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Geotechnical Engineering Report
Proposed Commercial Development
2315 Inter Ave
Puyallup, Washington
PN: 2105200140
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INTRODUCTION

This *Geotechnical Engineering Report* summarizes our site observations, subsurface explorations, laboratory testing and engineering analyses and provides geotechnical recommendations and design criteria for the proposed commercial development to be constructed at 2315 Inter Avenue in Puyallup, Washington. We prepared a *Soils Report* for the property on April 8, 2022 to address stormwater requirement per the City of Puyallup (the City). We understand that the structural engineer is requesting geotechnical recommendations per the 2018 International Building Code (IBC). The approximate site location is shown on the attached Site Location Map, Figure 1.

Our understanding of this project is based on our email correspondence with you, and representatives from Larson & Associates (Larson); our review of the provided Topographic Survey by Larson dated October 7, 2021; our understanding of the City of Puyallup development codes; and our experience in the area.

We understand the site consists of a single tax parcel and is currently developed with an existing building, paved parking areas, and utilities. We further understand that you propose to construct a new building that will add or replace about 5,000 square feet of hard surfacing. According to site plans provided by Castino Architecture the building will consist of an approximately 5,000 square foot metal framed warehouse.

SCOPE

The scope of our services was to evaluate the surface and subsurface conditions across the site as a basis for developing geotechnical recommendations and conclusions for construction of the proposed building. Specifically, the scope of services will include the following:

1. Reviewing the available geologic, hydrogeologic, and geotechnical data for the site area;
2. Reviewing subsurface conditions encountered in nearby explorations;
3. Describing surface and subsurface conditions, including soil type and depth to groundwater;

4. Providing seismic design parameters, including 2018 IBC site class;
5. Providing recommendations and design criteria for foundation and floor slab support, including conventional spread foundations, temporary shoring and subgrade retaining walls with lateral earth pressures;
6. Providing geotechnical recommendations for earthwork and grading activities including site preparation, subgrade preparation, fill placement criteria (including hillside grading), suitability of on-site soils for use as structural fill, and temporary and permanent cut and fill slopes;
7. Providing recommendations for erosion and sediment control during wet weather grading and construction; and
8. Preparing a *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 summarized in our *Proposal for Geotechnical Engineering Services* dated February 15, 2023. We received written authorization to proceed with our scope of work from you on February 18, 2023.

SITE CONDITIONS

Surface Conditions

As stated in our original stormwater report, the site is located at 2315 Inter Avenue in Puyallup, Washington within an area of existing commercial development. Based on information obtained from the Pierce County Public GIS website, the site is generally rectangular in shape, measures approximately 200 feet wide (east to west) by 400 to 405 feet long (north to south) and encompasses about 1.86 acres. The site is bounded by Inter Avenue to the south, single-family residence to the east, and by existing commercial development to the north and west.

The site generally flat with less than 2 feet of topographic relief. The vegetation in the area of the proposed development had been generally cleared and consisted of grasses, brambles, and other low lying native and invasive species. No areas of surficial erosion, seeps, or springs were observed at the time of our reconnaissance. Standing water was not observed in the existing pond/depressions on the northwest and portions of the site at the time of our December 2021 site visit. The existing site topography is shown on the Site Exploration Map, Figure 2.

Site Soils

The USDA Natural Resource Conservation Services (NRCS) Web Soil Survey maps the site as being underlain by Briscot loam soils (6A). A copy of the referenced NRCS Soils Map for the site area is included as Figure 3.

- *Briscot loam soils (6A)*: This soil is derived from alluvium, forms on slopes of 0 to 2 percent and has a “slight” potential for erosion when exposed. The upper, weathered soil horizons are listed in hydrologic soils group B, while the deeper soil horizons are listed in hydrologic soils group D.

Site Geology

The draft of the *Geologic Map of the Puyallup 7.5-Minute Quadrangle, Washington* (Troost et al.) maps the site and surrounding area as being underlain by alluvium (Qal). An excerpt of the above referenced map is included as Figure 4.

- *Alluvium (Qal)*: Alluvium generally consists of a well graded, lightly stratified mixture of silts and sands that may contain localized deposits of clay and gravel that were deposited by fluvial processes. The alluvial deposits are considered normally consolidated and generally have moderate strength and compressibility characteristics where undisturbed.

Subsurface Explorations

On December 21, 2021, we visited the site and monitored the excavation of three test pits to depths of 6.5 to 8 feet below the existing ground surface, one of which was completed as a PIT. We also reviewed Cone Penetrometer Test (CPT) data from a nearby site for which GeoResources prepared a geotechnical engineering report. A second nearby report that GeoResources was involved include the descriptive logs of two deep borings. Table 1, below, summarizes the approximate functional locations, surface elevations, and termination depths of our explorations.

TABLE 1:
APPROXIMATE LOCATIONS, ELEVATIONS, AND DEPTHS OF EXPLORATIONS

Exploration Number	Functional Location	Surface Elevation (feet)	Termination Depth (feet)	Termination Elevation[1] (feet)
TP-1	East portion of proposed development	60	6.5	53.5
TP-2	West portion of proposed development	60	8.0	52.0
PIT-1	Central portion of proposed development	60	5.0	55.0
CPT-01	Lat: 47.18003 Long: -122.27807	58	28.5	29.5
CPT-02	Lat: 47.18059 Long: -122.27805	58	30.8	27.2
B-1	1701 E Main Street	46	50.5	-4.5
B-2	1701 E Main Street	46	21.5	24.5

Notes:
1 = Surface elevation estimated from the provided by the Pierce County Public GIS contours based on NAVD 88

Test Pits:

The test pits were excavated by a small track-mounted excavator operated by a licensed earthwork contractor working for GeoResources. Soil densities presented on the logs were based on the difficulty of excavation and our experience. Representative soil samples obtained from the test pits were placed in sealed plastic containers and then taken to our laboratory for further examination and testing as deemed necessary. The test pits were then backfilled with the excavated soils and bucket tamped, but not otherwise compacted.

CPTs

The CPT were be completed using a track mounted rig operated by an independent firm working under subcontract to Georesources. The CPT was be pushed to depths of about 50 to 75 feet below existing grades. . This testing procedure involves pushing an electric piezocone into the soil with a hydraulic ram. The cone consisted of a standard design having a 60-degree tip apex, a 10-cm^2 projected area at the tip, a 150-cm^2 sleeve, and a porous element at the tip. The cone was advanced at a rate of approximately 2 cm per second, and the cone tip resistance (qT), sleeve friction (fs), and penetration pore water pressure (u2) were recorded in one inch increments during the test. As the penetrometer is pushed downward, the tip resistance, sleeve friction, and pore water pressure are measured electronically and plotted as a function of depth. Through interpretation and correlation, the resulting graphs can reveal soil types and groundwater levels, as well as the relative density of granular soils and the relative consistency of cohesive soils. After the CPT was completed, the exploration was backfilled with bentonite chips.

Borings

The nearby borings were drilled by a licensed drilling contractor operating a small track-mounted drill rig working under contract for GeoResources. 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: D-1586. 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 borings were backfilled by the driller in accordance with Washington State law.

