimize disturbance of the near-surface, silt and clay subgrade soils, we recommend demolition, site stripping, and all excavations be completed with excavators equipped with smooth-edged buckets. Upon completion of demolition, site stripping, and excavation to subgrade level, the resulting subgrade should be observed by a qualified member of STRATA's geotechnical engineering staff. Any soft areas or areas of unsuitable material should be overexcavated to undisturbed soil and to the satisfaction of STRATA's geotechnical engineering staff and backfilled with structural fill.

The on-site soils consist of fine-grained silt and clay soils that are moisture sensitive. When these soils exceed 3% to 4% of their optimum moisture content, they typically become weak and unstable when disturbed and remolded by construction traffic. For this reason, we recommend, if possible, all site preparation and earthwork be accomplished during the dry summer months, typically extending from mid-May to mid-October. If construction is to proceed during the wet months of the year or if the moisture content of subgrade soils is significantly above optimum, we recommend construction equipment not traffic the fine-grained subgrade soils. Site earthwork and subgrade preparation should not be completed during freezing conditions, except for mass excavation to the subgrade design elevations. Protection of the subgrade is the responsibility of the contractor. Construction of granular haul roads to the project site entrance may help reduce further damage to the pavement and disturbance of site soils. In our opinion, a 12-inch-thick granular work pad should be sufficient to prevent disturbance of the subgrade by lighter construction equipment. A granular work pad on the order of 18 inches to 24 inches thick is typically required to protect silty subgrade soils from disturbance by repetitive heavy construction loads. The imported granular material should be placed in one lift over the prepared undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. A geotextile fabric should be used to separate the subgrade from the imported granular material in areas of repeated construction traffic. Depending on site conditions, the geotextile should meet Washington State Department of Transportation (WSDOT) SS 9-33.2 Geosynthetic Properties for soil separation or stabilization. The geotextile should be installed in conformance with WSDOT SS 2-12.3 Construction Geosynthetic (Construction Requirements) and, as applicable, WSDOT SS 2-12.3(2) Separation or WSDOT SS 2-12.3(3) Stabilization. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with granular structural fill.

3.3 Temporary Excavation

Construction of temporary cut slopes in the adjacent soft soils may be problematic and should be shored or carefully carried out. Cuts on the north abutment may be into non-excavatable rock. Permanent cut and fill slopes should be constructed at 2H:1V or flatter. The near-surface soils on the south abutment of the site can be excavated with conventional earthwork equipment. Rock excavation may be encountered on the north abutment depending on the ultimate bridge abutment location and grading plans. Sloughing and caving should be anticipated. Excavation below the water table will be difficult without dewatering or shoring installed. Excavation adjacent to the active creek channel will likely be problematic. The creek flows should be captured and channelized or piped through the construction zone to prevent a potential washout.

All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The contractor is solely responsible for adherence to the OSHA requirements. Trench cuts should stand relatively vertical to a depth of approximately 4 feet bgs, provided no groundwater seepage is present in the trench walls. Open excavation techniques may be used provided the excavation is configured in accordance with the OSHA requirements, groundwater seepage is not present, and with the understanding that some sloughing may occur. Trenches/excavations should be flattened if sloughing occurs or seepage is present. Use of a trench shield or other approved temporary shoring is recommended if vertical walls are desired for cuts deeper than 4 feet bgs. The method of excavation and design of excavation support are the responsibilities of the contractor and should conform to applicable local, state, and federal regulations. If dewatering is used, we recommend that the type and design of the dewatering system be the responsibility of the contractor, who is in the best position to choose systems that fit the overall plan of operation.

3.4 Infiltration Testing

Encased falling-head infiltration testing was conducted at a depth of 5 feet bgs, designated I-1 in the approximate locations shown on the Site Plan, Figures 2. This test location was requested by the designers. The test was conducted in general conformance with the 2017 Cowlitz County Stormwater Drainage Manual. A more detailed description of the testing is provided in Appendix A. The results of the field infiltration testing was analyzed, and the falling-head infiltration rate was calculated from the field-test data. The recommended field infiltration rate for the test location, I-1, is 37 inches per hour. We also understand that the groundwater needs to be a minimum of 5 feet below the ground surface at the indicated location. Although free water was not encountered in the boring at the time of testing, there is a lake nearby the surface of which is at or very near the 5-foot-deep threshold as discussed with the designers.

Compaction of the subgrade soils beneath the infiltration facility could reduce the field permeability to values significantly less than reported above. Reduction of permeability due to compaction of subgrade soils may be a significant consideration in the design of permeable pavements, if used. Based on the soils encountered in borings B-1 and B-2, we anticipate lower-permeability materials may be encountered on the site. Additional explorations and infiltration testing can be completed in these areas if requested by the project team.

It should be noted the proposed locations and depths of the stormwater facilities had not been determined at the time testing was completed. STRATA should review the actual locations and depths with respect to the field-test results when that information becomes available. Additional testing may be required if subsurface conditions at the proposed facility locations are substantially different than those encountered during the testing.

3.5 Structural Fill

3.5.1 General

The extent of site grading is currently unknown; however, STRATA estimates that cuts and fills will be on the order of up to 5 feet deep as part of the bridge abutment and wingwall construction. Note that excavation into or near the groundwater level will be difficult. Structural fill should be placed over subgrade that has been prepared in conformance with the Site Preparation section of this report. Structural fill material should consist of relatively well-graded soil, or an approved rock product that is free of organic material and debris and contains particles not greater than 1.5 inches nominal dimension.

The suitability of soil for use as compacted structural fill will depend on the gradation and moisture content of the soil when it is placed. As the amount of fines (material finer than the US Standard No. 200 Sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and compaction becomes more difficult to achieve. Soils containing more than about 5 percent fines cannot consistently be compacted to a dense, non-yielding condition when the water content is significantly greater (or significantly less) than optimum.

If fill and excavated material will be placed on soil slopes steeper than 5H:1V, these must be keyed/benched into the existing slopes and installed in horizontal lifts. Vertical steps between benches should be approximately 2 feet.

3.5.2 On-Site Soil

On-site soils encountered in our explorations are not suitable for placement as structural fill for general site grading without significant moisture conditioning treatment. A suitable borrow site from the upland areas to the north may be identified based on our previous test pit work.

Fill placement should take place during moderate, dry weather when moisture content can be maintained by air drying and/or addition of water. The fine-grained fraction of the site soils are moisture sensitive, and during wet weather, may become unworkable because of excess moisture content. In order to reduce moisture content, some aerating and drying of fine-grained soils may be required. The material should be placed in lifts with a maximum uncompacted thickness of approximately 6 inches and compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor).

3.5.3 Select Granular Fill

Imported granular material used during periods of wet weather or for haul roads and staging areas should be pit or quarry run rock, crushed rock, or crushed gravel and sand, and should meet the specifications provided in WSDOT SS 9-03.14(2) – Select Borrow. In addition, the imported granular material should be well graded between coarse and fine, and of the fraction passing the US Standard No. 4 Sieve, less than 5 percent by dry weight should pass the US Standard No. 200 Sieve.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 9 inches and be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D698.

During wet conditions, where imported granular material is placed over potentially soft-soil subgrades, we recommend a geotextile be placed between the subgrade and imported granular material. Depending on site conditions, the geotextile should meet WSDOT SS 9-33.2 – Geosynthetic Properties for soil separation or stabilization. The geotextile should be installed in conformance with WSDOT SS 2-12.3 – Construction Geosynthetic (Construction Requirements) and, as applicable, WSDOT SS 2-12.3(2) – Separation or WSDOT SS 2-12.3(3) – Stabilization.

3.5.4 Abutment and Wingwall Structural Backfill and Drainage

Backfilling behind the abutments and wingwalls should conform to Section 6-11.3 of the WSDOT Standard Specifications (SS). Compaction techniques can significantly affect the actual lateral earth pressure against a wall and over compaction of the backfill behind walls should be avoided. Backfill within about 3 feet of the wall should be compacted using lightweight compaction equipment to about 95% of the maximum dry density as determined by ASTM D698 (AASHTO T-99). Beyond 3 feet of the wall, the backfill should be compacted to at least 95% of the maximum dry density as determined by ASTM D698 (AASHTO T-99).

