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Report of
Geotechnical Investigation
Ron Tonkin Gran Turismo Lamborghini Dealership
Lot South of 25195 SW Parkway Avenue
Wilsonville, Oregon

CGT Project Number G2306033

Prepared for

Celia Tonkin
Ron Tonkin Gran Torismo
25300 SW Parkway Avenue
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May 13, 2024

City of Wilsonville
Exhibit B9 DB24-0006

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Dear Celia Tonkin:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our geotechnical investigation for the proposed Ron Tonkin Gran Turismo Lamborghini Dealership project. The site is located directly south of 25195 SW Parkway Avenue in Wilsonville, Oregon. We performed our work in general accordance with CGT Proposal GP23-302R1, dated November 7, 2023. Written authorization for our services was received on November 9, 2023. A draft version of this report was submitted on December 27, 2023.

We appreciate the opportunity to work with you on this project. Please contact us at (503) 601-8250 if you have any questions regarding this report.

Respectfully Submitted,
CARLSON GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
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1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our geotechnical investigation for the proposed Ron Tonkin Gran Turismo Lamborghini Dealership project. The site is located directly south of 25195 SW Parkway Avenue in Wilsonville, Oregon, as shown on the attached Site Location, Figure 1.

1.1 Project Information

CGT developed an understanding of the proposed project based on our correspondence with the project architect, Axis Design Group (Axis), and review of the provided preliminary project plan set prepared by Axis, dated October 4, 2023, and a survey map, prepared by Westlake Consultants, Inc. Based on our review, we understand the project will include:

- Construction of a new showroom and service building within the northwest portion of the site. The building will be three-stories, metal- and steel-framed, will incorporate a slab on grade ground floor, and include a partially below-grade vehicle storage level. The ground floor of the building will be established at elevation 269.50 feet. Based on information provided by Eric Esqueda, P.E., of VLMK Engineering, maximum column and continuous wall loads will be on the order of 395 kips and 12 kips per lineal foot (klf), respectively. Uniform floor slab loads are anticipated to be less than 250 pounds per square foot (psf).
- Construction of paved passenger car parking areas located east of the showroom and service building, and along the north and east margins of the site. We assume new pavements will be surfaced with asphalt concrete (AC), while loading docks and driveway aprons will be surfaced with Portland Cement Concrete (PCC).
- If conditions allow, stormwater collected from new impervious areas at the site will be disposed of, at least in part, via onsite infiltration. Infiltration testing was requested at three locations as part of this assignment. As described later in this report, due to the presence of shallow groundwater, infiltration testing was not performed at the site.
- Although no grading plans have been provided, we anticipate permanent grade changes at the site will include minimal fills. Cuts up to about 6 feet in depth are anticipated in the planned building pad to achieve desired ground floor elevation(s).
- No development or grading is anticipated to occur within a designated wetland (identified by others) within the south central portion of the site.

1.2 Scope of Services

Our scope of work included the following:

- Contact the Oregon Utilities Notification Center to mark the locations of public utilities within a 20-foot radius of our explorations at the site.
- Explore subsurface conditions at the site by advancing five drilled borings to depths of up to about 26½ feet below ground surface (bgs). Details of the subsurface investigation are presented in Appendix A.
- Classify the soils encountered in the explorations in general accordance with ASTM D2488 (Visual-Manual Procedure).
- Provide a technical narrative describing surface and subsurface deposits, and local geology of the site, based on the results of our explorations and published geologic mapping.

- Provide recommendations for the Seismic Site Class, mapped maximum considered earthquake spectral response accelerations, and site seismic coefficients.
- Provide a qualitative evaluation of seismic hazards at the site, including earthquake-induced liquefaction, landsliding, and surface rupture due to faulting or lateral spread.
- Provide geotechnical recommendations for site preparation and earthwork.
- Provide geotechnical engineering recommendations for use in design and construction of shallow foundations deriving support from improved ground, floor slabs, retaining walls, and pavements.
- Provide this written report summarizing the results of our geotechnical investigation and recommendations for the project.

2.0 SITE DESCRIPTION

2.1 Site Geology

Based on available geologic mapping[1,2] of the area, the site is underlain by basalt bedrock. The basalt bedrock unit is composed of lava flows associated with the Columbia River Basalt group. The Columbia River basalt group consists of numerous fine-grained lava flows that primarily erupted from fissures in eastern Washington and Oregon and western Idaho during the Miocene (23.8 to 5.3 million years ago). Many individual flows are interbedded with thin paleosols that consist of clay-rich soils or sediments formed during period of volcanic inactivity. The basalt can weather in place to form clay and silt rich residual soils that overly the intact basalt bedrock. When intact, the basalt features jointed patterns ranging from columnar to entablature/colonnade, and is described as having fresh exposures that are dark gray to black, while weathered exposures area greenish-gray to grayish-black. Based on results of the drilled borings advanced at the site (described below) and review of local well logs, we anticipate that residual soils (fully decomposed bedrock) extend to depths of about 30 to 60 feet bgs, and are underlain by intact basalt bedrock.

2.2 Site Surface Conditions

The site is bordered by SW Parkway Avenue to the east, an on-ramp to Interstate 5 to the west, and commercial properties to the north and south. At the time of our field investigation, the north, west, and east perimeters of the site descended towards its center at gradients up to 4 horizontal:1 vertical (4H:1V). The south-central portion of the site is mapped (by others) as wetlands. Vegetation on the southern portion of the site consisted of grasses and scattered coniferous and deciduous trees. The northern and western portions of the site were densely vegetated with brush and trees. The western portion of the site exhibited moderately dense vegetation and resulted in limited access for exploration equipment. Site layout and surface conditions at the time of our field investigation are shown on the attached Site Plan (Figure 2) and Site Photographs (Figure 3).

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[1] Madin, I.P., 2004. Geologic mapping and database for the Portland area fault studies: Final report, Clackamas, Multnomah, and Washington Counties, Oregon: Oregon Department of Geology and Mineral Industries, Open-File Report O-04-02, scale 1:100,000.

[2] Beeson, M.H., Tolan, T.L., and Madin, I.P., 1991. Geologic map of the Portland quadrangle, Multnomah and Washington counties, Oregon, and Clark County, Washington: Oregon Department of Geology and Mineral Industries, Geological Map Series 75, scale 1:24,000.

2.3 Subsurface Conditions

2.3.1 Subsurface Investigation & Laboratory Testing

Our subsurface investigation consisted of five drilled borings (B-1 through B-5) completed on December 4, 2023. The approximate boring locations are shown on the Site Plan, attached as Figure 2. In summary, the borings were advanced to depths ranging from about 6½ to 26½ feet bgs. Details regarding the subsurface investigation, logs of the explorations, and results of laboratory testing are presented in Appendix A. Subsurface conditions encountered during our investigation are summarized below.

2.3.2 Subsurface Materials

Logs of the explorations are presented in Appendix A. The following describes each of the subsurface materials encountered at the site.

Organic Soil (OL)
Organic soil was encountered at the surface of each boring. The organic soil was typically dark brown, moist, exhibited low plasticity, and contained varying amounts of rootlets. This soil extended to depths of about ¼-foot bgs in the borings.

Elastic Silt (MH)
Elastic silt was encountered below the organic soil in each boring. The elastic silt was typically brown, moist, exhibited medium plasticity, and contained varying amounts of weathered rock fragments up to ¼-inch in diameter. In terms of consistency, this soil was very soft in the upper 5 feet in borings B-1 and B-2. Below that depth and in the remaining borings, this soil was typically medium stiff to stiff. This soil extended to depths of about 7 to 10 feet bgs in borings B-1 through B-4, and to the full depth explored in boring B-5, about 6½ feet bgs.

Silty Sand (SM)
Underlying the elastic silt in borings B-1 through B-4 was silty sand. The silty sand was typically medium dense, multicolored, moist to wet, fine- to coarse-grained, and contained medium plasticity fines and varying amounts of weathered rock fragments up to ½-inch in diameter. This soil extended to the full depths explored in those borings, about 9 to 26½ feet bgs. This soil was interpreted to consist of residual soils.

2.3.3 Groundwater

As shown on the attached logs and on the attached Site Plan, Figure 2, the groundwater level (phreatic surface) was encountered at variable depths (ranging from 1 to 12 feet bgs) within borings B-1 through B-5 during our investigation in early December 2023. To determine approximate regional groundwater levels in the area, we researched well logs available on the Oregon Water Resources Department (OWRD)[3] website for wells located within Section 02, Township 03 South, Range 01 West, Willamette Meridian. Our review indicated that groundwater levels in the area generally ranged from about 12½ to 25 feet bgs. It should be noted groundwater levels vary with local topography. In addition, the groundwater levels reported on the OWRD logs often reflect the purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced

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[3] Oregon Water Resources Department, 2023. Well Log Records, *accessed December 2023*, from OWRD web site: http://apps.wrd.state.or.us/apps/gw/well_log/.

above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the project site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. Additionally, the on-site fine-grained (silty) soils are conducive to formation of perched groundwater.

3.0 SEISMIC CONSIDERATIONS

3.1 Seismic Design

Section 1613.2.2 of the 2022 Oregon Structural Specialty Code (2022 OSSC) requires that the determination of the seismic site class be in accordance with Chapter 20 of the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7-16). We have assigned the site as Site Class D ("Stiff Soil") based on geologic mapping and subsurface conditions encountered during our investigation.

Earthquake ground motion parameters for the site were obtained in accordance with the 2022 OSSC using the Seismic Hazards by Location calculator on the ATC website[4]. The site Latitude 45.337419° North and Longitude 122.767954° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

Table 1 Seismic Ground Motion Values

Parameter		Value
Mapped Acceleration Parameters	Spectral Acceleration, 0.2 second (S_s)	0.826g
	Spectral Acceleration, 1.0 second (S_1)	0.384g
Coefficients (Site Class D)	Site Coefficient, 0.2 second (F_A)	1.169
	Site Coefficient, 1.0 second (F_V)[1]	1.916
Adjusted MCE Spectral Response Parameters	MCE Spectral Acceleration, 0.2 second (S_{MS})	0.966g
	MCE Spectral Acceleration, 1.0 second (S_{M1})	0.736g
Design Spectral Response Accelerations	Design Spectral Acceleration, 0.2 second (S_{DS})	0.644g
	Design Spectral Acceleration, 1.0 second (S_{D1})	0.491g
Seismic Design Category (Risk Category II)		D
[1] Value determined from 2022 OSSC Table 1613.2.3(2).		

3.2 Seismic Hazards

3.2.1 Liquefaction

In general, liquefaction occurs when deposits of loose/soft, saturated, cohesionless soils, generally sands and silts, are subjected to strong earthquake shaking. If these deposits cannot drain quickly enough, pore water pressures can increase, approaching the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. When the pore water pressure increases to the value of the overburden pressure, the shear strength of the soil approaches zero, and the soil can liquefy. The

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[4] Applied Technology Council (ATC), 2023. USGS seismic design parameters determined using "Seismic Hazards by Location," *accessed December 2023*, from the ATC website https://hazards.atcouncil.org/.

liquefied soils can undergo rapid consolidation or, if unconfined, can flow as a liquid. Structures supported by the liquefied soils can experience rapid, excessive settlement, shearing, or even catastrophic failure.