The specific number, locations, and depths of our explorations were selected based on the configuration of the proposed development and were adjusted in the field based on consideration for underground utilities, existing site conditions, site access limitations and encountered stratigraphy. The subsurface explorations excavated as part of this evaluation indicate the subsurface conditions at specific locations only, as actual subsurface conditions can vary across the site. Furthermore, the nature and extent of such variation would not become evident until additional explorations are performed or until construction activities have begun. Based on our experience and extent of prior explorations in the area, it is our opinion that the soils encountered in the explorations are generally representative of the soils at the site.

The approximate locations and numbers of our test pits are shown on the attached Site Exploration Map, Figure 2. The soils encountered were visually classified in accordance with the Unified Soil Classification System (USCS) and ASTM D2488. The USCS is included in Appendix A as Figure A-1, while the descriptive logs of our test pits are included as Figure A-2.

We also reviewed soil logs from nearby projects. Two CPT explorations were performed about ¾ mile SW of the site, and two borings were performed about ¼ mile NW of the site.

Subsurface Conditions

At the locations of our explorations, we encountered generally uniform subsurface conditions that in our opinion confirmed the mapped stratigraphy at the site. In general, our test pit explorations encountered about 1.2 to 1.4 feet of brown topsoil in a loose, moist condition mantling about 3.8 to 4.2 feet of iron-oxide stained brown to dark grey silty sand in a loose, moist condition. These surficial soils were underlain by iron-oxide stained mottled dark grey to black silty sand in a loose and wet condition to the full depth explored. We interpret the soils encountered at the site to be consistent with alluvium deposits. Table 2 below summarizes the soils encountered in our explorations.

Deeper soils observed in the nearby CPTs encountered 1 to 1.5 feet of surficial muck / clays that mantled about 24 to 25 feet of variable sands with occasional 0.5 to 3 foot thick lenses of silt. Gravelly sands were encountered at depths of 25 to 27 feet below the ground surface and extending to the full depth explored, where the CPTs encountered refusal (defined as qT ≥ 400 tsf). The boring encountered several inches to approximately 2.5 feet of fill and angular gravel from the existing parking lot. Below these surficial soils, the borings encountered a medium sand in a loose to medium dense, moist to saturated condition to approximately 15 to 21 feet below the existing ground surface. These soils were underlain by a thin layer of dark grey sandy gravel in a medium dense, saturated condition. Boring B-2 terminated in these gravelly soils while below the gravel, boring B-1 encountered approximately 5 feet of dark grey silty fine sand in a saturated, medium dense condition. These upper soils appear to be consistent with alluvial deposits.

Laboratory Testing

Geotechnical laboratory tests were performed on select samples retrieved from the test pits to estimate index engineering properties of the soils encountered. Laboratory testing included visual soil classification per ASTM D2488 and ASTM D2489, moisture content determinations per ASTM D2216, and grain size analyses per ASTM D6913 standard procedures. The results of the laboratory tests are summarized below in Table 3 and graphical outputs are included in Appendix B.

TABLE 3:
LABORATORY TEST RESULTS FOR ON-SITE SOILS

Sample	Soil Type	Lab ID	Gravel Content (percent)	Sand Content (percent)	Silt/Clay Content (percent)	D10 Ratio (mm)
PIT-1, S-2, 5’	SM	103087	0	82.1	17.9	>0.075

Groundwater Conditions

Groundwater seepage was observed at the time of our explorations. It is our opinion that the groundwater encountered is indicative of a seasonal or fluctuating high perched groundwater table. Perched groundwater typically develops when the vertical infiltration of precipitation through a more permeable soil is slowed at depth by a deeper, less permeable soil type. We anticipate fluctuations in the local groundwater levels will occur in response to precipitation patterns, off-site construction activities, and site utilization. As such, water level observations made at the time of our field investigation may vary from those encountered during the construction phase.

ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our data review, site reconnaissance, subsurface explorations and our experience in the area, it is our opinion that the site is suitable for the proposed development. Pertinent conclusions and geotechnical recommendations regarding the design and construction of the proposed development are presented below.

Because of the potential for liquefaction at the site, we recommend that ground improvements or deep foundations be utilized in order to mitigate against potential seismically induced settlements. These options could include stiffening the upper site soils with non-liquefiable structural fill, aggregate piers, and pile foundations. Information regarding these methods are provided below.

Seismic Design

The site is located in the Puget Sound region of western Washington, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate at the Cascadia Subduction Zone (CSZ). This produces both intercrustal (between plates) and intracrustal (within a plate) earthquakes. In the following sections we discuss the design criteria and potential hazards associated with the regional seismicity.

Seismic Site Class

Based on our observations and the subsurface 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 documents and American Society of Civil Engineers (ASCE) standard 7-16 Chapter 20 Table 20.3-1. This is based on the reviewed SPT (Standard Penetration Test) and CPT data from the neighboring deep explorations, which we interpret to be representative for the subject site based on the geologic mapping.

However, per 20.3.1.1 of ASCE Chapter 20, if the period of the structure is less than 0.5 seconds, then Site Class “D” (default) can be used, which we have used in our analyses.

Design parameters

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 and 2008. We used the *ATC Hazard by Location* website to estimate seismic design parameters at the site. Table 4, below, summarizes the recommended design parameters.

TABLE 4:
2018 IBC PARAMETERS FOR DESIGN OF SEISMIC STRUCTURES

Spectral Response Acceleration (SRA) and Site Coefficients	Short Period
Mapped SRA	S_s = 1.259g
Site Coefficients (Site Class D)	F_a = 1.000
Maximum Considered Earthquake SRA	S_{MS} = 1.259
Design SRA	S_{DS} = 0.839g

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Peak Ground Acceleration

The mapped peak ground acceleration (PGA) for this site is 0.50g. 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 1/2 of the PGA_M, or 0 0.28g.

Seismic Hazards

Earthquake-induced geologic hazards may include liquefaction, lateral spreading, slope instability, and ground surface fault rupture. Liquefaction is a phenomenon where there is a reduction or complete loss of soil strength due to an increase in pore water pressure in soils. The increase in pore water pressure is induced by seismic vibrations. Liquefaction primarily affects geologically recent deposits of loose, uniformly graded, fine-grained sands and granular silts that are below the groundwater table. The site is mapped as having a "High" liquefaction susceptibility by the *Liquefaction Susceptibility Map of Pierce County, Washington* (2004); an excerpt of this map is included as Figure 5.

The soils encountered in our explorations and detailed in adjacent boring and CPT logs consist of loose to medium dense sand and soft to medium stiff silt down to depths of about 30 feet where refusal was met. In our opinion, these soils could be prone to liquefaction induced settlements during a seismic event.

The ground surface at the project site is generally level and the offsite slopes to the west are mapped as bedrock; therefore, in our opinion, the potential for earthquake-induced slope instability on the site is low. According to the Department of Natural Resources Geologic Hazards Map (Geologic Information Portal), the site is located about 4 miles south of the Tacoma Fault Zone. The USGS Interactive Fault Map for the general area is included as Figure 6. No evidence of ground fault rupture was observed in the subsurface explorations or out site reconnaissance. Therefore, in our opinion, the proposed structure should have no greater risk for ground fault rupture than other structures located in the area.

