Geotechnical Engineering Services Report

Lynnwood Auto Dealership
188th Street SW and SR 99
Lynnwood, Washington

for

Holman Automotive Group, Inc.
c/o 3J Consulting

March 3, 2021

GeoEngineers
Earth Science + Technology

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1.0 INTRODUCTION

This report presents the results of GeoEngineers' geotechnical engineering services for the proposed Lynnwood Auto Dealership located at 18624 Highway 99 in Lynnwood, Washington (Parcel No. 00374300500201). The site is shown relative to surrounding physical features in Figure 1, Vicinity Map and Figures 2A and 2B, Existing and Proposed Exploration and Site Plans, respectively.

The purpose of this report is to provide geotechnical engineering conclusions and recommendations for the design of the Lynnwood Auto Dealership. GeoEngineers' geotechnical engineering services have been completed in general accordance with our services agreement executed October 28, 2020.

2.0 SCOPE OF SERVICES

GeoEngineers' scope of work for this report included:

- Review of existing subsurface and geologic information available for the site and surrounding area.
- Completed 11 borings across the site of the proposed auto dealership to evaluate soil conditions for the proposed buildings, parking garages and potential retaining walls to be part of site grading and development.
- Completed dynamic cone penetrometer (DCP) tests.
- Evaluated pertinent physical and engineering characteristics of the site soils by completing a laboratory testing program on samples obtained from the borings. The laboratory tests included moisture content determination, grain-size distribution and percent fines content.
- Provided recommendations for earthwork, including use of on-site and imported structural fill and fill placement and compaction requirements.
- Provided geotechnical recommendations, including shallow foundations, temporary shoring, slab-on-grade, temporary construction dewatering considerations, permanent drainage, and permanent below-grade wall recommendations.
- Geotechnical design recommendations for site retaining walls for walls less than 15 feet tall, including static and seismic active earth pressures, and drainage and backfill recommendations. Our scope did not include stability analyses, settlement, or design of walls.
- Recommendations for constructing asphaltic concrete (AC) pavements for the proposed new parking areas, including subgrade, drainage, base rock, and pavement section.
- Prepared this draft geotechnical engineering report.
- Finalize the geotechnical report after review and comment from the project team is provided.

3.0 PROJECT DESCRIPTION

Our understanding of the project is based on discussions with, and information provided by the project team, including the proposed building layout and grading plan provided to us via email on December 7, 2020. review of available soil information for the site, and our experience on similar projects.

The proposed auto dealership site is located on an approximate 6.15-acre site bounded by 186th Place SW to the north, Pacific Highway/Highway 99 to the east, 5615 188th SW and 7511 188th St SW to the south and residential housing and property to the west. Based on the information provided, we understand the proposed project will consist of two main buildings for uses, including showroom, offices, and service support along with a potential parking garage. The proposed showrooms and service centers will be two story buildings with car lifts. The proposed parking garage will include either three or four levels. At-grade surface parking is also planned along the western portion of the site as well as around the perimeter of the two proposed buildings.

4.0 FIELD EXPLORATIONS AND LABORATORY TESTING

4.1. Field Explorations

Subsurface conditions at the site were evaluated by completed 11 geotechnical borings (GEI-01 through GEI-11). Exploratory borings extended to depths ranging from 21½ to 41½ feet below ground surface (bgs) depending on location relative to the proposed development. Exploration locations are shown in Figures 2A and 2B. Descriptions of the field exploration program and detailed boring logs are presented in Appendix A of this report.

4.2. Laboratory Testing

Soil samples were obtained from geotechnical borings during the subsurface exploration program and transported to GeoEngineers' laboratory in Redmond, Washington for further evaluation. Selected samples were tested for determination of grain-size distribution (sieve analysis), fines content (percent passing U.S. No. 200 sieve) and moisture content. A description of the laboratory testing program and test results are presented in Appendix A. Laboratory results are also presented as part of the boring logs in Figures A-2 through A-12.

5.0 SITE CONDITIONS

5.1. Site Geology

The Puget Sound basin is a region of Quaternary (last 3 million years) sediments that range in thickness between 800 and 2,400 feet. The basin area has been repeatedly overridden by Pleistocene (between 11,000 and 3 million years ago) continental glacial ice depositing till, glacial sand and gravel. As the glacial ice retreated to the north, glaciofluvial sediment was deposited in the outwash channels. The most recent glacial cycle of sediment deposits is referred to as the Vashon Drift, occurring between 13,500 and 15,000 years ago.

We reviewed a United States Geological Survey (USGS) map for the project area, "Geologic Map of the Edmonds East and Part of the Edmonds West Quadrangles, Washington" (Minard 1983). Surficial geologic deposits in the site vicinity are mapped as recessional outwash, glacial till, and advance outwash. Recessional outwash typically consists of sand and gravel with varying amounts of silt that was deposited by meltwater from the stagnating and receding glacier. These soils are typically medium dense.

Glacial till is a generally heterogeneous mixture of sand, gravel, cobbles and occasional boulders in a silt and clay matrix that was deposited beneath a glacier. Advance outwash typically consists of well-stratified sand with variable amounts of gravel and cobbles. The advance outwash is usually exposed where the overlying glacial till cap has been eroded away, typically in ravines and bluff margins. Both the glacial till and advance outwash have been overridden by thousands of feet of ice, and are typically dense to very dense. The Whidbey Formation underlies the advance outwash and typically consists of dense sands and gravels overlying or interbedded with stiff to hard silts. Subsurface soils encountered in our explorations are consistent with the geologic mapping. Specific details of subsurface conditions encountered in our explorations are presented in the sections below.

5.2. Geologic Hazards and Considerations

GeoEngineers has reviewed Environmentally Critical Area (ECA) maps available online through Snohomish County geographic information system (GIS). Based on our review, a portion of the site is located within a steep slope ECA, as shown in Figure 2A.

5.2.1. Steep Slope Definitions

Per City of Lynnwood (City) Municipal Code Chapter 17.10 Environmentally Critical Areas, steep slope hazard areas are defined as "areas with slopes steeper than 40 percent…" Based on our site reconnaissance and review of the existing topography, the northeast portion of the site has slopes steeper than 40 percent. Based on on-site explorations, the slope consists of fill likely placed as part of previous site development.

5.2.2. On-Site Slopes

GeoEngineers investigated subsurface conditions at the site and we estimate that approximately 5 to 10 feet of fill is present at the site. We understand that during previous development of the trailer park portion of the property, surficial soil was locally transported throughout the site to create an asphalt road that looped through the property as well as to create level benches for the trailer parking slips. In the northwest corner of the site, the existing slope was cut into and shored up with retaining walls that remain in place. We observed remnants of a demolished structure just to the east of the existing retaining walls. Based on current general topography, existing fill and near slope improvements, we conclude that the steep slope present on the site was completed as part of grading activities associated with previous site development. Although portions of the existing site are mapped as steep slopes in the City Municipal Code, the proposed grading plan for the Lynnwood Auto Dealership shows the slope will be regraded and reduced to significantly less than 40 percent and include an engineered retaining wall system. Regrading and slope modification may require civil permitting per Section 17.10 of the Lynnwood Municipal Code.

5.3. Surface Conditions

Existing site grades drop about 50 feet from west to east across the site, ranging from approximate Elevation 420 feet North American Vertical Datum of 1988 (NAVD 88) at the northwest portion of the site to approximately Elevation 370 feet along the east extent along SR 99. Low grasses, shrubs , and semi-mature to mature trees exist throughout the site. Within the northern portion of the site, we observed concrete rubble pile and a demolished structure adjacent to an existing concrete wall and an existing rockery wall. We observed asphalt pavement throughout the middle portion, southern portion, and the northeastern corner of the site.

Existing buried utilities are anticipated within and near the footprint of the project and within the public right-of-way along SR 99 and 186th Place SW. These utilities may include, but are not limited to, gas, electricity, sanitary sewer, storm drain, fiber optic, telecommunications and water.

5.4. Subsurface Soil Conditions

GeoEngineers' understanding of subsurface conditions is based on the 11 exploratory borings completed during the subsurface investigation. Approximate locations of the borings completed as part of this study are presented in Figures 2A and 2B.

Geologic units identified at the site generally consist of fill, weathered glacial till, glacial till and advanced outwash. The weathered till, glacial till and advanced outwash are glacially consolidated consistent with geologic mapping as described in Section 5.1 above. The weathered till and glacial till geologic units represent competent foundation bearing soils. The interpreted subsurface soil conditions are presented in Figure 3, Cross Section A-A', Figure 4, Cross Section B-B' and Figure 5, Cross Section C-C'.

- **Asphalt Concrete (AC) Pavement and Base Course:** Boring GEI-05 encountered AC pavement at the ground surface, as it was completed within the driveway loop in the central portion of the site. The AC measured approximately 1½ inches thick at the boring location. Existing asphalt thickness will likely vary across the site.
- **Fill and Topsoil:** The topsoil and fill layers, where encountered, generally extend to depths ranging from ground surface to 9 feet bgs. The fill and topsoil generally consisted of silty fine to course sand with variable organics and gravel.
- **Glacially Consolidated Soils**: Glacially consolidated soils were encountered below the fill and topsoil layers, where those units were present. Based on our experience in the vicinity, our review of existing information, and our borings completed for this project, three glacially consolidated units were encountered in the explorations: weathered till, glacial till, and advanced outwash as described below.

  The weathered and glacial till generally overlie the advanced outwash, but layers of till-like deposits are also interbedded within the advanced outwash unit. It is important to note that boulders are commonly present in glacially consolidated soils in nearby excavations, and may be present at the site.

  - **Weathered Till**: Weathered till is present below the pavement and shallow fill, and generally consists of medium dense to dense sand with variable silt and gravel. The weathered till extends to depths ranging from approximately 5 to 12 feet bgs.
  - **Glacial Till**: Glacial till is present below the weathered till layer and generally consists of very dense silty sand with variable gravel with occasional sand with variable silt and gravel interbeds. The glacial till extends to depths ranging from approximately 5 to 29 feet bgs.
  - **Advanced Outwash**: Advanced outwash is present underlying the glacial till deposits and generally consists of very dense sand with variable silt and gravel as well as hard silt with variable sand. All 11 borings terminated in the advanced outwash unit.

