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Attn: Mark Nelson

Geotechnical Engineering Report
Proposed Commercial Development
300 River Road
Puyallup, Washington
PN: 0420214010, 0420214027, 0420281154
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INTRODUCTION

This geotechnical engineering report presents the results of our site observations, subsurface explorations, laboratory test results, literature review, engineering analyses, geotechnical recommendations, and design criteria for the proposed showroom to be constructed on the above referenced parcels at 300 River Road in Puyallup, Washington. The approximate site location is shown on the Site Location Map, Figure 1.

Our understanding of the project is based on our conversations with Jim Castino of Castino Architecture and Marc Pudists of Momentum Civil Engineering Consultants, the provided *Proposed Site Plan* prepared by Goree dated March 23, 2021, our understanding of the City of Puyallup (the City) development codes, and our experience in the area. We understand that you propose to demolish the northern portion of the existing showroom, then construct a larger showroom that extends north and west of the existing footprint. We further understand the structure will be a lightly loaded one-story, metal framed structure. A copy of the proposed site plan is attached as our Site & Exploration Plan, Figure 2.

Given the encountered subsurface conditions, we anticipate that shallow foundations will be sufficient to support the proposed structure if the subgrade is improved. In addition, we anticipate that a portion of the subsurface soils could be susceptible to liquefaction during an earthquake. Because of the amount of proposed hard surfacing associated with the project, the City requires a *Soils Report* be prepared in accordance with the *2014 Stormwater Management Manual for Western Washington* (SWMMWW), which includes in-situ infiltration testing and wet season groundwater monitoring.

PURPOSE & SCOPE

The purpose of our services was to evaluate the surface and subsurface conditions across the site as a basis for providing geotechnical recommendations and conclusions for the proposed development. Specifically, the scope of services included the following:

1. Reviewing the available geologic, hydrogeologic, and geotechnical data for the site area;
2. Exploring surface and subsurface conditions by reconnoitering the site and monitoring the drilling of 4 borings at selected locations across the site;
3. Describing surface and subsurface conditions, including soil type, depth to groundwater, and an estimate of seasonal high groundwater levels;
4. Addressing the City of Puyallup Critical Areas Ordinance (Title 21), as appropriate;

5. Providing geotechnical conclusions and recommendations regarding seismic hazards, including liquefaction analysis;
6. Providing geotechnical conclusions and recommendations regarding site grading activities, including site preparation, subgrade preparation, fill placement criteria, suitability of on-site soils for use as structural fill, temporary and permanent cut slopes and drainage and erosion control measures;
7. Providing conclusions regarding shallow foundations and floor slab support and design criteria, including bearing capacity and subgrade modulus as appropriate;
8. Providing conclusions regarding the feasibility of typical ground improvement methods, as appropriate;
9. Providing our opinion about the feasibility of onsite infiltration in accordance with the 2014 *Stormwater Management Manual for Western Washington* (SMMWW) and City Municipal code (City of Puyallup Municipal Code Chapter 21.10), including a preliminary design infiltration rate based on grain size analysis, as applicable; as applicable;
10. Performing groundwater monitoring during the wet season defined by the 2014 SMMWW (December 21 through March 21)
11. Providing a standard duty hot mix asphalt (HMA), heavy duty Portland cement concrete (PCC), pervious concrete, and porous asphalt pavement section designs based on traffic data provided by you;
12. Providing recommendations for erosion and sediment control during wet weather grading and construction; and,
13. Preparing a written *Geotechnical Engineering Report* summarizing our site observations and conclusions, and our geotechnical recommendations and design criteria, along with the supporting data.

The above scope of work was completed in accordance with our *Proposal for Geotechnical Engineering Services* dated April 14, 2021. We received written authorization to proceed with our scope of services from you on April 16, 2021.

SITE CONDITIONS

Surface Conditions

The site consists of three contiguous parcels at 300 River Road in Puyallup, Washington within an area of commercial and residential development. The site is currently developed with existing showroom and service buildings and paved parking areas. Based on the Pierce County Public GIS website, the subject parcels when combined, are irregular in shape and measure approximately 150 to 405 feet wide (east to west) by 175 to 510 feet long (north to south), and encompass about 3.05 acres. The site is bounded by commercial development to the east, residences to the south, 4th Street Northwest to the west, and River Road to the north.

Based on Pierce County GIS data and our site observations, the ground surface at the site gently slopes down to the north at inclinations of less than 2 percent. Total topographic relief across the site is on the order of 8 feet, and topographic relief across the project area is on the order of 4 feet. The site is paved. The existing site configuration and topography is shown on the Site Vicinity Map, Figure 3.

The site is developed with buildings and paved parking areas. No planters or vegetation were present on site. No seeps, springs, or standing water was observed, nor were any areas of exposed soils or active erosion observed at the site at the time of our April 23, 2021 site reconnaissance.

Site Soils

The USDA Natural Resource Conservation Survey (NRCS) Web Soil Survey maps most of the site as underlain by Xerothrents fill (48A) soils and the southwest portion of the site as underlain by Puyallup fine sandy loam (31A) soils. An excerpt from the NRCS soils map for the site area is included as Figure 4.

The Xerothrents fill soils typically consist of modified ground or artificial fill associated with past site grading activities, and form on slopes of 0 to 1 percent. These soils are listed as having “no” to a “slight” erosion hazard when exposed and are not listed in a hydrologic soils group. The Puyallup soils are derived from alluvium and have a “slight” erosion hazard when exposed. These soils form on slopes of 0 to 3 percent, and are included in hydrologic soils group A.

Site Geology

The draft *Geologic Map of the Puyallup 7.5-Minute Quadrangle, Washington* (Troost, in review) maps the site as being underlain by alluvium (Qal). An excerpt of the above referenced map is included as Figure 5.

The alluvium soils typically consist of a poorly sorted, lightly stratified mixture of silt and sand that may contain localized deposits of clay and gravel that were deposited by fluvial processes. The alluvial deposits are considered normally consolidated and can have a range of infiltration potential. No areas of landslides or landslide debris are mapped on or within the vicinity of the site.

Subsurface Explorations

On April 23, 2021, we monitored the drilling of four borings to depths of about 16 to 51 feet below the existing ground surface. The borings were drilled by a licensed drilling contractor operating a track-mounted drill rig working under subcontract to GeoResources. Table 1 below summarizes the location, depth, and elevations of our borings.

TABLE 1:
APPROXIMATE LOCATION, DEPTH, AND ELEVATION OF BORINGS

Boring Number	Approximate Location	Ground Surface Elevation[1] (feet)	Depth Explored (feet)	Termination Elevation[1] (feet)
B-1	Proposed showroom	43	51.5	-8.5
B-2	East of showroom	43	16.5	26.5
B-3	West of service building	43	16.5	26.5
B-4	Southwest parking area	43	16.5	26.5

Notes: [1]Surface elevation estimated from Pierce County Public GIS datum: NAVD 88

The specific location and depth of our borings were determined in the field based on the proposed development and was adjusted based on site access limitations. A field representative from our office continuously monitored the drilling, maintained a log of the subsurface conditions encountered, and obtained representative soil samples. Our field personnel also observed pertinent site features on and adjacent to the site. Representative soil samples obtained from the explorations were placed in sealed plastic bags and taken to our laboratory for further examination and testing as deemed necessary. Borings B-1 and B-3 were backfilled with bentonite chips and abandoned by the driller in accordance with Washington Department of Ecology requirements. Borings B-2 and B-4 were completed as groundwater observation wells so that we can monitor groundwater elevations during the wet winter months (October through April) as required by the City.

During drilling, soil samples were obtained at 2½ and 5 foot depth intervals in accordance with Standard Penetration Test (SPT) as per the test method outlined by ASTM D1586. The SPT method consists of driving a standard 2-inch-diameter split-spoon sampler 18-inches into the soil with a 140-pound hammer. The number of blows required to drive the sampler through each 6-inch interval is counted, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or “SPT blow count”. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The approximate location of our exploration is shown on the attached Site & Exploration Plan, Figure 3. The locations indicated on Figure 3 were estimated based on taping and pacing from locatable site features. Our subsurface explorations indicate the subsurface conditions at specific locations only, as actual subsurface conditions can vary across the site. Furthermore, the nature and extent of any variation would not become evident until additional explorations are performed or until construction activities have begun. Surface elevations were interpolated based on available topographic information. As such, our boring locations and elevations should only be considered accurate to the degree implied by our measuring methods. The soils encountered were visually classified in accordance with the Unified Soil Classification System (USCS) and ASTM D: 2488. The USCS is included in Appendix A as Figure A-1, while the descriptive logs of our borings are included as Figures A-2 through A-5.

Subsurface Conditions

At the locations of our explorations we encountered subsurface conditions that generally confirmed the mapped stratigraphy at the site. In general, our borings encountered about 4 inches of asphalt over several inches of sand and gravel fill. Boring B-2 encountered about 1 foot of gravel with silt, sand, and cobbles in a medium dense, moist condition underlying the pavement section, which we interpret to be fill. Underlying the fill, our borings encountered up to 30 feet of interbedded silt and silty sand in a medium stiff or loose to medium dense, moist to wet condition. We interpret these soils to be consistent with the mapped alluvial deposits.

Borings B-2, B-3, and B-4 terminated in the fine grained alluvial deposits. Boring B-1 extended through the fine grained deposits and encountered about 19 feet of brown well-graded sand with silt and gravel in a medium dense, wet condition. Underlying the well-graded sand, boring B-1 encountered grayish brown silty sand in a loose, wet condition to the full depth explored. We interpret these soils to be consistent with coarse grained alluvial deposits.

Laboratory Testing

Geotechnical laboratory tests were performed on select samples retrieved from the borings to estimate index engineering properties of the encountered soils. Laboratory testing included visual soil classification per ASTM D2488 and ASTM D2487, moisture content determinations per ASTM D2216, and grain size analyses per ASTM D6913 standard procedures. Samples were also submitted to a third party analytical laboratory for organic content testing per ASTM D2974 and cation exchange capacity (CEC) testing per SW846 9081. Test results are included in Appendix B and summarized below in Table 2.

TABLE 2:

LABORATORY TEST RESULTS FOR ON-SITE SOILS

Soil Type	Sample	Gravel Content (percent)	Sand Content (percent)	Silt/Clay Content (percent)	Organic Matter (percent)	CEC (mEQ/ 100g)
Sandy silt (ML)	B-1, S-5, D: 15 ft	0.2	38.9	60.9	ND	ND
Silty sand (SM)	B-1, S-7, D: 25 ft	0.0	61.4	38.6	ND	ND
Sand with silt (SW-SM)	B-1, S-10, D: 40 ft	33.7	59.1	7.2	ND	ND
Sandy silt (ML)	B-3, S-2, D: 5 ft	0.0	45.6	54.4	2.16	11.0
Sandy silt (ML)	B-4, S-2, D: 5 ft	0.0	25.7	72.6	2.39	7.20
Notes: ND = Not Determined						

Two samples of the near surface soils were submitted to an independent analytical laboratory to determine the potential of the site soils to provide water quality treatment. The near-surface soils were determined to have an organic content of 2.16 to 2.39 percent and a cation exchange capacity (CEC) of 7.20 to 11.0 milliequivalents per 100 grams, and therefore meets treatment requirements.

Groundwater Conditions

Groundwater was encountered in all of our borings at approximately 11 feet below existing grades. However, variability observed in the upper site soils, including interbedded silt and sand alluvial soils, likely results in a complex shallow groundwater regime at the site. Table 3 below summarizes the depths and elevations of groundwater encountered in our explorations. We anticipate fluctuations in the local groundwater levels will occur in response to precipitation patterns, off site construction activities, and site utilization.

Groundwater monitoring wells were installed in borings B-2 and B-4 and will be monitored through the 2021 to 2022 wet season. We will prepare a report addendum at the end of the wet season with additional readings.

TABLE 3:
APPROXIMATE DEPTHS AND ELEVATIONS OF GROUNDWATER ENCOUNTERED IN EXPLORATIONS

Boring Number	Depth to Groundwater (feet)	Elevation of Groundwater (feet)	Date Observed
B-1	11.0	32.0	ATD (April 23, 2021)
B-2	11.0	32.0	ATD (April 23, 2021)
B-3	11.0	32.0	ATD (April 23, 2021)
B-4	11.0	32.0	ATD (April 23, 2021)

Notes:
[1] = Surface elevation estimated by interpolating between contours presented on the Pierce County GIS website
ATD = At time of drilling

ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

Based on our site observations and data review, subsurface explorations and our engineering analysis, it is our opinion that the proposed redevelopment of the site is feasible from a geotechnical standpoint, provided the recommendations included herein are incorporated into the project plans. Infiltration of stormwater does not appear feasible at the existing site grades.

Erosion Hazards per PMC 21.06.1210(3)(a)

The Puyallup Municipal Code defines erosion hazard areas that include those identified by the U.S. Department of Agriculture Natural Resources Conservation Service as having a moderate to severe, severe, or very severe erosion hazard because of natural characteristics, including vegetative cover, soil texture, slope, gradient, and rainfall patterns, or human-induced changes to natural characteristics.

Based on the NRCS Web Soil Survey, the site is underlain by Xerothents fill (48A) and Puyallup fine sandy loam (31A), which are described as having "slight" erosion hazards when exposed. Based on the above, it is our opinion that the site does <u>not</u> meet the technical definition of an erosion hazard area.