Liquefaction Analysis

After compiling the generalized subsurface conditions, we performed liquefaction analyses using the computer Program "Liquefy Pro" from CivilTech Corporation, with seismic inputs for the site of a MCE_G per ASCE 7-16 of 0.507g and a magnitude of 7.2. Groundwater was assumed to be 4 feet below the existing ground surface, which was the shallowest depth measured at the site. We assumed medium dense non-saturated soils underly the site at 2 feet below existing grades and very dense soils underlie the site at 4 feet below ground surface based on our borings and our experience in the area.

Based on these assumptions, we estimate the potential total settlement that could result from liquefaction due to the maximum credible earthquake to as much as 8 inches. Based on these analyses, the majority of the soil below the groundwater table to a depth of about 4 feet is predicted to liquefy in the maximum considered event.

Estimating total vertical and horizontal displacements during a design seismic event as a result of liquefaction-induced settlement and lateral spreading would require additional explorations and detailed site-specific analyses and is outside the current scope of this report. If more detailed estimates are required to support structural mitigation or the design of ground improvements, we can provide these estimates under a separate scope of work at your request.

Liquefaction Mitigation

Liquefaction mitigation typically involves transferring the load to a non-liquefiable soil at depth, densifying the soils identified as liquefiable, dissipating excess pore water pressure, or bridging the liquefiable soils. The potential for liquefaction induced settlement can also be reduced by stiffening the upper layer of soil and/or stiffening the foundation elements. Paved areas or lightly loaded structures not supported or bearing on improved ground may still be damaged during a seismic event.

Shallow Foundation Recommendations

Because of the risk of settlement during a seismic event we recommend that isolated spread footings <u>not</u> be used and continuous strip footings be utilized instead. In addition, seismic ties, grade beams or other approved methods should be used in the footings to reduce the potential for differential settlement.

<u>Geogrid Gravel Raft</u>

Geotechnical research suggests that a layer of non-liquefiable or densified soils directly below the foundation elements can mitigate the potential damage from liquefaction induced settlement (Ishihara and Seed, 1998). To this end, a geogrid gravel raft could be constructed to support the proposed foundations. This methodology can help reduce the magnitude of differential settlement, but may have limited effect on the total magnitude of liquefaction induced settlement.

The area below the structure should be excavated at least 3 feet below design bearing elevation, structural geogrid be placed (such as Tensar Triax® TX140-375 or 475, InterAx® NX650, or an approved equivalent), and structural fill placed above the geogrid to reestablish the design bearing surface. In addition to being placed at the bottom of the over-excavation, we recommend that a layer of geogrid be placed approximately every 18 inches within the structural fill. Geogrid placed at the bottom of the over-excavation should be overlapped a minimum of 3 feet in order to mitigate movement of the geogrid during fill placement and compaction. Overlap for additional layers should be a minimum of 1 foot. 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 over-excavation should extend 5 to 10 feet out from the footing edges.

Where the over-excavation extends below the water table, we recommend quarry spalls be placed on top of the geogrid and bucket-tamped until firmly set. Since groundwater levels extend to the ground surface, we recommend that structural fill used in the geogrid gravel raft consist of quarry spalls per "*Quarry Spalls*" per WSDOT 9-13.1(5). This work should be completed during the dry season, where groundwater levels are at their lowest. Dewatering may be necessary to complete excavations and install geogrid.

<u>Shallow Foundation Design</u>

Foundation bearing surfaces prepared as described above can be designed for an allowable bearing pressure of up to 2,500 pounds per square foot (psf). Bearing pressures can be increased by up to one-third for seismic and wind loads. Minimum footing widths should be 24 inches for continuous spread footings. Exterior footings should be at least 18 inches below the lowest adjacent grade for frost protection. All loose or soft soil and soil containing organics should be removed from beneath footing and areas to receive structural fill.

Lateral Resistance of Shallow Foundations

For portions of the structure founded on shallow continuous footings, lateral loads may be resisted by a combination of base friction and passive pressure against the footings and buried portions of the wall. In our opinion, passive earth pressures developed from properly compacted structural fill should be based on an allowable equivalent fluid density of 250 pounds per cubic foot (pcf). This passive resistance value assumes that the footings extend at least 18 inches below the lowest adjacent grade and that the ground surface is horizontal for a minimum distance of 1.5 times the embedment depth. The above passive earth pressure includes a factor of safety (FS) of 1.5 to limit lateral deflections. We recommend a coefficient of friction of 0.30 be used between cast-in-place concrete and structural fill for calculating the resistance to sliding at the base of the footings. The friction factor also includes a FS of 1.5.

Settlement

We were not provided with design loads prior to preparing this report. We have assumed column loads on the order of 25 kips. Based on the assumed loading conditions, we estimate post-construction consolidation settlements (non-seismic) of footings designed and constructed as recommended to be between about 1 and 2 inches, with differential settlements (between adjacent footings or over a 50-foot span of continuous footing) between ½ and 1 inch. The actual settlement will be dependent on the actual loads and footing widths. These settlements are expected to occur as loads are being applied and over the first few years after construction because of the fine grained soils at the site.

Construction Considerations

We recommend that exposed footing subgrades be evaluated by a representative from GeoResources, LLC to confirm soil conditions and provide recommendations where unanticipated conditions are found. Native soils that are disturbed during footing excavation should be removed prior to the placement of the concrete forms and reinforcement.

Subgrade soil improvements, such as the geogrid gravel raft 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.

Ground Improvement

If the estimated settlements due to the consolidated or liquefaction-induced settlements utilizing a geogrid gravel raft and stiffened foundation system are not acceptable, then ground improvement techniques, such as rammed aggregate piers or stone columns can be used to reduce the potential magnitude of both consolidation and liquefaction-induced settlement. In these methods, a hollow steel mandrel is driven to the design depth, as the mandrel is withdrawn the aggregate is injected into the ground through the hollow core of the mandrel. 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 elements, the potential magnitude of the liquefaction induced settlement can be reduced by varying amounts. Typical aggregate pier diameters range from about 24 to 36 inches. Additionally, elements can be used to reduce the magnitude of consolidation-settlement by transferring the structural load below settlement sensitive layers. Once the grid of aggregate pier elements has been installed, the shallow foundation elements can be constructed directly on top of the piers.

Determination of required pier diameter, depth, and spacing is beyond our current scope and is typically completed by the specialty design-build contractor. Rammed aggregate piers should be designed to limit total post-construction settlement to less than 1 inch and differential settlement to less than ½ inch over 50 feet.

Because of the equipment used to install aggregate piers, there is typically a large mobilization fee rendering small projects less cost effective, and in our opinion, the subgrade stabilization and conventional foundations will be more economical option

Augercast Piles

In lieu of soil densification and conventional shallow foundations, the buildings can be supported on deep pile foundations, such as augercast piles in order to reduce the potential magnitude of both consolidation and liquefaction-induced settlement. If higher load capacities are required and because of the potential for soil liquefaction, the use of deep piles may be an appropriate and cost effective solution. If requested, we can provide recommendation for deep foundations.

Floor Slab Support

Slab-on-grade floors should be supported on the native soils or on structural fill prepared as described above. Areas of old fill material should be evaluated during grading activity for suitability of structural support. Areas of significant organic debris should be removed.