For fine-grained soils, susceptibility to liquefaction is evaluated based on penetration resistance and plasticity, among other characteristics. Criteria for identifying non-liquefiable, fine-grained soils are constantly evolving. Current practice to identify non-liquefiable, fine-grained soils is based on moisture content and plasticity characteristics of the soils[5,6,7]. The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on penetration resistance, as measured using SPTs, CPTs, or Becker Hammer Penetration tests (BPTs).

The Oregon Department of Geology and Mineral Industries' Oregon Statewide Geohazards Viewer (HazVu)[8] shows a *low* hazard for liquefaction at the site. The Oregon Hazard Explorer for Lifelines Program (O-HELP)[9] show a *very low* hazard for liquefaction for the site or immediate vicinity due to a M9.0 Cascadia Subduction Zone earthquake.

Based on its plasticity, the native elastic silt (MH) is not susceptible to liquefaction. Based on the plasticity characteristics of the fines and its classification as residual sols (fully decomposed rock), the silty sand (SM) encountered within our explorations is considered non-liquefiable. Based on review of geologic mapping and our previous experience in the area, we do not anticipate liquefiable conditions are present at depths below those explored as part of this assignment.

3.2.2 Slope Instability

We did not observe any obvious signs of past or on-going slope instability at the site. Review of the Statewide Landslide Information Database for Oregon (SLIDO), available at the DOGAMI website[10], shows no historic or prehistoric landslides at or in the immediate vicinity of the site. HazVu shows a *low* hazard for landslides at the site. O-HELP shows a *very low* probability of seismically-induced landslides at the site due to a M9.0 Cascadia Subduction Zone earthquake. Given the relatively gentle site grades, the lack of evidence of previous landslides in the vicinity, and the generally low hazard indicated by the hazard mapping, the risk of seismically-induced slope instability occurring at the site is considered very low. The proposed grading includes relatively minimal planned changes in site grades and is not anticipated to significantly increase this risk.

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[5] Seed, R.B. et al., 2003. Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. Earthquake Engineering Research Center Report No. EERC 2003-06.

[6] Bray, Jonathan D., Sancio, Rodolfo B., et al., 2006. Liquefaction Susceptibility of Fine-Grained Soils, Journal of Geotechnical and Geoenvironmental Engineering, Volume 132, Issue 9, September 2006.

[7] Idriss, I.M., Boulanger, R.W., 2008. Soil Liquefaction During Earthquakes, Earthquakes Engineering Research Institute Monograph MNO-12.

[8] Oregon Department of Geology and Mineral Industries, 2023. Oregon Statewide Geohazards Viewer, *accessed December 2023*, from DOGAMI web site: http://www.oregongeology.org/sub/hazvu/index.htm.

[9] Oregon State University College of Engineering, 2023. Oregon Hazard Explorer for Lifelines Program (O-HELP), *accessed December 2023*, from O-HELP web site: http://ohelp.oregonstate.edu/#&ui-state=dialog.

[10] Oregon Department of Geology and Mineral Industries, 2023. Statewide Landslide Information Database for Oregon (SLIDO), *accessed December 2023*, from DOGAMI web site: https://gis.dogami.oregon.gov/maps/slido/.

3.2.3 Surface Rupture

3.2.3.1 Faulting

Although the site is situated in a region of the country with known active faults and historic seismic activity, no known faults exist on or immediately adjacent to the site. Therefore, the risk of surface rupture at the site due to faulting is considered low.

3.2.3.2 Lateral Spread

Surface rupture due to lateral spread can occur on sites underlain by liquefiable soils that are located on or immediately adjacent to slopes steeper than about 3 degrees (20H:1V), and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. During lateral spread, the materials overlying the liquefied soils are subject to lateral movement downslope or toward the free face. Based on the relatively level topography at the site and the discontinuous nature of the liquefiable soil layers, the risk of damage associated with lateral spread is negligible.

4.0 FOUNDATION SETTLEMENT ANALYSES

CGT performed settlement analyses to estimate post-construction settlements of conventional shallow spread foundations supporting structural loads for the proposed building. The analyses were based on subsurface data collected from the drilled borings, laboratory testing performed on collected soil samples, the loadings detailed in Section 1.1, and the following assumptions:

- Building shallow foundations are designed assuming a maximum soil bearing pressure of 2,000 psf.
- Building shallow foundations are established on the native silty soils (MH, SM) at a depth of about 4 feet below existing site grades and no subgrade improvement is performed.

The following table presents the results of our settlement analyses for shallow foundations supporting the proposed building.

Table 2 Estimated Foundation Settlements from Structural Loads

Foundation Type	Maximum Loading[1]	Foundation Bearing Pressure Used	Estimated Settlement (inches)[2]
Individual (Column Pad)	395 kips	2,000 psf	Up to 2½
	200 kips	2,000 psf	Up to 1¾
	100 kips	2,000 psf	Up to 1½
Continuous Wall	12 kips per lineal foot	2,000 psf	Up to 1½
	6 kips per lineal foot	2,000 psf	Up to 1

[1] Consistent with loading described in Section 1.1 of this report and considers dead and long-term live loading. If increased loads are estimated for the building, the geotechnical engineer should be consulted to review loading conditions.

[2] Estimated settlement resulting from consolidation/densification of subgrade soils (from sustained loading).

Based on our experience with similar projects, we anticipate the maximum allowable total post-construction settlements of building foundations is 1 inch. Similarly, we anticipate the maximum allowable differential settlement of foundations (considering adjacent columns and/or walls) is ½ inch. To determine if the settlements could be reduced to those levels, we modeled “granular pads” below the column pad and heavier wall foundations. Our analyses showed the required subgrade improvement (taking the form of over-excavation and replacement with granular structural fill) would need to extend to considerable depths[11] and, recognizing the presence of relatively shallow groundwater and other considerations, is not anticipated to be cost effective for the project. As an alternative, we recommend an alternative form of ground improvement [granular piers (GPs)] be considered to mitigate the excessive settlements.

5.0 CONCLUSIONS

Based on the results of our field explorations and analyses, the site may be developed as described in Section 1.1 of this report, provided the recommendations presented in this report are incorporated into the design and development. The primary geotechnical considerations for the project are summarized in the following sections.

5.1 Consolidation (Settlement) Potential from Building Loads

As indicated in Section 4.0 above, our analyses indicated that consolidation settlements from sustained structural loads associated with the planned building will be up to about 2½ inches. In the absence of ground improvement, the estimated total and differential settlements are <u>not</u> expected to be tolerable for the proposed building if supported on conventional shallow spread foundations.

Subsequent to completion of our analyses, but prior to issuance of this written report, we reviewed this consideration with the project design team members. Based on recent discussions, the project team indicated their preference to proceed with shallow foundations supported on granular piers (GPs). GPs are an intermediate, foundation system that consists of nominally spaced, aggregate piers that provide shallow foundation bearing support and assist with controlling settlement. Through proper design and construction, we anticipate this approach should help reduce total and differential, consolidation settlements to a level acceptable for supporting the building on conventional shallow foundations. Geotechnical recommendations for use in design and construction of GPs are presented in Section 6.5 of this report.

5.2 Moisture Sensitive Soils

The near surface fine-grained silty soils (MH, SM) are susceptible to disturbance during wet weather. Trafficability of these soils may be difficult, and significant damage to the subgrade could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. In the event that construction occurs during wet weather, CGT recommends that measures be implemented to protect the fine-grained subgrade in areas of repeated construction traffic and within footing excavations. Geotechnical recommendations for wet weather construction are presented in Section 6.3 of this report.

5.3 Shallow Groundwater

As indicated in Section 2.3.3 above, the groundwater level (phreatic surface) was encountered at depths of about 1 to 12 feet bgs in the borings advanced at the site in early December 2023. The following geotechnical conclusions are presented relative to the groundwater levels observed at this site:

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[11] For the maximum column loading indicated in Table 2, our analyses indicates granular pads for ground-level column pad foundations would need to be 5+ feet in depth to reduce post-construction settlement to an acceptable level.

- Some seasonal and annual fluctuation[12] of the groundwater level should be anticipated at this site. With regard to the building pad, we recommend the “seasonal high groundwater level” be assigned at an elevation of 265 feet. Although not anticipated based on provided information, in the event the building ground floor will be established within 2 feet of that elevation, the geotechnical engineer should be consulted to review the proposed construction and provide supplemental recommendations for waterproofing and/or underslab drainage, if warranted.
- Within planned pavement areas, we recommend site grades be maintained at their current elevations to the extent possible. Permanent cuts at the site extending below a depth of 1-foot bgs, if proposed, should be reviewed by the geotechnical engineer.
- The relatively shallow groundwater effectively precludes infiltration of stormwater collected from new impervious areas of the site. Notwithstanding the preceding, in the event stormwater infiltration facilities(ies) are to be pursued at this site, the geotechnical engineer should be consulted to review potential siting and depth(s) of those facilities.
- With regard to construction, depending on the time of year (and the area of the site) that site work takes place, groundwater may be encountered when excavations extend below a few feet below existing ground surface and should be factored. Dewatering plans will rest with the project contractor. Additional discussion of dewatering considerations is presented in Section 6.2.2 of this report.

6.0 RECOMMENDATIONS

The recommendations presented in this report are based on the information provided to us, results of our field investigation and analyses, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation. CGT should be consulted for further recommendations if the design of the proposed development changes and/or variations or undesirable geotechnical conditions are encountered during site development.

6.1 Site Preparation

6.1.1 Stripping & Grubbing

Existing vegetation, topsoil, and rooted soils (OL) should be removed from within, and for a minimum 5-foot margin around, proposed building pad, structural fill, and pavement areas. Based on the results of our field explorations, topsoil stripping depths are anticipated to be on the order of about ¼-foot bgs. These materials may be deeper or shallower at locations away from the completed explorations. The geotechnical engineer’s representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped surface vegetation and rooted soils should be transported off-site for disposal, or stockpiled for later use in landscaped areas.

Grubbing of trees should include the removal of the root mass and roots greater than ½ inch in diameter. Grubbed materials should be transported off-site for disposal. Root masses from larger trees may extend greater than 3 feet bgs. Where root masses are removed, the resulting excavation should be properly backfilled with structural fill in conformance with Section 6.4 of this report.

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[12] The client is advised that monitoring of the groundwater level at the site could be performed at the site via periodic explorations (e.g. hand auger borings) and/or through the installation of piezometers. Such services are outside the scope of this current assignment, but could be provided, upon request, for an additional fee.