The abutments and wingwalls should have a minimum-12-inch-wide drainage zone of free-draining granular material conforming to Section 2-09.3(1)E of the WSDOT SS and should be provided with a perforated drainpipe or weepholes at the bottom of the backfill. A non-woven geotextile filter fabric, meeting the requirements of WSDOT SS Section 9-33.2 for drainage geotextile, should be placed between the drainage blanket and general wall backfill. Section 7-01.3 of the WSDOT SS also provides guidelines for appropriate drainpipe materials and construction.

3.6 Seismic Considerations

3.6.1 General

We anticipate the proposed bridge will be seismically designed in accordance with current AASHTO LRFD BDS and WSDOT requirements. The current WSDOT *Bridge Design Manual* (BDM) and the WSDOT *Geotechnical Design Manual* (GDM) require bridges to be designed to withstand seismic loading in accordance with the 2011 AASHTO *Guide Specification for LRFD Seismic Bridge Design* (AASHTO SBD) except as modified by the WSDOT BDM. These standards consider a Life Safety design criteria. Based on the AASHTO LRFD BDS, bridges must be designed for a "low probability of collapse but may suffer significant damage and disruption to service" in response to a 1,000-year return-interval earthquake (7% probability of exceedance in 75 years). The 1,000-year return-interval "No Collapse" or "Life-Safety" criteria requires bridge foundation and approach fills to withstand the forces and soil displacements caused by the earthquake without collapse of any portion of the bridge structure.

Ground-motion parameters for the 1,000-year (Life Safety) hazard level is based on the 2014 U.S. Geological Survey (USGS) seismic-hazard maps (Petersen et al., 2014). The 1,000-year acceleration response spectrum is based on three spectral response parameters: peak ground acceleration (PGA), S_S, and S_1, corresponding to periods of 0.0 seconds, 0.2 second, and 1.0 second. The appropriate site class factors, designated F_{PGA}, F_a, and F_v, can be used to adjust the bedrock spectral accelerations to ground-surface spectral accelerations. Based on review of the soil conditions at the site relative to site class definitions provided in Section 3.10.3 of the AASHTO LRFD BDS, the bridge structure can be designed using Site Class D. Section 3.10.2 of AASHTO LRFD BDS requires a site-specific procedure for projects located in a region where long-duration earthquakes are expected or within 6 miles of an active fault. We have assumed the bridge will be a single span with a short fundamental period of vibration, and in our opinion, Site Class D is appropriate for seismic design of the bridge. Therefore, a ground-motion hazard analysis was not completed for the project. A summary of the seismic parameters, including the zero-period peak ground-surface spectral acceleration, the 0.2- and 1.0-second coefficients for the 1,000-year/Life Safety Event hazard level adjusted for Site Class D condition are provided for the project site (i.e., site coordinates of 45.9285° N and 122.7479° W) in Table 3-1 below.

Table 3-1: Recommended 1000-year Earthquake Seismic Design Parameters (Site Class D)

Parameter	0 Second	Short Period	1 Second
Mapped Acceleration (B/C Boundary)	PGA = 0.26 g	S_s = 0.57 g	S_1 = 0.22 g
Site Class	D		
Site Coefficient	F_{PGA} = 1.34	F_a = 1.31	F_v = 2.16

g= Acceleration due to gravity

3.6.2 Cyclic Softening

Cyclic softening is a term that describes a relatively gradual and progressive increase in shear strain with load cycles. Excess pore pressures may increase due to cyclic loading but will generally not approach the total overburden stress. Shear strains accumulate with additional loading cycles, but an abrupt or sudden decrease in shear stiffness is not typically expected. Settlement due to post-seismic consolidation can occur, particularly in lower-plasticity silts. Large shear strains can develop, and strength loss related to soil sensitivity may be a concern. The potential for cyclic softening is typically estimated using a simplified method that compares the cyclic shear stresses induced by the earthquake (demand) to the cyclic shear strength of the soil available to resist these stresses (resistance).

The potential for cyclic softening at the site was estimated based on our observations of subsurface conditions at the site and experience with similar materials. 2014 USGS Probabilistic Seismic Hazard Analysis (PSHA) deaggregation data for the 1,000-year return period indicate the Cascadia Subduction Zone Earthquake (CSZE) generally controls the seismic hazards at the site. Our evaluation indicates the very soft to medium-stiff silt and clay soils overlying the decomposed or weathered basalt to a depth of about 13 feet bgs are susceptible to minor cyclic softening during ground motions associated with the code-level earthquake.

3.6.3 Other Seismic Design Considerations

Based on our review of the Quaternary Fault and Fold Database for the United States published by the U.S. Geological Survey (USGS 2006), the nearest fault zone is the Portland Hills fault zone, which is located about 13 miles from the project site. Therefore, due to the absence of mapped crustal faults near the site, it is our opinion the potential for surface rupture to affect the project site following a seismic event is low.

Based on the subsurface conditions, topography, and site location, it is our opinion the risk of earthquake-induced liquefaction and lateral spreading at the site is low. In our opinion, the risk of seiche at the site is low and the risk of tsunami at the site is absent.

3.7 Bridge Foundation Support

3.7.1 General

Considering the relatively soft soil that mantles the site, the risk of significant static and seismic settlement, spread footings are not considered appropriate for this site. Due to this risk, we recommend that the new bridge abutments and wingwalls be supported on a system of deep foundations. The estimated maximum factored pile loads for the Service, Strength and Extreme limit states for the abutments are not known at this time.

Based on our experience and discussions with local contractors and based on the site conditions as we understand it, it is our opinion that pin piles would provide the least cost option for the bridge support. The following subsections provide preliminary design and construction criteria for steel pin pile foundations based on our subsurface investigation and understanding of the project site.

3.7.2 Pin Piles

We anticipate open-end, steel, 8-inch-diameter pin piles will develop the nominal resistances provided in the table below within about 2 to 5 feet of embedment into the underlying weathered basalt unit. The degree of weathering of the siltstone is variable and the actual penetrations required to achieve the design capacities could be more or less than estimated due to variation in the subsurface materials and conditions.

Table 3-2: Preliminary Estimated Nominal Pile Resistances and Downdrag Loads for Open-Ended Steel Pin Piles Driven a Minimum of 2 Feet into Underlying Weathered Basalt

	Nominal Resistance, kips		Downdrag Load, kips	
Pin Pile Diameter, in.	Strength Limit State	Extreme Limit State	Strength Limit State	Extreme Limit State
8	90	90	5	5

The nominal resistance in Table 3-2 is based on the factored loads provided for the piles. Larger nominal resistances for piles bearing on the weathered basalt could be achieved if necessary. The steel pin piles will develop supporting capacity from a combination of skin friction and tip resistance. Based on Table 10.5.5.2.3-1 of the AASHTO BDS, a resistance factor of 0.45 is appropriate for piles with end bearing in rock. Based on Subsection 10.5.5.3.3 of AASHTO BDS, a resistance factor of 1.0 is appropriate for design of pile foundations in compression for the Extreme Limit State. STRATA recommends center-to-center pile spacing be at minimum 4D, where D is the pile diameter.

The maximum fill heights associated with construction of the bridge abutments are not known at this time. However, settlement of the soft silt soils due to placement of new fills will induce static downdrag loads on the pin piles. Fill associated with construction of the abutments and wing walls will also impose lateral loading on the pin pile caps due to lateral earth pressures on the walls. In addition, the very soft to medium-stiff silt and clay soils above the decomposed and weathered basalt could soften during a design-level earthquake and induce down drag loads on the piles during the Extreme Limit State. The estimated nominal resistances for the Strength and Extreme limit states shown in Table 3-2 do not include contribution from side resistance in the potential zone of static settlements or soil softening because of the potential downward movement of the soil.