Seismic Hazards per PMC 21.06.1210(3)(b)

The City of Puyallup Municipal Code Chapter 21.06 defines seismic hazard areas as "areas subject to severe risk of damage as a result of earthquake-induced ground shaking, slope failure, settlement, soil liquefaction, lateral spreading, or surface faulting. Settlement and soil liquefaction conditions occur in areas underlain by cohesionless, loose, or soft-saturated soils of low density, typically in association with a shallow ground water table".

Liquefaction is a phenomenon where there is a reduction or complete loss of soil strength due to an increase in pore water pressure. The increase in pore water pressure is induced by seismic vibrations. Liquefaction mainly affects geologically recent deposits of loose, fine-grained sands and granular silts that are below the groundwater table. The soils encountered at the site generally consisted of silty sands and silts in a loose to medium dense/medium stiff to stiff condition to depths of about 31 feet below the ground surface. Groundwater levels have been observed or interpreted to be about 11 feet below the existing ground surface at the site. Because the site is underlain by loose to medium dense, saturated sands, it is our opinion that the site meets the technical definition of a seismic hazard area and has the potential to liquefy during a seismic event. Additional recommendations regarding the potential for liquefaction at the site are included in the "**Liquefaction Analysis**" portion of this report.

Volcanic Hazards per PMC 21.06.1210(3)(c)

The PMC Chapter 21.06 defines volcanic hazard areas as "those areas subject to pyroclastic flows, lava flows, debris avalanche, and inundation by debris flows, lahars, mudflows, or related flooding resulting from volcanic activity". Volcanic hazard areas shall be classified as Case I or Case II lahars, as identified in the report *Sedimentology, Behavior, and Hazards of Debris Flows at Mount Rainier, Washington,* U.S. Geological Survey Professional Paper 1547, 1995. The site is mapped as being located with an Inundation Zone for Case II Lahars. In our opinion, the site is at similar risk of inundation via lahar, mudflow, or lava flow as the existing development in the area.

Seismic Design

Based on the encountered subsurface conditions and the geologic units mapped at the site, we interpret the structural site conditions to correspond to a Seismic Site Class "F" in accordance with the 2018 IBC (International Building Code) and ASCE 7-16, Chapter 20, Section 20.3. Seismic Site Class "F" is defined by the average standard shear wave velocity of the upper 100 feet of soil being less than 600 feet per second or where liquefaction is likely to occur in the design seismic event. Provided the buildings have a resonant frequency of 0.5 seconds or less and our recommendations to mitigate the liquefaction hazard are incorporated, the values for Site Class "D" may be used and a site response analysis is not required.

The U.S. Geological Survey (USGS) completed probabilistic seismic hazard analyses (PSHA) for the entire country in November 1996, which were updated and republished in 2002, 2008, and 2014. The PSHA ground motion results were obtained from the *ATC Hazard by Location* website. The results are summarized in Table 4, below, with the relevant parameters as provided by the 2018 IBC design.

TABLE 4:
2018 IBC Parameters for Design of Seismic Structures

Spectral Response Acceleration (SRA) and Site Coefficients	Short Period
Mapped SRA	S_s = 1.276
Site Coefficients (Site Class D)	F_a = 1.000
Maximum Considered Earthquake SRA	S_{MS} = 1.276
Design SRA	S_{DS} = 0.850

Peak Ground Acceleration

The mapped peak ground acceleration (PGA) for this site is 0.5g. To account for site class, the PGA is multiplied by a site amplification factor (F_{PGA}) of 1.1. The resulting site modified peak ground acceleration (PGA_M) is 0.55g. In general, estimating seismic earth pressures (k_h) by the Mononobe-Okabe method or seismic inputs for slope stability analysis are taken as 33 to 50 percent of the PGA_M, or 0.18g to 0.27g.

Liquefaction Analysis

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in saturated soils and subsequent loss of strength in the deposit of soil so affected. In general, soils that are most susceptible to liquefaction include loose to medium dense "clean" to silty sands and granular silts that are below the water table. A review of the *Liquefaction Susceptibility Map of Pierce County, Washington* indicates the site soils have a "high" liquefaction potential (Figure 6). Details of our Liquefaction Analysis are included in Appendix C.

We performed liquefaction analyses using the computer program "Liquefy Pro" from CivilTech Corporation, with seismic inputs for the site for the mapped maximum considered geometric mean (MCE_G) peak ground acceleration, per ASCE 7-16 of 0.50g and a magnitude of 7.2. Groundwater was assumed to be at 11 feet below existing grades, based on the groundwater levels measured in our subsurface explorations. Based on these assumptions, we estimate a potential total settlement on the order of 10 to 12 inches could result from liquefaction during the maximum considered earthquake. It is our opinion that liquefaction can be partially mitigated during foundation preparation as described in the "**Liquefaction Mitigation Considerations**" section of this report.

Our liquefaction analyses only account for approximately the upper 50 feet of the site subsurface profile. Potentially liquefiable soils may underlie the soils observed in our borings within 100 feet of the ground surface. There may be potential for additional liquefaction induced settlements on the order of 6 inches above the above estimate if this condition is correct.

Our explorations indicate the subsurface conditions at specific locations only, and the subsurface conditions can vary across the site. It is our opinion that the above listed assumptions are suitable for the site, and that the subsurface conditions encountered are representative. If subsurface conditions that vary from our explorations exist at the site, the above assumptions and associated calculated settlements may no longer be valid. Should variable subsurface conditions be encountered during construction and earthwork activities, we should be notified and allowed to review and revise our assumptions and calculations.

Liquefaction Mitigation Considerations

As discussed above, liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in saturated soils and subsequent loss of strength in the deposit affected.

In general, soils that are susceptible to liquefaction include loose to medium dense "clean" to silty sands and granular silts that are below the water table. Two general approaches to mitigation of liquefaction induced settlement are to address the causes or to address the results. Addressing the causes typically involves ground improvement techniques that densify the soil or provide a means to dissipate the excess pore water pressure. Addressing the results of liquefaction induced settlement typically involves stiffening the upper soil layers and the foundation elements to reduce the potential differential settlement. Below we discuss options for the mitigation of liquefaction induced settlement.

Stiffened Foundation

The potential for liquefaction induced settlement can be partially mitigated by stiffening the upper layer of soil and/or stiffening the foundation elements. Geotechnical research suggests that a layer of non-liquefiable soils directly below the foundation elements can partially mitigate the potential damage from liquefaction induced settlement (Ishihara and Seed, 1998).

We recommend a mat of structural fill be constructed to support the footings. Such a mat could be constructed by either raising grades after stripping the structural areas or by overexcavating and replacing. Using either method, we recommend a structural biaxial or triaxial geogrid with a minimum allowable tensile strength of 1,000 plf be placed on the exposed native soils and minimum thickness of 3 feet of non-liquefiable structural fill placed above the geogrid. The fill should be compacted with a large mechanical compactor such as a vibratory roller or hoe-pack in accordance with the "**Structural Fill**" section of this report. The structural fill mat should extend a minimum horizontal distance of at least 5 feet beyond the footing edges. Where the spacing between foundation elements is greater than 10 feet, we recommend the mat area be extended laterally to create a continuous mat of structural fill below the building. Where excavations extend below the groundwater table, or where the soils are wet, 4 to 6 inch quarry spalls should be placed on top of the geogrid and bucket-tamped until firmly set. The quarry spalls should extend at least 1 foot above the groundwater table.

In addition to soil replacement, seismic ties, grade beams, or other approved methods should be used to stiffen the foundation to reduce the potential for differential settlement. A typical overexcavation detail is included as Figure 7.

Subgrade soil improvements, as described above, can help to reduce the overall and differential settlement within a building footprint during a liquefaction event; however, the soils below the improvements still have the potential to liquefy, and therefore the risk of settlement is not completely eliminated. We recommend that, at a minimum, the shallow soils at the site are removed and a structural fill mat as described above should be placed below shallow foundations for the proposed residences.

Soil Densification

The potential for liquefaction induced settlement can be mitigated by densifying the soils susceptible to liquefaction by using a ground improvement technique, such as aggregate piers (stone columns). Aggregate piers consist of constructing a pattern of subsurface columns comprised of coarse aggregate to displace and densify surrounding soils. Regardless of type or contractor, aggregate piers are installed by driving down to the design depth and backfilling the cavity with compacted granular soil. The aggregate is deposited in lifts and compacted using vertical dynamic impact energy. This process is repeated lift by lift until a column of aggregate is constructed from the design depth to the ground surface.

By adjusting the spacing, diameter, and depth of the aggregate piers, the potential magnitude of the liquefaction induced settlement can be reduced by varying amounts. Typical aggregate pier dimensions range from about 24 to 36 inches. In our opinion, aggregate piers would provide favorable support for spread footings and slab-on-grade floors, thereby eliminating the need for overexcavation and replacement. We recommend that the aggregate pier designer ensure that the piers have sufficient depths and widths to provide the bearing capacities for the design loads. Once the grid of aggregate piers has been installed, the shallow foundation elements can be constructed directly on top of the piers.

Because of the equipment used to install aggregate pier elements, there is typically a large mobilization cost that makes this option have a higher installation cost, but the amount of structural fill and off-haul is considerably less, providing an offset cost savings.

Shallow Foundation Support

We do not recommend that shallow foundation elements be founded directly on the native alluvial soils encountered at the site. Instead, we recommend shallow foundations be supported as described in the "**Liquefaction Mitigation Considerations**" on either a reinforced earth fill or on improved ground (soil densification or stone columns) as describe above. We recommend a minimum width of at least 16 inches for continuous wall footings. Because of the risk of settlement during a seismic event we recommend that isolated spread footings not be used. As stated above, we recommend grade beams or other approved methods should be used to reduce the potential for differential settlement.

Footings founded on non-liquefiable structural fill can be designed using an allowable soil bearing capacity of 1,500 psf (pounds per square foot); footings founded on aggregate piers can be designed using an allowable soil bearing capacity of 2,500 psf. These values are for combined dead and long-term live loads. The weight of the footing and any overlying backfill may be neglected. The allowable bearing value may be increased by one-third for transient loads such as those induced by seismic events or wind loads.

All exterior footing elements should be embedded at least 18 inches below grade for frost protection. Lateral loads may be resisted by friction on the base of footings and floor slabs and as passive pressure on the sides of footings. We recommend that an allowable coefficient of friction of 0.30 be used to calculate friction between the concrete and the appropriately prepared structural fill. Passive pressure may be determined using an allowable equivalent fluid density of 250 pcf (pounds per cubic foot) for foundations backfilled with adequately compacted structural fill that extends a minimum horizontal distance of 3 feet beyond the edge of footing. Factors of safety have been applied to these values.

Post construction settlement below footings designed and constructed as recommended herein should be on the order of 1 inch for the anticipated load conditions, with differential settlements along 50 feet of continuous footings of 0.75 inches or less.

The post construction consolidation settlement is separate from potential liquefaction induced settlement. Because the majority of the upper soils encountered at the site were granular, most of the settlement should occur essentially as loads are being applied; however, some fine grained soils were encountered in the upper 15 feet of our explorations, and these soils have the potential to consolidate over a longer period of time. Based on our experience with similar soils, we anticipate that the majority of the post construction consolidation settlement should occur within 3 to 5 months of completion of construction, and may be on the order of 2 inches where the native soils are not over-excavated or aggregate piers are not used.

Floor Slab Support

Slab-on-grade floors, where constructed, should be supported on the improved subgrade soils prepared as described above. Areas of significant organics should be removed. If a soil densification technique such as aggregate piers is used, a structural slab may be used to span between the aggregate piers.

We recommend that floor slabs be directly underlain by a minimum 4 inch thickness capillary break material such as coarse sand, pea gravel, or crushed rock containing less than 2 percent fines. The capillary break material should be placed in one lift and compacted to an unyielding condition.

A synthetic vapor retarder is recommended to control moisture migration through the slabs. This is of particular importance where the slab elements are underlain by the silty alluvial subgrade, or where moisture migration through the slab is an issue, such as where adhesives are used to anchor carpet or tile to the slab or where slabs are present below heated, enclosed spaces.

A subgrade modulus of 200 pci (pounds per cubic inch) may be used for floor slab design. We estimate that settlement of the floor slabs designed and constructed as recommended, will be ½- inch or less over a span of 50 feet.

Pavement Recommendations

We understand that either flexible pavement consisting of hot mix asphalt (HMA) or rigid pavement consisting of Portland cement concrete (PCC) may be used for the new onsite pavement associated with the development.

Pavement Subgrades

Pavement subgrade areas should be prepared by removing any soft or deleterious material down to firm and unyielding soils in accordance with the "**Site Preparation**" section of this report. The prepared subgrade should be evaluated by proof-rolling with a fully-loaded dump truck or equivalent point load equipment. Soft, loose, or wet areas that are identified should be recompacted or removed, as appropriate. Over-excavated areas should be backfilled with compacted structural fill. Where fill is placed, the upper 2 feet of roadway subgrade should have a maximum dry density of at least 95 percent, as determined in accordance with the ASTM D1557.

Pavement Section Design

We have prepared this analysis in accordance with the 1993 AASHTO flexible and rigid pavement design methods. The AASHTO 93 design method quantifies traffic loading in terms of 18-Kip ESALs (equivalent single axle loads). The estimated ESALs over the entire design life were determined using the assumed traffic data and vehicle loads, and extending the daily value over a 20- or 40-year design life.

We understand that the proposed paved surfaces will consist of either hot mix asphalt (HMA) or Portland cement concrete (PCC) pavement. The pavement sections are designed to support traffic loading from personal vehicles and delivery trucks, as well as two daily trips from heavier vehicles, such as car carrier trucks/trailers. These assumptions should be verified prior to construction, and, if the assumptions contained herein are not correct, we should be notified and allowed to review our calculations. Additional loading may contribute to shortened design life of the pavement section.