6.1.2 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath the new building, pavements, and hardscaping features should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill in conformance with Section 6.4 this report. Buried structures (i.e. footings, foundation walls, retaining walls, slabs-on-grade, tanks, etc.), if encountered during site development, should be completely removed and replaced with structural fill in conformance with Section 6.4 of this report.

6.1.3 Subgrade Preparation - Building Pad & Pavement Areas

After site preparation as recommended above, but prior to placement of structural fill and/or aggregate base, the geotechnical engineer’s representative should observe the exposed subgrade soils in order to identify areas of excessive yielding through either proof rolling or probing. Proof rolling of subgrade soils is typically conducted during dry weather using a fully-loaded, 10- to 12-cubic-yard, tandem-axle, tire-mounted, dump truck or equivalent weighted water truck. Areas of limited access or that appear too soft or wet to support proof rolling equipment should be evaluated by probing. During wet weather, subgrade preparation should be performed in general accordance with the recommendations presented in Section 6.3 of this report. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 6.4.2 of this report.

The elastic silt (MH) soils should be kept moist, near optimum moisture content, and not allowed to dry out. If allowed to dry below optimum moisture content, to a point where surface cracking appears in the subgrade, the affected material should be over-excavated and replaced with imported granular structural fill.

6.1.4 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable City, County, and State regulations.

6.2 Temporary Excavations

6.2.1 Overview

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for the anticipated site cuts as described earlier in this report. All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A “competent person,” as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT’s current role on the project does not include review or oversight of excavation safety.

6.2.2 Dewatering

As indicated in Section 2.3.3 above, groundwater was encountered at depths of approximately 1 to 12 feet bgs within the borings advanced at the site in early December 2023. The soils encountered at these depths exhibited relatively high fines content and are anticipated to exhibit low to moderate rates of transmissivity. Accordingly, we would expect low to moderate seepage when excavations extend below the groundwater level. Pumping from sumps may be effective in removing groundwater within shallow or localized excavations at the site. Pumping from multiple well points will likely be required for larger excavations and those extending below the groundwater level. The sumps or wells should be installed to remove water to a depth of at least 2 feet below the lowest elevation of the excavation, and should be installed and put into operation prior to commencing excavation. With regards to temporary dewatering, the contractor or his representative should determine the appropriate size, number, and location of sump pumps or wells. The project civil engineer should evaluate requirements for disposal of the resultant discharge.

6.2.3 OSHA Soil Types

For use in the planning and construction of temporary excavations up to 10 feet in depth, an OSHA soil type “A” may be used for the native elastic silt (MH) encountered near the surface of the site. In the event groundwater seepage is observed within temporary excavations within this soil, the sidewalls should be flattened in accordance with OSHA soil type “C”. Similarly, an OSHA soil type “C” should be used for the native silty sand (SM) encountered at depth in the borings.

6.2.4 Utility Trenches

Temporary trench cuts should stand near vertical to depths of approximately 4 feet in the native, elastic silt (MH) encountered near the surface of the site. If groundwater seepage undermines the stability of the trench, or if sidewall caving is observed during excavation, the sidewalls should be flattened or shored. Depending on the time of year trench excavations occur, trench dewatering may be required in order to maintain dry working conditions. A discussion of dewatering of temporary excavations is presented in Section 6.2.2 above. If groundwater is encountered, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 6.4.3.

6.2.5 Excavations Near Foundations

Excavations near footings should not extend within a 1 horizontal to 1 vertical (1H:1V) plane projected out and down from the outside, bottom edge of the footings. In the event excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

6.3 Wet Weather Considerations

For planning purposes, the wet season should be considered to extend from late September to late June. It is our experience that dry weather working conditions should prevail between early July and mid-September. Notwithstanding the above, soil conditions should be evaluated in the field by the geotechnical engineer’s representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

6.3.1 Overview

Due to their fines content, the on-site silty soils (MH, SM) are susceptible to disturbance during wet weather. Trafficability of these soils may be difficult, and significant damage to subgrade soils could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. For wet weather construction, site preparation activities may need to be accomplished using track-mounted equipment, loading removed material onto trucks supported on granular haul roads, or other methods to limit soil disturbance. The geotechnical engineer's representative should evaluate the subgrade during excavation by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 6.4.2.

6.3.2 Geotextile Separation Fabric

We recommend a geotextile separation fabric be placed to serve as a barrier between the prepared subgrade and granular fill/base rock in areas of repeated or heavy construction traffic. The geotextile fabric should meet the requirements presented in the current Oregon Department of Transportation (ODOT) Standard Specification for Construction (ODOT SSC), Section 02320.

6.3.3 Granular Working Surfaces (Haul Roads & Staging Areas)

Haul roads subjected to repeated heavy, tire-mounted, construction traffic (e.g. dump trucks, concrete trucks, etc.) will require a minimum of 18 inches of imported granular material. For light staging areas, 12 inches of imported granular material is typically sufficient. Additional granular material or geo-grid reinforcement may be recommended based on site conditions and/or loading at the time of construction. The imported granular material should be in conformance with Section 6.4.2 and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric (Section 6.3.2) prior to placement of the imported granular material. The imported granular material should be placed in a single lift (up to 24 inches deep) and compacted using a smooth-drum, non-vibratory roller until well-keyed.

6.3.4 Footing Subgrade Protection

A minimum of 3 inches of imported granular material (crushed rock) is recommended to protect fine-grained (silty), footing subgrades from foot traffic during inclement weather. The imported granular material should be in conformance with Section 6.4.2. The maximum particle size should be limited to 1 inch. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using non-vibratory equipment until well keyed.

Surface water should not be allowed to collect in footing excavations. The excavations should be draped and/or provided with sumps to preclude water accumulation during inclement weather.

6.4 Structural Fill

The geotechnical engineer should be provided the opportunity to review all materials considered for use as structural fill (prior to placement). Samples of the proposed fill materials should be submitted to the geotechnical engineer a minimum of 5 business days prior their use on site[13]. The geotechnical engineer's representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction may take the form of in-place density tests and/or proof roll tests with suitable equipment. Structural fill should be evaluated at intervals not exceeding every 2 vertical feet as the fill is being placed.

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[13] Laboratory testing for moisture density relationship (Proctor) is required. Tests for gradation may be required.

6.4.1 On-Site Soils – General Use

6.4.1.1 Elastic Silt (MH), Silty Sand (SM)

Re-use of these soils as structural fill may be difficult because these soils are sensitive to small changes in moisture content and are difficult, if not impossible, to adequately compact during wet weather. We anticipate the moisture content of these soils will be higher than the optimum moisture content for satisfactory compaction. Therefore, moisture conditioning (drying) should be expected in order to achieve adequate compaction. If used as structural fill, these soils should be free of organic matter, debris, and particles larger than 4 inches. When used as structural fill, these soils should be placed in lifts with a maximum pre-compaction thickness of about 8 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 92 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor).

If the on-site materials cannot be properly moisture-conditioned and/or processed, we recommend using imported granular material for structural fill.

6.4.2 Imported Granular Structural Fill – General Use

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 4 inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. For fine-grading purposes, the maximum particle size should be limited to 1½ inches. The percentage of fines can be increased to 12 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. Imported granular fill material should be placed in lifts with a maximum thickness of about 12 inches, and compacted to not less than 95 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor). Proper moisture conditioning and the use of vibratory equipment will facilitate compaction of these materials.

Granular fill materials with high percentages of particle sizes in excess of 1½ inches are considered non-moisture-density testable materials. As an alternative to conventional density testing, compaction of these materials should be evaluated by proof roll test observation (deflection tests), where accepted by the geotechnical engineer.

6.4.3 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of a minimum of 1 foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

6.4.4 Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch-thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

Table 3 Utility Trench Backfill Compaction Recommendations

Backfill Zone	Recommended Minimum Relative Compaction	
	Structural Areas[1,2]	Landscaping Areas
Pipe Base and Within Pipe Zone	90% ASTM D1557 or pipe manufacturer's recommendation	85% ASTM D1557 or pipe manufacturer's recommendation
Above Pipe Zone	92% ASTM D1557	88% ASTM D1557
Within 3 Feet of Design Subgrade	95% ASTM D1557	90% ASTM D1557

[1] Includes proposed building, pavement areas, structural fill areas, exterior hardscaping, etc.
[2] Or as specified by the local jurisdiction where located within the public right of way.

6.4.5 Controlled Low-Strength Material (CLSM)

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. CLSM is sometimes referred to as "controlled density fill" or CDF. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM be in conformance with Section 00442 of the most recent, ODOT SSC. The geotechnical engineer's representative should observe placement of the CLSM and obtain samples for compression testing in accordance with ASTM D4832. As a guideline, for each day's placement, two compressive strength specimens from the same CLSM sample should be tested. The results of the two individual compressive strength tests should be averaged to obtain the reported 28-day compressive strength. If CLSM is considered for use on this site, please contact the geotechnical engineer for site-specific and application-specific recommendations.

6.5 Building Foundations

As indicated in Section 5.1 above, we recommend granular piers (GPs) be used to support shallow foundations associated with the proposed building. GPs are an intermediate foundation system that consists of nominally spaced aggregate piers that provide shallow foundation bearing support and assist with controlling settlement. We recommend GPs be designed and installed by an experienced, qualified, design-build firm specialized in this ground improvement technique. GPs and shallow foundations supported by GPs should be constructed in accordance with plans, details, and specifications provided by the GP design-build firm.

6.5.1 Recommended Foundation Design & Performance Criteria

For the purposes of planning and design, subject to review of the design team, we recommend the following criteria be used for design and construction of shallow foundations associated with the proposed building and supported on GP-improved ground:

Table 4 Design & Performance Criteria for Shallow Foundations

Foundation Soil Parameter		Recommended Value for Design
Allowable net soil bearing pressure[1] (considering dead + long-term live loads)		6,000 psf
Allowable net soil bearing pressure[1] (considering dead + long-term live + transient loads)		8,000 psf
Maximum Allowable Settlement (from building loads)[1,2]		Total = 1 inch; Differential = ½ inch
Minimum Footing Width	Continuous Walls	18 inches
	Individual (Column Pad)	24 inches
Minimum Footing Embedment[3]		18 inches
Ultimate Sliding Coefficient[1,4]		0.40
Allowable Passive Lateral (Equivalent Fluid) Pressure[4]		150 pcf

[1] Recommended design objective for the granular pier (GP) improvement plans.

[2] Or as specified by the building structural engineer. Differential settlements should be measured between adjacent columns and/or walls.

[3] Relative to the lowest, permanent adjacent grade next to the subject foundation.

[4] Refer to Section 6.5.3 below for additional discussion.

6.5.2 Subgrade Preparation

Subgrade preparation of shallow foundations supported on GP-improved ground should be in conformance with the approved GP-design plans.