Downdrag loads and wall loads on the piles will need to be included in the pile load combinations. Based on guidelines in Section 8.6.2 of the WSDOT GDM, load factors of 1.1 and 1.0 should be applied to the down drag load for the Strength and Extreme limit states, respectively, provided in Table 3-2. The pile down drag load does not need to be subtracted from the resistance side of the equation because nominal side resistance in the zone of potential settlement or soil softening was not included in the estimated nominal capacity.

We understand a pile-supported abutment wall will be used to retain the embankment fills. The static- and seismically induced down drag loads provided in Table 3-2 do not consider down drag load on the abutment wall due to settlement caused by fill placement. Based on our understanding of the project, it is our opinion these loads will be insignificant.

3.7.3 Settlement

Settlement of the steel pin piles driven into the weathered basalt and designed based on the nominal resistances provided in Table 3-2 are anticipated to be limited to the elastic shortening of the pile plus about ¼ inch for Service Limit State loads. To avoid damage to the pile during installation, driving stresses should not exceed 0.9 fy for steel piles. We recommend open-ended pin piles, if selected, be fitted with tip protection. The pin pile installation equipment and methods of installation proposed by the contractor should be reviewed and approved by the project team prior to pile installation. We do not anticipate existing utilities to be present in the planned location of the bridge. However, if existing utilities are present near the new piles, the piles can be pre-bored below the bottom of the utilities to reduce potential damage from lateral pressure developed by soil displacement around the pile. If preboring is used, the prebored hole should not exceed the diameter of the pile.

For preliminary design recommendations, we have assumed the bridge wingwalls will be constructed as part of the bridge abutment walls. If the bridge wingwalls are constructed separately from the abutment walls, significant static and seismic settlement could occur at the wingwalls if not supported by a deep foundation system. STRATA should be consulted to provide additional recommendations once final grading and structural plans are made available. Additionally, we have assumed the bridge will be single span.

As discussed, the bridge will function as an aid to construction while supporting heavily loaded off highway trucks over a construction season. After that the design load would by typical residential which is likely limited to potential fire truck traffic which is approximately half the loading of a loaded off-highway truck. As a cost savings based on significantly lower support requirements, temporary mid-span bents supported by temporary pin piles may be contemplated to support the anticipated heavy truck traffic. The temporary piles can be removed for long term bridge operations.

Subgrade for abutment and wingwall pile caps should be prepared in conformance with our recommendations provided in the Site Preparation and Grading section of this report.

3.7.4 Lateral Loading

Lateral forces in the longitudinal direction can be provided partially or fully by passive earth pressure against the pile cap/abutment. We anticipate the passive earth pressure and modulus values used in design will be for the most extreme loading case, which occurs during a seismic event. In our opinion, the methods described in Section 4.2.11.2 of the WSDOT BDM are appropriate for computing resistance to lateral forces during the seismic event and are suitable for rapid, short-term, transient loading. Alternatively or additionally, pin piles or micropiles can be battered to partially resist lateral forces during the extreme loading case.

4 ADDITIONAL SERVICES AND CONSTRUCTION OBSERVATIONS

In most cases, other services beyond completion of a final geotechnical engineering report are necessary or desirable to complete the project. Occasionally, conditions or circumstances arise that require additional work that was not anticipated when the geotechnical report was written. Strata offers a range of environmental, geological, geotechnical, and construction services to suit the varying needs of our clients.

STRATA should be retained to review the plans and specifications for this project before they are finalized. Such a review allows us to verify that our recommendations and concerns have been adequately addressed in the design.

Satisfactory earthwork performance depends on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that Strata be retained to observe general excavation, stripping, fill placement, footing subgrades, and/or pile installation. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

5 LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers, for aiding in the design and construction of the proposed development and is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without express written consent of the client and Strata. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, field explorations, laboratory testing, and engineering analyses. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Strata is notified immediately so that we may reevaluate the recommendations of this report.

Unanticipated fill, soil and rock conditions, and seasonal soil moisture and groundwater variations are commonly encountered and cannot be fully determined by merely taking soil samples or completing explorations such as soil borings or test pits. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly constructed project; therefore, we recommend a contingency fund to accommodate such potential extra costs.

The scope of work for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations presented herein. Land use, site conditions (both on and off site), or other factors may change over time and could materially affect our findings; therefore, this report should not be relied upon after three years from its issue, or if the site conditions change.

6 REFERENCES

American Association of State Transportation Officials (AASHTO), 2020, Load and resistance factor design (LRFD) bridge design specifications, 9th Edition.

AASHTO, 2011, Guide specification for LRFD seismic bridge design, Second Edition.

Allen, Burns, & Burns, "Cataclysms on the Columbia", Ooligan Press-Portland State University, 2009.

Evarts, R.C., 2004, Geologic map of the Woodland quadrangle, Clark and Cowlitz Counties, Washington, U.S. Geological Survey, Scientific Investigations Map SIM-2827, 1:24,000.

Petersen, M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y., Rezaeian, S., Harmsen, S. C., Boyd, O. S., Field, N., Chen, R., Rukstales, K. S., Nico, L., Wheeler, R. L., Williams, R. A., and Olsen, A. H., 2014, Documentation for the 2014 update of the United States national seismic hazard maps, U.S. Geological Survey, Open-File Report 2014-1091, 243 pages, http://dx.doi.org/10.3133/ofr20141091.

U.S. Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed January 13, 2021, from USGS website: https://www.usgs.gov/naturalhazards/earthquake-hazards/faults.

Washington State Department of Transportation (WDOT BDM), 2020, Bridge Design Manual, M 23-50.20.

Washington State Department of Transportation (WDOT GDM), 2020, Geotechnical Design Manual, M 46-03.13

Washington State Department of Transportation (WSDOT SS), 2021, Standard Specifications for Road, Bridge, and Municipal Construction, M 41-10, Olympia, Washington.

Wegmann, Karl W., 2006, Report of Investigations 35 Digital Landslide Inventory for the Cowlitz County Urban Corridor, Washington version 1.0. Washington State Department of Natural Resources (DNR) Division of Geology and Earth Resources.

Geotechnical Engineering Report
New Bridge Foundation – Adjacent to Green Mountain Road
Woodland, WA

Prepared for:
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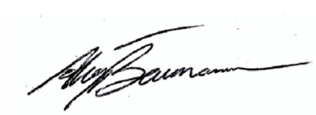
18518 NW 41st Avenue
Ridgefield, WA 98642