We anticipate subgrade soils will consist of in-situ or recompacted native alluvial soils. Table 5, below, summarizes our assumptions and inputs for the design of the concrete and asphalt sections, and Table 6, below, summarizes the recommended pavement section thickness.

TABLE 5:
Input Data for Pavement Design

Parameter	HMA Section	PCC Section
Design Life (years)	20	40
Design Traffic Load (ESALs)	33,000	84,000
Initial Serviceability	4.2	4.5
Terminal Serviceability	2.3	2.5
Reliability, R	85%	80%
Elastic Modulus, E (ksi)	N/A	4,000
Modulus of Subgrade Reaction, k (pci)	N/A	200
Resilient Modulus, Base Course (ksi)	28	N/A
Resilient Modulus, Subgrade (ksi)	6	N/A
Layer Coefficient, HMA (a_1)	0.44	N/A
Layer Coefficient Base Course (a_2)	0.13	N/A
Drainage Coefficient (Cd)	1.0	1.0

Notes:
ESALs - Equivalent Single Axle Loads
ksi – kips per square inch
pci – pounds per cubic inch

TABLE 6:
Minimum Section Thickness Recommendations

Section	Standard HMA	Standard PCC
Pavement	3	6
CSBC or CSTC	7	6[1]

Notes:
CSBC – Crushed Surface Base Course
CSTC – Crushed Surface Top Course
[1] Leveling course as needed below PCC (typically about 4 to 6 inches of crushed rock)

The above recommended section thickness meets the AASHTO 93 design standards based on the assumed traffic loading. Additional loading may contribute to premature failure of the pavement section.

Pavement Frost Conditions

Frost-susceptible soils are generally considered as having greater than 3 percent particle size (by weight) finer than 0.02 millimeter (mm). Soil with a fines content not exceeding 7 percent passing the No. 200 sieve, based on the minus ¾-inch fraction, can normally be expected to have 3 percent or less finer than 0.02 mm. Based on the soils observed during our construction monitoring, most of the near-surface soils could be considered frost-susceptible. Based on information provided in the WSDOT Pavement Policy, we recommend assuming the frost depth would be about 18 inches. For both rigid and flexible pavements, WSDOT recommends that the total depth of the pavement section be at least 50 percent of the frost depth. Our recommended pavement section are thicker than 9-inches and therefore should provide adequate frost protection.

Pavement Materials and Construction

In general, the aggregate base course, HMA, and PCC should be constructed in accordance with WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (WSDOT Standard Specifications, 2020). HMA should conform to Section 5-04 in the WSDOT Standard Specifications and the PCC should conform to Section 5-05 of the WSDOT Standard Specifications. We recommend that crushed rock used as CSBC in pavement sections consist of material of approximately the same quality as "crushed surfacing (base course)" (or better) described in Section 9-03.9(3) of the WSDOT Standard Specifications. We further recommend that CSBC material be compacted to at least 95 percent of the MDD based on the modified Proctor procedure (ASTM D1577).

Site Drainage

All ground surfaces, pavements and sidewalks at the site should be sloped away from structures. The site should also be carefully graded to ensure positive drainage away from all structures and property lines. We recommend that foundation drains are installed for any new structures in accordance with IBC 1805.4.2. The roof drains should not be connected to the foundation drains.

Infiltration Recommendations

Per the 2012 Stormwater Management Manual for Western Washington (SWMMWW), Volume III, Section 3.1.1, infiltration facilities require a minimum vertical separation of 3 feet from the bottom of the proposed facility to the top of a seasonal high groundwater table or other low permeability surface. Additionally, pervious pavement shall not create saturated conditions within 1 foot of the bottom of the proposed facility per Volume V, BMP T5.15.

The soils encountered in our borings are consistent with soils unconsolidated by glacial advance. We therefore used the grain size analysis method (Massmann, 2003) to calculate design infiltration rates for the native site soils. The shallow soils generally consist of silt with somewhat variable fines contents of 54 to 73 percent. Based on the results of the grain size analyses, the siltier soils encountered have an infiltration rate of less than 0.3 inches per hour and meet the criteria for a hydraulic restriction layer. Based on the above, onsite infiltration does <u>not</u> appear feasible.

Per the 2012 SWMMWW, a minimum cation exchange capacity of 5 milliequivalents per 100 grams of soil and 1 percent organic content is required for soils to provide adequate water quality treatment to the stormwater. The near-surface soils were determined to have an organic content of 2.16 to 2.39 percent and a cation exchange capacity (CEC) of 7.20 to 11.0 milliequivalents per 100 grams, and therefore meets treatment requirements.

Alternative stormwater management methods, such as detention or dispersion, should be considered for this project in accordance with the 2012 SWMMWW. All minimum setback requirements and infeasibility criteria per the 2012 SWMMWW should be considered prior to the selection of any stormwater facility for the proposed development.

EARTHWORK RECOMMENDATIONS

Site Preparation

As the site is already developed, it appears that site has been stripped of organic surface soils. Any area to be filled, graded, or developed should be cleared of any other deleterious materials including any existing structures, pavements, foundations, or abandoned utility lines. We anticipate that stripping depths on the order of 12 inches may be required to remove the pavement and gravel base encountered across the site. Stripping depths may be deeper where topographic depressions exist. Where placement of fill material is required, the stripped and exposed subgrade areas should be compacted to a firm and unyielding surface prior to placement of any fill. Excavations for debris removal should be backfilled with structural fill compacted to the densities described in the "**Structural Fill**" section of this report.

We recommend that a member of our staff evaluate the exposed subgrade conditions after stripping is completed and prior to placement of structural fill and or base coarse material. The exposed subgrade soil should be proof-rolled with heavy rubber-tired equipment during dry weather or probed with a ½-inch-diameter steel T-probe during wet weather conditions.

Any soft, loose, or otherwise unsuitable areas delineated during proof-rolling or probing should be recompacted, if practical, or over-excavated and replaced with structural fill. The depth and extent of overexcavation should be evaluated by our field representative at the time of construction. Any areas of old fill material encountered should be evaluated during grading operations to determine if they need mitigation, recompaction, or removal.

Structural Fill

All material placed as fill associated with mass grading, as utility trench backfill, under building areas, or under roadways should be placed as structural fill. The structural fill should be placed in horizontal lifts of appropriate thickness to allow adequate and uniform compaction of each lift. Fill should be compacted to at least 95 percent of the maximum dry density (MDD) as determined by the Modified Proctor test in accordance with ASTM D1557.

The appropriate lift thickness will depend on the fill characteristics and the compaction equipment used. We recommend that the appropriate lift thickness be evaluated by our field representative during construction, and that our representative be present during site grading activities to observe the work and perform field density tests, as appropriate.

The suitability of material for use as structural fill will depend on the gradation and moisture content of the soil. Structural fill placed should below foundations, at a minimum, should consist of granular, non-liquefiable material. As the amount of fines (material passing US No. 200 sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. During wet weather, we recommend use of well-graded sand and gravel with less than 5 percent (by weight) passing the US No. 200 sieve based on that fraction passing the 3/4-inch sieve, such as "*Gravel Backfill for Walls*" (9-03.12(2)) or "*Bank Run Gravel for Trench Backfill*" (9-03.19). If prolonged dry weather prevails during the earthwork and foundation installation phase of construction, material containing up to 8 percent fines (material passing the No. 200 sieve, based on the minus ¾-inch fraction) will be acceptable; the fines should be non-plastic. GeoResources should review submittals for import fill to assess the liquefaction potential.

Material placed for structural fill should be free of debris, organic matter, trash, and cobbles greater than 6-inches in diameter. The moisture content of the fill material should be adjusted as necessary for proper compaction.

Suitability of On-Site Materials as Fill

During dry weather construction, granular, non-organic, onsite soil may be considered for use as structural fill, provided it meets the criteria described above in the "**Structural Fill**" section of this report and can be compacted as recommended. As stated above, soils with moderate to high fines content should not be re-used as structural fill below proposed structures. If the soil material is over-optimum in moisture content when excavated, it will be necessary to aerate or dry the soil prior to placement as structural fill. The soils encountered in our explorations were generally observed to be moist to saturated, and will likely require aeration if used as structural fill. We recommend that native soils not be used as structural fill for the stiffened foundation option because they do not meet the definition of a non-liquefiable soil.

We recommend that completed graded-areas be restricted from traffic or protected prior to wet weather conditions. The graded areas may be protected by paving, placing asphalt-treated base, a layer of free-draining material such as pit run sand and gravel or clean crushed rock material containing less than 5 percent fines, or some combination of the above.

Temporary Excavations

All job site safety issues and precautions are the responsibility of the contractor providing services/work. The following cut/fill slope guidelines are provided for planning purposes only. Temporary cut slopes will likely be necessary during grading operations or utility installation.

All excavations at the site associated with confined spaces, such as utility trenches and retaining walls, must be completed in accordance with local, state, or federal requirements. Based on current Washington Industrial Safety and Health Act (WISHA, WAC 296-155-66401) regulations, the soils on the site would be classified as Type C soils.

According to WISHA, for temporary excavations of less than 20 feet in depth, the side slopes in Type C soils should be laid back at a slope inclination of 1.5H:1V (Horizontal: Vertical), or flatter from the toe to top of the slope. It should be recognized that slopes of this nature do ravel and require occasional maintenance. All exposed slope faces should be covered with a durable reinforced plastic membrane, jute matting, or other erosion control mats during construction to prevent slope raveling and rutting during periods of precipitation. These guidelines assume that all surface loads are kept at a minimum distance of at least one half the depth of the cut away from the top of the slope and that significant seepage is not present on the slope face. Flatter cut slopes will be necessary where significant raveling or seepage occurs, or if construction materials will be stockpiled along the top of the slope.

Where it is not feasible to slope the site soils back at the recommended inclinations, a shoring system should be considered. Where retaining structures are greater than 4-feet in height (bottom of footing to top of structure) or have slopes of greater than 15 percent above them, they should be engineered per Washington Administrative Code (WAC 51-16-080 item 5).

This information is provided solely for the benefit of the owner and other design consultants, and should not be construed to imply that GeoResources assumes responsibility for job site safety. It is understood that job site safety is the sole responsibility of the project contractor.

Dewatering Considerations

Depending on the depth of utilities to be installed at the site and the timing of construction, we anticipate some trenches may be below seasonal high groundwater levels. During the winter months, October through May, static groundwater may be less than 8 feet below the ground surface. This level can change based on seasonal variation in precipitation. Dewatering may be necessary where significant groundwater is encountered. We recommend that earthwork activities, including utility trenching, occur during the drier summer months, June through September.

Where groundwater seepage levels within trench excavations exceeds levels that can be easily mitigated with conventional dewatering sumps/pumps, other methodology should be utilized. This may include reducing the open trench area, larger pumps, well points, or dewatering wells. Based on the time of year and the site-specific conditions encountered, additional and more specific recommendations can be provided. If dewatering volumes become significant, permits may be required for discharge. A dewatering design is not included in our scope of work or provided in this report.

Utility Trench Construction

Based on the level of groundwater and moisture content of the site soils at the time of construction, it may be necessary to mitigate soft or wet soil conditions within the trench excavations and use a select granular backfill. If soft or wet soil conditions are encountered in the trench area or at the trench bottom, we recommend the follow mitigation options be considered:

- Geotextile fabric placed on the bottom of the trench and covered with the normal bedding material. A common geotextile used in this application is a US Fabrics US200 (or an approved equivalent), commonly referred to as a Driveway Fabric.
- Pipe-sleds are commonly placed on the trench bottom where wet soft/wet soils are encountered. This typically requires a minor over-excavation to accommodate the thickness of the sled.
- Similar to pipe-sleds, quarry spall wraps consist of approximately 12 inches of 2- to 4-inch quarry spalls (crushed rock) placed on and wrapped with a geotextile fabric. A specific fabric type is determined at the time of excavation based on the ground conditions. Bedding material is typically placed above the spalls and fabric.
- Over-excavate and replace, typically with a select sand and gravel or crushed rock with a fabric wrap. The thickness of select material and type of fabric are determined based on ground conditions.

The goal of ground improvement for utility support is to provide sound support for the utility pipe and minimize potential differential settlement, which could result in deflections, “bellies” or depressions in the utility pipe. At the same time, the supporting media should not add significant additional weight relative to the soil it replaces, which could induce additional settlement.

Erosion Control

Erosion protection measures should be in place prior to beginning construction or earthwork activities. Erosion hazards can be mitigated by implementing appropriate Best Management Practices outlined in the 2012 SWMMWW.

Wet Weather Earthwork Recommendations

In the Puget Sound area, wet weather generally begins about mid-October and continues through about May, although rainy periods could occur at any time of year. It is encouraged that earthwork be scheduled during the dry weather months of June through September. Most of the soils at the site contain sufficient fines to produce an unstable mixture when wet. Such soil is highly susceptible to changes in water content and tends to become unstable and impossible to proof-roll and compact if the moisture content exceeds the optimum.