5.5. Groundwater Conditions

Table 1 below shows groundwater levels observed within the explorations at the time of drilling. Depths to groundwater noted at the time of drilling represent a short-term condition observed and may not represent the true static groundwater level. Based on the soils encountered and hydraulic conductivity, it can take hours, even days for the groundwater level observed in a boring to reach equilibrium. Groundwater levels are expected to fluctuate seasonally, with varied precipitation and are likely influenced by Scriber Creek located to the east of Highway 99.

TABLE 1. OBSERVED GROUNDWATER LEVELS FROM EXPLORATIONS

Exploration	Estimated Surface Elevation (feet)	Date	Approximate Depth to Groundwater (feet)	Approximate Groundwater Elevation (feet)
GEI-01	388	11/30/2020	21	367
GEI-02	397	11/30/2020	26.5	370.5
GEI-03	409	11/30/2020	Not Observed	-
GEI-04	416	11/30/2020	Not Observed	-
GEI-05	409	12/01/2020	35	374
GEI-06	403	12/01/2020	30	373
GEI-07	400	12/01/2020	30	370
GEI-08	406	12/01/2020	36	370
GEI-09	409	12/02/2020	31.5	377.5
GEI-10	377	12/02/2020	Not Observed	-
GEI-11	372	12/02/2020	8	364

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1. Summary of Key Geotechnical Issues

The following is a summary of key geotechnical issues related to site development. Design level recommendations for each of these items, as well as other important geotechnical recommendations for the proposed project, are contained in subsequent sections of this report. The entire "Conclusions and Recommendations" Section 6.0 should be reviewed to obtain pertinent geotechnical recommendations.

- Based on the relatively thin layers of fill overlying dense to very dense glacially consolidated soils, the site is classified as Site Class C. The 2018 International Building Code (IBC) seismic design parameters are provided in a subsequent section.
- We recommend that the proposed buildings be supported on conventional spread footings bearing on undisturbed dense to very dense glacially consolidated soils or on compacted structural fill.
- We recommend an allowable soil bearing pressure of 12 kips per square foot (psf) where footings are supported on relatively undisturbed glacial till.
- Temporary dewatering may be required during construction activities to control perched groundwater conditions encountered in the northeast portion of the site. Based on the conditions encountered in the site explorations, we expect groundwater can be adequately managed by diverting to gravel lined ditches and the use of sumps and pumps.
- Permanent drainage layers should be provided behind all below-grade walls.

- If deep excavations are required in the native glacially deposited soils, they will require more effort and may involve heavy ripping. Large cobbles and occasional boulders should also be anticipated within the glacial deposits at the site.
- We recommend temporary slopes be inclined at 1½H:1V (horizontal to vertical) or flatter. Slope inclinations may have to be modified by the contractor if localized sloughing occurs or if seepage is present (particularly if loose fill soils are encountered). We recommend the Geotechnical Engineer evaluate the stability of cut slopes to confirm subsurface soils are as anticipated.
- The existing fill and native soils at the site contain sufficient fines content such that they are moisture sensitive and will be easily disturbed during the wet season. We recommend site preparation and fill placement be completed during extended periods of dry weather if practical. Wet weather construction will require the use of import soils that contain less than 5 percent fines passing the U.S. No. 200 sieve and export of wet native soils.

6.2. Foundation Support

We recommend that the proposed Lynnwood Auto Dealership buildings be supported on shallow spread footings supported on native, undisturbed glacial till, compacted structural fill or control density fill (CDF) extending to these soils. Proposed finished footing elevations for the two planned Auto Dealership buildings as shown on the information provided to us, is at approximate Elevation 378 feet for Building 1, and approximate Elevation 380 feet for Building 2.

At the time this report was prepared, building and pavement traffic loads were not provided. We have assumed typical loads for the type of structures discussed to develop preliminary geotechnical design considerations. We have assumed column and wall loads for Buildings 1 and 2 will be on the order of 110 kips per column and 5 kips per lineal foot (klf) of wall, and floor loads on the order of 100 psf. We have assumed that column and wall loads for the parking garage will be on the order of 240 kips per column and 5 klf of wall, and floor slabs on the order of 150 psf.

We anticipate that design loads will be provided to us by the project team prior to issuing our final geotechnical report, or that assumed loads as noted are greater than final design loads.

6.2.1. Foundation Subgrade Preparation

We suggest that the excavations for the footings be accomplished with a smooth bucket to minimize subgrade disturbance. Any loose or disturbed material should be removed from the excavation. The silty footing subgrade soils will be susceptible to disturbance when wet. We recommend that a representative of GeoEngineers observe the subgrade to determine if fill or any unsuitable soils are present and observe any overexcavation. If any overexcavation is required, it is critical that structural fill be placed properly for the recommended allowable bearing pressure. All structural fill below foundations should be placed and compacted to 95 percent of the maximum dry density (MDD) in accordance with ASTM International (ASTM) Standard Practices Test Method D 1557. The structural fill should extend horizontally beyond the edges of the footing a distance equivalent to the depth of the overexcavation. If the overexcavation will be backfilled with CDF or lean concrete, the overexcavation need only be nominally larger than the size of the footing.

6.2.2. Allowable Bearing Pressures

Proposed structures can be supported on shallow foundations bearing on native dense to very dense native till soils or on compacted crushed rock structural fill over native soils, or on upper medium dense material overlying the native till soils. Allowable bearing pressures will be different, depending on the underlying soils.

Based on proposed footing elevations provided to us by the project team, we anticipate that most of the footings will be founded directly on very dense glacial till. Foundations founded on very dense glacial till may be designed using an allowable soil bearing pressure of 12 kips per square foot (ksf).

Across the southern portion of the building, we anticipate that the footings will likely be supported on existing fill material overlying the glacial till soils. Based on the condition of the fill encountered in our borings, the fill appears to be medium dense. We recommend that the upper 18 inches of the exposed existing fill be re-compacted to a minimum of 95 percent of the MDD in accordance with ASTM D 1557 modified proctor. Where footings will be founded on compacted structural fill, we recommend an allowable soil bearing pressure of 3 ksf be utilized for design.

All exterior footings should be founded at least 18 inches below the lowest adjacent finished grade, while interior footings may be founded a minimum of 12 inches below top of slab. We recommend minimum footing widths of 18 inches and 3 feet for continuous and isolated footings, respectively.

The recommended bearing pressure has a factor of safety of at least 3 for dead plus long-term live loads. For short-term transient loads (e.g., seismic, wind loads), the allowable bearing capacity can be increased by one-third if permitted by the design standard.

6.2.3. Modulus of Subgrade Reaction

Large footings, such as large shear wall or brace-frame footings may be designed as mat foundations, which requires a soil subgrade modulus. Where the mat foundations bear on very dense glacial till, or a limited thickness of compacted structural fill extending to the glacial till, the subgrade may be assumed to have a subgrade modulus on the order of 85 pounds per cubic inch (pci). Existing undocumented fill below the foundation elements, if present, should be overexcavated and the subgrade prepared as described above.

6.2.4. Foundation Settlement

Provided all loose soil is removed and the subgrade is prepared as recommended under the "Construction Considerations" Section 6.2.6 below, we estimate the total settlement of shallow foundations will be about 1 inch or less. The settlements will occur rapidly, essentially as loads are applied. We anticipate differential settlements between footings could be half of the expected total settlement. Note that smaller settlements will result from lower applied loads.

6.2.5. Lateral Resistance

Lateral loads can be resisted by passive resistance on the sides of the footings and by friction on the base of the footings. Passive resistance should be evaluated using an equivalent fluid density of 370 pounds per cubic foot (pcf) where footings are poured neat against native soil or are surrounded by structural fill compacted to at least 95 percent of MDD, as recommended. Resistance to passive pressure should be calculated from the bottom of adjacent floor slabs and paving or below a depth of 1 foot where the adjacent area is unpaved, as appropriate. Frictional resistance can be evaluated using 0.35 for the coefficient of base friction against footings. The above values incorporate a factor of safety of about 1.5.

If soils adjacent to footings are disturbed during construction, the disturbed soils must be recompacted or replaced with compacted structural fill, otherwise the lateral passive resistance value must be reduced.

6.2.6. Construction Considerations

We recommend the condition of all subgrade areas be observed by GeoEngineers to evaluate whether the work is completed in accordance with our recommendations and whether the subsurface conditions are consistent with our geotechnical report.

If soft areas are present at the footing subgrade elevation, the soft areas should be removed and replaced with lean concrete or structural fill at the recommendation of GeoEngineers. In such instances, the zone of structural fill should extend laterally beyond the footing edges a horizontal distance at least equal to the thickness of the fill.

We recommend the contractor consider leaving the subgrade for the foundations as much as 6 to 12 inches high, depending on soil and weather conditions, until excavation to final subgrade level is required for placing foundation reinforcement. Temporarily leaving the subgrade high will help reduce damage to the subgrade resulting from inclement weather, construction traffic, or other activities on site.

6.2.7. Footing Drains

We recommend that footing drains be installed around the perimeter of the proposed building. The perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of, and surrounded by, 6 inches of drainage material enclosed in a non-woven geotextile such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drain material. We recommend against using flexible tubing for footing drainpipes. The perimeter drains should be sloped to drain by gravity to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered and placed in flush-mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

6.3. Slab-on-Grade Floors

All slab subgrade areas should be evaluated as recommended in the “Site Preparation” Section 7.1 of the report. All loose soils should be removed below the slab footprint and the subgrade should be compacted to a minimum 95 percent of the MDD. We recommend a capillary break be placed beneath the slab to consist of a 6-inch-thick layer of clean crushed gravel with a maximum particle size of 1½ inches and negligible sand or silt (similar to American Association of State Highway and Transportation Officials [AASHTO] Grading No. 57). The subgrade should be evaluated by the geotechnical engineer prior to placing the recommended capillary break material. We recommend a subgrade modulus of 150 pci for slabs supported on compacted structural fill underlain by very dense glacial till, or directly on the capillary break material over the glacial till.