Prepared by:
Strata Design, LLC

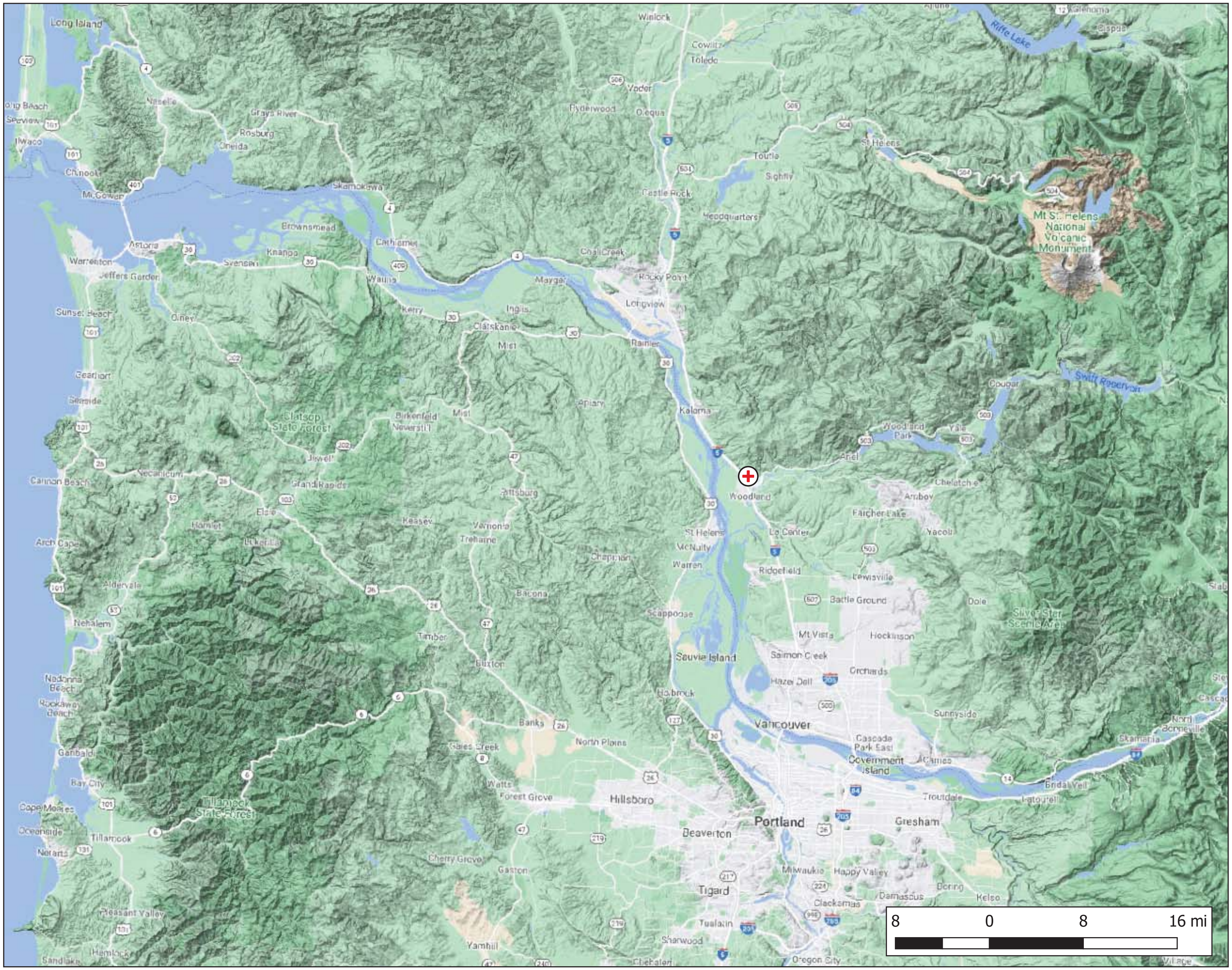


Alex K. Baumann, PE
Senior Geotechnical Engineer

Reviewed by:
Strata Design, LLC

Frederick G. Thrall, PE, GE
Senior Geotechnical Engineer

FIGURES



VICINITY MAP

NEW BRIDGE FOUNDATION - ADJACENT TO GREEN MOUNTAIN ROAD, WOODLAND, WA

STRATA DESIGN

Legend

Site Location

Job #21-0379

Figure 1

AKS SITE PLAN

NEW BRIDGE FOUNDATION - ADJACENT TO GREEN MOUNTAIN ROAD, WOODLAND, WA

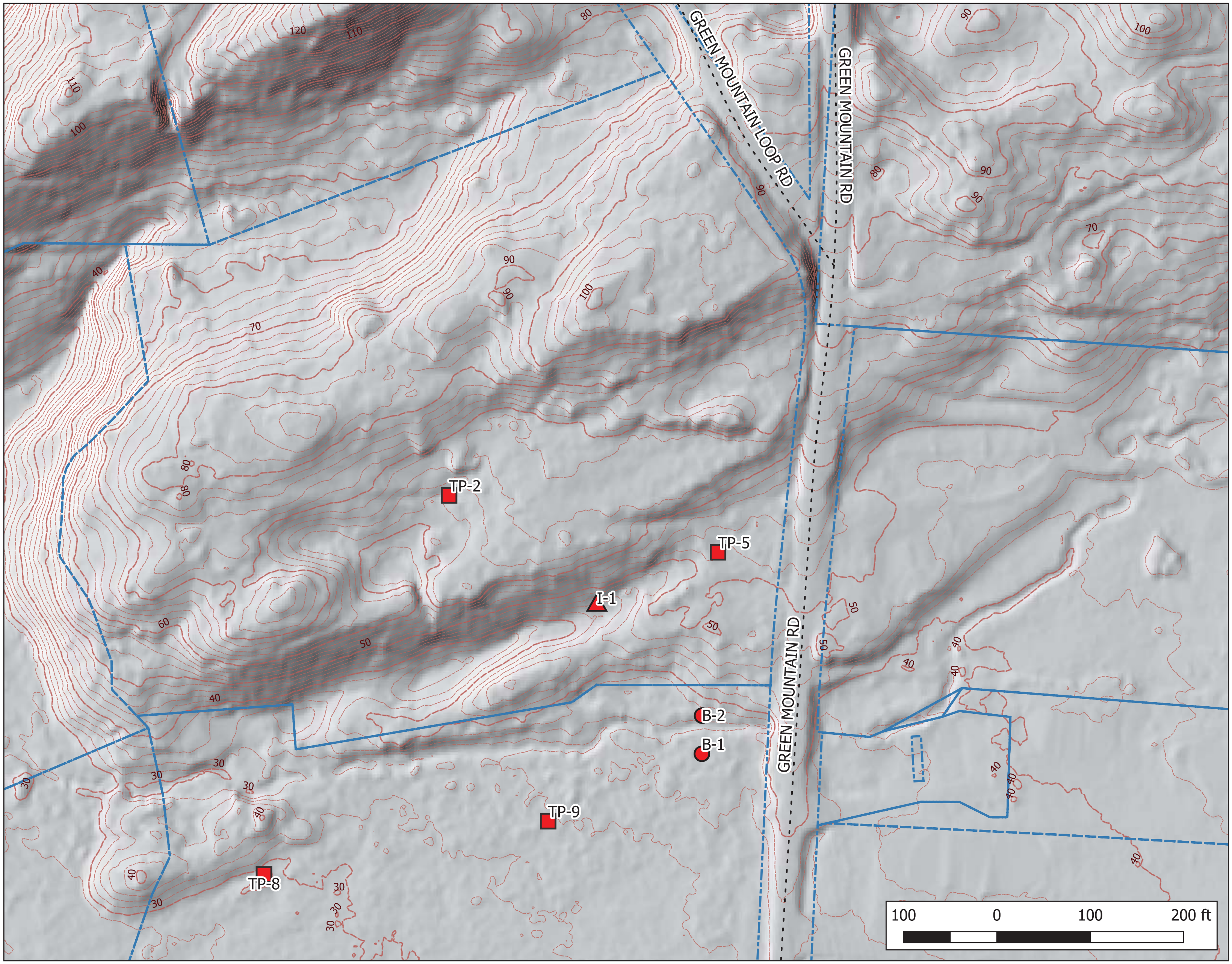
STRATA DESIGN

Legend

- Test Pit Locations
- Boring Locations
- Infiltration Locations

Job #21-0379

Figure 2



LIDAR TOPOGRAPHY SITE PLAN

NEW BRIDGE FOUNDATION - ADJACENT TO GREEN MOUNTAIN ROAD, WOODLAND, WA

Contours derived from 2019 LiDAR (WA-DNR)

STRATA DESIGN

Legend

Contours
- 10-ft
- 2-ft

Test Sites
- Test Pit Locations
- Boring Locations
- Infiltration Locations

Cowlitz County GIS
- Cowlitz Road Centerlines
- Cowlitz County Parcels

Lot lines shown are from Cowlitz County GIS

Figure 3

APPENDIX A

Field Explorations

APPENDIX A: FIELD EXPLORATIONS

A1 GENERAL

STRATA explored subsurface conditions at the project site by advancing two borings. The borings were advanced to depths of up to approximately 12.5 to 20 feet bgs on March 20, 2021. The approximate locations of the explorations are shown on the Site Plan, Figure 2. The procedures used to advance the borings, collect samples, and other field techniques are described in detail in the following paragraphs. Unless otherwise noted, all soil sampling and classification procedures followed engineering practices in general accordance with relevant ASTM procedures. "General accordance" means that certain local drilling/excavation and descriptive practices and methodologies have been followed.

A2 BORINGS

A2.1 Drilling

Borings were advanced using a small, crawler style rubber-tired drill rig provided and operated by Geoservices Northwest . Borings B-1 and B-2 were advanced using hollow-stem auger techniques. The borings were observed by a member of the STRATA geotechnical staff, who maintained a log of the subsurface conditions and materials encountered during the course of the work.