In addition, during wet weather months, the groundwater levels could increase, resulting in seepage into site excavations. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, construction traffic, and handling of wet soil. However, should wet weather/wet condition earthwork be unavoidable, the following recommendations are provided:

- The ground surface in and surrounding the construction area should be sloped to promote positive drainage away from work areas, structures, and property lines, and to prevent ponding of water.
- Work areas or slopes should be covered with plastic when not being worked. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill could be accomplished on the same day. The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with a backhoe, or equivalent, and locate them so that equipment does not pass over the excavated area. Thus, subgrade disturbance caused by equipment traffic would be minimized.
- Fill material should consist of clean, well-graded sand and gravel, of which not more than 5 percent fines by dry weight passes the No. 200 mesh sieve, based on wet sieving the fraction passing the ¾-inch mesh sieve. The gravel content should range from between 20 and 50 percent retained on a No. 4 mesh sieve. The fines should be non-plastic.
- No exposed soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.
- In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil (see soil gradation requirements in the "**Structural Fill**" section of this report).
- Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer (or representative) experienced in wet weather/wet condition earthwork to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
- Grading and earthwork should not be completed during periods of heavy, continuous rainfall.

We recommend that the above requirements for wet weather/wet condition earthwork be incorporated into the contract specifications.

Additional Services and Construction Observation

Additionally, we recommend GeoResouces be retained to observe the geotechnical aspects of construction, particularly the ground improvements, fill placement and compaction, and drainage activities, including the drainage facilities. This observation would allow us to verify the subsurface conditions as they are exposed during construction and to determine that work is accomplished in accordance with our recommendations. If conditions encountered during construction differ from those anticipated, we can provide recommendations for the conditions encountered.

LIMITATIONS

We have prepared this report for Larson Automotive, Castino Architecture, Momentum Civil Engineering Consultants, and other members of the design team for use in evaluating a portion of this project. The data used in preparing this report and this report should be provided to prospective contractors. Our report, conclusions and interpretations are based on data from others and limited site reconnaissance, and should not be construed as a warranty of the subsurface conditions.

Variations in subsurface conditions are possible between the explorations and may also occur with time. A contingency for unanticipated conditions should be included in the budget and schedule. Sufficient monitoring, testing and consultation should be provided by our firm during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork and foundation installation activities comply with contract plans and specifications.

The scope of our services does not include services related to environmental remediation and construction safety precautions. Our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

If there are any changes in the loads, grades, locations, configurations or type of facilities to be constructed, the conclusions and recommendations presented in this report may not be fully applicable. If such changes are made, we should be given the opportunity to review our recommendations and provide written modifications or verifications, as appropriate.

◆ ◆ ◆

We have appreciated working for you on this project. Please do not hesitate to call at your earliest convenience if you have any questions or comments.

Respectfully submitted,
GeoResources, LLC

NEIL ANDREW FERGUSON
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8/25/2021

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Project Geotechnical Engineer

Engineering Geologist
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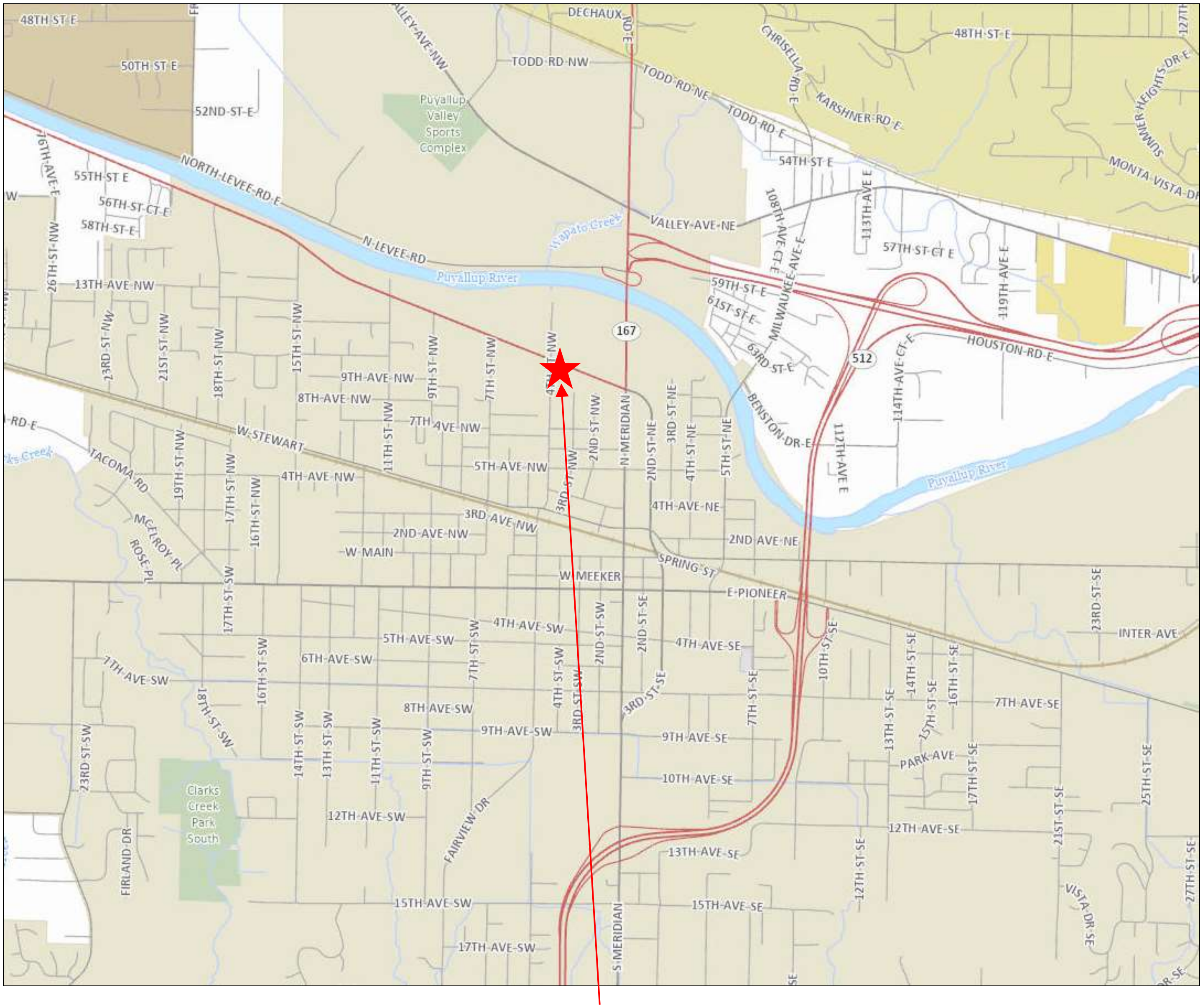
Senior Geotechnical Engineer

NAF:KSS:EWH/naf
Doc ID: LarsonAutomotive.LarsonJeep.RG
Attachments:
- Figure 1: Site Location Map
- Figure 2: Site Vicinity Map
- Figure 3: Site and Exploration Plan
- Figure 4: NRCS Soils Map
- Figure 5: Geologic Map
- Figure 6: Liquefaction Susceptibility Map
- Figure 7: Typical Over Excavation Detail
- Appendix A –Subsurface Explorations
- Appendix B – Laboratory Test Results
- Appendix C – Liquefaction Analysis



Approximate Site Location

Map created from Pierce County Public GIS (https://matterhornwab.co.pierce.wa.us/publicgis/)

N
W E
S

Not to Scale

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Site Location Map

Proposed Commercial Development
300 River Road
Puyallup, Washington
PN: 0420214010

Doc ID: LarsonAutomotive.Jeep.F	May 2021	Figure 1

Notes:

⊕ Approximate location and number of boring

N
W E
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Scale: 1" = 60'

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Site & Exploration Plan

Proposed Commercial Development
300 River Road
Puyallup, Washington
PN: 0420214010, 0420214027, 0420281154

Doc ID: LarsonAutomotive.Jeep.F	May 2021	Figure 2

Approximate Site Location

Map created from Pierce County Public GIS (https://matterhornwab.co.pierce.wa.us/publicgis/)

Not to Scale

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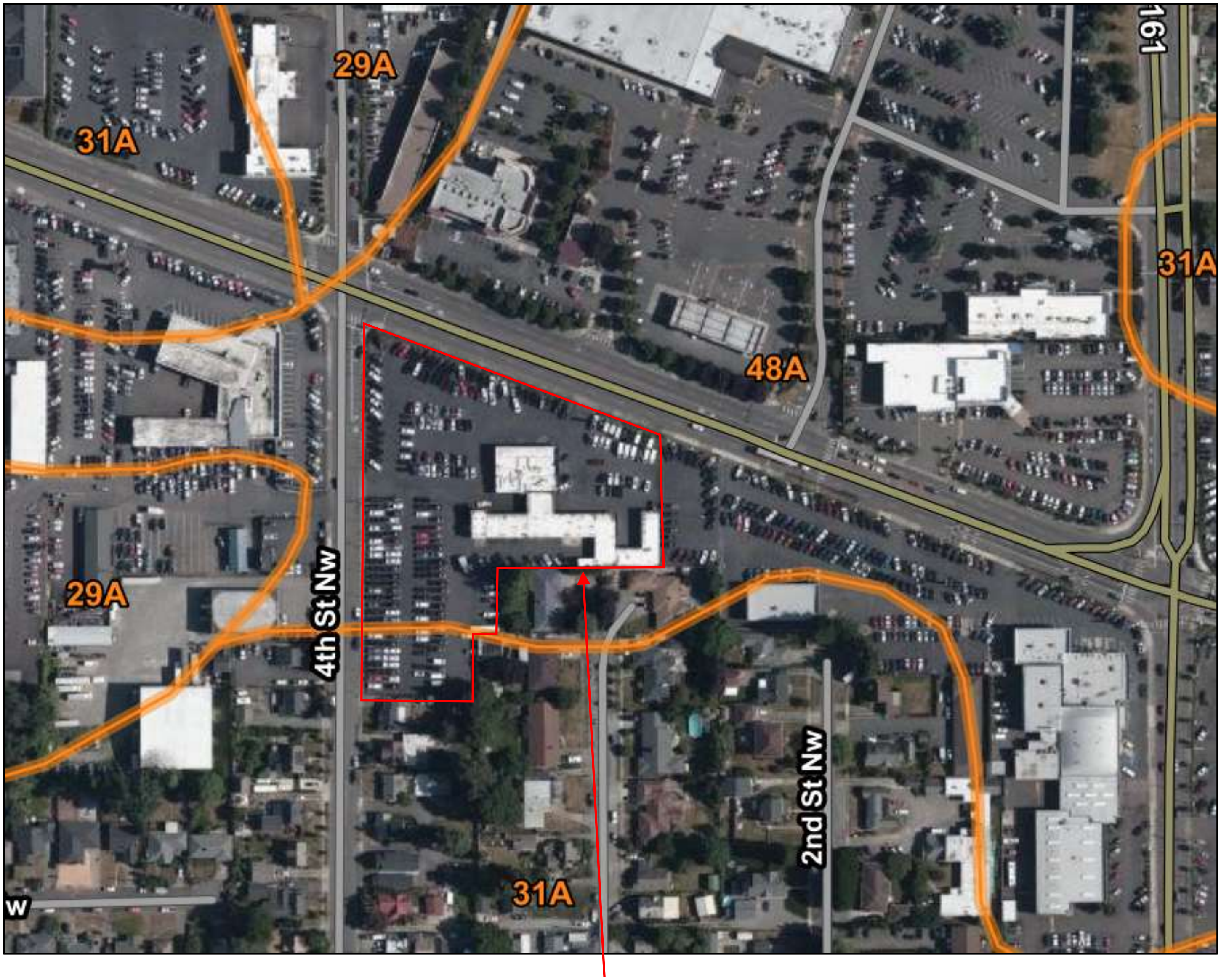
Site Vicinity Map

Proposed Commercial Development
300 River Road
Puyallup, Washington
PN: 0420214010, 0420214027, 0420281154

Doc ID: LarsonAutomotive.Jeep.F	May 2021	Figure 3



Approximate Site Location

Map created from Web Soil Survey (http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx)

Soil Type	Soil Name	Parent Material	Slopes	Erosion Hazard	Hydrologic Soils Group
31A	Puyallup fine sandy loam	Alluvium	0 to 3	Slight	A
48A	Xerothrents fill	Artificial fill and/or dredge spoils	0 to 1	Slight	N/A