6.4. Excavation Support and Retaining Walls

Based on our discussions with the project team and review of preliminary plans, the project site will be regraded and incorporate several retaining walls. The planned wall heights are variable across the site and range from 5 to 16 feet in height. We recommend cantilever soldier pile and soldier pile and tieback walls for use as temporary shoring or permanent retaining walls, as well as permanent below grade walls and cast-in-place walls. Geotechnical design recommendations are presented in the following sections.

6.4.1. Cantilever Soldier Pile Wall

Soldier pile walls consist of steel beams set in concrete shafts in drilled vertical holes located along the wall alignment, typically about 8 feet on center with timber or concrete lagging between pile elements. We recommend that the Earth Pressure Diagram for Cantilever Soldier Pile Walls shown in Figure 6 be used for retaining walls where the height of the excavation (H) is less than 10 feet. Retaining walls that are taller than 10 feet should be designed using tiedback soldier pile systems.

6.4.2. Soldier Pile and Tieback Walls

For taller wall sections, tiedback support is recommended for the soldier pile section. After excavation to specified elevations on the soldier pile wall, tiebacks are installed at specified depths, depending on wall design. Once tiebacks are installed, the pullout capacity of each tieback is tested, and the tieback is structurally connected (locked off) to the soldier pile at or near the design tieback load. Tiebacks typically consist of steel strands that are installed into pre-drilled holes and then either tremie or pressure grouted. Timber lagging is typically installed behind the flanges of the steel beams to retain the soil located between the soldier piles.

Geotechnical design recommendations for each of these components of the soldier pile and tieback wall system are presented in the following sections.

6.4.2.1. Soldier Piles

We recommend soldier pile walls be designed using the earth pressure diagram presented in Figure 6. The pressures represent the estimated loads that will be applied to the wall system for various wall heights. Earth pressures shown are per foot of wall width. Pile design will depend on center to center spacing and tributary area to each pile.

Earth pressures presented in Figure 6 include loading from typical traffic surcharge. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be considered on a case-by-case basis in accordance with the recommendations presented in Figure 8, Recommended Surcharge Pressure. Seismic pressures included in Figure 6 should only be applied to permanent walls.

We recommend the embedded portion of the soldier piles be at least 2 feet in diameter and extend a minimum distance of 10 feet below the base of the excavation to resist “kick-out.” Axial capacity of the soldier piles must resist the downward component of tieback anchor loads and other vertical loads, as appropriate. We recommend using an allowable end bearing value of 30 ksf for piles supported on the glacially consolidated soils. The allowable end bearing value should be applied to the base area of the concrete shaft into which the soldier pile is embedded. This value includes a factor of safety of about 2.0. The allowable end bearing value assumes the shaft bottom is cleaned out immediately prior to concrete placement. If necessary, an allowable pile skin friction value of 1.5 ksf in glacially consolidated soils may be used on the embedded portion of the soldier piles to resist the vertical loads.

6.4.2.2. Timber Lagging

We recommend the temporary timber lagging be sized using the procedures outlined in the Federal Highway Administration’s Geotechnical Engineering Circular No. 4 (1999). The site soils are best described as competent soils. The following table presents GeoEngineers' recommended lagging thicknesses (rough-cut) as a function of soldier pile clear span and depth.

TABLE 2. RECOMMENDED LAGGING THICKNESS

Depth (feet)	Recommended Lagging Thickness (rough-cut) for clear spans of:					
	5 feet	6 feet	7 feet	8 feet	9 feet	10 feet
0 to 25	2 inches	3 inches	3 inches	3 inches	4 inches	4 inches

Lagging should be installed promptly after excavation, especially in areas where groundwater is present or where clean sand and gravel soils are present and caving soil conditions are likely. The workmanship associated with lagging installation is important for maintaining the integrity of the excavation.

The space behind the lagging should be filled with soil as soon as practical. Voids should be backfilled immediately or within a single shift, depending on the selected method of backfilling. Placement of this material will help reduce the risk of voids developing behind the wall and damage to existing improvements located behind the wall.

Material used as backfill in voids located behind the lagging should not cause buildup of hydrostatic pressure behind the wall. Lean concrete is a suitable option for the use of backfill behind the walls. Lean concrete will reduce the volume of voids present behind the wall. Alternatively, lean concrete may be used as backfill behind the upper 10 feet of the excavation to limit caving and sloughing of the upper soils, with on-site soils used to backfill the voids for the remainder of the excavation. Based on our experience, the voids between each lean concrete lift are sufficient for preventing the buildup of hydrostatic pressure behind the wall.

6.4.2.3. Tiebacks

Tieback anchors should extend far enough behind the wall to develop anchorage beyond the "no-load" zone and within a stable soil mass. As shown in Figure 6, the no-load zone is defined to extend horizontally behind the base of the wall a distance of H/4, where H is the height of the wall, and then up to the ground surface away from the wall face at an angle of 60 degrees measured from horizontal. The anchors should be inclined downward at 15 to 25 degrees below the horizontal. Corrosion protection is not required for temporary tiebacks.

Centralizers should be used to keep the tieback in the center of the hole during grouting. Structural grout or concrete should be used to fill the bond zone of the tiebacks. A bond breaker such as plastic sheathing should be placed around the portion of the tieback located within the no-load zone if the shoring contractor plans to grout both the bond zone and unbonded zone of the tiebacks in a single stage. If the shoring contractor does not plan to use a bond breaker to isolate the no-load zone, GeoEngineers should be contacted to provide recommendations.

Loose soil and slough should be removed from the holes drilled for tieback anchors prior to installing the tieback. The contractor should take necessary precautions to minimize loss of ground and prevent disturbance to previously installed anchors and existing improvements in the site vicinity. Holes drilled for tiebacks should be grouted/filled promptly to reduce the potential for loss of ground.

Tieback anchors should develop anchorage in the glacially consolidated soils. We recommend the spacing between tiebacks be at least three times the diameter of the anchor hole to reduce the potential for group interaction. We recommend a preliminary design load transfer value between the anchor and soil of 4 kips per foot for glacially consolidated soils.

The tieback anchors should be verification- and proof-tested to confirm the tiebacks have adequate pullout capacity. Pullout resistance of tiebacks should be designed using a factor of safety of 2.0. The pullout resistance should be verified by completing at least two successful verification tests in each soil type and at least four total tests for the project. Each tieback should be proof-tested to 133 percent of the design load. Verification and proof tests should be completed as described in Appendix B.

The tieback layout and inclination should be checked to confirm the tiebacks do not interfere with adjacent buildings, buried utilities, Washington Department of Transportation (WSDOT) structure foundations and retaining walls. The City and WSDOT minimum clearances between ground anchors and existing utilities and structures should be maintained.

6.4.2.4. Soldier Pile Wall Drainage

Drainage for soldier pile and lagging walls is achieved through seepage between the timber lagging boards. Seepage flows at the bottom of the excavation should be contained and controlled to prevent loss of soil from behind the lagging. Drainage should be incorporated into the permanent dewatering system as described in the "Permanent Below-Grade Walls Against Shoring" Section 6.4.3 of this report.

6.4.2.5. Construction Considerations

Temporary casing or drilling fluid may be required to install the soldier piles and casing may be necessary for tiebacks where:

- Loose fill is present;
- The native soils do not have adequate cementation or cohesion to prevent caving or raveling; and/or
- Groundwater is present.

GeoEngineers should be allowed to observe and document the installation and testing of the shoring elements to verify conformance with the design assumptions and recommendations.

6.4.3. Permanent Below-Grade Walls Against Shoring

Permanent below-grade walls constructed adjacent to temporary shoring walls should be designed for the same earth pressures as the temporary shoring walls with the addition of a rectangular seismic earth pressure equal to 9H psf (where H is the height of the wall in feet). Foundation surcharge loads and traffic surcharge loads should be incorporated into the design of the below-grade walls using the surcharge pressures presented in Figure 8. Other surcharge loads, such as from construction equipment or construction staging areas, should be considered on a case-by-case basis. We can provide the estimated lateral pressures from these surcharge loads as the design progresses.

Soil pressures recommended above assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as described in the "Footing Drains" Section 6.2.7 and the "Slabs-on-Grade Floors" Section 6.3 of this report.

6.4.4. Other Cast-in-Place and Modular Block Walls

Conventional cast-in-place or modular block walls may be necessary for retaining structures or below-grade utility vaults located on site. Lateral soil pressures acting on conventional cast-in-place or precast subsurface walls will depend on the nature, density and configuration of the soil behind the wall and the amount of lateral wall movement that can occur as backfill is placed.

For walls that are free to yield at the top at least 0.1 percent of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. Assuming that the walls are backfilled and drainage is provided as outlined in the following paragraphs, we recommend that yielding walls supporting horizonal backfill be designed using an equivalent fluid density of 35 pcf (triangular distribution), while non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf (triangular distribution). In addition, where applicable, utility vaults should be designed for full hydrostatic pressures unless adequate drainage as described below can be provided and the wall drainpipes can be tightlined to a suitable discharge location. For hydrostatic conditions, the wall should be designed using an equivalent fluid density of 90 pcf. For seismic loading conditions, a rectangular earth pressure equal to 9H psf (where H is the height of the wall in feet) should be added to the active/at-rest pressures. Other surcharge loading should be applied as appropriate.

Lateral resistance for conventional cast-in-place walls can be provided by frictional resistance along the base of the wall and passive resistance in front of the wall. For walls founded on native soils, the allowable friction resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces. The allowable passive resistance may be computed using an equivalent fluid density of 370 pcf (triangular distribution). The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

6.4.5. Permanent Below-Grade Wall Drainage

Drainage behind the permanent below-grade walls constructed in front of shoring walls is typically provided using a prefabricated drainage mat attached to the shoring walls. The drainage mat should be connected to weep pipes that extend through the permanent below-grade building walls at the footing elevation. The weep pipes through the permanent below-grade walls should be spaced no more than 12 feet on center and should have a minimum diameter of 2 inches. The weep pipes should be connected to perimeter footing drains (like those described under the "Footing Drains" Section 6.2.7 above), which are in turn tightlined to the underslab drainage system and permanent sump.