We recommend that floor slabs be directly underlain by a minimum 4 inch thick pea gravel or washed 5/8 inch crushed rock and should contain less than 2 percent fines. This layer should be placed in a single 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 moisture migration through the slab is an issue, such as where adhesives are used to anchor carpet or tile to the slab.

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 1/2 inch or less over a span of 50 feet.

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 including Washington Administrative Code (WAC) and Washington Industrial Safety and Health Administration (WISHA). Excavation, trenching, and shoring is covered under WAC 296-155 Part N.

Based on WAC 296-155-66401, it is our opinion that the medium dense recessional and advance outwash soils on the site would be classified as Type C soils. According to WAC 296-155-66403, for temporary excavations of less than 20 feet in depth, the side slopes in Type C soils should be sloped at a maximum inclination of 1½H:1V or flatter from the toe to top of the slope. All exposed slope faces should be covered with a durable reinforced plastic membrane 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 slope crest.

Where it is not feasible to slope the site soils back at these inclinations, a retaining structure should be considered. Retaining structures greater than 4 feet in height (bottom of footing to top of structure) or that have slopes of greater than 15 percent above them, 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.

Site Drainage

All ground surfaces, pavements and sidewalks at the site should be sloped to direct surface water away from the structures and property lines. Surface water runoff should be controlled by a system of curbs, berms, drainage swales, and or catch basins, and conveyed to an appropriate discharge point.

We recommend that footing drains are installed for the residence in accordance with IBC 1805.4.2, and basement walls (if utilized) have a wall drain as describe above. The downspouts should not be connected to directly the footings drains until they are combined to tightline to the discharge point. Figure 7 shows typical wall drainage and backfilling details. If the basement cut extends below the adjacent municipal stormwater system, a sump and pump system may be required.

Dewatering Considerations

Depending on the depth of utilities to be installed at the site, we anticipate some trenches may be below seasonal high groundwater levels. During the winter months, October through June, the groundwater may be 4 feet or less 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 the trench excavation exceed 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.

EARTHWORK RECOMMENDATIONS

Site Preparation

All structural areas on the site to be graded should be stripped of vegetation, organic surface soils, and other deleterious materials including existing structures, foundations or abandoned utility lines. Organic topsoil is not suitable for use as structural fill, but may be used for limited depths in non-structural areas. Stripping depths ranging from 4 to 12 inches should be expected to remove these unsuitable soils. Areas of thicker topsoil or organic debris may be encountered in areas of heavy vegetation or depressions.

Where placement of fill material is required, the stripped/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 removal of vegetation and topsoil stripping is completed and prior to placement of structural fill. The exposed subgrade soil should be proof-rolled with heavy rubber-tired equipment during dry weather or probed with a ½ inch diameter steel rod during wet weather conditions.

Soft, loose, or otherwise unsuitable areas delineated during proofrolling 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. The areas of old fill material should be evaluated during grading operations to determine if they need mitigation; recompaction or removal.

Structural Fill

All material placed as fill for the proposed wall should be placed as structural fill. Material placed as structural fill should be free of debris, organic matter, trash, and cobbles greater than 4-inches in diameter. The moisture content of the fill material should be adjusted as necessary for proper compaction.

The suitability of material for use as structural fill will depend on the gradation and moisture content of the soil. 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 ¾-inch sieve, such as *Gravel Backfill for Walls* (WSDOT 9-03.12(2)). If prolonged dry weather prevails during the wall construction, higher fines content (up to 10 to 12 percent) may be acceptable.

The appropriate lift thickness will depend on the structural fill characteristics and compaction equipment used, but it is typically limited to 4 to 6 inches for hand operated equipment; thicker lifts may be appropriate for larger equipment. For larger equipment such as a hoe-pac or drum roller, we recommend a maximum loose-lift thickness of 12 inches. Structural fill should be compacted to at least 95 percent of the MDD as determined by the Modified Proctor (ASTM D1557), except for within 12 inches of the back of the wall, as described in the "**Wall Drainage**" section of this report. Additionally, the moisture content should be maintained within 3 percent of the optimum moisture content in accordance with ASTM D1557.

Suitability of On-Site Materials as Fill

During dry weather construction, non-organic on-site soil may be considered for use as structural fill; provided it meets the criteria described above in the "**Structural Fill"** section and can be compacted as recommended. 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. We generally did not observe the site soils to be excessively moist at the time of our subsurface exploration program.

The native sand is generally comparable to "common borrow" material and should be suitable for use as structural fill provided the moisture content is maintained within 2 percent of the optimum moisture level. The native silt material contains a significate fraction f fine material which will make this material difficult to impossible to work when wet.

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.

Erosion Control

Weathering, erosion and the resulting surficial sloughing and shallow land sliding are natural processes. As noted, no evidence of surficial raveling or sloughing was observed at the site. To manage and reduce the potential for these natural processes, we recommend erosion protection measures will need to be in place prior to grading activity on the site. Erosion hazards can be mitigated by applying Best Management Practices (BMP's) outlined in the 2019 Stormwater management Manual for Western Washington (SWMMWW).

LIMITATIONS

We have prepared this report for use by CIMCO and other members of the design team, for use in the design of a portion of this project. The data used in preparing this report and this report should be provided to prospective contractors for their bidding or estimating purposes only. Our report, conclusions and interpretations are based on our subsurface explorations, 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 the opportunity to be of service to you on this project. If you have any questions or comments, please do not hesitate to call at your earliest convenience.