6.5.3 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings cast neat into excavations in suitable native soil or confined by imported granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.40 may be used when calculating resistance to sliding for footings founded on GP-improved ground. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

6.5.4 Subsurface Drainage

Subject to review of the GP designer, recognizing the predominantly fine-grained (silty) soils encountered at this site, we recommend placing foundation drains at the exterior, base elevations of perimeter continuous wall footings. Foundation drains should consist of a minimum 4-inch diameter, perforated, PVC drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should also be encased in a geotextile fabric in order to provide separation from the surrounding fine-grained soils. Foundation drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer's representative should observe the drains prior to backfilling. Roof drains should not be tied into foundation drains.

6.5.5 Soil Strength Parameters

We have provided recommended values for soil strength parameters, including drained friction angle (Φ'), effective cohesion (c'), total unit weight (γ_T), and undrained shear strength (S_u), for use in design of GPs in the following table. The parameters provided below were based on the results of the subsurface explorations, laboratory testing, published correlations with SPT and laboratory (index) test data, and our experience with similar soils.

Table 5 Soil Parameters Recommended for Use in Granular Pier Design

Depth (feet bgs)[1]	Description[2,3]	Recommended Soil Type	Soil Shear Strength Parameter[2,3]			
			Φ' (degrees)	c' (psf)	γ_T (pcf)	S_u (psf)
0 to 10	Native, Med. Stiff to Stiff, Elastic Silt (MH)	Phi + c'	30	100	115	900
10+	Native, Medium Dense, Silty Sand (SM)	Cohesionless	36	0	120	0

[1] Depth measured relative to existing site grades.
[2] Soil profile from boring B-2 were used for this model. If additional parameters are required, the geotechnical engineer should be consulted.
[3] We recommend modeling groundwater at elevation 265 feet MSL at this site.

6.6 Rigid Retaining Walls

The recommendations that follow are presented for use in design and construction of "site" retaining walls (i.e. walls that are not structurally-connected to, or relied upon for vertical support of structural loads associated with, the planned building). Retaining walls that will be structurally-connected to the building should be supported similarly to that selected for the building in accordance with Section 6.5 of this report.

6.6.1 Footings

6.6.1.1 Subgrade Preparation

Satisfactory subgrade support for retaining wall foundations can be obtained from the native, medium stiff to better elastic silt (MH), the native, medium dense to better silty sand (SM), or new structural fill that is properly placed and compacted on these materials during construction. These materials were first encountered at depths of about 5 feet bgs within our borings (B-1 and B-2) advanced in the vicinity of the building pad. The geotechnical engineer's representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or granular backfill (if required). If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 6.4.2. The maximum particle size of over-excavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

6.6.1.2 Minimum Footing Width & Embedment

Minimum footing widths should be in conformance with the most recent, Oregon Structural Specialty Code (OSSC). We recommend continuous wall footings have a minimum width of 18 inches. All footings should be founded at least 18 inches below the lowest, permanent adjacent grade.

6.6.1.3 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads. For foundations founded as recommended above and considering static loading only, total settlement of foundations is anticipated to be less than 1 inch.

6.6.1.4 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings cast neat into excavations in suitable native soil or confined by imported granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.35 may be used when calculating resistance to sliding for footings founded as described above. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

6.6.2 Wall Drains

We recommend placing retaining wall drains at the base elevation of the heel of retaining wall footings. Retaining wall drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile fabric in order to provide separation from the surrounding soils. Retaining wall drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer's representative should be contacted to observe the drains prior to backfilling. Roof or area drains should not be tied into retaining wall drains.

6.6.3 Wall Backfill

Retaining walls should be backfilled with imported granular structural fill in conformance with Section 6.4.2 and contain less than 5 percent passing the U.S. Standard No. 200 Sieve. The backfill should be compacted to a minimum of 90 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). When placing fill behind walls, care must be taken to minimize undue lateral loads on the walls. Heavy compaction equipment should be kept at least "H" feet from the back of the walls, where "H" is the height of the wall. Light mechanical or hand tamping equipment should be used for compaction of backfill materials within "H" feet of the back of the walls.

6.6.4 Design Parameters & Limitations

For rigid retaining walls founded, backfilled, and drained as recommended above, the following table presents parameters recommended for design.

Table 6 Design Parameters for Rigid Retaining Walls

Retaining Wall Condition	Modeled Backfill Condition	Static Equivalent Fluid Pressure (S_A)[1]	Seismic Equivalent Fluid Pressure (S_{AE}) [1,2]	Surcharge from Uniform Load, q, Acting on Backfill Behind Retaining Wall
Not Restrained from Rotation	Level (i=0)	28 pcf	38 pcf	0.22*q
Restrained from Rotation	Level (i=0)	50 pcf	52 pcf	0.38*q

[1] Refer to the attached Figure 4 for a graphical representation of static and seismic loading conditions. Seismic resultant force acts at 0.6H above the base of the wall.

[2] Seismic (dynamic) lateral loads were computed using the Mononobe-Okabe Equation as presented in the 1997 Federal Highway Administration (FHWA) design manual. Static and seismic equivalent fluid pressures are not additive.

The above design recommendations are based on the assumptions that:

- The walls consist of concrete cantilevered retaining walls (β = 0 and δ = 24 degrees, see Figure 4).
- The walls are 10 feet or less in height.
- The backfill is drained and consists of imported granular structural fill (ϕ = 38 degrees).
- No point, line, or strip load surcharges are imposed behind the walls.
- The grade behind the wall is level, or sloping down and away from the wall, for a distance of 15 feet or more from the wall.
- The grade in front of the walls is level or ascending for a distance of at least 5 feet from the wall.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

6.6.5 Surcharge Loads

Where present, surcharges from adjacent site features (i.e. buildings, slabs, pavements, etc.) should be evaluated in design of retaining walls at the site. Methods for calculating lateral pressures on rigid retaining walls from strip, line, and vertical point loads are presented on the attached Figure 5.

6.7 Floor Slabs

6.7.1 Subgrade Preparation

Satisfactory subgrade support for slabs constructed on grade, supporting up to 250 psf area loading, can be obtained from the native, medium stiff to better elastic silt (MH), the native, medium dense to better silty sand (SM), or new structural fill that is properly placed and compacted on these materials during construction. The geotechnical engineer's representative should be contacted to observe subgrade conditions prior to placement of structural fill or aggregate base. If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 6.4.2.

6.7.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 6-inch-thick layer of crushed rock (base rock). Floor slab base rock should consist of well-graded granular material (crushed rock) containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Floor slab base rock should be placed in one lift and compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). We recommend "choking" the surface of the base rock with sand just prior to concrete placement. Choking means the voids between the largest aggregate particles are filled with sand, but does not provide a layer of sand above the base rock. Choking the base rock surface reduces the lateral restraint on the bottom of the concrete during curing. Choking the base rock also reduces punctures in vapor retarding membranes due to foot traffic where such membranes are used.

6.7.3 Design Considerations

For floor slabs constructed as recommended, an effective modulus of subgrade reaction of 150 pounds per cubic inch (pci) is recommended for the design of the floor slab. A higher effective modulus of subgrade reaction can be obtained by increasing the base rock thickness. Please contact the geotechnical engineer for additional recommendations if a higher modulus is desired. Floor slabs constructed as recommended will likely settle less than ½ inch. For general floor slab construction, slabs should be jointed around columns and walls to permit slabs and foundations to settle differentially.

6.7.4 Subgrade Moisture Considerations

Liquid moisture and moisture vapor should be expected at the subgrade surface. The recommended crushed rock base is anticipated to provide protection against liquid moisture. Where moisture vapor emission through the slab must be minimized, e.g. impervious floor coverings, storage of moisture sensitive materials directly on the slab surface, etc., a vapor retarding membrane or vapor barrier below the slab should be considered. Factors such as cost, special considerations for construction, floor coverings, and end use suggest that the decision regarding a vapor retarding membrane or vapor barrier be made by the architect and owner.

If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction. In some cases, this indicates placement of concrete directly on the vapor retarder or barrier. Please note that the placement of concrete directly on impervious membranes increases the risk of plastic shrinkage cracking and slab curling in the concrete. Construction practices to reduce or eliminate such risk, as described in ACI 302, should be employed during concrete placement.

6.8 Pavements

6.8.1 Subgrade Preparation

Satisfactory subgrade support for pavements can be obtained from the native, medium stiff to better elastic silt (MH), the native, medium dense to better silty sand (SM), or new structural fill that is properly placed and compacted on these materials during construction. The geotechnical engineer's representative should be contacted to observe pavement subgrade conditions prior to placement of structural fill or aggregate base. If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 6.4.2. Pavement subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer.

6.8.2 Traffic Classifications

Recognizing that traffic data has not been provided, CGT has considered four levels of traffic demand for review and design of pavement sections. We modeled the following four design cases (traffic levels) developed from the Asphalt Pavement Association of Oregon (APAO):

- *APAO Level I (Very Light):* This design case considers typical average daily truck traffic (ADTT) of 1 per day over 20 years. Among others, examples under this loading consist of passenger car parking stalls, residential driveways, and seasonal recreational roads.
- *APAO Level II (Light):* This design case considers typical ADTT of 2 to 7 per day over 20 years. Examples under this loading consist of residential streets and parking lots of less than 500 stalls.
- *APAO Level III (Low Moderate):* This design case considers typical ADTT of 7 to 14 per day over 20 years. Among others, examples under this loading consist of urban minor collector streets and parking lots with more than 500 stalls.
- *APAO Level IV (Moderate):* This design case considers typical ADTT of 14 to 35 per day over 20 years. Among others, examples under this loading consist of urban minor arterial streets and residential streets with bus routes.

We recommend the owner and design team review the traffic levels presented above and select those that most accurately represent anticipated daily truck traffic for select new pavements.

6.8.3 Asphalt Concrete Pavements

6.8.3.1 Input Parameters

Design of the asphalt concrete (AC) pavement sections presented below were based on the parameters presented in the following table, the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" manual, and pavement design manuals presented by APAO and ODOT[14]. If any of the items listed need revision, please contact us and we will reassess the provided design sections.

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[14] Oregon Department of Transportation (ODOT) Pavement Design Guide, January 2019.

Table 7 Input Parameters Used in AC Pavement Design

Input Parameter	Design Value[1]	Input Parameter		Design Value[1]
Pavement Design Life	20 years	Resilient Modulus	Subgrade (Native Silty Soils)[4]	5,000 psi
Annual Percent Growth	0 percent		Crushed Aggregate Base[2]	20,000 psi
Initial Serviceability[2]	4.2	Structural Coefficient[2]	Crushed Aggregate Base	0.10
Terminal Serviceability[2]	2.5		Asphalt	0.42
Reliability[2]	75 percent	Vehicle Traffic (range in ESAL[5])	APAO Level I (Very Light)	Less than 10,000
Standard Deviation[2]	0.49		APAO Level II (Light)	Less than 50,000
Drainage Factor[3]	1.0		APAO Level III (Low Moderate)	Less than 100,000
---	---		APAO Level IV (Moderate)	Less than 250,000

[1] If any of the above parameters are incorrect, please contact us so that we may revise our recommendations, if warranted.