Full wall face coverage will be needed to minimize seepage and/or wet areas at the face of the permanent walls and to reduce the buildup of hydrostatic pressures acting on the basement walls. The drainage mat full face wall coverage should extend from the weep pipe elevation up to about 3 feet below the top of the wall to reduce the potential for surface water to enter the wall drainage system. The tops of the drainage mats should be sealed to prevent soil and water entry.

Although the use of full wall face coverage will reduce the likelihood of seepage and/or wet areas at the face of the permanent wall, there is still a potential for these conditions to occur. If this is a concern, waterproofing should be specified.

For permanent walls constructed in open cut areas, positive drainage should be provided behind cast-in-place and modular bock retaining walls by placing a minimum 2-foot-wide zone of Gravel Backfill for Walls, Section 9-03.12(2) of the WSDOT Standard Specifications. A perforated or slotted drainpipe should be placed near the base of the retaining wall to provide drainage. The drainpipe should be surrounded by at least 6 inches of WSDOT Gravel Backfill for Drains, Section 9-03.12(4) or an alternative approved by GeoEngineers. The drainage material should be wrapped with a geotextile filter fabric meeting the requirements of Construction Geotextile for Underground Drainage, WSDOT Standard Specification 9-33. The wall drainpipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger diameter pipe will allow for easier maintenance of drainage systems.

6.5. Pavement Recommendations

6.5.1. General

Pavement subgrades should be prepared in accordance with Section 7.0 of this report. If the subgrade soils are excessively loose or soft, it may be necessary to excavate localized areas and replace them with additional gravel borrow or gravel base material. Pavement subgrade conditions should be observed and proof-rolled during construction and prior to placing the subbase materials in order to properly evaluate the presence of unsuitable subgrade soils and the need for over excavation and placement of a geotextile separator.

Our pavement recommendations assume that traffic at the site will consist of occasional truck traffic and passenger cars. We do not have specific information on the frequency and type of vehicles that will use the area; however, we have based our design analysis on traffic consisting of two heavy trucks per day to account for delivery- and service-type vehicles and passenger car traffic for the heavy-duty pavement sections, and passenger car traffic only for the light-duty pavement sections. In addition, we have provided pavement sections for typical local access and collector streets from Lynwood Washington Public Works.

6.5.2. Dynamic Cone Penetrometer (DCP) Testing

In order to estimate subgrade resilient modulus (M_R) for the upper weathered till material, we conducted DCP tests in general accordance with ASTM D 6951 at the two locations where we encountered sandy gravel weathered till (GEI-10 and GEI-11). We recorded penetration depth of the cone versus hammer blow count and terminated testing after penetrating depths of approximately 17.5 inches (DCP-1) and 24.5 inches (DCP-2).

We plotted depth of penetration versus blow count and visually assessed regions where slopes of the data were relatively constant to estimate the moduli. Table 3 lists our estimate of the subgrade resilient modulus at each test location based on data obtained in the upper 18 inches below the existing ground surface.

For areas where exposed subgrade consists of native weathered till, the estimated M_R from Table 3 are appropriate. For fill placed over native subgrade as part of site grading (compacted as recommended for structural fill in this report) an M_R of 5,500 pounds per square inch (psi) was used for design of pavement sections.

TABLE 3. ESTIMATED SUBGRADE RESILIENT MODULI BASED ON DCP TESTING

Boring Number	Estimated Resilient Modulus (psi)
DCP-1	8,100
DCP-2	9,200

6.5.3. Pavement Drainage

Long-term performance of pavements is influenced significantly by drainage conditions beneath the pavement section. Positive drainage can be accomplished by crowning the subgrade with a minimum 2 percent cross slope and establishing grades to promote drainage.

6.5.4. AASHTO Input Parameters

Input parameters used in pavement thickness design were selected based on review of typical values found in the AASHTO Design Guide. The following parameters were used:

- On-site soil subgrade below proposed fill placed to raise site grades or below aggregate base sections has been prepared as described in Section 7.0 of this report, and observations indicate that subgrade is in a firm and unyielding condition.
- Fill placed to raise site grades should be compacted as recommended for structural fill in this report, placed in lifts and compacted to a minimum of 95 percent of MDD per ASTM D 1557 (modified proctor).
- A resilient modulus of 20,000 psi was estimated for base rock prepared and compacted as recommended.
- A resilient modulus of 5,500 psi was estimated for firm in-place soils or structural fill placed on firm native soils for consistency.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviations of 85 percent and 0.45, respectively.
- Structural coefficients of 0.41 and 0.10 for the asphalt and base rock, respectively.
- A 20-year design life.
- Truck traffic consists of an even distribution of two-axle service trucks/vans and large, four-axle trucks per day.

6.6. Asphalt Concrete (AC) Pavement Sections

If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate.

It is our understanding that the majority of the project paving will consist of surface parking lots and drive aisles on site and will remain private. However, some improvements may need to meet City Standards for typical Roadway Section Arterials and Neighborhood Street. Recommended minimum pavement sections are provided in Table 4 below.

TABLE 4. RECOMMENDED PAVEMENT SECTIONS

Street Type	Thickness Wearing Course[1]	Thickness HMA[2]	Thickness of crushed surfacing base course CSBC[3]	Design Standard
Automobile Parking	3 inches	-	4 inches	
Heavy Duty Pavement (truck traffic and delivery areas)	4 inches	-	6 inches	
Typical Roadway Section Neighborhood Street	-	4 inches	4 inches	Lynnwood Washington Public Works, STD3-3
Typical Roadway Section Arterials	-	6 inches	4 inches	Lynnwood Washington Public Works, STD3-2

Notes:

[1] ½-inch hot mix asphalt (HMA) (PG 58-22) per WSDOT Standard Specification Section 5-04 and 9-03.

[2] 1-inch HMA (PG 5822) per WSDOT Standard Specification Section 5-04 and 9-03.

[3] Crushed surfacing base course (CSBC) per WSDOT Standard Specification Section 9-03.9(3).

The base course should be compacted to at least 95 percent of the MDD (ASTM D 1557). We recommend that a proof-roll of the compacted base course be observed by a representative from our firm prior to paving. Soft or yielding areas observed during proof-rolling may require overexcavation and replacement with compacted structural fill.

Crushed surfacing base course should conform to applicable sections of 4-04 and 9-03.9(3) of the WSDOT Standard Specifications.

6.6.1. New Hot Mix Asphalt (HMA) Pavement

HMA should conform to applicable sections of 5-04, 9-02 and 9-03 of the WSDOT Standard Specifications. The AC binder should be PG 64-22 grade meeting WSDOT Standard Specifications. AC pavement should be compacted to 91.0 percent at Maximum Theoretical Unit Weight (Rice Gravity) of AASHTO T-209.

The recommended pavement sections assume that final improvements surrounding the pavement will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the crushed base.

6.6.2. Portland Cement Concrete (PCC) Pavement

The recommend PCC pavement section shall consist of 6 to 8 inches of PCC over a minimum of 6 inches of CSBC. The base course should be compacted to at least 95 percent MDD.

We recommend PCC pavements incorporate construction joints and/or crack control joints spaced maximum distances of 12 feet apart, center-to-center, in both the longitudinal and transverse directions. Crack control joints may be created by placing an insert or groove into the fresh concrete surface during finishing, or by sawcutting the concrete after it has initially set-up. We recommend the depth of the crack control joints be approximately one-fourth the thickness of the concrete; or 1½ and 2 inches deep for the recommended concrete thickness of 6 and 8 inches, respectively. We also recommend the crack control joints be sealed with an appropriate sealant to help restrict water infiltration into the joints.

6.7. Seismic Parameters

6.7.1. Seismicity

The Puget Sound region is located at the convergent continental boundary known as the Cascadia Subduction Zone (CSZ), which extends from mid-Vancouver Island to Northern California. The CSZ is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The interaction of these two plates results in three potential seismic source zones: (1) a shallow crustal source zone; (2) the Benioff source zone; and (3) the CSZ interplate source zone.

The shallow crustal source zone is used to characterize shallow crustal earthquake activity within the North American Plate at depths ranging from 3 to 19 miles below the ground surface. The closest known fault is the South Whidbey Island Fault, mapped approximately 2 miles north of the site.

The Benioff source zone is used to characterize intraplate, intraslab or deep subcrustal earthquakes. Benioff source zone earthquakes occur within the subducting Juan de Fuca Plate at depths between 20 and 40 miles. In recent years, three large Benioff source zone earthquakes occurred that resulted in some liquefaction in loose alluvial deposits and significant damage to some structures. The first earthquake, which was centered in the Olympia area, occurred in 1949 and had a Richter magnitude of 7.1. The second earthquake, which was centered between Seattle and Tacoma, occurred in 1965 and had a Richter magnitude of 6.5. The third earthquake, which was located in the Nisqually valley north of Olympia, occurred in 2001 and had a Richter magnitude of 6.8.

The CSZ interplate source zone is used to characterize rupture of the convergent boundary between the subducting Juan de Fuca Plate and the overriding North American Plate. The depth of CSZ earthquakes is greater than 40 miles. No earthquakes on the CSZ have been instrumentally recorded; however, through the geologic record and historical records of tsunamis in Japan, it is believed that the most recent CSZ event occurred in 1700.

6.7.2. Geologic Hazards

Based on our review of the City's critical areas code (Chapter 17.10) and Snohomish County Geologic Hazards Landslide Hazard Areas map, the site is not located in a geologic hazard area, except for areas where slopes are greater than 40 percent as discussed in Section 5.2.2 of this report. Those areas will be regraded as part of site development.

Based on our review of the borings, the site soils have a low risk of liquefaction, lateral spread, or seismically induced landslide. Additionally, due to the distance to the closest mapped fault (South Whidbey Fault), the site has a low risk of fault rupture.

6.7.3. 2018 IBC Seismic Design Information

The 2018 IBC is the current building code. The 2018 IBC references the 2016 Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers [ASCE] 7-16). We recommend that the site be classified as Site Class C – Very Dense Soil and Soft Rock. Our recommended seismic design parameters are presented in Table 5 below.