A2.2 Sampling

Disturbed soil samples were taken in the borings at selected depth intervals. The samples were obtained using a standard 2-inch outside diameter (OD), split-spoon sampler following procedures prescribed for the standard penetration test (SPT). Using the SPT, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance (N-value). The N-value provides a measure of the relative density of granular soils such as sands and gravels, and the consistency of cohesive soils such as clays and plastic silts. The disturbed soil samples were examined by a member of the STRATA geotechnical staff and then sealed in plastic bags for further examination and physical testing in our laboratory.

A2.3 Boring Logs

The boring logs show the various types of materials that were encountered in the borings and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during drilling, along with their sample identification number, are shown to the right of the classification of materials. The N-values are shown further to the right.

A.3 MATERIAL DESCRIPTION

Initially, samples were classified visually in the field. Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the soil samples were noted. Afterward, the samples were reexamined in the STRATA laboratory and the field classifications were modified where necessary. The terminology used in the soil classifications and other modifiers are defined in Table A-1, Guidelines for Classification of Soil.

A.4 Falling-Head Infiltration Testing

Falling-head infiltration tests were conducted at a depth of 5 feet in the approximate locations shown on the Site Plan, Figure 2, designated I-1. The testing was conducted in general conformance with the

2017 Cowlitz County Stormwater Drainage Manual. The infiltration tests were conducted using a 4-inch-diameter, hollow-stem auger installed by Geoservices Northwest. The infiltration test was conducted by filling the auger with water to have about 12 inches of head and conducting a falling-head infiltration test through the auger. The bottom of the auger is installed into the soil at the bottom of the hole using a track-mounted drill rig to prevent flow of water below the auger. Prior to conducting the tests, the bottom of each auger was soaked by maintaining an approximate 12- to 24-inch head of water for a minimum of two hours. The infiltration test holes were backfilled with bentonite chips after removing the augers. Results of the infiltration testing are provided in the Infiltration Testing section of this report.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values), blows/ft
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values), blows/ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification

Boulders:
>12 in.

Cobbles:
3-12 in.

Gravel:
¼ - ¾ in. (fine)
¾ - 3 in. (coarse)

Sand:
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay:
Pass No. 200 sieve

Modifier for Subclassification

Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
	Percentage of Other Material (By Weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

Borehole: B-1 South Sitde of Creek

Project:	New Bridge Foundation - Adjacent to Green Mtn. Rd.	Date start:	3/20/2021
Project ID:	Field Drilling	Date end:	3/20/2021
Location:	South Side of Creek		
Client	Jeffries	Easting:	1068443.82
Drilling Co.:	GSNW	Northing:	225174.82
Method of drilling:	HSA	Ground Elevation:	36.00 (Approximate)
Logged by:	rgt	Checked by:	
		Altitude system:	
Notes:		Scale:	

Depth [ft]	Samples and GWT	Recovery [%]	RQD [%]	Number of Blows [BL/12 in]	USCS Classification	Graphical log	Layers description	Environmental Data - Vapor
0.00–1.00	3"	100.0		1	OL		Brown to black, Clayey SILT (OL), very soft, contains organics	
1.00–5.50	12.5"			4	ML-CL		Grey-brown, clayey SILT (ML-CL), moist, soft to medium stiff	
5.50–13.00	8.50 during drilling; 13"; 18"			1	ML-CL		Gray, sandy silty CLAY (ML-CL), moist, very soft, fine- to coarse-grained sand	
13.00–19.00	12"			30	Basalt		Red-brown, sandy SILT with some gravel (Decomposed Basalt), fine- to coarse-grained sand, subangular to subrounded gravel	
19.00–20.00	6"			50	Basalt		Gray, BASALT, slightly weathered, very soft to soft (R1 to R2)	

Ground elevation marker: 36.00

Key:

GWT bored

disturbed

Borehole: B-2 North Sitde of Creek (2)

Project:	New Bridge Foundation - Adjacent to Green Mtn. Rd.	Date start:	3/20/2021
Project ID:	Field Drilling	Date end:	3/20/2021
Location:	North Side of Creek		
Client	Jeffries	Easting:	1068448.53
Drilling Co.:	GSNW	Northing:	225206.06
Method of drilling:	HSA	Ground Elevation:	36.00 (Approximate)
Logged by:	rgt Checked by:	Altitude system:	
Notes:		Scale:	

Depth [ft]	Samples and GWT	Recovery [%]	RQD [%]	Number of Blows [BL/12 in]	USCS Classification	Graphical log	Layers description	Environmental Data - Vapor
						36.00		
0.00 – 1.00	8"			1	OL		Brown to black. clayey SILT (OL), very soft, contains organics	
1.00 – 11.50	12."; 5.00 during drilling; 12"	100.0		7	ML-CL		Gray-brown, clayey SILT (ML-CL), moist, medium stiff	
11.50 – 12.50	6"			50	Basalt		Gray, BASALT, slightly weathered, very soft to soft (R1 to R2)	

Depth scale [ft]: 0.00, 1.00, 2.00, 3.00, 4.00, 5.00, 6.00, 7.00, 8.00, 9.00, 10.00, 11.00, 12.00, 12.50

Key:

GWT bored   disturbed

APPENDIX B

August 2020 Draft Test Pit Logs (close to bridge site)

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ML	Stiff, brown SILT (TOPSOIL) with organics (roots); moist. (6-inch thick heavily rooted zone at the ground surface)					
		ML	Stiff, brown, fine SANDY SILT with trace clay, rounded gravel and cobbles; moist.					
		ML	Stiff, brown SILT; moist.					
5			End at 5 feet in stiff silt. No caving and no groundwater to the depth explored.					
10								
15								
20								

Station:

LOGGED BY: Rick Thrall, PE

Approximate Elevation: 65 +/-

Excavator: TB-1140 Trackhoe

Excavation Started: 8/24/20

Excavation Completed: 8/24/20

20-0262 Jeffries

2117 NE Oregon Street #502
Portland OR 97232
Tel 503-819-4423

STRATA DESIGN

LOG OF TEST PIT

TP-2

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ML	Stiff, brown-black SILT with gravel; moist.(up to 12-inch thick heavily rooted zone at the ground surface)					
		ML	Stiff, brown, fine to coarse SANDY SILT with fine to medium gravel; moist.	1				
				2				
5		RK	Hard, weathered SILTSTONE; moist.					
			End at 10 feet in hard siltstone bedrock. No caving and no groundwater to the depth explored.					
10								
15								
20								

Station:

LOGGED BY: Rick Thrall, PE

Approximate Elevation: 52 +/-

Excavator: TB-1140 Trackhoe

Excavation Started: 8/24/20

Excavation Completed: 8/24/20

20-0262 Jeffries

2117 NE Oregon Street #502
Portland OR 97232
Tel 503-819-4423

STRATA DESIGN

LOG OF TEST PIT

TP-5

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ML	Stiff, brown SILT; moist. (8-inch thick heavily rooted zone at the ground surface)					
		ML	Stiff, mottled brown, fine to medium SANDY SILT; moist.					
5				1				
		ML	Soft, gray SILTY CLAY with fine to coarse sand; wet.	2		▼		
10			End at 9 feet in soft silty clay. Severe caving observed at 8 feet. Groundwater observed at 8 feet					
15								
20								

Station:

LOGGED BY: Rick Thrall, PE

Approximate Elevation: 30 +/-

Excavator: TB-1140 Trackhoe

Excavation Started: 8/24/20

Excavation Completed: 8/24/20

DEPTH (feet bgs)	GRAPHIC LOG	USCS SYMBOL	SOIL DESCRIPTION	SAMPLE	WATER CONTENT (%)	GROUND WATER	FIELD TESTING	TESTING AND LABORATORY DATA
0		ML	Stiff, brown SILT (TOPSOIL) with organics(roots), moist ; up to 12- inch thick heavily rooted zone at the ground surface)					
		ML	Stiff, brown, fine SANDY SILT with trace clay; moist.	1				
5		SC	Medium dense, mottled gray, fine to coarse CLAYEY SAND; wet.	2		▼		
10			End at 8 feet in medium dense clayey sand. No caving observed during exploration. Groundwater observed at 7 feet during site exploration. Standing water at 7 feet.					
15								
20								