Respectfully submitted,
GeoResources, LLC

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STATE OF WASHINGTON
43471
REGISTERED
PROFESSIONAL ENGINEER
3/1/2023

State of Washington
Engineering Geologist
925
Licensed Geologist
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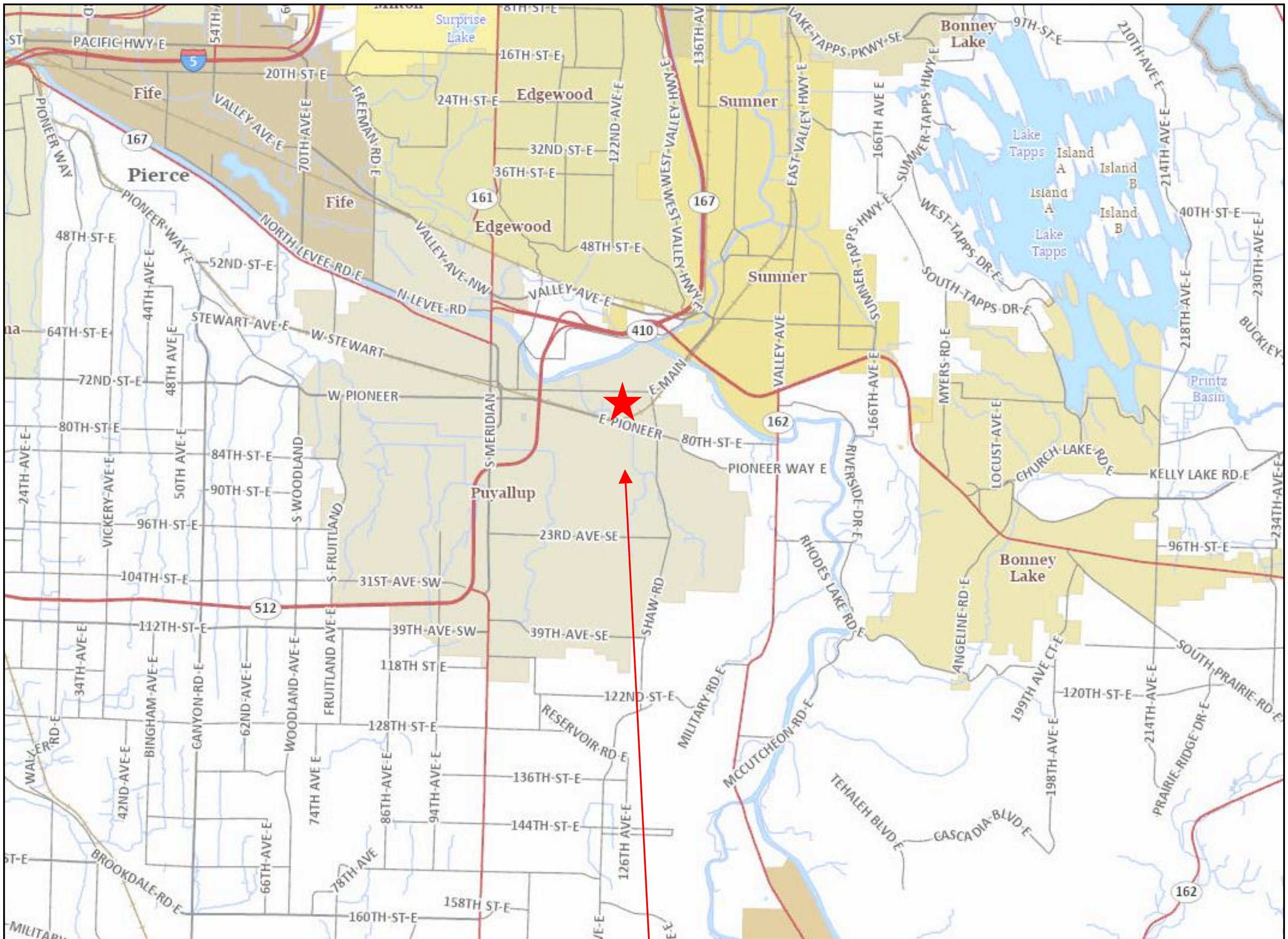
Senior Geotechnical Engineer

Keith S. Schembs, LEG
Principal

EWH:KSS/ewh
DocID: CIMCO.InterAve.RG
Attachments:
- Figure 1: Site Location Map
- Figure 2: Site Exploration Map
- Figure 3: NRCS Soils Map
- Figure 4: Geologic Map
- Figure 5: Liquefaction Susceptibility Map
- Figure 6: Seismic Hazards Map
- Figure 7: Typical Wall Drainage & Backfill Detail
- Appendix A: Subsurface Explorations
- Appendix B: Laboratory Results



Approximate Site Location

(map created from Pierce County Public GIS http://matterhorn3.co.pierce.wa.us/publicgis/)

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Not to Scale

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Site Location Map

Proposed Commercial Development
2315 Inter Avenue
Puyallup, Washington
PN: 2105200140

Doc ID: CIMCO.InterAve.F	March 2023	Figure 1

Approximate Site Location

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

Number and approximate location of test pit/Pilot Infiltration Test

Not to Scale

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Site & Exploration Map

Proposed Commercial Development
2315 Inter Avenue
Puyallup, Washington
PN: 2105200140

Doc ID: CIMCO.InterAve.F	March 2023	Figure 2

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
6A	Briscot loam	Alluvium	-	Slight	D
31A	Puyallup fine sandy loam	Alluvium	0 to 3	Slight	B

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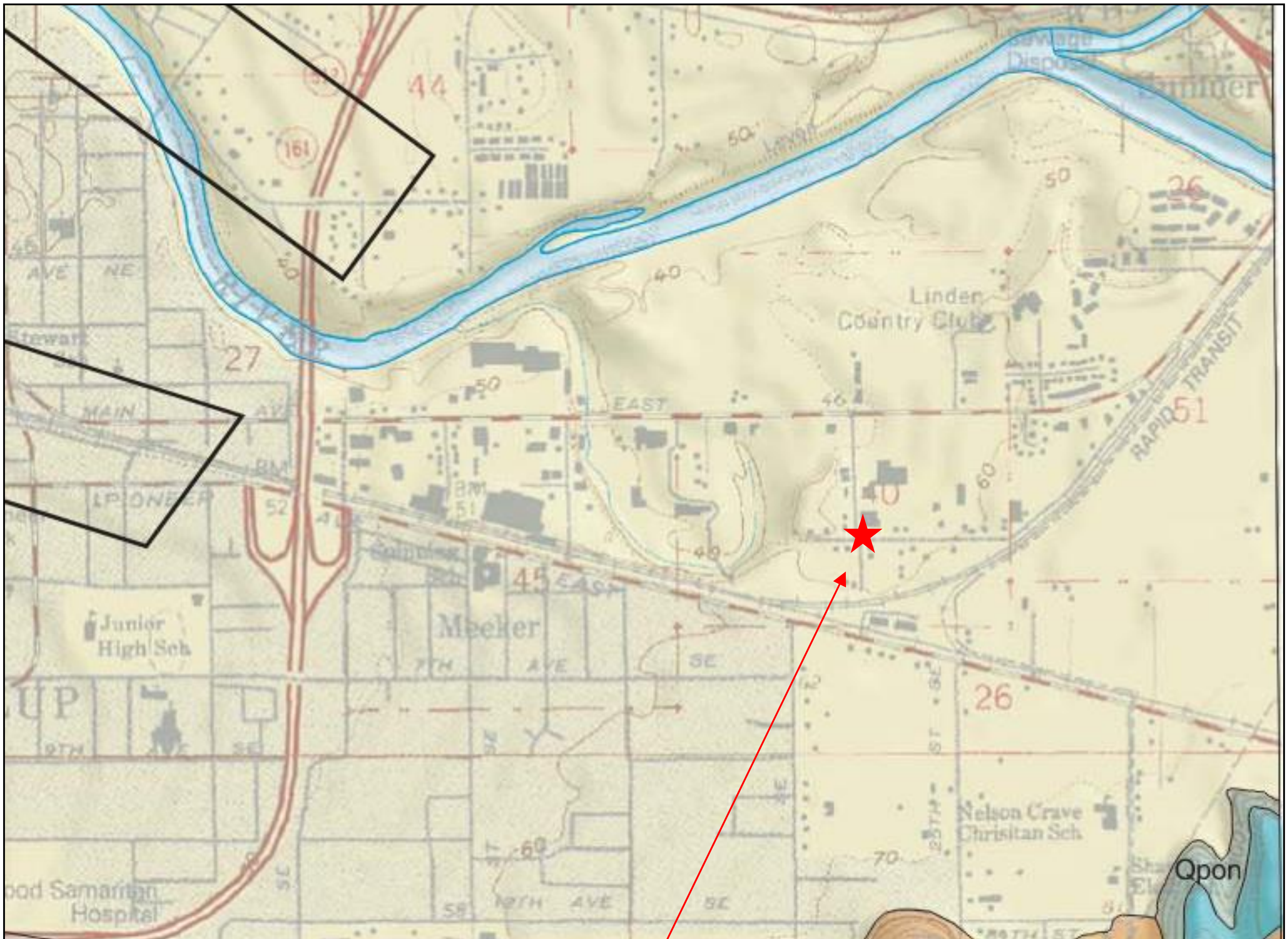
NRCS Soils Map