TABLE 5. MAPPED 2018 IBC SEISMIC DESIGN PARAMETERS

Seismic Design Parameters	Recommended Value[1, 2]
Site Class	C
Mapped Spectral Response Acceleration at Short Period (S_s)	1.303g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.51g
Site Modified Peak Ground Acceleration (PGA_M)	0.534g
Site Amplification Factor at 0.2 second period (F_a)[2]	1.0
Site Amplification Factor at 1.0 second period (F_v)[2]	1.3
Design Spectral Acceleration at 0.2 second period (S_{DS})[2]	0.869g
Design Spectral Acceleration at 1.0 second period (S_{D1})[2]	0.442g

Notes:

[1] Parameters developed based on Latitude N47.8287302° and Longitude -122.309308° using the ATC Hazards online tool.

7.0 EARTHWORK RECOMMENDATIONS

7.1. Site Preparation

In general, site development will require demolition of any existing structures, concrete sidewalks and surface pavement, new foundation construction, and installation of subsurface utilities. The existing surficial soils consist primarily of silty sand and contain high fines (silt) content such that repeated construction traffic will result in considerable disturbance during wet weather construction. If wet weather construction occurs, it may be necessary to provide a layer of quarry spalls, crushed rock or pit run sand and gravel if the on-site soils become wet and disturbed. Additional wet weather considerations are included below.

7.1.1. Demolition

All existing structural elements should be excavated and removed from proposed structural areas. If present, existing utilities that will be abandoned on site should be identified prior to project construction. Abandoned utility lines larger than 4 inches in diameter that are located beneath proposed structural areas should be completely removed or filled with grout if abandoned and left in place in order to reduce potential settlement or caving in the future. Materials generated during demolition of existing structural improvements should be transported off site for disposal.

Previously developed sites often have remnant buried features from previous uses such as fuel storage or other types of buried tanks, cisterns, former basements or storages, or other remnant debris from foundations and slabs, and some of those elements may be present on this site. During demolition and site grading, such buried features may be encountered and should be excavated and removed within proposed development areas.

Materials generated during demolition of existing improvements should be transported off site for disposal. Existing voids and new depressions created during site preparation, and resulting from removal of existing utilities or other subsurface elements, should be cleaned of loose soil or debris down to firm soil and backfilled with compacted structural fill. Disturbance to a greater depth should be expected if site preparation and earthwork are conducted during periods of wet weather.

7.1.2. Stripping

In landscape areas or areas of wild vegetative growth stripping of surface organics and roots will be required. In general upper organics should be stripped to depths of 6 to 8 inches with increased depths in areas of thicker vegetation. Greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal unless otherwise allowed by project specifications for other uses such as landscaping. Clearing and grubbing recommendations provided below should be used in areas where moderate to heavy vegetation are present, or where surface disturbance from prior use has occurred.

Where thicker vegetation is present, more extensive site clearing will be required to remove site vegetation, including thick grass, shrubs and trees. Following clearing, grubbing and excavations up to several feet may be required to remove the root zones of thick shrubs and trees. Deeper excavations, up to 5 or 6 feet may be required to remove the root zones of large trees. Roots larger than ½ inch in diameter should be removed. Excavations to remove root zones should be done with a smooth bucket to minimize subgrade disturbance. Portions of the site are heavily vegetated and previously buried roots may be present, even in the current grassy areas of the site. Grubbed materials should be hauled off site and properly disposed unless otherwise allowed by the project specifications for other uses such as landscaping, stockpiling or on-site burning.

Existing voids and new depressions created during demolition, clearing, grubbing or other site preparation activities, should be excavated to firm soil and backfilled with Imported Select Structural Fill. Greater depths of disturbance should be expected if site preparation and earthwork are conducted during periods of wet weather.

7.1.3. Subgrade Improvement

In areas where additional fill is to be placed to raise site grades the subgrade should be compacted in place prior to fill placement. After demolition and in areas that have been stripped of the root zone and organics, and where some upper disturbed material remains in-place, we recommend that the upper soil be improved by compaction. Subgrade improvement can be accomplished by removing and replacing or scarifying and recompacting the upper 18 inches of in place fill soil prior to placing site grading fill or base rock materials. Scarification is typically performed by ripping with agricultural discs and aerating the soils to dry. This is best performed during the dry season when rain is less likely to occur. Considerable soil processing, including moisture conditioning (likely drying), should be expected at most times of the year in order to adequately compact the on-site silty soil. If the soil cannot be properly moisture conditioned (dried-back), the subgrade should be removed down to firm material and replaced with granular fill.

7.2. Subgrade Preparation and Evaluation

Upon completion of site preparation activities, exposed subgrades should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tired construction equipment where space allows to identify soft, loose or unsuitable areas. Probing may be used for evaluating smaller areas or where proof-rolling is not practical. Proof-rolling and probing should be conducted prior to placing fill, and should be performed by a representative of GeoEngineers who will evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. If soft or loose zones are identified during proof-rolling or probing, these areas should be excavated to the extent indicated by our representative and replaced with structural fill.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations, probing and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing should be removed and replaced with compacted structural fill.

7.3. Footing and Basement Excavations

Following footing excavation, we recommend that exposed subgrade be evaluated by a representative from our firm to assess the adequacy of the subgrade conditions and to confirm subsurface soils as are anticipated. Any disturbed or otherwise unsuitable areas identified should be re-compacted, if practical or removed.

Excavations are anticipated during construction in relation to spread footings, permanent below-grade walls and underground utilities. We anticipate portions of the excavations can likely be made as temporary open cut slopes depending on the site constraints. Where temporary cut slopes are not feasible, temporary shoring will be required. The stability of open cut slopes is a function of soil type, groundwater seepage, slope inclination, slope height and nearby surface/surcharge loads. The use of inadequately designed open cuts could impact the stability of adjacent work areas, existing utilities and endanger personnel.

The contractor performing the work has the primary responsibility for protection of workmen and adjacent improvements. In our opinion, the contractor will be the in the best position to observe subsurface conditions continuously through the construction process and to respond to variable soil and groundwater conditions. Therefore, the contractor should have the primary responsibility for deciding whether or not to use open cut slopes for much of the excavation rather than some form of temporary excavation support, and for establishing the safe inclination of the cut slope. Acceptable slope inclinations for utilities and ancillary excavations should be determined during construction. Because of the diversity of construction techniques and available shoring systems, the design of the temporary shoring is most appropriately left up to the contractor proposing to complete the installation. Temporary cut slopes and shoring must comply with the provisions of Title 296-155 Washington Administrative Code (WAC), Part N, "Excavation, Trenching and Shoring."

7.4. Utility Trenches

Trench excavations, pipe bedding and trench backfilling should be completed using the general procedures described in the 2020 WSDOT Standard Specifications or other suitable procedures specified by the project civil engineer. The native glacial deposits and fill soils encountered at the site are generally low corrosivity, based on our experience in the Puget Sound region.

Utility trench backfill should consist of structural fill and should be placed in lifts of 6 inches or less (loose thickness) if hand-operated compaction equipment is utilized. Imported backfill, containing less than 5 percent fines, may be compacted in loose lifts not exceeding 12 inches, depending on the compaction equipment used. Each lift must be compacted prior to placing the subsequent lift. Prior to compaction, the backfill should be moisture-conditioned to within 3 percent of the optimum moisture content, if necessary.

The backfill should be compacted in accordance with the criteria discussed in the "Fill Placement and Compaction Criteria" Section 7.6.3 below.

7.5. Slopes

7.5.1. Temporary Cut Slopes

For planning purposes, temporary unsupported cut slopes more than 4 feet high may be inclined at 1½H:1V or flatter within the fill or recent deposits. Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs. For open cuts at the site, we recommend that:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least one-half the slope height from the top of the cut.
- Exposed soil along the slope be protected from surface erosion by using waterproof tarps or plastic sheeting.
- Construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practicable.
- Erosion control measures be implemented, as appropriate, such that runoff from the site is reduced to the extent practicable.
- Surface water be diverted away from the slope.
- The general condition of the slopes be observed periodically by the geotechnical engineer to confirm adequate stability.

More restrictive requirements may apply depending on specific site conditions, which should be continuously assessed by the contractor.

If temporary sloping is not feasible based on site spatial constraints, excavations could be supported by internally braced shoring systems, such as a trench box or other temporary shoring. There are a variety of options available. We recommend that the contractor be responsible for selecting the type of shoring system to apply.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

7.5.2. Permanent Cut and Fill Slopes

Permanent slopes may be constructed at inclinations of 2H:1V or flatter. Fill to create permanent slopes should be compacted to at least 95 percent of MDD. To achieve uniform compaction, we recommend that fill slopes be slightly overbuilt and cut back to expose well-compacted fill.

To reduce erosion, newly constructed slopes and disturbed existing slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and raveling of the slopes should be expected. This may necessitate localized repairs and reseeding. Temporary covering, such as clear heavy plastic sheeting, or erosion control blankets (such as American Excelsior Curlex 1 or North American Green SC150) could be used to protect the slopes during periods of rainfall.

7.5.3. Slope Drainage

If seepage is encountered at the face of permanent or temporary slopes, it will be necessary to flatten the slopes or install a subdrain to collect the water. We should be contacted to evaluate such conditions on a case-by-case basis.

7.6. Structural Fill and Backfill

Structural areas include areas beneath foundations, floor slabs, pavements, and any other areas intended to support structures or within the influence zone of structures should generally meet the criteria for structural fill presented below. All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches (3-inch maximum particle size in building footprints), and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

7.6.1. Materials

Fill placed to support structures, behind retaining structures, and below pavements and sidewalks is specified as structural fill as described below:

- Structural fill used to support buildings, roadways, utilities, and hardscapes may contain an increased fines content during dry weather provided it can be moisture conditioned and compacted to the minimum standard. Fill placed during the wet season should consist of Gravel Borrow, WSDOT Standard Specification 9-03.14(1) with the added restriction that it contains less than 5 percent passing the U.S. No. 200 sieve.
- Structural fill placed as capillary break material should meet the requirements of WSDOT Standard Specification 9-03.1(4)C, Grading No. 57.
- Structural fill placed behind retaining walls should meet the requirements of Gravel Backfill for Walls, WSDOT Specification 9-03.12(2).
- Structural fill placed around perimeter footing drains, underslab drains and cast-in-place wall drains should meet the requirements of Gravel Backfill for Drains, 9-03.12(4).
- Structural fill placed as CSBC below pavements and sidewalks should meet the requirements of Crushed Surfacing Base Course, WSDOT Specification 9-03.9(3).