[2] Value based on guidelines presented in the ODOT Pavement Design Guide.

[3] Assumes good drainage away from pavement, base, and subgrade is achieved by proper crowning of subgrades.

[4] Values based on experience with similar soils in the region.

[5] ESAL = Total 18-Kip equivalent single axle load. Traffic levels taken from Table 3.1 of APAO manual. If actual traffic levels will be above those identified above, the geotechnical engineer should be consulted.

6.8.3.2 Recommended Minimum Sections

The following table presents the minimum AC pavement sections for various traffic loads indicated in the preceding table, based on the referenced AASHTO procedures.

Table 5 Recommended Minimum AC Pavement Sections

Material	APAO Traffic Loading			
	Level I	Level II	Level III	Level IV
Asphalt Pavement (inches)	3	3½	4	4½
Crushed Aggregate Base (inches)[1]	6	8	10	11
Subgrade Soils	Prepared in conformance with Section 6.8.1 of this report.			

[1] Thickness shown assumes dry weather construction. A granular sub-base section and/or a geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade. Refer to Section 6.3 for additional discussion.

6.8.3.3 AC Pavement Materials

Aggregate Base: We recommend pavement aggregate base consist of dense-graded aggregate in conformance with Section 02630.10 of the most recent ODOT SSC, with the following additional considerations. We recommend the material consist of crushed rock or gravel, have a maximum particle size of 1½ inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Aggregate base should be compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor).

Asphalt Concrete: We recommend asphalt pavement consist of Level 2, ½-inch, dense-graded AC in conformance with the most recent ODOT SSC. Asphalt pavement should be compacted to at least 91 percent of the material's theoretical maximum density as determined in general accordance with ASTM D2041 (Rice Specific Gravity).

6.8.4 Rigid (Concrete) Pavements

6.8.4.1 Input Parameters

Design of the rigid (Portland Cement Concrete, PCC) pavement sections presented below was based on the parameters presented in the following table and the referenced AASHTO design manual. If any of the items listed need revision, please contact us and we will reassess the provided design sections. Jointing, reinforcement, and surface finish should be performed in accordance with the project civil engineer, architect, and owner requirements.

Table 6 Input Parameters Used in PCC Pavement Design

Parameter / Discussion		Design Value
Subgrade Modulus (k-value)		150 pci
Standard Deviation[1]		0.39
Load Transfer Devices incorporated?		Yes; Load Transfer Coefficient = 3.2
Minimum Concrete Modulus of Rupture		600 psi
Concrete Elastic Modulus		5.0 x 10^6 psi
Minimum Air-Entrained Concrete Compressive Strength		4,000 psi
Vehicle Traffic[2] (range in ESAL)	APAO Level I (Very Light)	Less than 10,000
	APAO Level II (Light)	Less than 50,000
	APAO Level III (Low Moderate)	Less than 100,000
	APAO Level IV (Moderate)	Less than 250,000

[1] Value based on guidelines presented in the ODOT Pavement Design Guide.

[2] ESAL = Total 18-Kip equivalent single axle load. If actual traffic levels will be above those identified above, the geotechnical engineer should be consulted.

6.8.4.2 Recommended Minimum Sections

The following table presents the recommended minimum concrete pavement sections based on the referenced AASHTO procedures.

Table 7 Recommended Minimum PCC Pavement Sections

Material	APAO Traffic Loading			
	Level I	Level II	Level III	Level IV
Portland Cement Concrete, PCC[1] (inches)	5	5½	6	7
All-Weather Base[2,3] (inches)	4	4	4	4
Subgrade Soils	Prepared in conformance with Section 6.8.1 of this report			

[1] Concrete strength and other properties should be in conformance with Table 6 above.

[2] All-weather base (base rock) should be a minimum of 4 inches thick.

[3] Thickness shown assumes dry weather construction. A granular sub-base section and/or a geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade. Refer to Section 6.3 for additional discussion.

6.8.4.3 PCC Pavement Materials

All-Weather Base: We recommend all-weather base consist of dense-graded aggregate in conformance with Section 02630.10 of the most recent ODOT SSC, with the following additional considerations. We recommend the material consist have a maximum particle size of ¾-inch and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Aggregate base should be compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor).

PCC Pavement: Portland cement concrete (PCC) pavement should be in conformance with Section 02001 of the most recent ODOT SSC and meet the properties detailed in Table 6 above.

6.9 Additional Considerations

6.9.1 Drainage

Subsurface drains should be connected to the nearest storm drain or other suitable discharge point. Paved surfaces and grading near or adjacent to the building should be sloped to drain away from the building. Surface water from paved surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains, retaining wall drains, or onto site slopes.

6.9.2 Expansive Potential

The near surface native soils consist mostly of moderate plasticity elastic silt soils. Based on our experience with similar soils in the vicinity of the site, these soils are not considered to be susceptible to appreciable movements from changes in moisture content. Accordingly, no special considerations are required to mitigate expansive potential of the near surface soils at the site.

7.0 RECOMMENDED ADDITIONAL SERVICES

7.1 Design Review

Geotechnical design review is of paramount importance. We recommend the geotechnical design review take place prior to releasing bid packets to contractors.

7.2 Observation of Construction

Satisfactory earthwork, foundation, floor slab, and pavement performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report. We recommend geotechnical engineer's representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer's representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site Stripping and Grubbing
- Installation of Granular Piers (GPs)
- Subgrade Preparation for Shallow Foundations, Retaining Walls, Structural Fills, Floor Slabs, and Pavements
- Compaction of Structural Fill, Retaining Wall Backfill, and Utility Trench Backfill
- Compaction of Base Rock for Floor Slabs and Pavements
- Compaction of Asphalt Concrete for Pavements

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

8.0 LIMITATIONS

We have prepared this report for use by the owner/developer and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are forwarded to assist in the planning and design process and are not intended to be, nor should they be construed as, a warranty of subsurface conditions.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from our explorations. If subsurface conditions vary from those encountered in our site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for ensuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review and construction phase testing and observation services are beyond the scope of our current assignment, but will be provided for an additional fee.

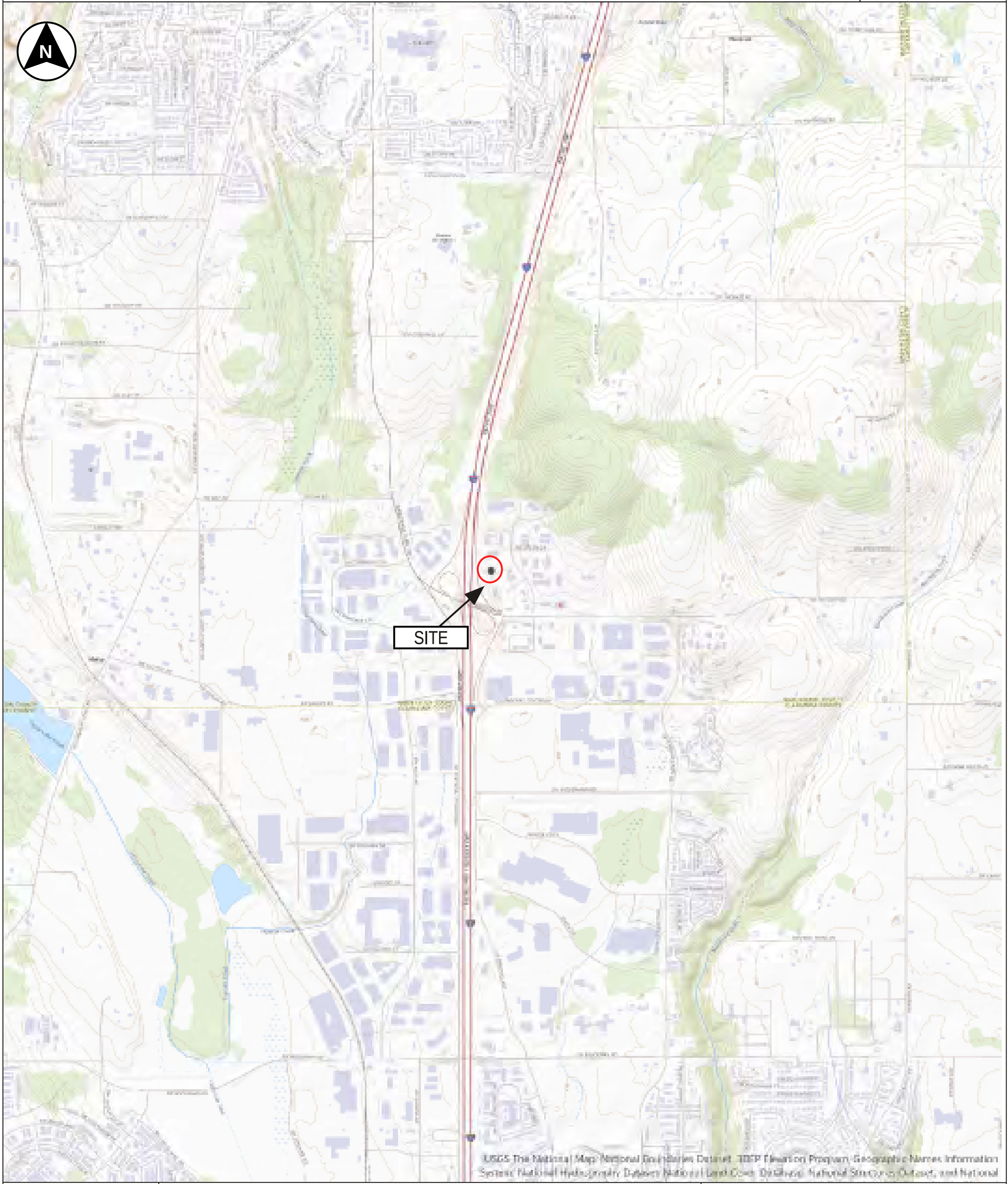
The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor’s methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of three years.

RON TONKIN GRAN TURISMO LAMBORGHINI DEALERSHIP - WILSONVILLE, OREGON
Project Number G2306033

FIGURE 1

Site Location



USGS The National Map: National Boundaries Dataset, 3DEP Elevation Program, Geographic Names Information System, National Hydrography Dataset, National Land Cover Database, National Structures Dataset, and National

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Drafted by: MDI

USGS Topographic base map created with The National Map, 2020, at https://viewer.nationalmap.gov/advanced-viewer/

Township 3 South, Range 1 West, Section 2, Willamette Meridian

Latitude: 45.337419° North
Longitude: 122.767954° West

1 Inch = 2,000 feet

0 2000 4000

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Project Number G2306033

FIGURE 2

Site Plan

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Drafted by: BG

LEGEND

B-1 (12') Hollow-stem auger boring. Depth to groundwater shown in ().