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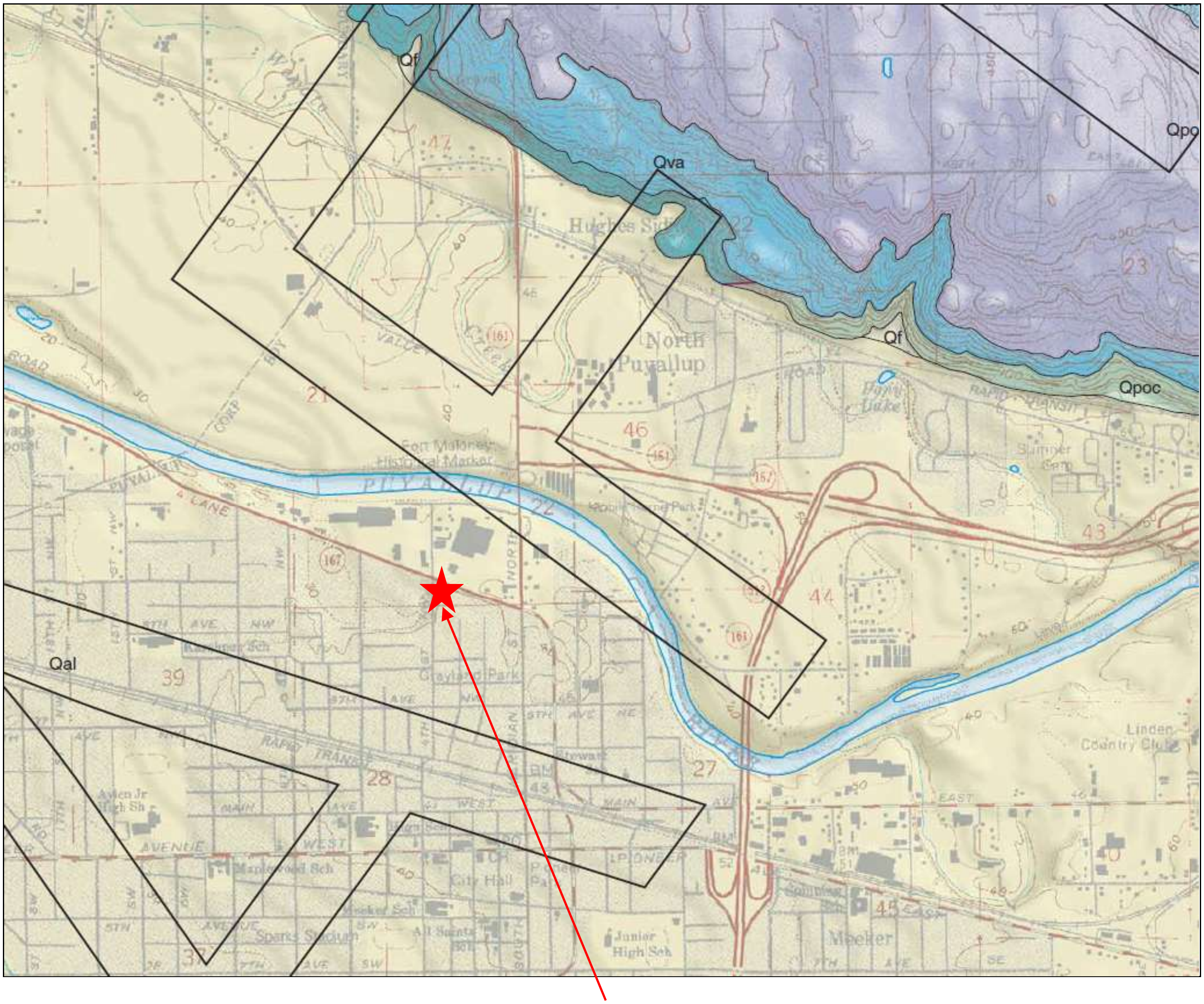
NRCS Soils Map

Proposed Commercial Development
300 River Road
Puyallup, Washington
PN: 0420214010, 0420214027, 0420281154

Doc ID: LarsonAutomotive.Jeep.F	May 2021	Figure 4



Approximate Site Location

An excerpt from the draft the *Geologic Map of the Puyallup 7.5-minute Quadrangle, Washington* by K.G. Troost (in review)

Qal	Alluvium

Not to Scale

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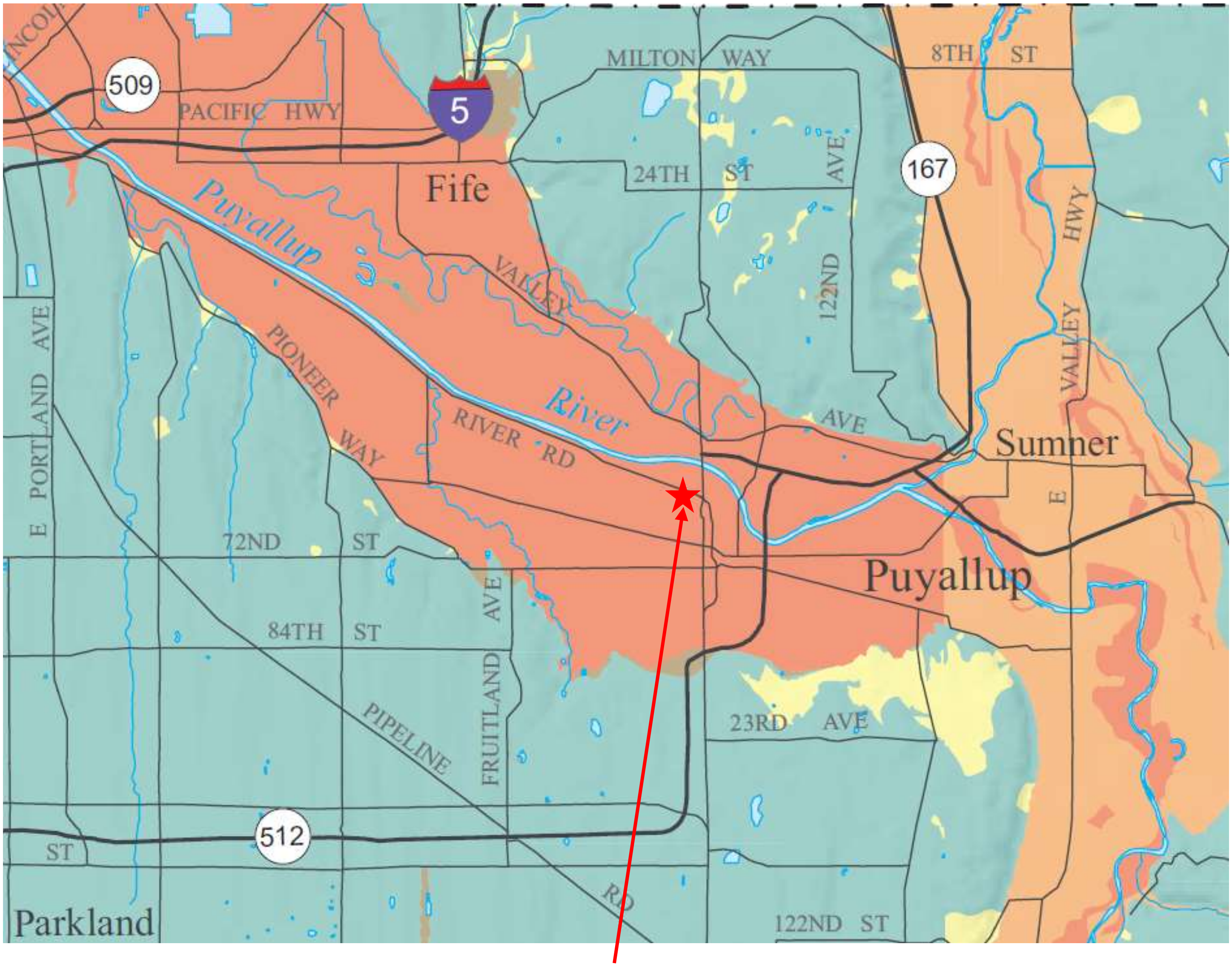
Geologic Map

Proposed Commercial Development
300 River Road
Puyallup, Washington
PN: 0420214010, 0420214027, 0420281154

Doc ID: LarsonAutomotive.Jeep.F	May 2021	Figure 5



Approximate Site Location

An excerpt from the *Liquefaction Susceptibility Map of Pierce County, Washington*
by Palmer et. Al., (September 2004)

- 🟥 Liquefaction susceptibility: HIGH
- 🟧 Liquefaction susceptibility: MODERATE to HIGH
- 🟧 Liquefaction susceptibility: MODERATE
- 🟨 Liquefaction susceptibility: LOW to MODERATE
- 🟩 Liquefaction susceptibility: LOW
- 🟩 Liquefaction susceptibility: VERY LOW to LOW
- 🟦 Liquefaction susceptibility: VERY LOW
- ▨ Bedrock
- 🟫 Peat deposit — Peat is not susceptible to liquefaction but may undergo permanent displacement or loss of strength as a result of earthquake shaking.
- 🟦 Water
- ☐ Ice
- ⬚ Extent of detailed study

This explanation is standardized for this series of county-based liquefaction maps; some categories may not appear on this map.

Not to Scale

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Liquefaction Susceptibility Map

Proposed Commercial Development
300 River Road
Puyallup, Washington
PN: 0420214010, 0420214027, 0420281154

Doc ID: LarsonAutomotive.Jeep.F	May 2021	Figure 6

Appendix A

Subsurface Explorations

SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS More than 50% Retained on No. 200 Sieve	GRAVEL More than 50% Of Coarse Fraction Retained on No. 4 Sieve	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND More than 50% Of Coarse Fraction Passes No. 4 Sieve	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS More than 50% Passes No. 200 Sieve	SILT AND CLAY Liquid Limit Less than 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY Liquid Limit 50 or more	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1. Field classification is based on visual examination of soil in general accordance with ASTM D2488-90.
2. Soil classification using laboratory tests is based on ASTM D2487-90.
3. Description of soil density or consistency are based on interpretation of blow count data, visual appearance of soils, and or test data.

SOIL MOISTURE MODIFIERS:

Dry-	Absence of moisture, dry to the touch
Moist-	Damp, but no visible water
Wet-	Visible free water or saturated, usually soil is obtained from below water table

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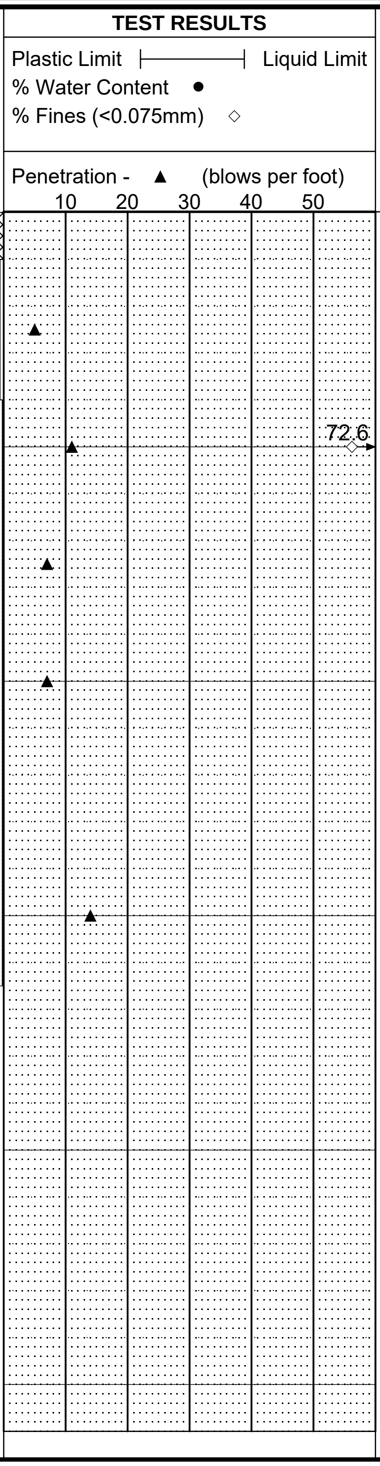
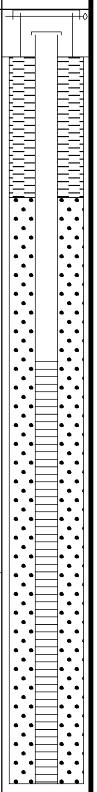
Unified Soils Classification System
Proposed Commercial Development
300 River Road
Puyallup, Washington
PN: 0420214010, 0420214027, 0420281154

Doc ID: LarsonAutomotive.Jeep.F | May 2021 | Figure A-1

TOTAL DEPTH: 16.5
DRILLING METHOD: Hollow-stem auger
LOGGED BY: NAF
TOP ELEVATION: 43
DRILLING COMPANY: Boretec
HAMMER TYPE: Cathead
LATITUDE:
DRILL RIG: TD 85
HAMMER WEIGHT: 140 lb
LONGITUDE:
NOTES:

Depth	Elevation	SOIL DESCRIPTION	DRILLING NOTES	Sample	Blow Count	Ground Water
0		Asphalt over sand and gravel (fill)				
		Tan mottled sandy SILT (medium stiff to stiff, moist) (alluvium) (ML)				
	40			1	3 2 3	
4		Gray silty fine SAND with interbedded mottled tan sandy SILT (medium dense, moist becomes wet at 11 feet) (alluvium) (SM/ML)				
				2	2 4 7	
	36					
8				3	2 3 4	
	32			4	4 2 5	
12						▽
	28			5	7 7 7	
16						
		Bottom of Boring Completed4/23/21				
	24					
20						
	20					
24						





NOTES
1. Refer to log key for definition of symbols, abbreviations and codes
2. USCS designation is based on visual manual classification and selected lab testing
3. Groundwater level, if indicated, is for the date shown and may vary
4. N.E. = Not Encountered