Proposed Commercial Development
2315 Inter Avenue
Puyallup, Washington
PN: 2105200140

Doc ID: CIMCO.InterAve.F	March 2023	Figure 3



Approximate Site Location

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

Qal	Alluvium (Holocene)

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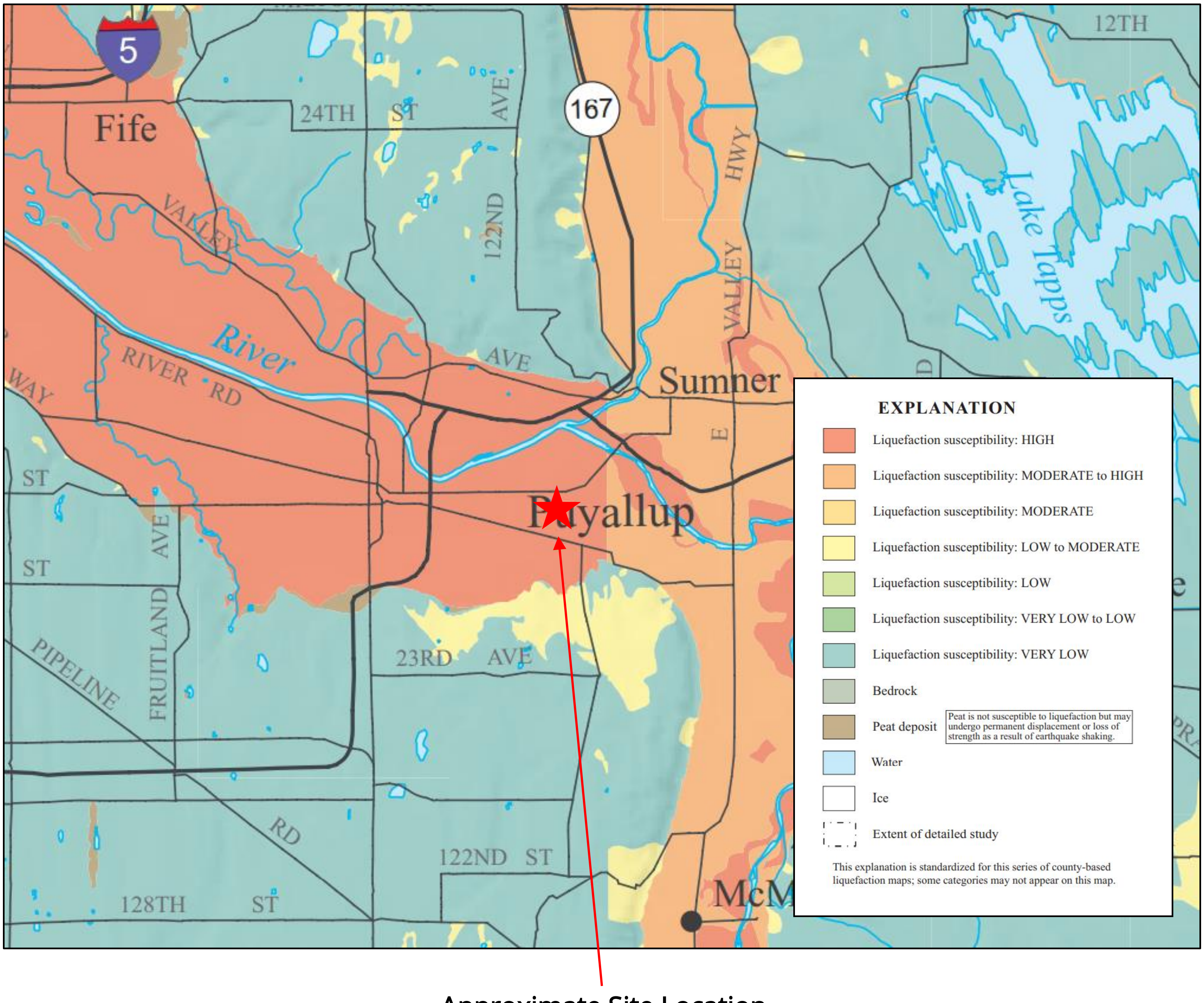
Geologic Map

Proposed Commercial Development
2315 Inter Avenue
Puyallup, Washington
PN: 2105200140

Doc ID: CIMCO.InterAve.F	March 2023	Figure 4



Map created from the Liquefaction Susceptibility Map of Pierce County, Washington by Stephen P. Palmer, Sammantha L. Magsino, Eric L. Bilderback, James L. Poelstra, Derek S. Folger, and Rebecca A. Niggemann (September 2004)

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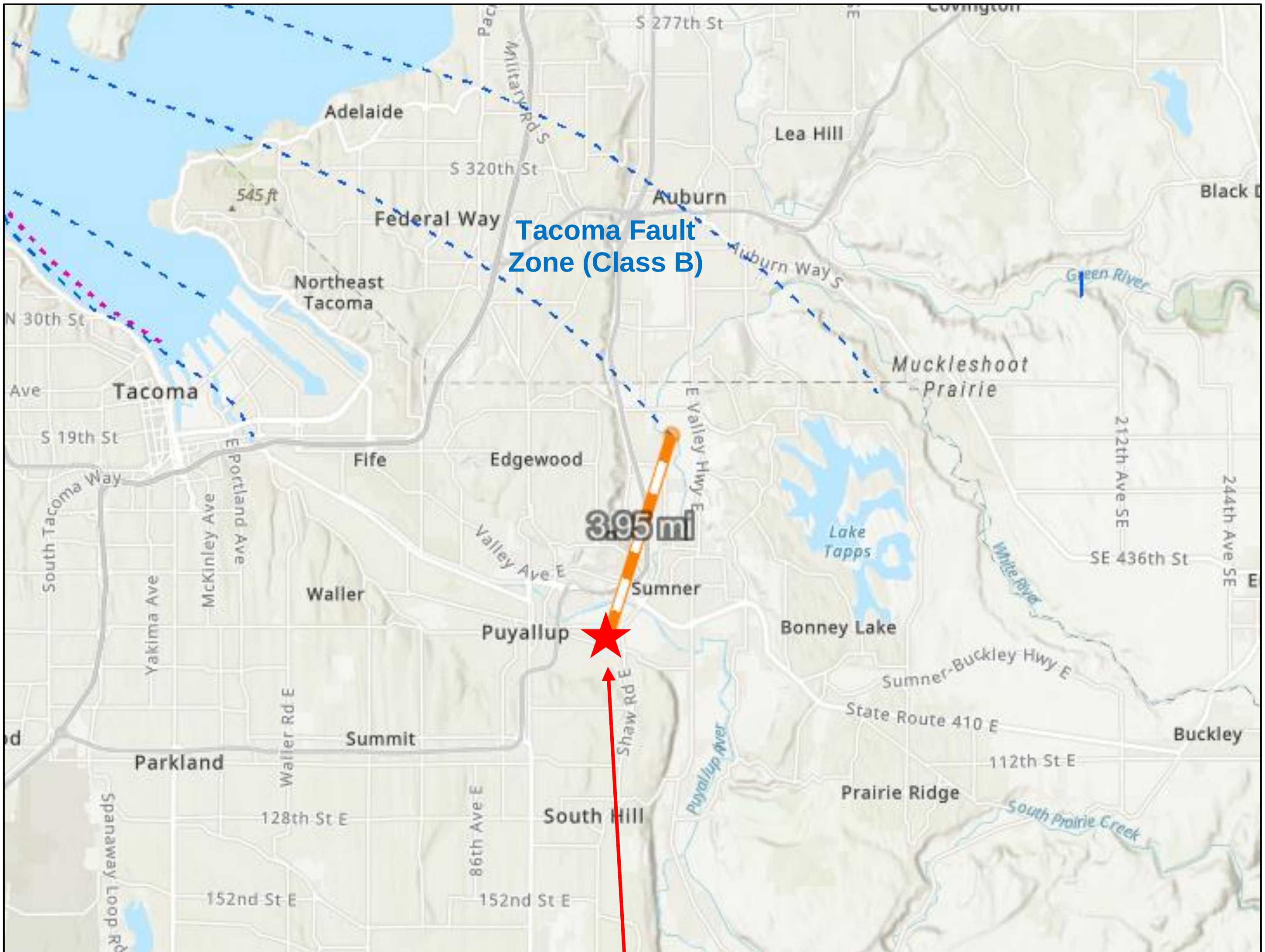
Liquefaction Susceptibility Map