1 Orientation of site photographs shown on Figure 3

1 Inch = 60 Feet

0 60 120

NOTES: Drawing based on sheet A-101, “Site Plan”, produced by Axis Design Group on 10/04/2023 and 2021 aerial imagery, provided by Wilsonville Maps, www.wilsonvillemaps.com, accessed December 2, 2023. All locations are approximate.

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Project Number G2306033

FIGURE 3

Site Photographs

Photograph 1

Photograph 2

Photograph 3

Photograph 4

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See Figure 2 for approximate photograph locations and directions. Photographs were taken at the time of our fieldwork.

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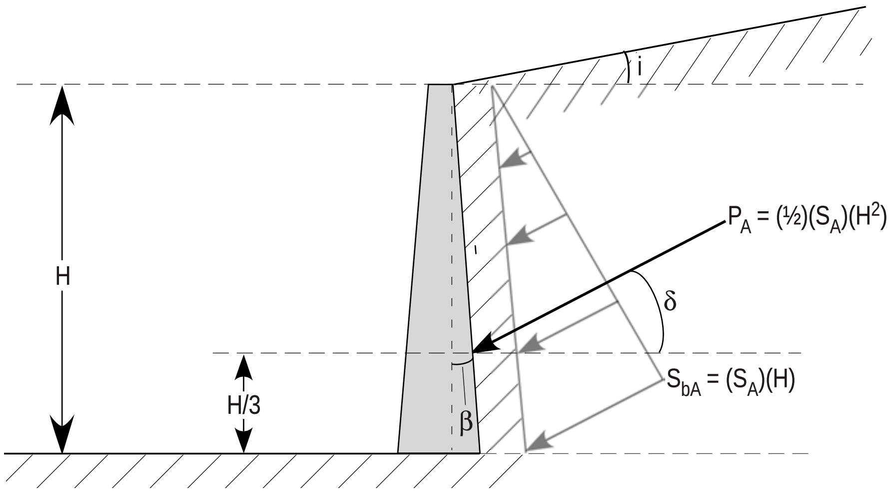
Project Number G2306033

FIGURE 4

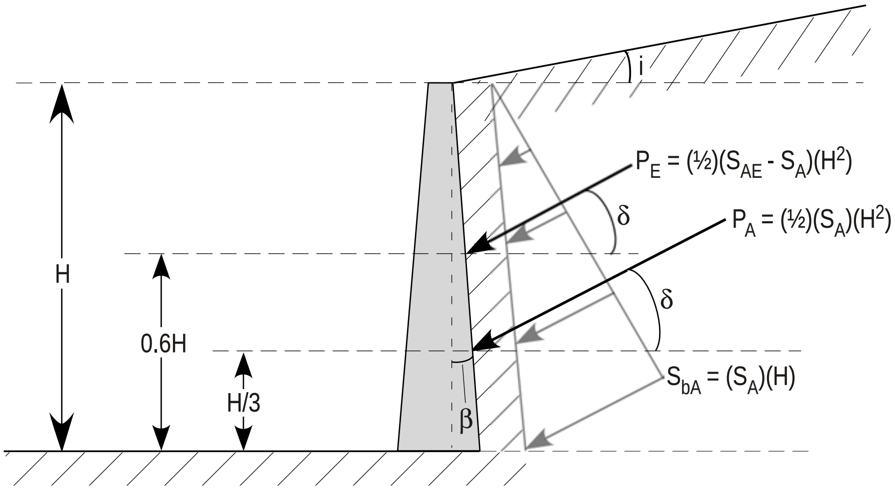
Retaining Walls

ACTIVE LATERAL PRESSURE DISTRIBUTION

STATIC LOADING CONDITIONS



SEISMIC LOADING CONDITIONS



LEGEND

S_A = Active lateral equivalent fluid pressure (lb/ft^3)*

S_{bA} = Active lateral earth pressure (static) at the bottom of wall (lb/ft^3)

S_{AE} = Active total (static + seismic) equivalent fluid pressure (lb/ft^3)*

i = Slope of backfill, relative to horizontal (degrees)**

β = Slope of back of wall, relative to vertical (degrees)**

P_A = Static active thrust force acting at H/3 from bottom of retaining wall (lb/ft)

P_E = Dynamic active thrust force acting at 0.6H from bottom of retaining wall (lb/ft)

δ = Angle from normal of back of wall (degrees). Based on friction developing between wall and backfill**

*Refer to report text for calculated values **Refer to report text for modeled/assumed values

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Notes

1. Uniform pressure distribution of seismic loading is based on empirical evaluations [Sherif et al, 1982 and Whitman, 1990].
2. Placement of seismic resultant force at 0.6H is based on wall behavior and model test results [Whitman, 1990].

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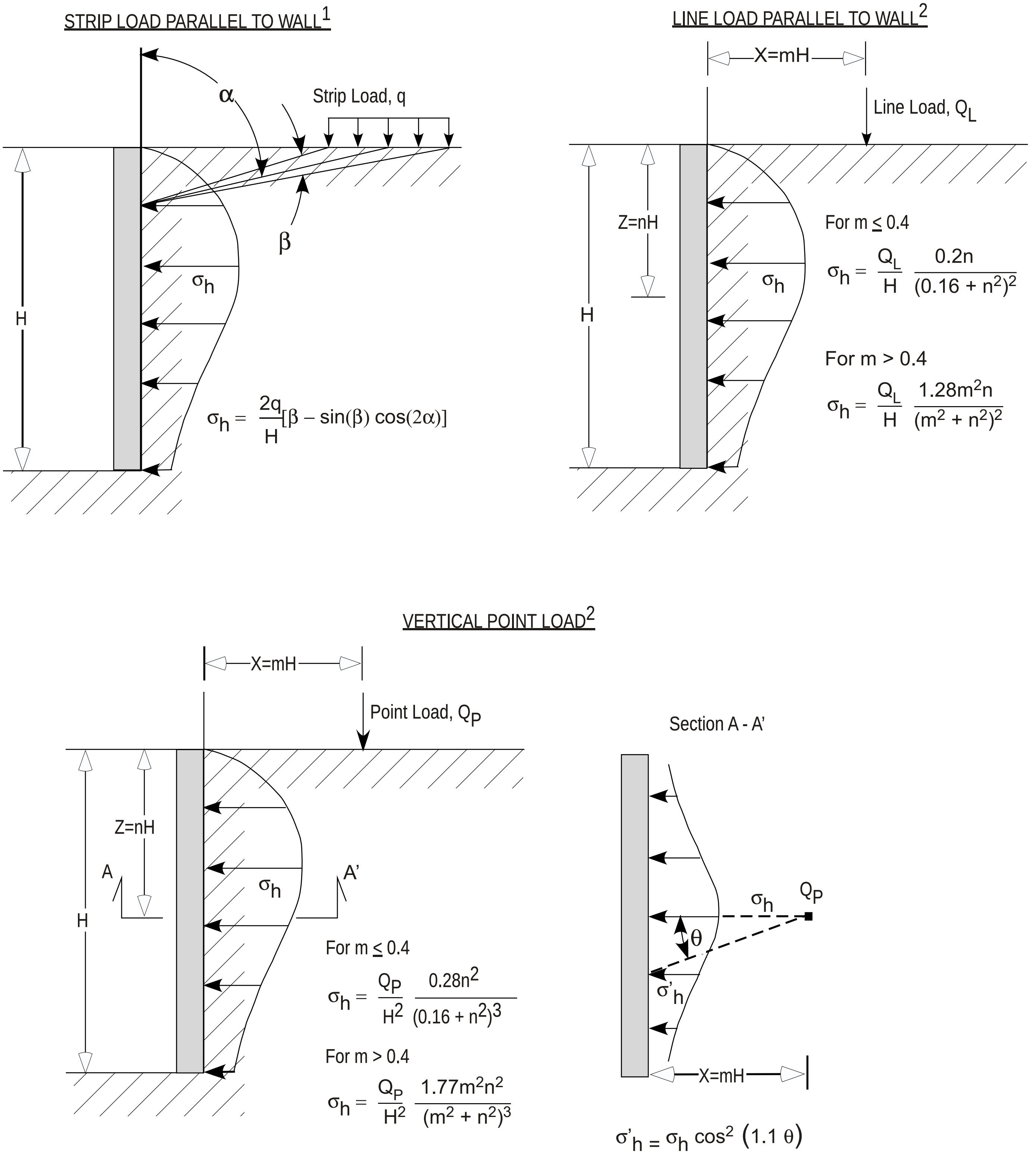
FIGURE 5
Retaining Wall Surcharge



Strip load parallel to wall:

$$\sigma_h = \frac{2q}{H}[\beta - \sin(\beta)\cos(2\alpha)]$$

Line load parallel to wall:

For $m \leq 0.4$

$$\sigma_h = \frac{Q_L}{H}\ \frac{0.2n}{(0.16 + n^2)^2}$$

For $m > 0.4$

$$\sigma_h = \frac{Q_L}{H}\ \frac{1.28m^2n}{(m^2 + n^2)^2}$$

Vertical point load:

For $m \leq 0.4$

$$\sigma_h = \frac{Q_P}{H^2}\ \frac{0.28n^2}{(0.16 + n^2)^3}$$

For $m > 0.4$

$$\sigma_h = \frac{Q_P}{H^2}\ \frac{1.77m^2n^2}{(m^2 + n^2)^3}$$

$$\sigma'_h = \sigma_h \cos^2(1.1\,\theta)$$

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Notes: 1. Das, Principles of Geotechnical Engineering, 1990 Edition.
2. NAVFAC Design Manual 7.06.

Refer to the referenced design manuals for additional guidance. Contact CGT if there are any questions with modeling surcharge loads.

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Appendix A: Subsurface Investigation and Laboratory Testing

Ron Tonkin Gran Turismo Lamborghini Dealership
Lot South of 25195 SW Parkway Avenue
Wilsonville, Oregon

CGT Project Number G2306033

May 13, 2024

Prepared For:

Celia Tonkin
Ron Tonkin Gran Turismo
25300 SW Parkway Avenue
Wilsonville, Oregon 97070

Prepared by
Carlson Geotechnical

Exploration Key...Figure A1
Soil Classification...Figure A2
Drilled Boring Logs ....Figures A3 – A7

A.1.0 SUBSURFACE INVESTIGATION

Our field investigation consisted of five drilled borings completed on December 4, 2023. The exploration locations are shown on the Site Plan, attached to the geotechnical report as Figure 2. The exploration locations shown therein were determined based on measurements from existing site features (trees, pavements, etc.) and are approximate. Surface elevations indicated on the logs were estimated based on the topographic contours (by others) shown on the topographic survey provided by our client, and are approximate. The attached figures detail the exploration methods (Figure A1), soil classification criteria (Figure A2), and present detailed logs of the explorations (Figures A3 through A7), as discussed below.