7.6.2. On-site Soils

The on-site soils are moisture sensitive and generally have natural moisture contents higher than the anticipated optimum moisture content for compaction. As a result, the on-site soils will likely require moisture-conditioning in order to meet the required compaction criteria during dry weather conditions and will not be suitable for reuse during wet weather. Furthermore, most of the fill soils required for this project have specific gradation requirements, and the on-site soils do not meet these gradation requirements. If the contractor wants to use on-site soils for structural fill, GeoEngineers can evaluate the on-site soils for suitability as structural fill following site preparation when these materials are exposed.

7.6.3. Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 1 foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

- Structural fill placed in building areas (supporting foundations, slab-on-grade floors or structural slabs) and in pavement and sidewalk areas (including utility trench backfill) should be compacted to at least 95 percent of the MDD estimated in general accordance with ASTM D 1557.
- Structural fill placed against subgrade walls should be compacted to between 90 and 92 percent. Care should be taken when compacting fill against subsurface walls to avoid overcompaction and hence, overstressing the walls.

We recommend that GeoEngineers be present during probing of the exposed subgrade soils in building and pavement areas, and during placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

7.7. Weather Considerations

The on-site soils contain a sufficient percentage of fines (silt and clay) to be moisture sensitive. When the moisture content of these soils is more than a few percentage points above the optimum moisture content, these soils become muddy and unstable, and operation of equipment on these soils is difficult. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. During wet weather, we recommend the following:

- The ground surface in and around the work area be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. The contractor should take measures to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unsuitable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance (as described in previous sections).
- Construction activities should be scheduled so that the length of time the soils are left exposed to moisture is reduced to the extent practicable.

7.8. Construction Dewatering

Based on the site explorations, the proposed excavation depths are not anticipated to require active dewatering using wells or well points. Passive dewatering systems using gravel-lined ditches routed to sumps and pumps are anticipated to be sufficient for the planned site excavations.

In addition to groundwater seepage and upward confining flow, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

A contingency should be carried in the budget for earthwork that may occur during periods of wet weather. Wet weather will result in delays and difficulty in achieving compaction of fill soils. Additionally, if wet weather occurs during any overexcavation work, pumps may need to run overnight and on weekends to keep any open excavations from filling.

7.9. Recommended Additional Geotechnical Services

GeoEngineeers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.

During construction, GeoEngineers should observe the installation of the soldier piles and tiebacks, evaluate the suitability of the foundation subgrades, observe installation of subsurface drainage measures, evaluate structural backfill, observe temporary cut slopes and provide a summary letter of our construction observation services. The purposes of GeoEngineers' construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons as described in Appendix C, Report Limitations and Guidelines for Use.

8.0 LIMITATIONS

We have prepared this report for the exclusive use of Holman Auto and their authorized agents for the Lynnwood Auto Dealership project in Lynnwood, Washington

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix C titled "Report Limitations and Guidelines for Use" for additional information pertaining to the use of this report.

9.0 REFERENCES

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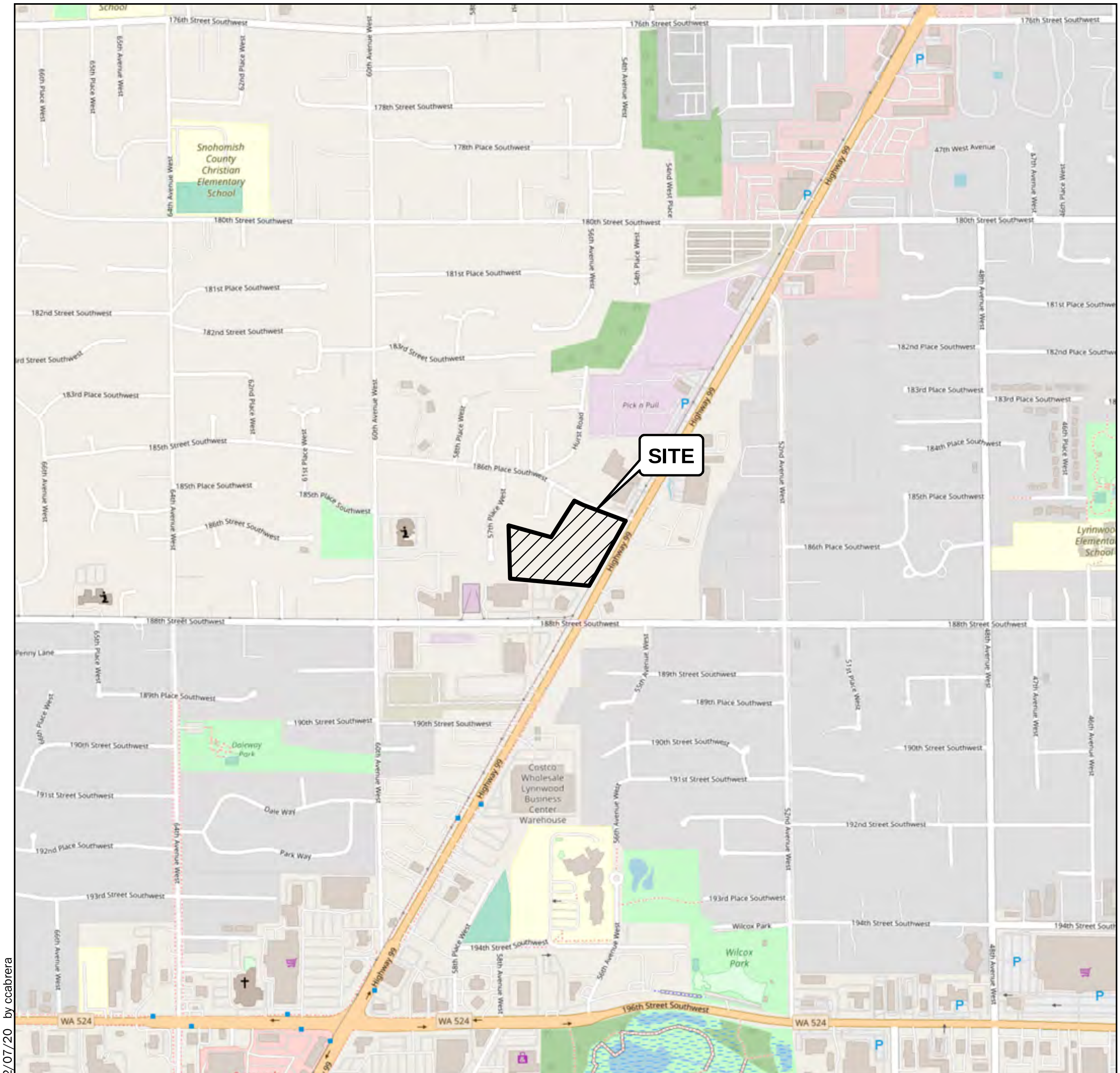
FIGURES

GeoEngineers



N
W E
S

1,000 0 1,000

Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Lynnwood Auto Dealer
Lynnwood, Washington

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Figure 1

P:\24\24789001\GIS\MXD\2478900100_F01_VicinityMap.mxd Date Exported: 12/07/20 by ccabrera

P:\24\24789001\CAD\00\GeoTech Report\24789001_F02-F05_Site Plan & Sections.dwg TAB:2478900100_F02A Existing Conditions Site and Exploration Plan Date Exported: 12/18/20 - 11:18 by mklym.tmp

Legend

GEI-01 Boring by GeoEngineers, Inc., 2020

A A' Cross Section Location

Area with Slope Greater than 40%

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Background from 3J Consulting dated 12/07/2020.

Projection: WA State Plane, North Zone, NAD83, US Foot

N
W E
S

80 0 80
Feet

Existing Conditions Site and Exploration Plan

Lynnwood Auto Dealership
Lynnwood, Washington

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Figure 2A

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Legend

GEI-01 Boring by GeoEngineers, Inc., 2020

A A' Cross Section Location

Notes:

1. The locations of all features shown are approximate.
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Data Source: Background from 3J Consulting dated 12/07/2020.