Proposed Commercial Development
2315 Inter Avenue
Puyallup, Washington
PN: 2105200140

Doc ID: CIMCO.InterAve.F	March 2023	Figure 5



Approximate Site Location

Map created from the Washington Geologic Information Portal (geologyportal.dnr.wa.gov)

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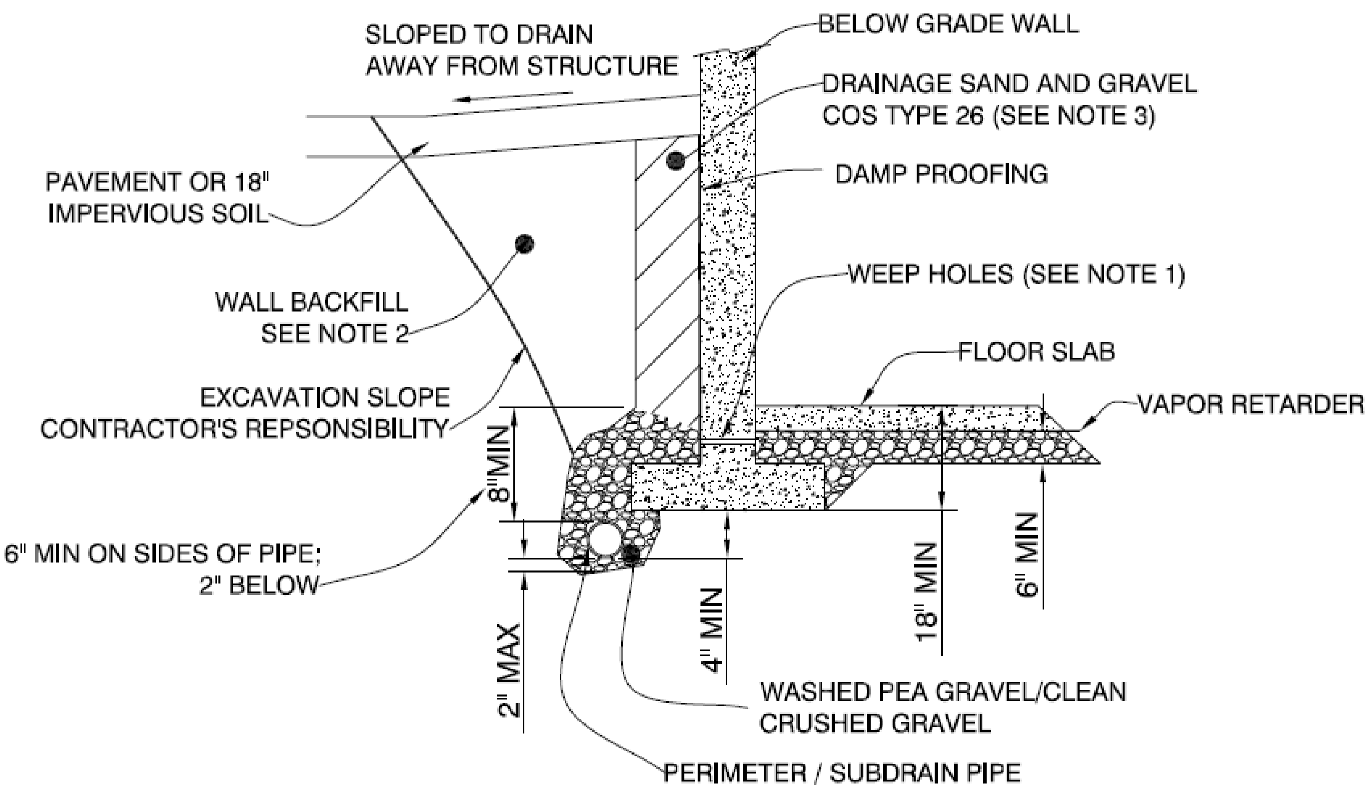
Fault Hazards Map

Proposed Commercial Development
2315 Inter Avenue
Puyallup, Washington
PN: 2105200140

Doc ID: CIMCO.InterAve.F	March 2023	Figure 6



1. Washed pea gravel/crushed rock beneath floor slab could be hydraulically connected to perimeter/subdrain pipe. Use of 1" diameter weep holes as shown is one applicable method. Crushed gravel should consist of 3/4" minus. Washed pea gravel should consist of 3/8" to No. 8 standard sieve.

2. Wall backfill should meet WSDOT Gravel Backfill for walls Specification 9-03-12(2).

3. Drainage sand and gravel backfill within 18" of wall should be compacted with hand-operated equipment. Heavy equipment should not be used for backfill, as such equipment operated near the wall could increase lateral earth pressures and possibly damage the wall. The table below presents the drainage sand and gravel gradation.

4. All wall back fill should be placed in layers not exceeding 4" loose thickness for light equipment and 8" for heavy equipment and should be densely compacted. Beneath paved or sidewalk areas, compact to at least 95% Modified Proctor maximum density (ASTM: 01557-70 Method C). In landscaping areas, compact to 90% minimum.

5. Drainage sand and gravel may be replaced with a geocomposite core sheet drain placed against the wall and connected to the subdrain pipe. The geocomposite core sheet should have a minimum transmissivity of 3.0 gallons/minute/foot when tested under a gradient of 1.0 according to ASTM 04716.

6. The subdrain should consist of 4" diameter (minimum), slotted or perforated plastic pipe meeting the requirements of AASHTO M 304; 1/8-inch maximum slot width; 3/16- to 3/8-inch perforated pipe holes in the lower half of pipe, with lower third segment unperforated for water flow; tight joints; sloped at a minimum of 6"/100' to drain; cleanouts to be provided at regular intervals.