Station:

Approximate Elevation: 36 +/-

Excavation Started: 8/24/20

LOGGED BY: Rick Thrall, PE

Excavator: TB-1140 Trackhoe

Excavation Completed: 8/24/20

20-0262 Jeffries

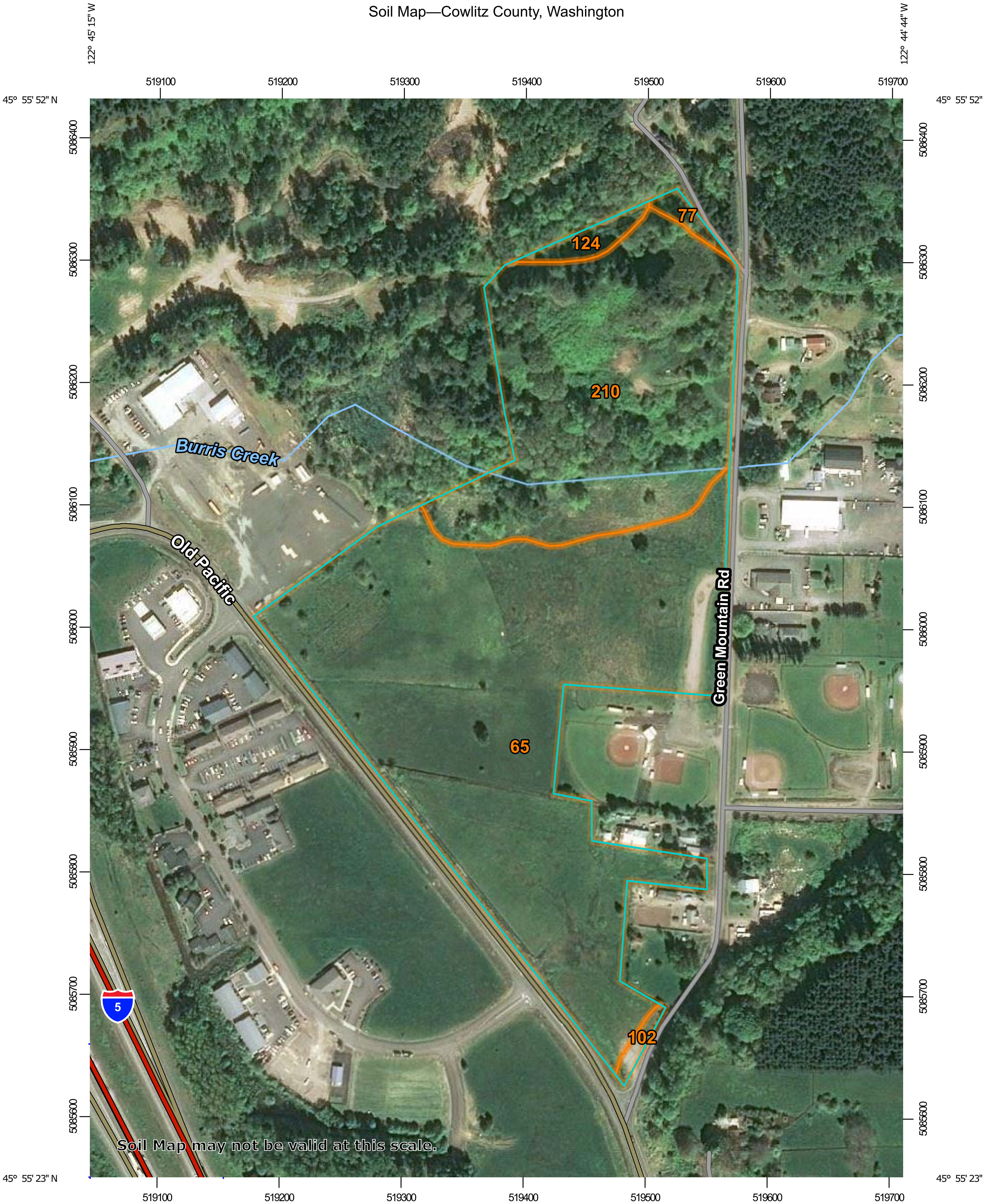
2117 NE Oregon Street #502
Portland OR 97232
Tel 503-819-4423

STRATA DESIGN

LOG OF TEST PIT

TP-9

Appendix G.2: NRCS Soils Report



Map Scale: 1:4,290 if printed on A portrait (8.5" x 11") sheet.

N

Meters: 0 50 100 200 300

Feet: 0 200 400 800 1200

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Cowlitz County, Washington
Survey Area Data: Version 21, Jun 4, 2020

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Apr 26, 2019—Jun 11, 2019

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
65	Godfrey silt loam, 0 to 3 percent slopes	20.9	62.5%
77	Hazeldell gravelly silt loam, 20 to 30 percent slopes	0.3	0.8%
102	Kelso silt loam, 15 to 30 percent slopes	0.2	0.6%
124	Mart silt loam, 8 to 20 percent slopes	0.5	1.4%
210	Stella silt loam, 15 to 30 percent slopes	11.6	34.8%
Totals for Area of Interest		**33.4**	**100.0%**

Engineering Properties

This table gives the engineering classifications and the range of engineering properties for the layers of each soil in the survey area.

Hydrologic soil group is a group of soils having similar runoff potential under similar storm and cover conditions. The criteria for determining Hydrologic soil group is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Listing HSGs by soil map unit component and not by soil series is a new concept for the engineers. Past engineering references contained lists of HSGs by soil series. Soil series are continually being defined and redefined, and the list of soil series names changes so frequently as to make the task of maintaining a single national list virtually impossible. Therefore, the criteria is now used to calculate the HSG using the component soil properties and no such national series lists will be maintained. All such references are obsolete and their use should be discontinued. Soil properties that influence runoff potential are those that influence the minimum rate of infiltration for a bare soil after prolonged wetting and when not frozen. These properties are depth to a seasonal high water table, saturated hydraulic conductivity after prolonged wetting, and depth to a layer with a very slow water transmission rate. Changes in soil properties caused by land management or climate changes also cause the hydrologic soil group to change. The influence of ground cover is treated independently. There are four hydrologic soil groups, A, B, C, and D, and three dual groups, A/D, B/D, and C/D. In the dual groups, the first letter is for drained areas and the second letter is for undrained areas.

The four hydrologic soil groups are described in the following paragraphs:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

Depth to the upper and lower boundaries of each layer is indicated.

Texture is given in the standard terms used by the U.S. Department of Agriculture. These terms are defined according to percentages of sand, silt, and clay in the fraction of the soil that is less than 2 millimeters in diameter. "Loam," for example, is soil that is 7 to 27 percent clay, 28 to 50 percent silt, and less than 52 percent sand. If the content of particles coarser than sand is 15 percent or more, an appropriate modifier is added, for example, "gravelly."

Classification of the soils is determined according to the Unified soil classification system (ASTM, 2005) and the system adopted by the American Association of State Highway and Transportation Officials (AASHTO, 2004).

The Unified system classifies soils according to properties that affect their use as construction material. Soils are classified according to particle-size distribution of the fraction less than 3 inches in diameter and according to plasticity index, liquid limit, and organic matter content. Sandy and gravelly soils are identified as GW, GP, GM, GC, SW, SP, SM, and SC; silty and clayey soils as ML, CL, OL, MH, CH, and OH; and highly organic soils as PT. Soils exhibiting engineering properties of two groups can have a dual classification, for example, CL-ML.

The AASHTO system classifies soils according to those properties that affect roadway construction and maintenance. In this system, the fraction of a mineral soil that is less than 3 inches in diameter is classified in one of seven groups from A-1 through A-7 on the basis of particle-size distribution, liquid limit, and plasticity index. Soils in group A-1 are coarse grained and low in content of fines (silt and clay). At the other extreme, soils in group A-7 are fine grained. Highly organic soils are classified in group A-8 on the basis of visual inspection.