A.1.1 Drilled Borings

CGT observed the advancement of five drilled borings (B-1 through B-5) at the site using a B58 track-mounted drill rig provided and operated by our subcontractor, PLI Systems of Hillsboro, Oregon. The borings were advanced using the hollow-stem auger drilling technique to depths ranging from approximately 6½ to 26½ feet below ground surface (bgs). Upon completion, the borings were backfilled with granular bentonite. Drilling wastes (cuttings and drilling fluids) were left onsite.

A.1.2 In-Situ Testing

A.1.2.1 Standard Penetration Tests (SPTs)

SPTs were conducted within the borings using a split-spoon sampler in general accordance with ASTM D1586. The SPTs were conducted at 2½- to 5-foot intervals to the termination depths of the borings. The SPT is described on the attached Exploration Key, Figure A1.

A.1.3 Material Classification & Sampling

Soil samples were obtained at selected intervals in the borings using the referenced split-spoon (SPT) sampler and thin-walled, steel (Shelby) tube samplers detailed on Figure A1. A qualified member of CGT's geological staff collected the samples and logged the soils in general accordance with the Visual-Manual Procedure (ASTM D2488). An explanation of this classification system is attached as Figure A2. The SPT samples were stored in sealable plastic bags and the Shelby tube samples were sealed with caps and tape and transported to our soils laboratory for further examination and testing. Our geotechnical staff visually examined all samples in order to refine the initial field classifications.

A.1.4 Subsurface Conditions

Subsurface conditions are summarized in Section 2.3 of the geotechnical report. Detailed logs of the explorations are presented on the attached exploration logs, Figures A3 through A7.

A.2.0 LABORATORY TESTING

Laboratory testing was performed on samples collected in the field to refine our initial field classifications and determine in-situ parameters. Laboratory testing included the following:

- Twelve moisture content determinations (ASTM D2216).
- Three percentage passing the U.S. Standard No. 200 Sieve tests (ASTM D1140).
- Three Atterberg limits (plasticity) tests (ASTM D4318).

Results of the laboratory tests are shown on the exploration logs.

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Project Number G2306033

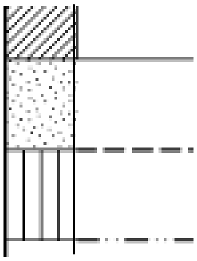
FIGURE A1
Exploration Key

PL ├──●──┤ LL MC	Atterberg limits (plasticity) test results (ASTM D4318): PL = Plastic Limit, LL = Liquid Limit, and MC= Moisture Content (ASTM D2216)
☐ FINES CONTENT (%)	Percentage passing the U.S. Standard No. 200 Sieve (ASTM D1140)

SAMPLING

GRAB	Grab sample
BULK	Bulk sample
⊠ SPT	**Standard Penetration Test** (SPT) consists of driving a 2-inch, outside-diameter, split-spoon sampler into the undisturbed formation with repeated blows of a 140-pound, hammer falling a vertical distance of 30 inches (ASTM D1586). The number of blows (N-value) required to drive the sampler the last 12 inches of an 18-inch sample interval is used to characterize the soil consistency or relative density. The drill rig was equipped with an cat-head or automatic hammer to conduct the SPTs. The observed N-values, hammer efficiency, and N_{60} are noted on the boring logs.
⧗ MC	**Modified California** sampling consists of 3-inch, outside-diameter, split-spoon sampler (ASTM G3550) driven similarly to the SPT sampling method described above. A sampler diameter correction factor of 0.44 is applied to calculate the equivalent SPT N_{60} value per Lacroix and Horn, 1973.
CORE	**Rock Coring** interval
■ SH	**Shelby Tube** is a 3-inch, inner-diameter, thin-walled, steel tube push sampler (ASTM D1587) used to collect relatively undisturbed samples of fine-grained soils.
WDCP	**Wildcat Dynamic Cone Penetrometer** (WDCP) test consists of driving 1.1-inch diameter, steel rods with a 1.4-inch diameter, cone tip into the ground using a 35-pound drop hammer with a 15-inch free-fall height. The number of blows required to drive the steel rods is recorded for each 10 centimeters (3.94 inches) of penetration. The blow count for each interval is then converted to the corresponding SPT N_{60} values.
DCP	**Dynamic Cone Penetrometer** (DCP) test consists of driving a 20-millimeter diameter, hardened steel cone on 16-millimeter diameter steel rods into the ground using a 10-kilogram drop hammer with a 460-millimeter free-fall height. The depth of penetration in millimeters is recorded for each drop of the hammer.
POCKET PEN. (tsf)	**Pocket Penetrometer** test is a hand-held instrument that provides an approximation of the unconfined compressive strength in tons per square foot (tsf) of cohesive, fine-grained soils.

CONTACTS



Observed (measured) contact between soil or rock units.

Inferred (approximate) contact between soil or rock units.

Transitional (gradational) contact between soil or rock units.

ADDITIONAL NOTATIONS

Italics	Notes drilling action or digging effort
{ Braces }	Interpretation of material origin/geologic formation (e.g. { Base Rock } or { Columbia River Basalt })

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All measurements are approximate.

RON TONKIN GRAN TURISMO LAMBORGHINI DEALERSHIP - WILSONVILLE, OREGON
Project Number G2306033

FIGURE A2

Soil Classification

Classification of Terms and Content

NAME: Group Name and Symbol
Relative Density or Consistency
Color
Moisture Content
Plasticity
Other Constituents
Other: Grain Shape, Approximate Gradation
Organics, Cement, Structure, Odor, etc.
Geologic Name or Formation

Grain Size

Grain Size		U.S. Standard Sieve
Fines		<#200 (0.075 mm)
Sand	Fine	#200 - #40 (0.425 mm)
	Medium	#40 - #10 (2 mm)
	Coarse	#10 - #4 (4.75 mm)
Gravel	Fine	#4 - 0.75 inch
	Coarse	0.75 inch - 3 inches
Cobbles		3 to 12 inches
Boulders		> 12 inches

Coarse-Grained (Granular) Soils

Relative Density

SPT N_{60}-Value	Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
>50	Very Dense

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace silt"
5 - 15%	"With" as part of group name	"**POORLY GRADED SAND WITH SILT**"
15 - 49%	Modifier to group name	"**SILTY SAND**"

Fine-Grained (Cohesive) Soils

SPT N_{60}-Value	Torvane tsf Shear Strength	Pocket Pen tsf Unconfined	Consistency	Manual Penetration Test
<2	<0.13	<0.25	Very Soft	Thumb penetrates more than 1 inch
2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Thumb penetrates about 1 inch
4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Thumb penetrates about ¼ inch
8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than ¼ inch
15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>30	>2.00	>4.00	Hard	Difficult to indent by thumbnail

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace fine-grained sand"
5 - 15%	"Some" as part of soil description	"some fine-grained sand"
15 - 30%	"With" as part of group name	"**SILT WITH SAND**"
30 - 49%	Modifier to group name	"**SANDY SILT**"

Moisture Content

Dry: Absence of moisture, dusty, dry to the touch

Moist: Leaves moisture on hand

Wet: Visible free water, likely from below water table

	Plasticity	Dry Strength	Dilatancy	Toughness
ML	Non to Low	Non to Low	Slow to Rapid	Low, can't roll
CL	Low to Medium	Medium to High	None to Slow	Medium
MH	Medium to High	Low to Medium	None to Slow	Low to Medium
CH	Medium to High	High to Very High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick

Laminated: Alternating layers < 6 mm thick

Fissured: Breaks along definite fracture planes

Slickensided: Striated, polished, or glossy fracture planes

Blocky: Cohesive soil that can be broken down into small angular lumps which resist further breakdown

Lenses: Has small pockets of different soils, note thickness

Homogeneous: Same color and appearance throughout

Visual-Manual Classification

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils: More than 50% retained on No. 200 sieve	Gravels: 50% or more *retained* on the No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel/sand mixtures, little or no fines
			GP	Poorly-graded gravels and gravel/sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel/sand/silt mixtures
			GC	Clayey gravels, gravel/sand/clay mixtures
	Sands: More than 50% *passing* the No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand/silt mixtures
			SC	Clayey sands, sand/clay mixtures
Fine-Grained Soils: 50% or more Passes No. 200 Sieve	Silt and Clays Low Plasticity Fines		ML	Inorganic silts, rock flour, clayey silts
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
			OL	Organic soil of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic soil of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

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References:

ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)

ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)

Terzaghi, K., and Peck, R.B., 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons.

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FIGURE A3

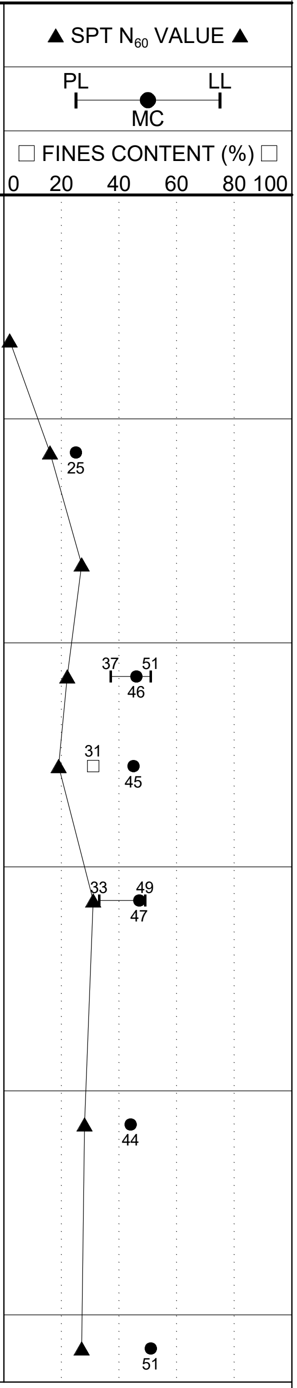
Boring B-1

PAGE 1 OF 1

CLIENT Celia Tonkin - Ron Tonkin Gran Turismo
PROJECT NAME Ron Tonkin Gran Turismo Lamborghini Dealership
PROJECT NUMBER G2306033
PROJECT LOCATION South of 25195 SW Parkway Ave. - Wilsonville, OR
DATE STARTED 12/4/23
GROUND ELEVATION 276 ft
ELEVATION DATUM From Survey Map Provided by Client
WEATHER Rain, 57F
SURFACE Shrubs
LOGGED BY BJG
REVIEWED BY BMW
DRILLING CONTRACTOR PLI Systems, Inc.
SEEPAGE ---
EQUIPMENT B58 Track Mounted Drill Rig
GROUNDWATER DURING DRILLING ---
DRILLING METHOD Hollow Stem 4¼-inch ID Auger
▼ **GROUNDWATER 4 HOURS AFTER DRILLING** 12.0 ft / El. 264.0 ft

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION
276–275.5	OL	**ORGANIC SOIL:** Dark brown, moist, low plasticity, with some rootlets.
275.5–267	MH	**ELASTIC SILT:** : Very soft, brown to red-brown, moist, medium plasticity, with weathered rock fragments and some rounded gravel up to ¼ inch diameter. Medium stiff to stiff, some weathered rock fragments up to ¼ inch diameter, trace fine-grained sand below about 4 feet bgs.
267–249.5	SM	**SILTY SAND:** Medium dense, red/orange/yellow/brown, moist, fine- to medium-grained, medium plasticity fines, with black weathered rock fragments up to ½ inch diameter. Wet below about 12 feet bgs.

SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.7%	DRY UNIT WT. (pcf)
SPT 1	33	0-0-2 (2)	2	
SPT 2	33	4-8-8 (16)	16	
SPT 3	89	6-12-16 (28)	27	
SPT 4	67	4-9-12 (21)	22	
SPT 5	89	5-9-9 (18)	19	
SPT 6	89	9-15-13 (28)	31	
SPT 7	89	5-10-13 (23)	28	
SPT 8	100	6-10-12 (22)	27	



- Boring terminated at 26½ feet bgs.
- Groundwater observed at about 12 feet bgs.
- No caving observed.
- Boring backfilled with granular bentonite upon completion.

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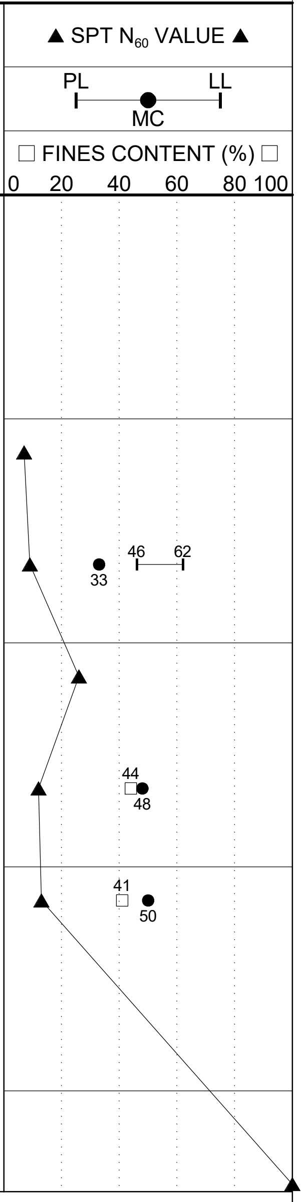
FIGURE A4

Boring B-2

CLIENT Celia Tonkin - Ron Tonkin Gran Turismo
PROJECT NAME Ron Tonkin Gran Turismo Lamborghini Dealership
PROJECT NUMBER G2306033
PROJECT LOCATION South of 25195 SW Parkway Ave. - Wilsonville, OR
DATE STARTED 12/4/23
GROUND ELEVATION 271 ft
ELEVATION DATUM From Survey Map Provided by Client
WEATHER Rain, 58F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
DRILLING CONTRACTOR PLI Systems, Inc.
SEEPAGE ---
EQUIPMENT B58 Track Mounted Drill Rig
GROUNDWATER DURING DRILLING ---
DRILLING METHOD Hollow Stem 4¼-inch ID Auger
▼ **GROUNDWATER 2 HOURS AFTER DRILLING** 7.5 ft / El. 263.5 ft



ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.7%	DRY UNIT WT. (pcf)	PL / MC / LL; FINES CONTENT (%)
270	OL	**ORGANIC SOIL:** Dark brown, moist, low plasticity, with some rootlets.		0						
	MH	**ELASTIC SILT:** Very soft, brown, moist, medium plasticity, with some weathered rock fragments up to ¼ inch diameter, trace rootlets.			SPT 1	33	0-0-0 (0)			
265		Medium stiff to stiff, red-brown with gray mottling, with some weathered rock fragments up to ¼ inch diameter, some medium-grained sand below about 4 feet bgs.		5	SPT 2	33	2-3-4 (7)	7		
		Wet below about 7½ feet bgs.	▼		SPT 3	67	4-4-5 (9)	9		PL 46, MC 33, LL 62
260	SM	**SILTY SAND:** Medium dense, red/orange/gray, wet, fine- to coarse-grained, medium plasticity fines, with some weathered rock fragments up to ½ inch diameter.		10	SPT 4	89	4-9-16 (25)	26		
					SPT 5	89	4-6-6 (12)	12		Fines 44, MC 48
255		Some yellow mottling below about 15 feet bgs.		15	SPT 6	89	2-6-6 (12)	13		Fines 41, MC 50
250				20	SH 7	25				
					SPT 8	100	50/2"	100		
245										

- Boring terminated at 22¼ feet bgs due to practical refusal on a boulder.
- Groundwater observed at about 7½ feet bgs.
- No caving observed.
- Boring backfilled with granular bentonite upon completion.

FIGURE A5

Boring B-3

PAGE 1 OF 1

CLIENT Celia Tonkin - Ron Tonkin Gran Turismo

PROJECT NAME Ron Tonkin Gran Turismo Lamborghini Dealership

PROJECT NUMBER G2306033

PROJECT LOCATION South of 25195 SW Parkway Ave. - Wilsonville, OR

DATE STARTED 12/4/23

GROUND ELEVATION 268 ft

ELEVATION DATUM From Survey Map Provided by Client

WEATHER Rain, 58F

SURFACE Grass

LOGGED BY BJG

REVIEWED BY BMW

DRILLING CONTRACTOR PLI Systems, Inc.

SEEPAGE ---

EQUIPMENT B58 Track Mounted Drill Rig

GROUNDWATER DURING DRILLING ---

DRILLING METHOD Hollow Stem 4¼-inch ID Auger

▼ **GROUNDWATER 1 HOURS AFTER DRILLING** 4.5 ft / El. 263.5 ft

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.7%	DRY UNIT WT. (pcf)	▲ SPT N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Dark brown, moist, low plasticity, with abundant rootlets.		0						
		MH	**ELASTIC SILT:** Medium stiff, brown, moist, high plasticity, with trace weathered rock fragments up to ¼ inch diameter, trace rootlets.								
265			Some fine-grained sand below about 3 feet bgs.			SPT 1	33	0-2-3 (5)	5		▲
			Wet below about 4½ feet bgs.	▼	5						
						SPT 2	100	1-3-3 (6)	6		▲ ● 34
260			Stiff below about 7½ feet bgs.			SPT 3	100	5-6-10 (16)	16		▲
					10						
		SM	**SILTY SAND:** *Medium dense*, red/brown/yellow, wet, fine- to coarse-grained, low plasticity fines,			SH 4	42				● 34
255											
250											
245											
240											

- Boring terminated at 12 feet bgs.
- Groundwater observed at about 4½ feet bgs.
- No caving observed.
- Boring backfilled with granular bentonite upon completion.

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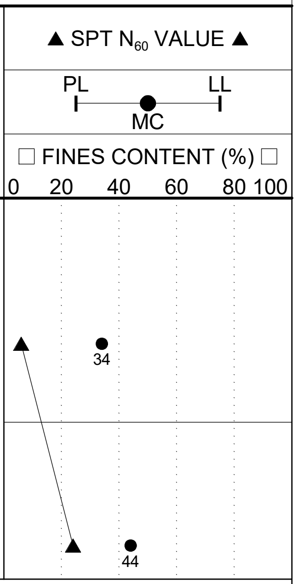
FIGURE A6

Boring B-4

CLIENT Celia Tonkin - Ron Tonkin Gran Turismo
PROJECT NAME Ron Tonkin Gran Turismo Lamborghini Dealership
PROJECT NUMBER G2306033
PROJECT LOCATION South of 25195 SW Parkway Ave. - Wilsonville, OR
DATE STARTED 12/4/23
GROUND ELEVATION 270 ft
ELEVATION DATUM From Survey Map Provided by Client
WEATHER Rain, 58F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
DRILLING CONTRACTOR PLI Systems, Inc.
SEEPAGE ---
EQUIPMENT B58 Track Mounted Drill Rig
GROUNDWATER DURING DRILLING ---
DRILLING METHOD Hollow Stem 4¼-inch ID Auger
▼ **GROUNDWATER .5 HOURS AFTER DRILLING** 1.0 ft / El. 269.0 ft

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.7%	DRY UNIT WT. (pcf)
					0					
		OL	**ORGANIC SOIL:** Dark brown, moist, low plasticity, with some rootlets.	▼						
		MH	**ELASTIC SILT:** Medium stiff, brown with red/gray mottling, wet, medium plasticity, some fine-grained sand. Wet below about 1-foot bgs.			SPT 1	100	1-3-3 (6)	6	
265					5					
						SH 2	83			
		SM	**SILTY SAND:** Medium dense, red/orange/brown, wet, fine- to coarse-grained, medium plasticity fines, with black weathered rock fragments up to ¼ inch diameter.			SPT 3	100	6-11-14 (25)	24	
260										
255										
250										
245										
240										



- Boring terminated at 9 feet bgs.
- Groundwater observed at about 1-foot bgs.
- No caving observed.
- Boring backfilled with granular bentonite upon completion.

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FIGURE A7

Boring B-5

CLIENT Celia Tonkin - Ron Tonkin Gran Turismo

PROJECT NAME Ron Tonkin Gran Turismo Lamborghini Dealership

PROJECT NUMBER G2306033

PROJECT LOCATION South of 25195 SW Parkway Ave. - Wilsonville, OR

DATE STARTED 12/4/23

GROUND ELEVATION 269 ft

ELEVATION DATUM From Survey Map Provided by Client

WEATHER Rain, 58F

SURFACE Grass

LOGGED BY BJG

REVIEWED BY BMW

DRILLING CONTRACTOR PLI Systems, Inc.

SEEPAGE ---

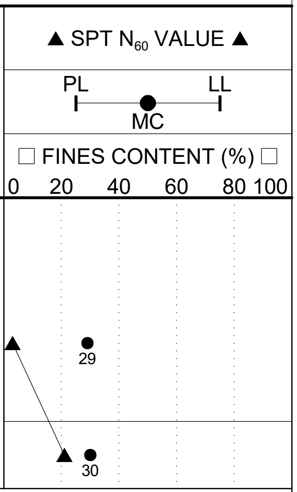
EQUIPMENT B58 Track Mounted Drill Rig

GROUNDWATER DURING DRILLING ---

DRILLING METHOD Hollow Stem 4¼-inch ID Auger

▼ **GROUNDWATER .5 HOURS AFTER DRILLING** 1.0 ft / El. 268.0 ft

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.7%	DRY UNIT WT. (pcf)
		OL	**ORGANIC SOIL:** Dark brown, moist, low plasticity, with some rootlets.	▼	0					
265		MH	**ELASTIC SILT:** Soft, brown with red/gray mottling, moist, medium plasticity, some fine-grained sand. Wet below about 1-foot bgs.			SPT 1	56	1-1-2 (3)	3	
			Very stiff below about 5 feet bgs.		5	SPT 2	56	9-10-12 (22)	21	



- Boring terminated at 6½ feet bgs.
- Groundwater observed at about 1-foot bgs.
- No caving observed.
- Boring backfilled with granular bentonite upon completion.

260

255

250

245

240