Proposed Commercial Development
300 River Rd
Puyallup, WA

LOG OF BORING B-4

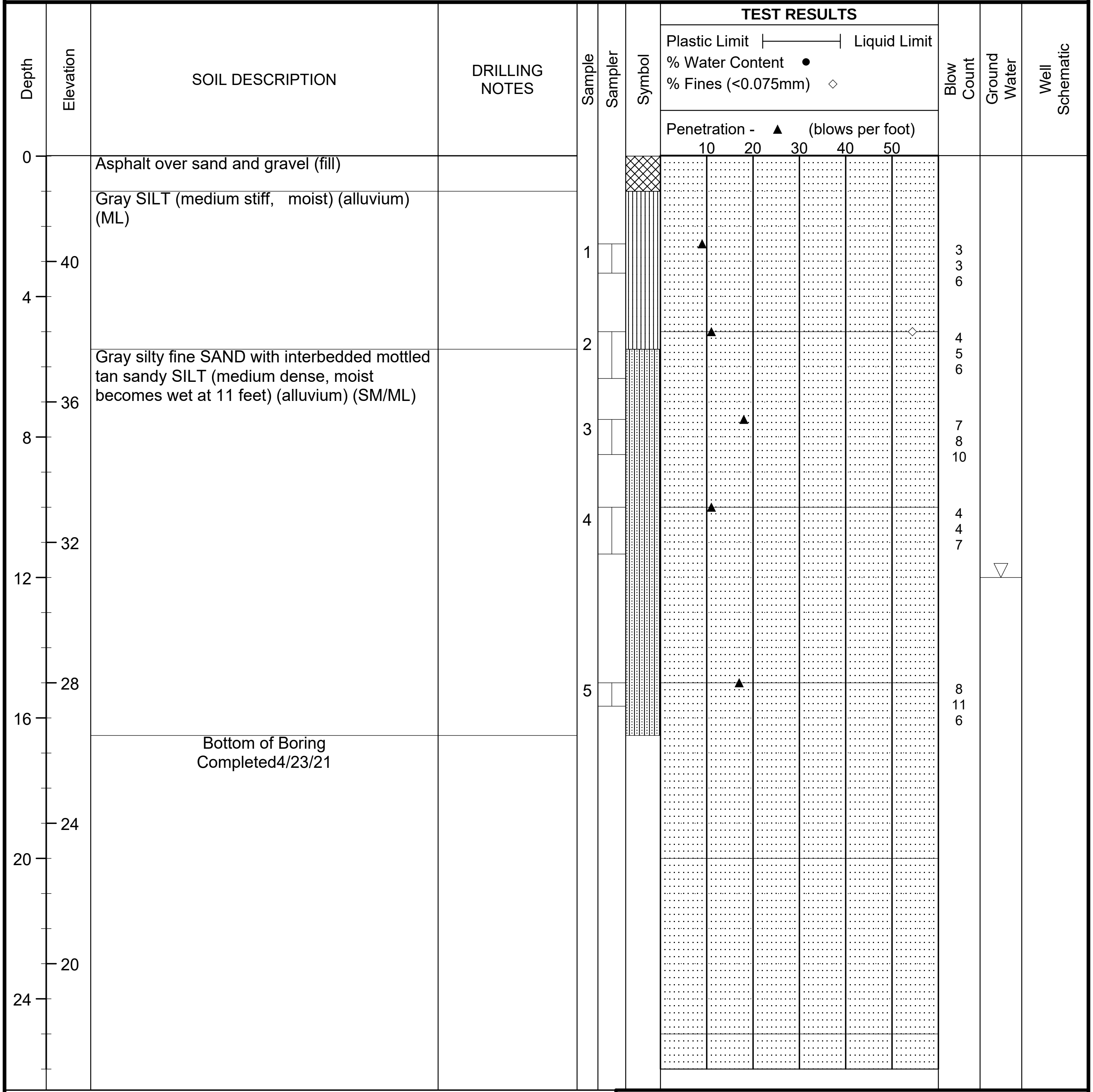
JOB: Larson.Jeep

Sheet of 5

GeoResources, LLC

FIG. A-5

TOTAL DEPTH: 16.5
DRILLING METHOD: Hollow-stem auger
LOGGED BY: NAF
TOP ELEVATION: 43
DRILLING COMPANY: Boretec
HAMMER TYPE: Cathead
LATITUDE:
DRILL RIG: TD 85
HAMMER WEIGHT: 140 lb
LONGITUDE:
NOTES:



Asphalt over sand and gravel (fill)

Gray SILT (medium stiff, moist) (alluvium) (ML)

Gray silty fine SAND with interbedded mottled tan sandy SILT (medium dense, moist becomes wet at 11 feet) (alluvium) (SM/ML)

Bottom of Boring
Completed4/23/21

Sample	Blow Count
1	3 3 6
2	4 5 6
3	7 8 10
4	4 4 7
5	8 11 6

NOTES
1. Refer to log key for definition of symbols, abbreviations and codes
2. USCS designation is based on visual manual classification
   and selected lab testing
3. Groundwater level, if indicated, is for the date shown and may vary
4. N.E. = Not Encountered

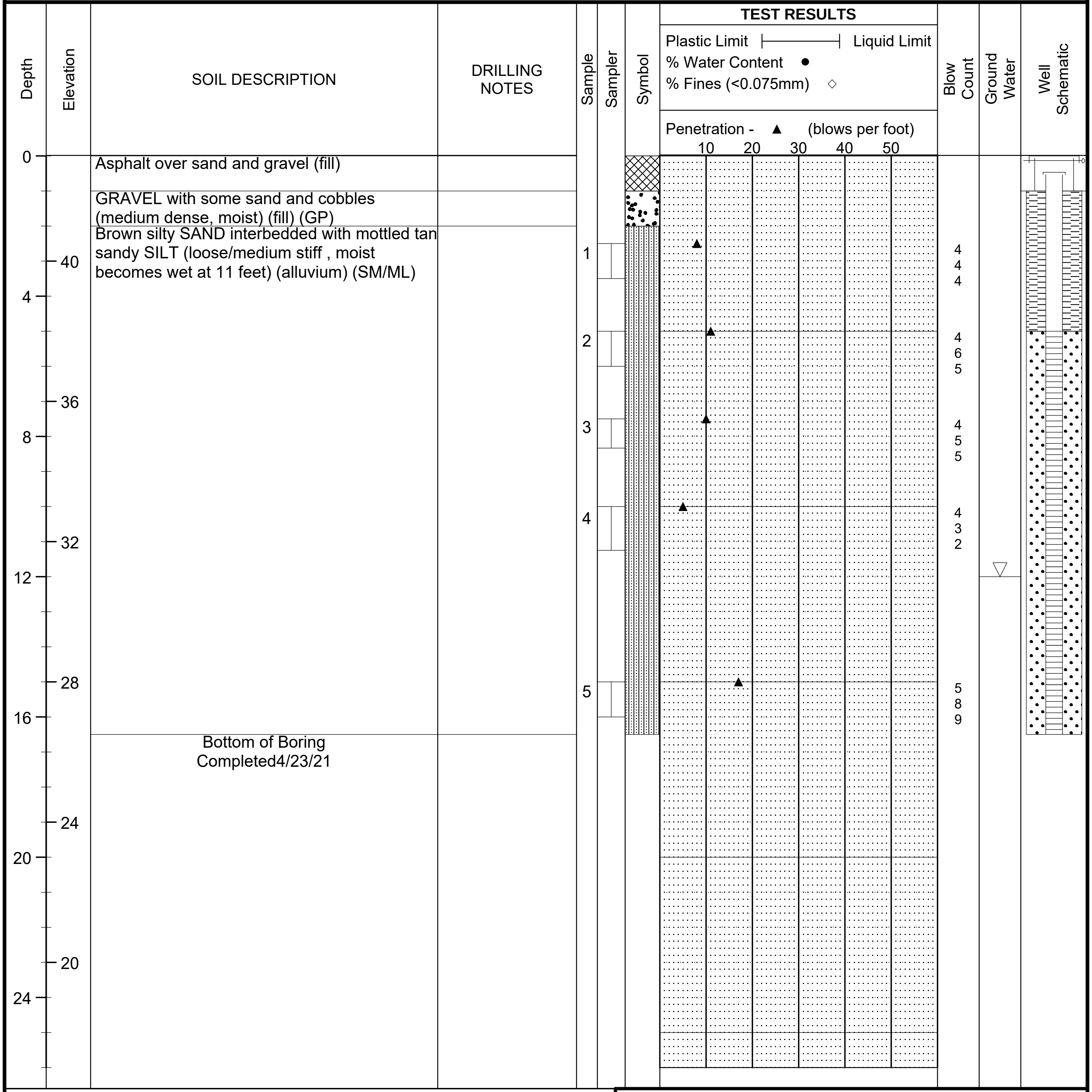
Proposed Commercial Development
300 River Rd
Puyallup, WA

LOG OF BORING B-3

JOB: Larson.Jeep
Sheet of 5

GeoResources, LLC
FIG. A-4

TOTAL DEPTH: 16.5
DRILLING METHOD: Hollow-stem auger
LOGGED BY: NAF
TOP ELEVATION: 43
DRILLING COMPANY: Boretec
HAMMER TYPE: Cathead
LATITUDE:
DRILL RIG: TD 85
HAMMER WEIGHT: 140 lb
LONGITUDE:
NOTES:



SOIL DESCRIPTION	DRILLING NOTES	Sample	Blow Count
Asphalt over sand and gravel (fill)			
GRAVEL with some sand and cobbles (medium dense, moist) (fill) (GP)			
Brown silty SAND interbedded with mottled tan sandy SILT (loose/medium stiff , moist becomes wet at 11 feet) (alluvium) (SM/ML)		1	4 4 4
		2	4 6 5
		3	4 5 5
		4	4 3 2
		5	5 8 9
Bottom of Boring Completed4/23/21			

NOTES
1. Refer to log key for definition of symbols, abbreviations and codes
2. USCS designation is based on visual manual classification and selected lab testing
3. Groundwater level, if indicated, is for the date shown and may vary
4. N.E. = Not Encountered

Proposed Commercial Development
300 River Rd
Puyallup, WA

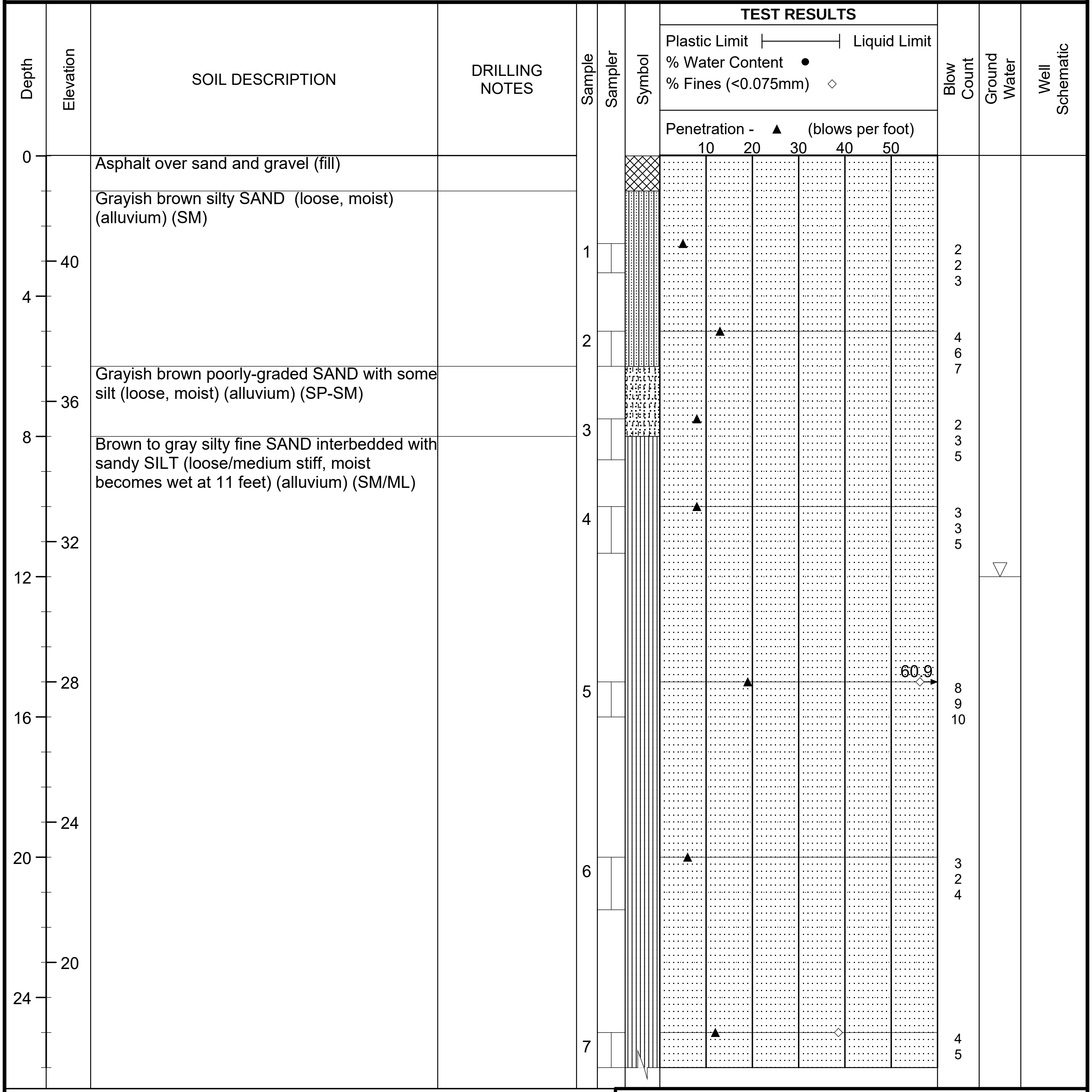
LOG OF BORING B-2

JOB: Larson.Jeep
Sheet of 5

GeoResources, LLC

FIG. A-3

TOTAL DEPTH: 51.5
TOP ELEVATION: 43
LATITUDE:
LONGITUDE:
DRILLING METHOD: Hollow-stem auger
DRILLING COMPANY: Boretec
DRILL RIG: TD 85
NOTES:
LOGGED BY: NAF
HAMMER TYPE: Cathead
HAMMER WEIGHT: 140 lb



Asphalt over sand and gravel (fill)

Grayish brown silty SAND (loose, moist) (alluvium) (SM)

Grayish brown poorly-graded SAND with some silt (loose, moist) (alluvium) (SP-SM)

Brown to gray silty fine SAND interbedded with sandy SILT (loose/medium stiff, moist becomes wet at 11 feet) (alluvium) (SM/ML)