If laboratory data are available, the A-1, A-2, and A-7 groups are further classified as A-1-a, A-1-b, A-2-4, A-2-5, A-2-6, A-2-7, A-7-5, or A-7-6. As an additional refinement, the suitability of a soil as subgrade material can be indicated by a group index number. Group index numbers range from 0 for the best subgrade material to 20 or higher for the poorest.

Percentage of rock fragments larger than 10 inches in diameter and 3 to 10 inches in diameter are indicated as a percentage of the total soil on a dry-weight basis. The percentages are estimates determined mainly by converting volume percentage in the field to weight percentage. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Percentage (of soil particles) passing designated sieves is the percentage of the soil fraction less than 3 inches in diameter based on an ovendry weight. The sieves, numbers 4, 10, 40, and 200 (USA Standard Series), have openings of 4.76, 2.00, 0.420, and 0.074 millimeters, respectively. Estimates are based on laboratory tests of soils sampled in the survey area and in nearby areas and on estimates made in the field. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Liquid limit and *plasticity index* (Atterberg limits) indicate the plasticity characteristics of a soil. The estimates are based on test data from the survey area or from nearby areas and on field examination. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

References:

American Association of State Highway and Transportation Officials (AASHTO). 2004. Standard specifications for transportation materials and methods of sampling and testing. 24th edition.

American Society for Testing and Materials (ASTM). 2005. Standard classification of soils for engineering purposes. ASTM Standard D2487-00.

Report—Engineering Properties

Absence of an entry indicates that the data were not estimated. The asterisk '*' denotes the representative texture; other possible textures follow the dash. The criteria for determining the hydrologic soil group for individual soil components is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Engineering Properties–Cowlitz County, Washington

Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
65—Godfrey silt loam, 0 to 3 percent slopes														
Godfrey	85	D	0-5	Silt loam	CL, CL-ML	A-4, A-6	0- 0- 0	0- 0- 0	100-100-100	100-100-100	90-95-100	80-88-95	25-30-35	5-10-15
			5-27	Silty clay loam, silty clay, clay	MH, ML	A-7	0- 0- 0	0- 0- 0	100-100-100	100-100-100	90-95-100	80-88-95	40-48-55	15-20-25
			27-60	Sandy clay, silty clay, clay	MH, ML	A-7	0- 0- 0	0- 0- 0	100-100-100	100-100-100	80-88-95	50-70-90	40-48-55	15-20-25
77—Hazeldell gravelly silt loam, 20 to 30 percent slopes														
Hazeldell	80	B	0-7	Gravelly silt loam	CL, ML	A-6, A-7	0- 0- 0	0- 0- 0	70-78-85	60-68-75	55-65-75	50-58-65	30-40-50	10-15-20
			7-28	Gravelly clay loam, gravelly loam	CL, ML, SC	A-6, A-7	0- 0- 0	0- 8- 15	75-80-85	60-68-75	55-63-70	40-45-50	30-40-50	10-15-20
			28-40	Gravelly clay loam, gravelly loam, very gravelly clay loam	SC, SM, GC	A-4, A-5, A-6, A-7	0- 3- 5	0- 8- 15	50-68-85	45-60-75	40-55-70	35-43-50	30-40-45	5-15-20
			40-60	Gravelly clay loam, gravelly clay, very gravelly clay loam	CL, GC, ML, SC	A-4, A-5, A-6, A-7	0- 0- 0	0- 3- 5	50-65-80	45-60-75	40-55-70	35-48-60	45-50-55	20-25-25

Engineering Properties–Cowlitz County, Washington														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
124—Mart silt loam, 8 to 20 percent slopes														
Mart	80	C	0-11	Silt loam	CL, CL-ML	A-4, A-6	0- 0- 0	0- 0- 0	90-95-100	85-93-100	85-90-95	85-90-95	25-30-35	5-10-15
			11-20	Silt loam, silty clay loam	CL, CL-ML	A-4, A-6	0- 0- 0	0- 0- 0	90-95-100	85-93-100	85-90-95	80-85-90	25-30-35	5-10-15
			20-40	Silty clay loam	CL	A-6, A-7	0- 0- 0	0- 0- 0	90-95-100	85-93-100	85-93-100	80-85-90	30-38-45	10-15-20
			40-72	Silty clay loam, silt loam	CL, CL-ML	A-4, A-6	0- 0- 0	0- 0- 0	90-95-100	85-93-100	85-90-95	80-85-90	25-30-35	5-10-15
210—Stella silt loam, 15 to 30 percent slopes														
Stella	80	C	0-11	Silt loam	CL	A-6	0- 0- 0	0- 0- 0	100-100-100	100-100-100	90-95-100	75-85-95	25-30-35	10-15-20
			11-25	Silt loam	CL	A-6	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	90-95-100	25-30-35	10-15-20
			25-48	Silty clay loam, silt loam	CL	A-6, A-7	0- 0- 0	0- 0- 0	100-100-100	100-100-100	90-95-100	85-90-95	35-40-45	15-18-20
			48-60	Silty clay, silty clay loam	CH, CL	A-7	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	90-95-100	45-60-75	20-33-45

Data Source Information

Soil Survey Area: Cowlitz County, Washington
Survey Area Data: Version 21, Jun 4, 2020

Physical Soil Properties

This table shows estimates of some physical characteristics and features that affect soil behavior. These estimates are given for the layers of each soil in the survey area. The estimates are based on field observations and on test data for these and similar soils.

Depth to the upper and lower boundaries of each layer is indicated.

Particle size is the effective diameter of a soil particle as measured by sedimentation, sieving, or micrometric methods. Particle sizes are expressed as classes with specific effective diameter class limits. The broad classes are sand, silt, and clay, ranging from the larger to the smaller.

Sand as a soil separate consists of mineral soil particles that are 0.05 millimeter to 2 millimeters in diameter. In this table, the estimated sand content of each soil layer is given as a percentage, by weight, of the soil material that is less than 2 millimeters in diameter.

Silt as a soil separate consists of mineral soil particles that are 0.002 to 0.05 millimeter in diameter. In this table, the estimated silt content of each soil layer is given as a percentage, by weight, of the soil material that is less than 2 millimeters in diameter.

Clay as a soil separate consists of mineral soil particles that are less than 0.002 millimeter in diameter. In this table, the estimated clay content of each soil layer is given as a percentage, by weight, of the soil material that is less than 2 millimeters in diameter.

The content of sand, silt, and clay affects the physical behavior of a soil. Particle size is important for engineering and agronomic interpretations, for determination of soil hydrologic qualities, and for soil classification.

The amount and kind of clay affect the fertility and physical condition of the soil and the ability of the soil to adsorb cations and to retain moisture. They influence shrink-swell potential, saturated hydraulic conductivity (Ksat), plasticity, the ease of soil dispersion, and other soil properties. The amount and kind of clay in a soil also affect tillage and earthmoving operations.

Moist bulk density is the weight of soil (ovendry) per unit volume. Volume is measured when the soil is at field moisture capacity, that is, the moisture content at 1/3- or 1/10-bar (33kPa or 10kPa) moisture tension. Weight is determined after the soil is dried at 105 degrees C. In the table, the estimated moist bulk density of each soil horizon is expressed in grams per cubic centimeter of soil material that is less than 2 millimeters in diameter. Bulk density data are used to compute linear extensibility, shrink-swell potential, available water capacity, total pore space, and other soil properties. The moist bulk density of a soil indicates the pore space available for water and roots. Depending on soil texture, a bulk density of more than 1.4 can restrict water storage and root penetration. Moist bulk density is influenced by texture, kind of clay, content of organic matter, and soil structure.

Saturated hydraulic conductivity (Ksat) refers to the ease with which pores in a saturated soil transmit water. The estimates in the table are expressed in terms of micrometers per second. They are based on soil characteristics observed in the field, particularly structure, porosity, and texture. Saturated hydraulic conductivity (Ksat) is considered in the design of soil drainage systems and septic tank absorption fields.