7. Surround subdrain pipe with 8 inches (minimum) of washed pea gravel (2" below pipe" or 5/8" minus clean crushed gravel. Washed pea gravel to be graded from 3/8-inch to No.8 standard sieve.

8. See text for floor slab subgrade preparation.

Materials

Drainage Sand and Gravel

Sieve Size	% Passing by Weight
3/4 "	100
No 4	28-56
No 8	20-50
No 50	3-12
No 100	0-2

3/4" Minus Crushed Gravel

Sieve Size	% Passing by Weight
3/4 "	100
1/2"	75 - 100
1/4"	0 - 25
No 100 (by wet sieving)	0 - 2 (non-plastic)

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Typical Drainage and Backfill Detail

Proposed Commercial Development
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PN: 2105200140

DocID: CIMCO.InterAve.F	March 2023	Figure 7

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

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Puyallup, Washington
PN: 2105200140

Doc ID: CIMCO.InterAve.F	March 2023	Figure A-1

Test Pit TP-1

Location: West portion of parcel
Approximate Elevation: 60' (NAVD88)

Depth (ft)			Soil Type	Soil Description
0	-	0.5	-	Topsoil
0.5	-	3.0	SM	Brown silty SAND (loose, moist) (alluvium)
3.0	-	5.5	SM/ML	Gray, orange iron oxide stained silty SAND, interbedded gray mottled silt (medium dense/stiff, moist to wet) (alluvium)
5.5	-	6.0	ML	Gray mottled SILT (stiff, wet) (alluvium)
6.0	-	6.5	SP	Gray SAND (medium dense, wet) (alluvium)

Terminated at 3.2 feet below ground surface.
No caving observed.
Slow groundwater seepage observed at 4 feet below existing grades.

Test Pit TP-2

Location: SW central portion of parcel
Approximate Elevation: 60' (NAVD88)

Depth (ft)			Soil Type	Soil Description
0	-	0.5	-	Topsoil
0.5	-	3.0	SM	Reddish brown silty SAND (loose to medium dense, moist) (alluvium)
3.0	-	3.5	ML	Light gray SILT (medium stiff, moist) (alluvium)
3.5	-	6.0	SP	Gray mottled SAND (medium dense, wet) (alluvium)
6.0	-	7.5	SM	Gray mottled silty SAND (medium dense, wet) (alluvium)
7.5	-	8.0	ML	Gray SILT (stiff, wet) (alluvium)

Terminated at 8.0 feet below ground surface.
No caving observed.
Slow groundwater observed at 4.5 feet below ground surface

.

Logged by: AES

Observed on: December 21, 2021

Pilot Infiltration Test PIT-1

Location: West portion of parcel
Approximate Elevation: 60' (NAVD88)

Depth (ft)			Soil Type	Soil Description
0	-	0.5	-	Topsoil
0.5	-	2.0	SM	Reddish brown silty SAND (loose to medium dense, moist) (alluvium)
2.0	-	3.0	SM	Gray silty SAND (medium dense, moist) (alluvium)
3.0	-	5.0	SM	Gray mottled silty SAND (medium dense, moist) (alluvium)

Infiltration testing performed at 2 feet below existing grades.
Measured 1 inch per hour.
Overexcavated to 5 feet below existing grades.
No caving observed.
No groundwater seepage observed.

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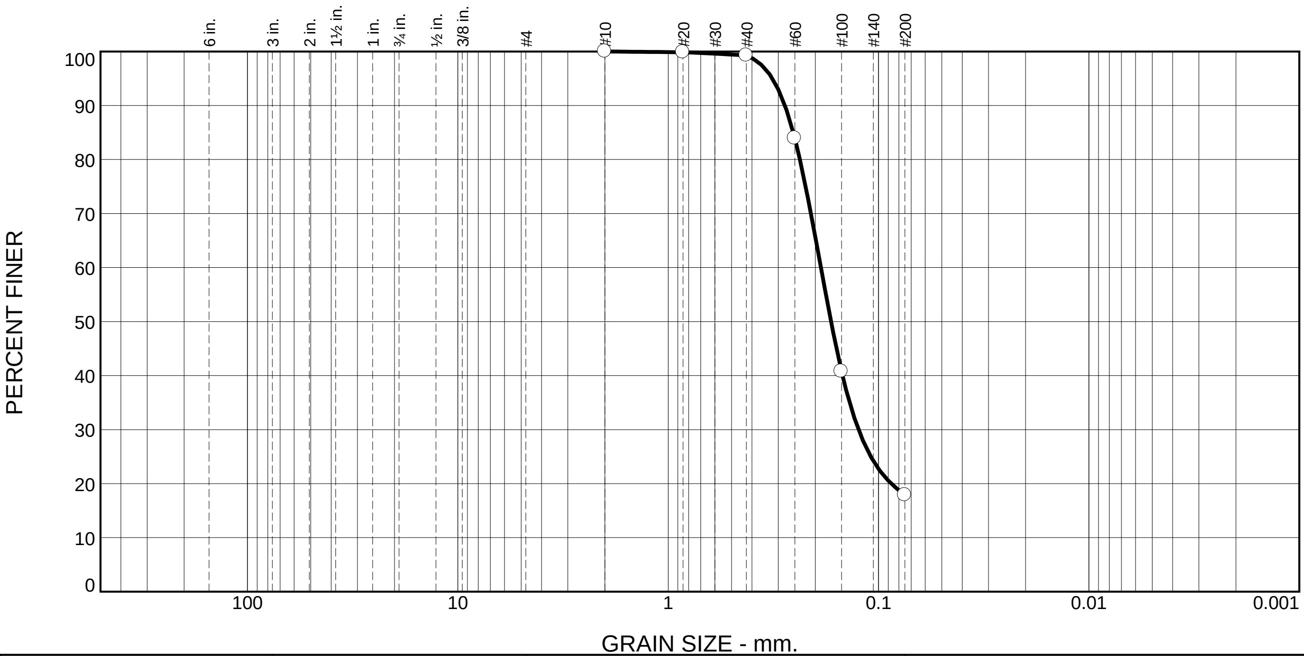
Logged by: AES **Observed on: December 21, 2021**

Appendix B
Laboratory Test Results

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.7	81.4	17.9	

Test Results (ASTM D 6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#20	99.9		
#40	99.3		
#60	83.9		
#100	40.8		
#200	17.9		

* (no specification provided)

Material Description

Silty SAND (SM)

Atterberg Limits (ASTM D 4318)

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

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

D_{90}= 0.2787 D_{85}= 0.2542 D_{60}= 0.1879
D_{50}= 0.1683 D_{30}= 0.1245 D_{15}=
D_{10}= C_u= C_c=

Remarks

Natural Moisture: 22%

Date Received: 12/21/21 **Date Tested:** 3/24/22

Tested By: MAW

Checked By: KSS

Title: PM

Location: PIT-1, S-2, 5'
Sample Number: 103087
Date Sampled: 12/21/21

GeoResources, LLC

Fife, WA

Client: CIMCO
Project: Proposed Commercial Development
Project No: CIMCO.InterAve **Figure** B-1

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:** ____________