NOTES

1. Refer to log key for definition of symbols, abbreviations and codes
2. USCS designation is based on visual manual classification and selected lab testing
3. Groundwater level, if indicated, is for the date shown and may vary
4. N.E. = Not Encountered

Proposed Commercial Development
300 River Rd
Puyallup, WA

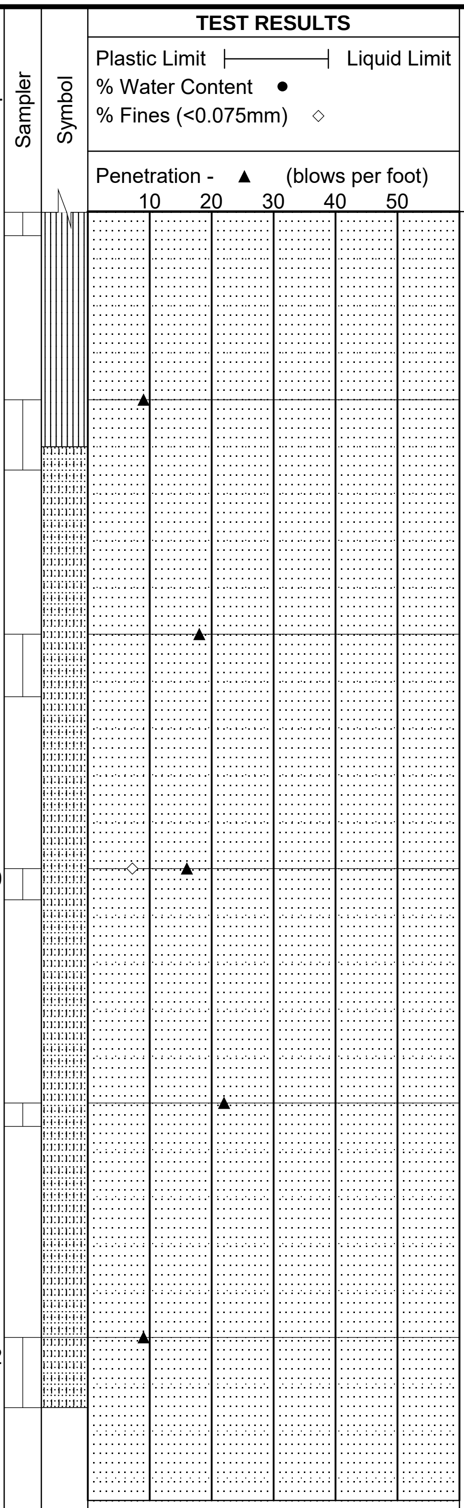
LOG OF BORING B-1

JOB: Larson.Jeep *Sheet of 5*

GeoResources, LLC **FIG. A-2**

TOTAL DEPTH: 51.5
TOP ELEVATION: 43
LATITUDE:
LONGITUDE:
DRILLING METHOD: Hollow-stem auger
DRILLING COMPANY: Boretec
DRILL RIG: TD 85
NOTES:
LOGGED BY: NAF
HAMMER TYPE: Cathead
HAMMER WEIGHT: 140 lb

Depth	Elevation	SOIL DESCRIPTION	DRILLING NOTES	Sample	Blow Count	Ground Water	Well Schematic
					7		
28	16						
				8	2 3 6		
32	12	Brown well-graded SAND with some silt and gravel (medium dense, wet) (coarse-grained alluvium) (SW-SM)					
36	8			9	13 8 10		
40	4			10	11 7 9		
44	0			11	15 13 9		
48	-4						
		Grayish brown silty SAND (loose, wet) (SM)		12	4 4 5		
52	-8	Bottom of Boring Completed4/23/21					



NOTES
1. Refer to log key for definition of symbols, abbreviations and codes
2. USCS designation is based on visual manual classification and selected lab testing
3. Groundwater level, if indicated, is for the date shown and may vary
4. N.E. = Not Encountered

Proposed Commercial Development
300 River Rd
Puyallup, WA

LOG OF BORING B-1

JOB: Larson.Jeep

Sheet of 5

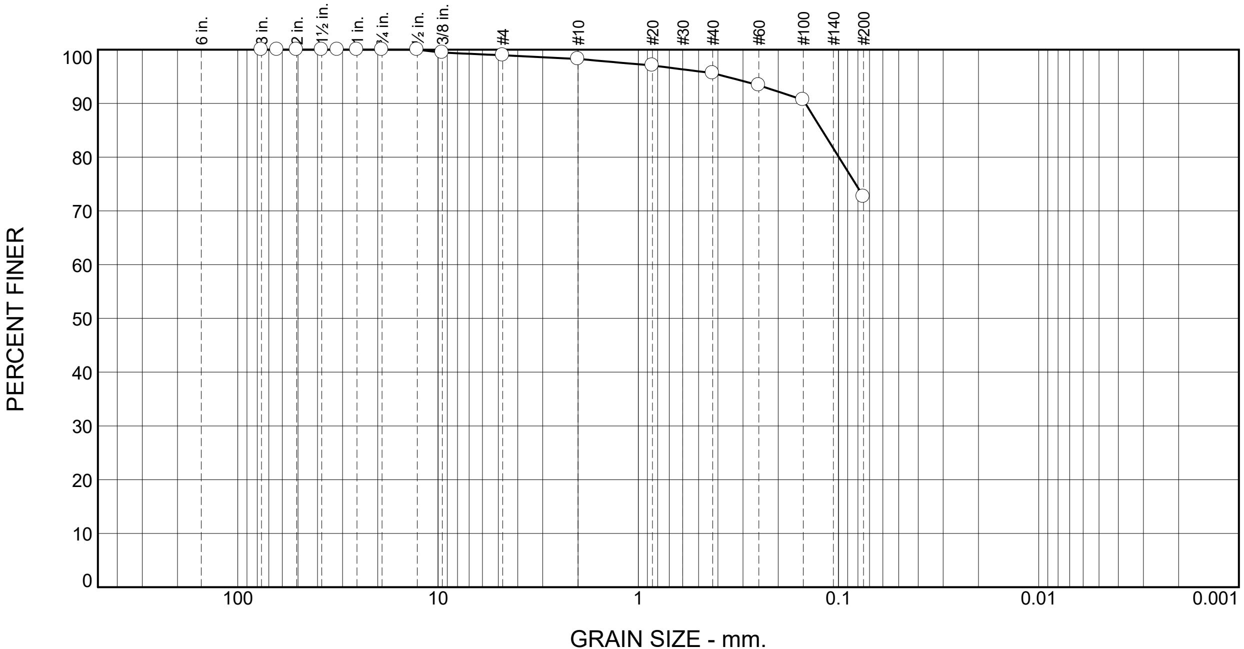
GeoResources, LLC

FIG. A-2

Appendix B

Laboratory Test Results

Particle Size Distribution Report



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	1.7	2.7	23.0	72.6	

Test Results (ASTM D 6913 & ASTM C 117)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3.0	100.0		
2.5	100.0		
2.0	100.0		
1.5	100.0		
1.25	100.0		
1	100.0		
.75	100.0		
.5	100.0		
0.375	99.5		
#4	99.0		
#10	98.3		
#20	97.1		
#40	95.6		
#60	93.4		
#100	90.6		
#200	72.6		

* (no specification provided)

Material Description

silt with sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.1463 D_{85}= 0.1207 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Natural Moisture: 28.2%

Date Received: 4/23/21 **Date Tested:** 4/27/21
Tested By: NAF/CJB
Checked By: KSS
Title: PM

Source of Sample: B-4 **Depth:** 5 **Date Sampled:** 4/23/21
Sample Number: 2

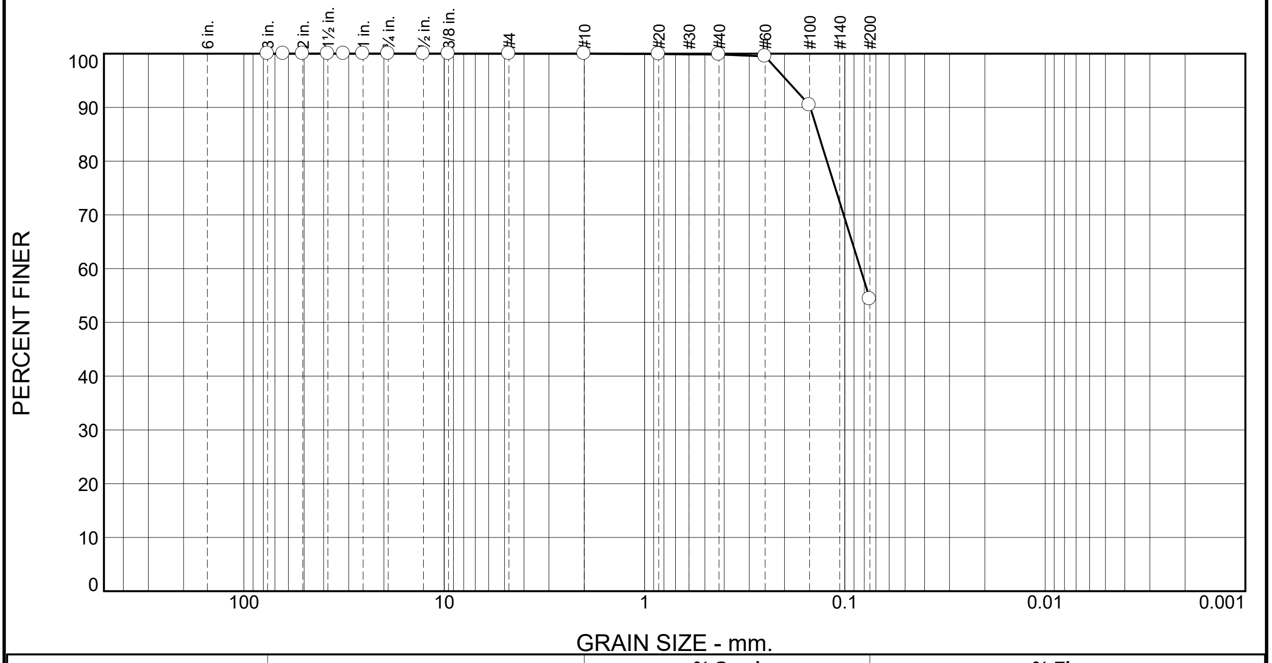
GeoResources, LLC

Fife, WA

Client: Larson Automotive
Project: Proposed Commercial Development
Project No: Larson.Jeep **Figure**

Tested By: ______________________ **Checked By:** ______________________

Particle Size Distribution Report



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	0.0	0.2	45.4	54.4	

Test Results (ASTM D 6913 & ASTM C 117)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3.0	100.0		
2.5	100.0		
2.0	100.0		
1.5	100.0		
1.25	100.0		
1	100.0		
.75	100.0		
.5	100.0		
0.375	100.0		
#4	100.0		
#10	100.0		
#20	99.9		
#40	99.8		
#60	99.5		
#100	90.4		
#200	54.4		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.1488 D_{85}= 0.1352 D_{60}= 0.0836
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Natural Moisture: 17.4%

Date Received: 4/23/21 **Date Tested:** 4/27/21
Tested By: NAF/CJB
Checked By: KSS
Title: PM

Source of Sample: B-3 **Depth:** 5 **Date Sampled:** 4/23/21
Sample Number: 2

GeoResources, LLC

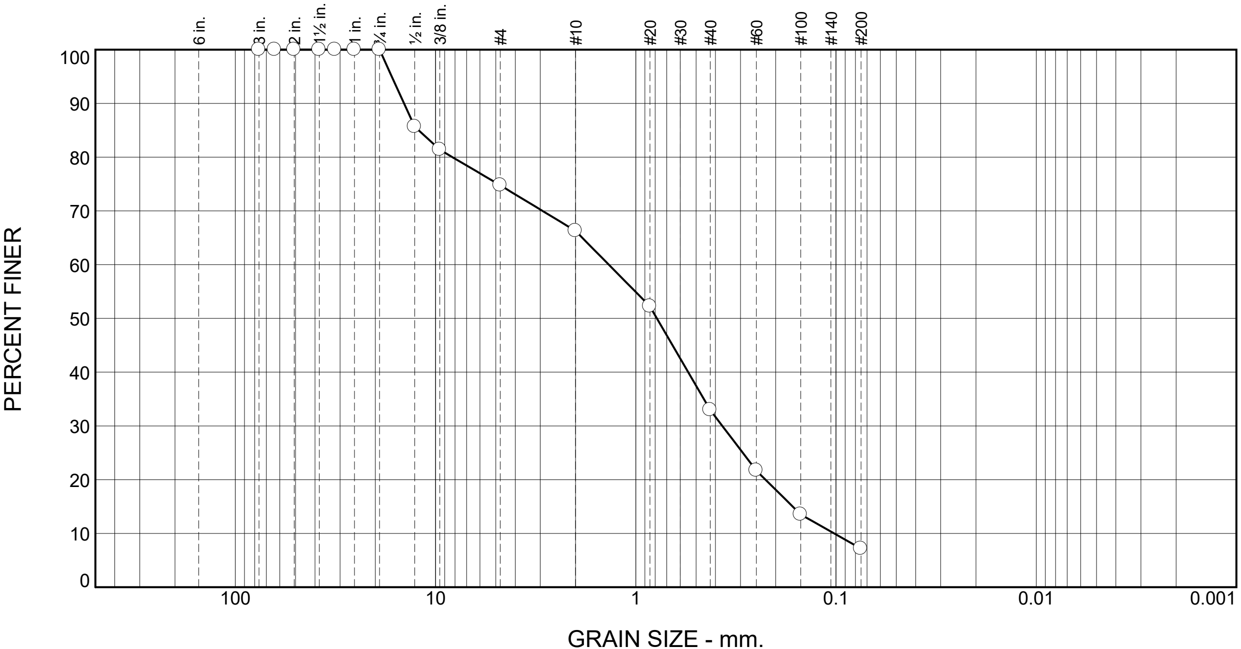
Fife, WA

Client: Larson Automotive
Project: Proposed Commercial Development
Project No: Larson.Jeep **Figure**

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical samples.