Available water capacity refers to the quantity of water that the soil is capable of storing for use by plants. The capacity for water storage is given in inches of water per inch of soil for each soil layer. The capacity varies, depending on soil properties that affect retention of water. The most important properties are the content of organic matter, soil texture, bulk density, and soil structure. Available water capacity is an important factor in the choice of plants or crops to be grown and in the design and management of irrigation systems. Available water capacity is not an estimate of the quantity of water actually available to plants at any given time.

Linear extensibility refers to the change in length of an unconfined clod as moisture content is decreased from a moist to a dry state. It is an expression of the volume change between the water content of the clod at 1/3- or 1/10-bar tension (33kPa or 10kPa tension) and oven dryness. The volume change is reported in the table as percent change for the whole soil. The amount and type of clay minerals in the soil influence volume change.

Linear extensibility is used to determine the shrink-swell potential of soils. The shrink-swell potential is low if the soil has a linear extensibility of less than 3 percent; moderate if 3 to 6 percent; high if 6 to 9 percent; and very high if more than 9 percent. If the linear extensibility is more than 3, shrinking and swelling can cause damage to buildings, roads, and other structures and to plant roots. Special design commonly is needed.

Organic matter is the plant and animal residue in the soil at various stages of decomposition. In this table, the estimated content of organic matter is expressed as a percentage, by weight, of the soil material that is less than 2 millimeters in diameter. The content of organic matter in a soil can be maintained by returning crop residue to the soil.

Organic matter has a positive effect on available water capacity, water infiltration, soil organism activity, and tilth. It is a source of nitrogen and other nutrients for crops and soil organisms.

Erosion factors are shown in the table as the K factor (Kw and Kf) and the T factor. Erosion factor K indicates the susceptibility of a soil to sheet and rill erosion by water. Factor K is one of six factors used in the Universal Soil Loss Equation (USLE) and the Revised Universal Soil Loss Equation (RUSLE) to predict the average annual rate of soil loss by sheet and rill erosion in tons per acre per year. The estimates are based primarily on percentage of silt, sand, and organic matter and on soil structure and Ksat. Values of K range from 0.02 to 0.69. Other factors being equal, the higher the value, the more susceptible the soil is to sheet and rill erosion by water.

Erosion factor Kw indicates the erodibility of the whole soil. The estimates are modified by the presence of rock fragments.

Erosion factor Kf indicates the erodibility of the fine-earth fraction, or the material less than 2 millimeters in size.

Erosion factor T is an estimate of the maximum average annual rate of soil erosion by wind and/or water that can occur without affecting crop productivity over a sustained period. The rate is in tons per acre per year.

Wind erodibility groups are made up of soils that have similar properties affecting their susceptibility to wind erosion in cultivated areas. The soils assigned to group 1 are the most susceptible to wind erosion, and those assigned to group 8 are the least susceptible. The groups are described in the "National Soil Survey Handbook."

Wind erodibility index is a numerical value indicating the susceptibility of soil to wind erosion, or the tons per acre per year that can be expected to be lost to wind erosion. There is a close correlation between wind erosion and the texture of the surface layer, the size and durability of surface clods, rock fragments, organic matter, and a calcareous reaction. Soil moisture and frozen soil layers also influence wind erosion.

Reference:

United States Department of Agriculture, Natural Resources Conservation Service. National soil survey handbook, title 430-VI. (http://soils.usda.gov)

Report—Physical Soil Properties

Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Physical Soil Properties–Cowlitz County, Washington														
Map symbol and soil name	Depth	Sand	Silt	Clay	Moist bulk density	Saturated hydraulic conductivity	Available water capacity	Linear extensibility	Organic matter	Erosion factors			Wind erodibility group	Wind erodibility index
										Kw	Kf	T		
	In	*Pct*	*Pct*	*Pct*	*g/cc*	*micro m/sec*	*In/In*	*Pct*	*Pct*					
65—Godfrey silt loam, 0 to 3 percent slopes														
Godfrey	0-5	-25-	-53-	18-23- 27	1.15-1.25 -1.35	4.00-9.00-14.00	0.20-0.22-0. 24	0.0- 1.5- 2.9	0.5- 1.3- 2.0	.49	.49	4	6	48
	5-27	- 7-	-51-	35-43- 50	1.15-1.28 -1.40	0.42-0.91-1.40	0.14-0.16-0. 18	6.0- 7.5- 8.9	0.5- 0.8- 1.0	.32	.32			
	27-60	-48-	- 7-	40-45- 50	1.15-1.28 -1.40	0.01-0.22-0.42	0.13-0.14-0. 15	6.0- 7.5- 8.9	0.5- 0.8- 1.0	.20	.20			
77—Hazeldell gravelly silt loam, 20 to 30 percent slopes														
Hazeldell	0-7	-25-	-52-	20-24- 27	1.10-1.23 -1.35	4.00-9.00-14.00	0.19-0.22-0. 24	3.0- 4.5- 5.9	5.0- 7.5-10. 0	.17	.28	5	7	38
	7-28	-36-	-39-	20-25- 30	1.10-1.23 -1.35	4.00-9.00-14.00	0.18-0.21-0. 24	3.0- 4.5- 5.9	1.0- 1.5- 2.0	.20	.32			
	28-40	-35-	-38-	20-28- 35	1.10-1.28 -1.45	4.00-9.00-14.00	0.13-0.17-0. 21	3.0- 4.5- 5.9	0.0- 0.5- 1.0	.15	.32			
	40-60	-30-	-31-	35-39- 70	1.25-1.35 -1.45	4.00-9.00-14.00	0.13-0.17-0. 20	3.0- 4.5- 5.9	0.0- 0.3- 0.5	.10	.24			

Physical Soil Properties–Cowlitz County, Washington														
Map symbol and soil name	Depth	Sand	Silt	Clay	Moist bulk density	Saturated hydraulic conductivity	Available water capacity	Linear extensibility	Organic matter	Erosion factors			Wind erodibility group	Wind erodibility index
										Kw	Kf	T		
	In	Pct	Pct	Pct	g/cc	micro m/sec	In/In	Pct	Pct					
124—Mart silt loam, 8 to 20 percent slopes														
Mart	0-11	-25-	-52-	20-24- 27	1.15-1.25 -1.35	4.00-9.00-14.00	0.18-0.20-0. 21	3.0- 4.5- 5.9	1.0- 3.0- 5.0	.28	.28	5	6	48
	11-20	-24-	-51-	20-25- 30	1.20-1.30 -1.40	4.00-9.00-14.00	0.18-0.20-0. 21	3.0- 4.5- 5.9	1.0- 1.5- 2.0	.43	.43			
	20-40	-17-	-48-	30-35- 40	1.20-1.30 -1.40	1.40-3.00-4.00	0.17-0.19-0. 20	3.0- 4.5- 5.9	0.0- 0.5- 1.0	.37	.37			
	40-72	-20-	-54-	25-26- 40	1.20-1.30 -1.40	1.40-3.00-4.00	0.09-0.11-0. 12	3.0- 4.5- 5.9	0.0- 0.3- 0.5	.49	.49			
210—Stella silt loam, 15 to 30 percent slopes														
Stella	0-11	-11-	-68-	15-21- 27	1.15-1.25 -1.35	4.00-9.00-14.00	0.19-0.20-0. 21	0.0- 1.5- 2.9	1.0- 3.0- 5.0	.37	.37	5	6	48
	11-25	- 9-	-67-	20-24- 27	1.30-1.38 -1.45	4.00-9.00-14.00	0.18-0.19-0. 20	0.0- 1.5- 2.9	1.0- 1.5- 2.0	.49	.49			
	25-48	- 7-	-65-	20-28- 35	1.20-1.25 -1.30	1.40-3.00-4.00	0.18-0.19-0. 20	3.0- 4.5- 5.9	0.0- 0.5- 1.0	.49	.49			
	48-60	- 7-	-48-	30-45- 60	1.15-1.25 -1.35	0.01-0.91-1.40	0.17-0.18-0. 19	3.0- 4.5- 5.9	0.0- 0.3- 0.5	.32	.32			

Data Source Information

Soil Survey Area: Cowlitz County, Washington
Survey Area Data: Version 21, Jun 4, 2020