Tested By: ____________________ **Checked By:** ____________________

Particle Size Distribution Report



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	33.7	33.3	25.8	7.2	

Test Results (ASTM D 6913 & ASTM C 117)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3	100.0		
2.5	100.0		
2.0	100.0		
1.5	100.0		
1.25	100.0		
1	100.0		
.75	100.0		
.5	85.7		
0.375	81.4		
#4	74.8		
#10	66.3		
#20	52.2		
#40	33.0		
#60	21.7		
#100	13.5		
#200	7.2		

* (no specification provided)

Material Description

well-graded sand with silt and gravel

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SW-SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 14.3597	D_{85}= 12.1511	D_{60}= 1.3636
D_{50}= 0.7841	D_{30}= 0.3696	D_{15}= 0.1645
D_{10}= 0.1022	C_u= 13.34	C_c= 0.98

Remarks

Natural Moisture: 13.3%

Date Received: 4/23/21 **Date Tested:** 4/28/21

Tested By: NAF/CJB

Checked By: KSS

Title: PM

Source of Sample: B-1 **Depth:** 40 **Date Sampled:** 4/23/21

Sample Number: 10

GeoResources, LLC

Fife, WA

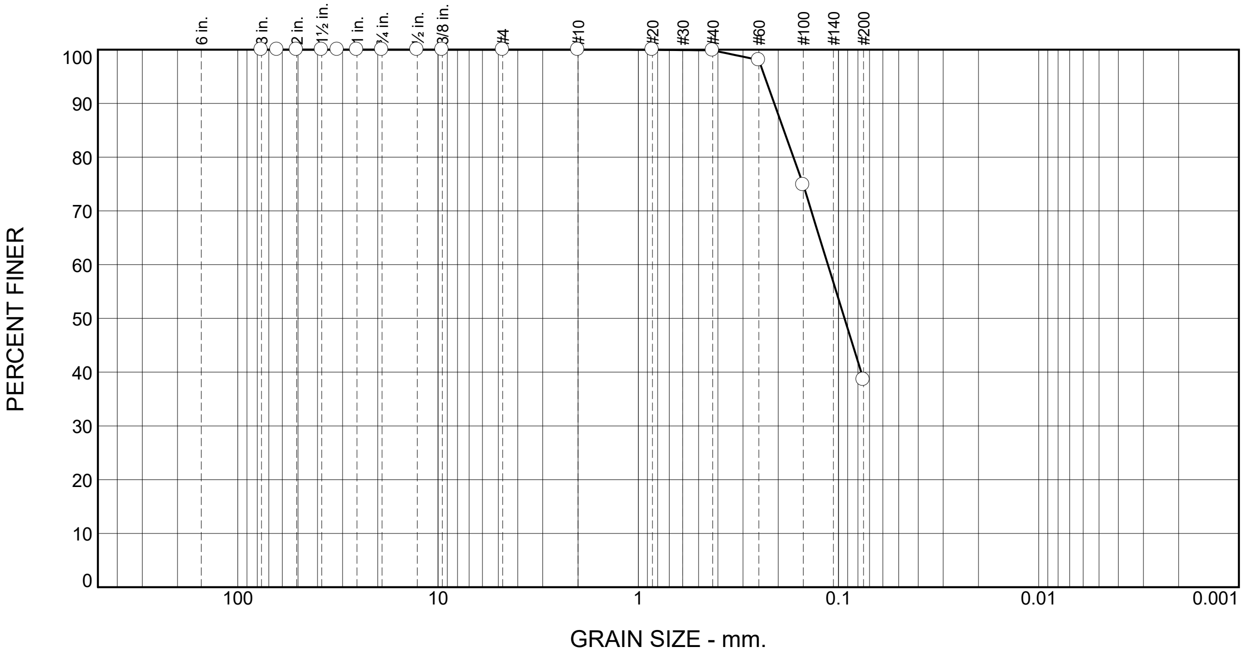
Client: Larson Automotive

Project: Proposed Commercial Development

Project No: Larson.Jeep **Figure**

Tested By: ____________________________ **Checked By:** ____________________________

Particle Size Distribution Report



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	0.0	0.1	61.3	38.6	

Test Results (ASTM D 6913 & ASTM C 117)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3.0	100.0		
2.5	100.0		
2.0	100.0		
1.5	100.0		
1.25	100.0		
1	100.0		
.75	100.0		
.5	100.0		
0.375	100.0		
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.9		
#60	98.1		
#100	74.9		
#200	38.6		

* (no specification provided)

Material Description

silty sand

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.2093 D_{85}= 0.1875 D_{60}= 0.1129
D_{50}= 0.0933 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Natural Moisture: 32.4%

Date Received: 4/23/21 **Date Tested:** 4/23/21

Tested By: NAF/CJB

Checked By: KSS

Title: PM

Source of Sample: B-1 **Depth:** 25 **Date Sampled:** 4/23/21
Sample Number: 7

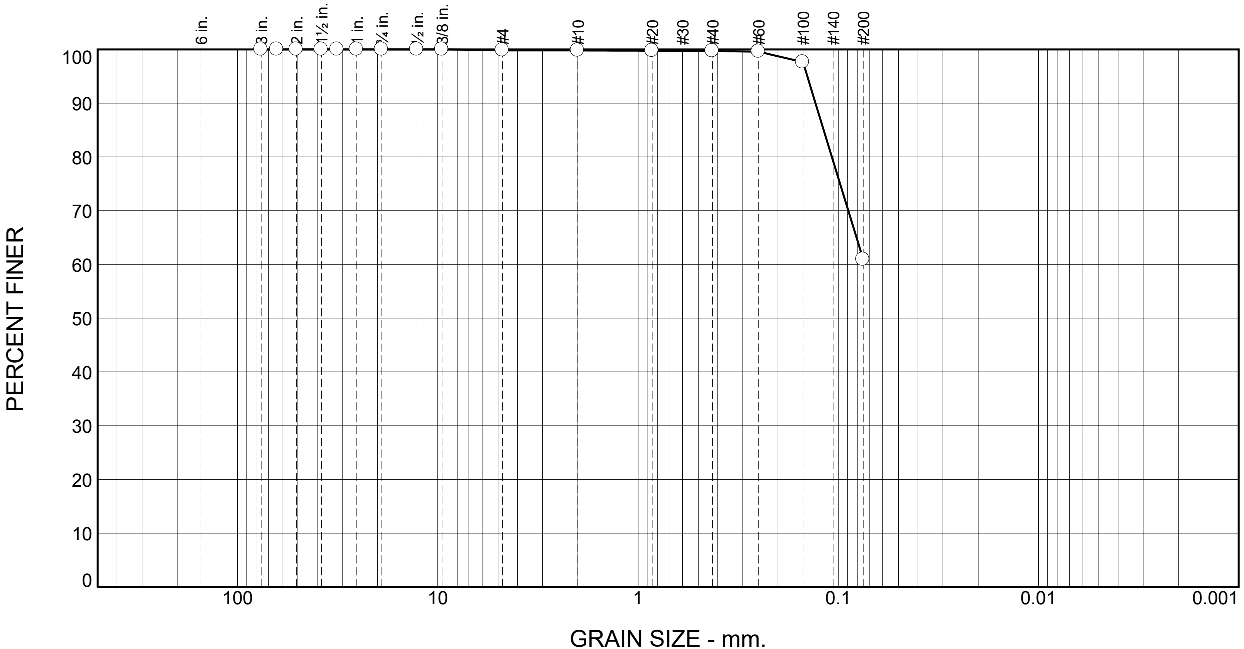
GeoResources, LLC

Fife, WA

Client: Larson Automotive
Project: Proposed Commercial Development
Project No: Larson.Jeep **Figure**

Tested By: ______________________________ **Checked By:** ______________________________

Particle Size Distribution Report



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0.0	0.2	0.1	38.8	60.9	

Test Results (ASTM D 6913 & ASTM C 117)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3.0	100.0		
2.5	100.0		
2.0	100.0		
1.5	100.0		
1.25	100.0		
1	100.0		
.75	100.0		
.5	100.0		
0.375	100.0		
#4	99.8		
#10	99.8		
#20	99.8		
#40	99.7		
#60	99.6		
#100	97.6		
#200	60.9		

* (no specification provided)

Material Description

sandy silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.1299	D_{85}= 0.1182	D_{60}=
D_{50}=	D_{30}=	D_{15}=
D_{10}=	C_u=	C_c=

Remarks

Natural Moisture: 30.75%

Date Received: 4/23/21 **Date Tested:** 4/28/21

Tested By: NAF/CJB

Checked By: KSS

Title: PM

Source of Sample: B-1 **Depth:** 15 **Date Sampled:** 4/23/21

Sample Number: 5

GeoResources, LLC

Fife, WA

Client: Larson Automotive

Project: Proposed Commercial Development

Project No: Larson.Jeep **Figure**

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical samples.

Tested By: ____________________ **Checked By:** ____________________

Appendix C

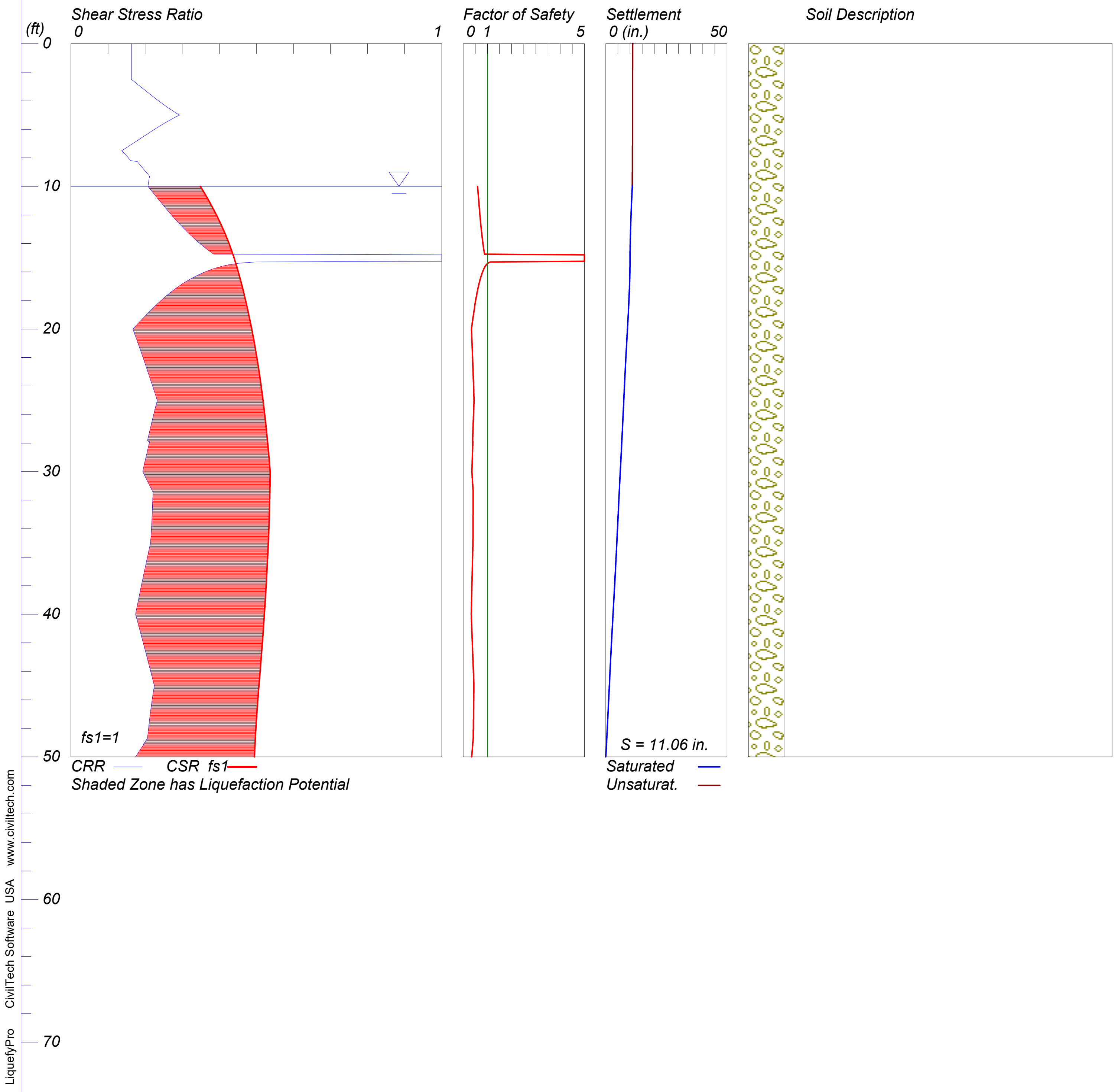
Liquefaction Analysis

LIQUEFACTION ANALYSIS

Larson Jeep

Hole No.=B-1 Water Depth=10 ft

Magnitude=7.2
Acceleration=0.55g



CivilTech Corporation

Figure C-1