Projection: WA State Plane, North Zone, NAD83, US Foot

N
W E
S

80 0 80
Feet

Proposed Conditions Site and Exploration Plan

Lynnwood Auto Dealership
Lynnwood, Washington

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Figure 2B

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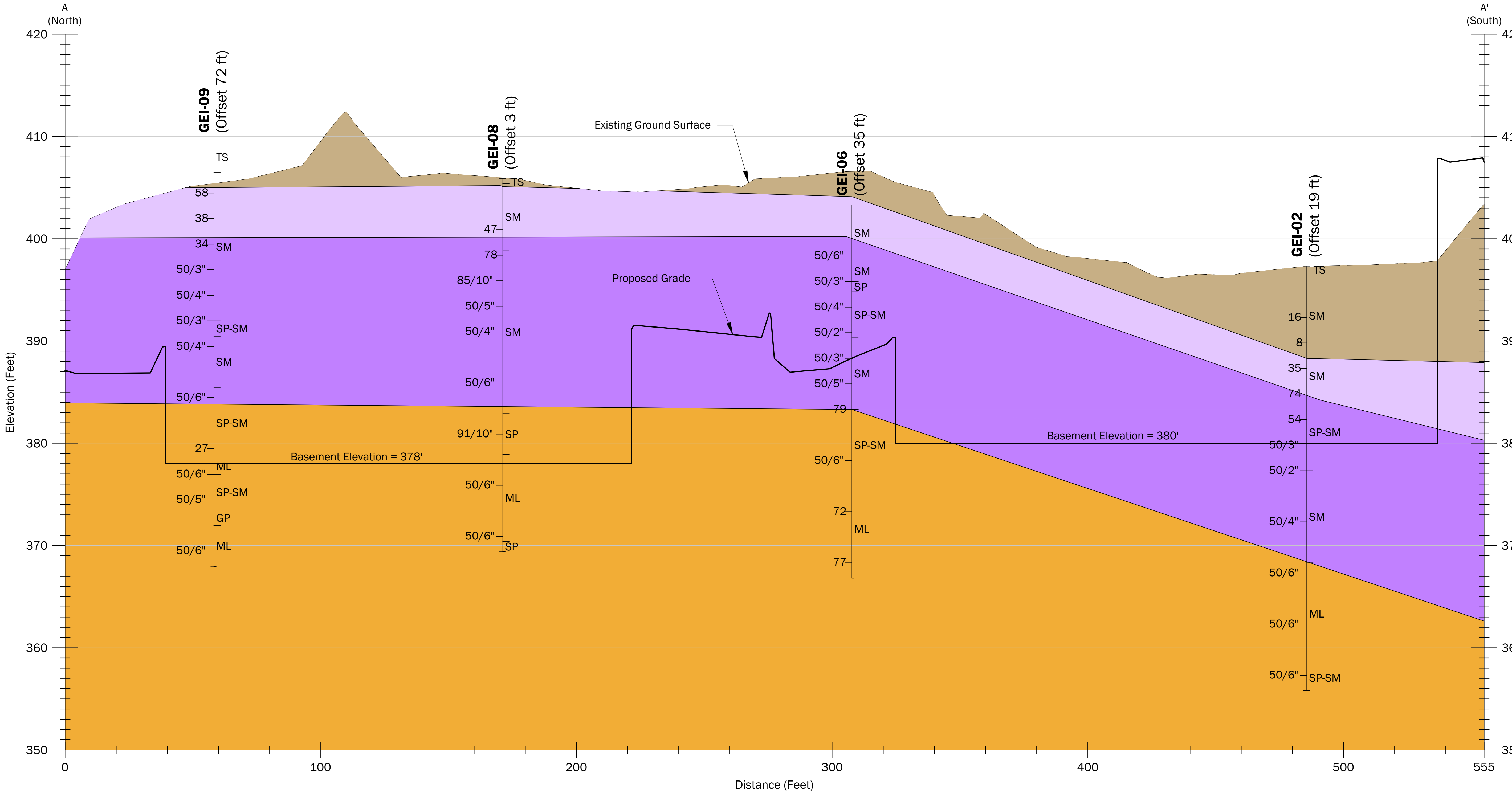
P:\24\24789001\CAD\00\GeoTech Report\2478900100_F02-F05_Site Plan & Sections.dwg TAB:2478900100_F02B Proposed Conditions Site and Exploration Plan Date Exported: 12/18/20 - 11:18 by mklym.tmp



Legend

Boring ID (Offset)

Boring

SM Soil Classification

Groundwater Measured in Piezometer

20 Blow Count

Fill Material/Topsoil

Weathered Till

Glacial Till

Advanced Outwash

40 0 40
Horizontal Scale in Feet

10 0 10
Vertical Scale in Feet
Vertical Exaggeration: 2.5X

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Background based on survey created by 3J Consultants dated 12/07/2020

Projection: WA State Plane, North Zone, NAD83, US Foot

Cross Section A-A'

Lynnwood Auto Dealership
Lynnwood, Washington

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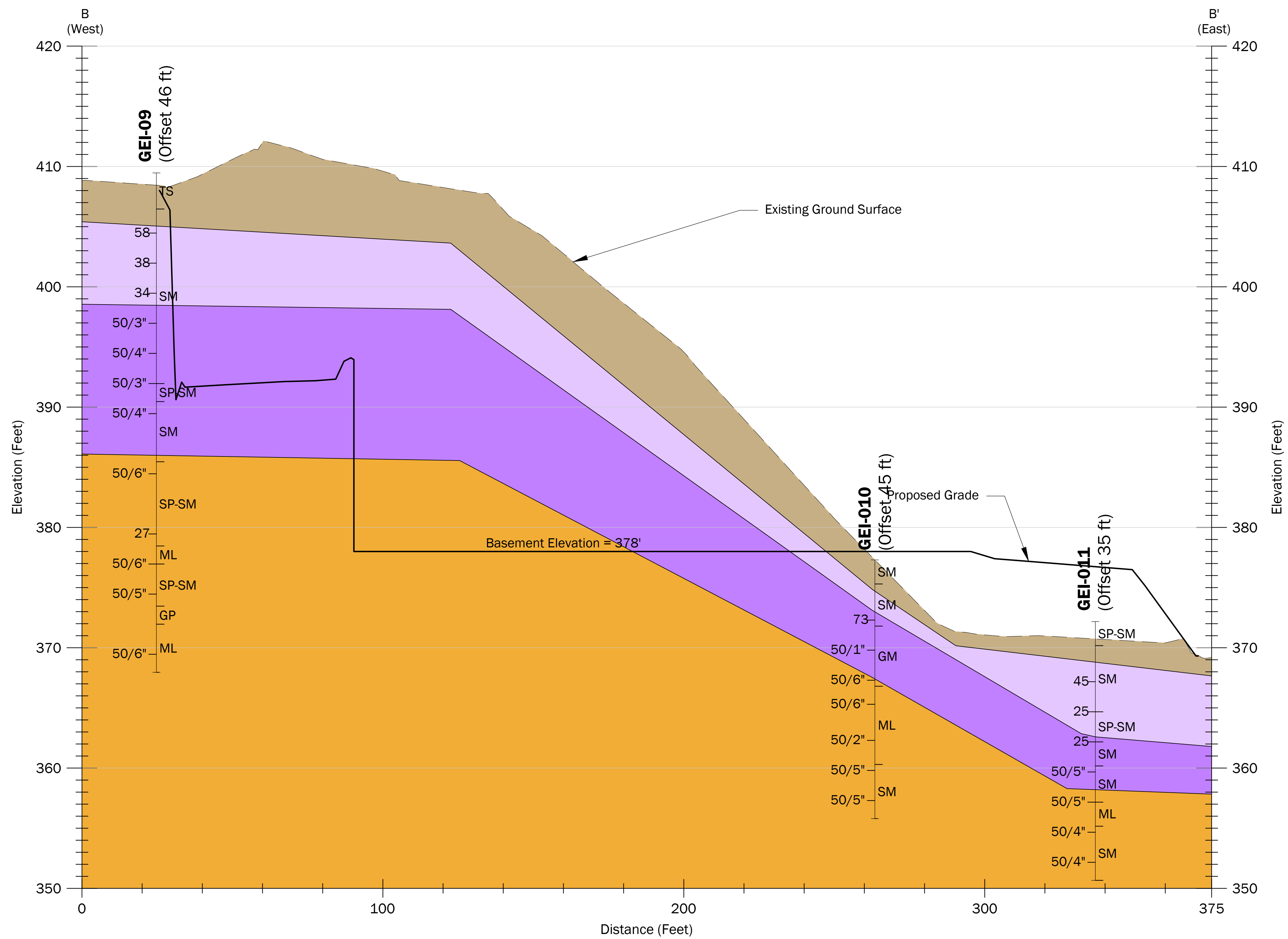
Figure 3



Legend

Boring ID (Offset)

Boring

SM Soil Classification

Groundwater Measured in Piezometer

20 Blow Count

Fill Material/Topsoil

Weathered Till

Glacial Till

Advanced Outwash

40 0 40
Horizontal Scale in Feet

10 0 10
Vertical Scale in Feet
Vertical Exaggeration: 2.5X

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Background based on survey created by 3J Consultants dated 12/07/2020

Projection: WA State Plane, North Zone, NAD83, US Foot

Cross Section B-B'

Lynnwood Auto Dealership
Lynnwood, Washington

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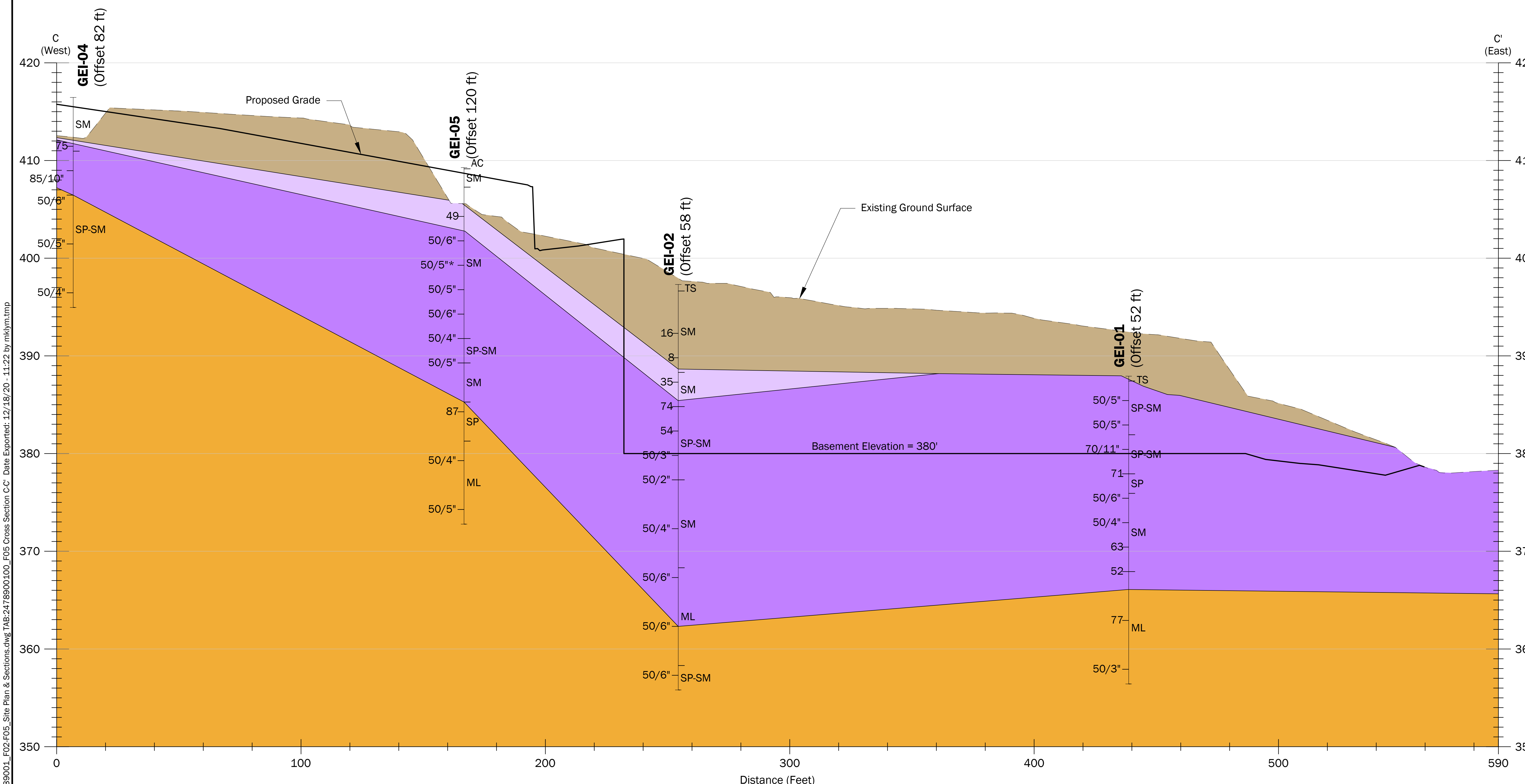
Figure 4



Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Background based on survey created by 3J Consultants dated 12/07/2020

Projection: WA State Plane, North Zone, NAD83, US Foot

Legend

Boring ID (Offset)

Boring

SM Soil Classification

Groundwater Measured in Piezometer

20 Blow Count

Fill Material/Topsoil

Weathered Till

Glacial Till

Advanced Outwash

40 0 40
Horizontal Scale in Feet

10 0 10
Vertical Scale in Feet
Vertical Exaggeration: 2.5X

Cross Section C-C'

Lynnwood Auto Dealership
Lynnwood, Washington

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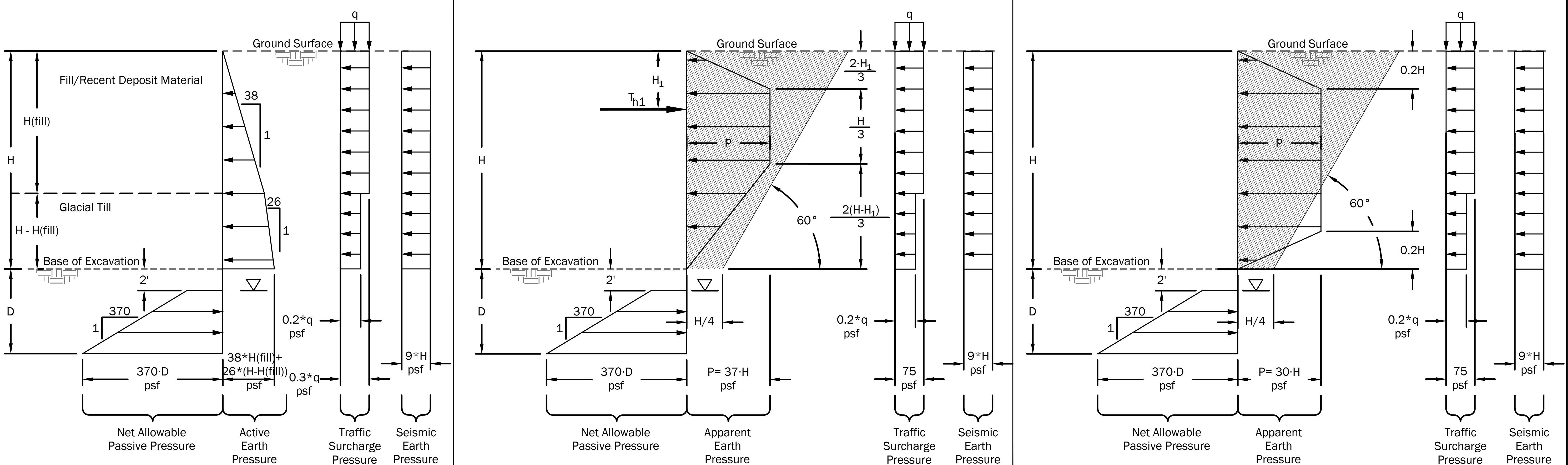
Figure 5

P:\24\24789001\CAD\00\GeoTech Report\24789001_F02-F05_Site Plan & Sections.dwg TAB:24789001_F05 Cross Section C-C' Date Exported: 12/18/20 - 11:22 by mklym.tmp

Cantilever Soldier Pile | Conventional Soldier Pile Wall with One Level of Tiebacks | Conventional Soldier Pile Wall with Multiple Levels of Tiebacks



Notes:

1. The static earth pressures do not include a factor of safety.
2. Active/apparent earth pressure and traffic surcharge pressure act over the pile spacing above the base of the excavation.
3. Passive earth pressure acts over 2.5 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
4. Passive pressure includes a factor of safety of 1.5
5. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 8.
6. This pressure diagram is appropriate for temporary soldier pile and tieback walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.
7. Seismic Earth Pressures do not need to be considered for temporary walls.

Legend

- No Load Zone
- H = Height of Excavation, Feet
- D = Soldier Pile Embedment Depth, Feet
- H_1 = Distance From Ground Surface to Uppermost Tieback, Feet
- H(fill) = Fill/Recent Deposits thickness behind wall
- T_{h1} = Horizontal Load in Uppermost Ground Anchor
- P = Maximum Apparent Earth Pressure, Pounds per Square Foot
- q = Traffic Surcharge
- ▽ Design Groundwater Elevation for Drained Walls/ Passive Resistance Design

Not To Scale

Earth Pressure Diagrams - Soldier Pile & Tieback Wall

Lynnwood Auto Dealership
Lynnwood, Washington

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Figure 6

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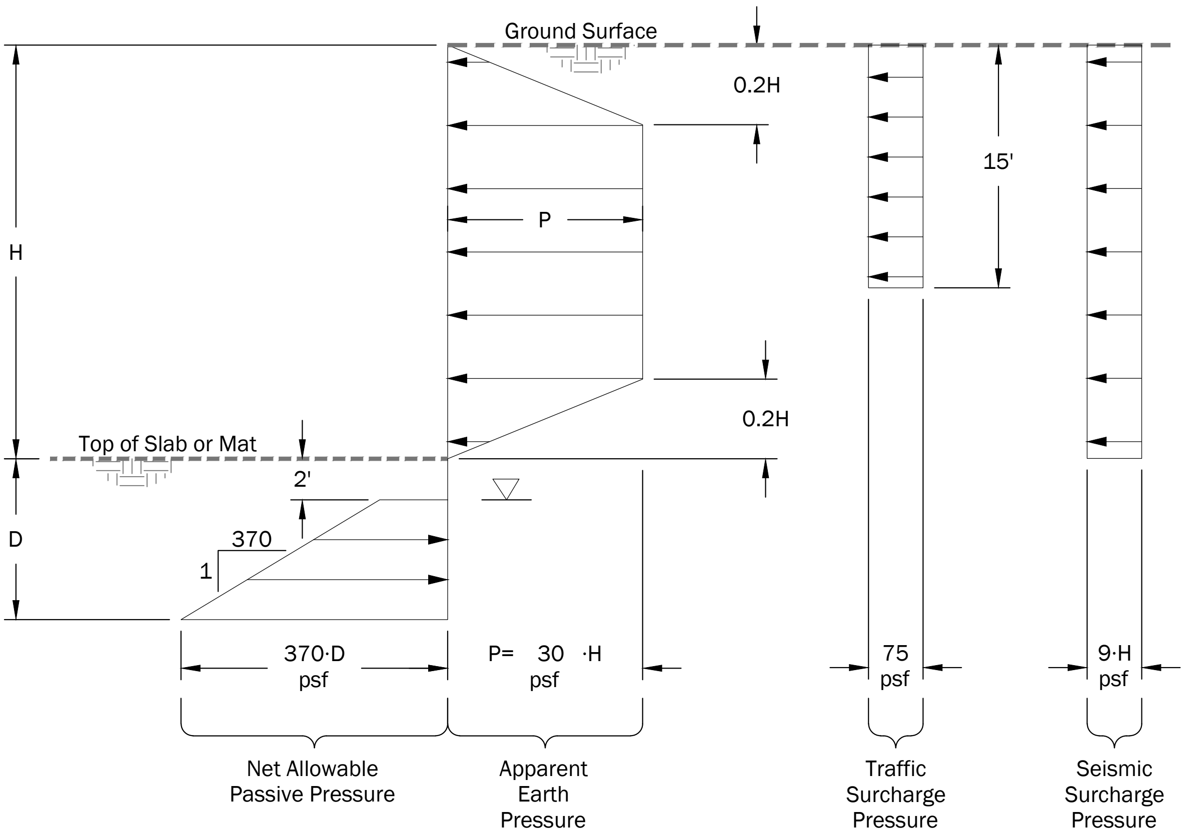
Permanent Basement Wall Against Temporary Shoring



Legend

H = Height of Basement Wall, Feet

D = Foundation Embedment Depth, Feet

P = Maximum Apparent Earth Pressure, Pounds per Square Foot

▽ Design Groundwater Elevation for Drained Walls/ Passive Resistance Design

Not To Scale

Notes:

1. Passive earth pressure includes a factor of safety of 1.5
2. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 8.
3. This pressure diagram is appropriate for permanent basement walls constructed in front of temporary shoring walls with tieback or soil nail anchors. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.
4. The static earth pressure does not include a factor of safety and represents the actual anticipated static earth pressure.

Earth Pressure Diagram
Permanent Below Grade Walls

Lynnwood Auto Dealer
Lynnwood, Washington

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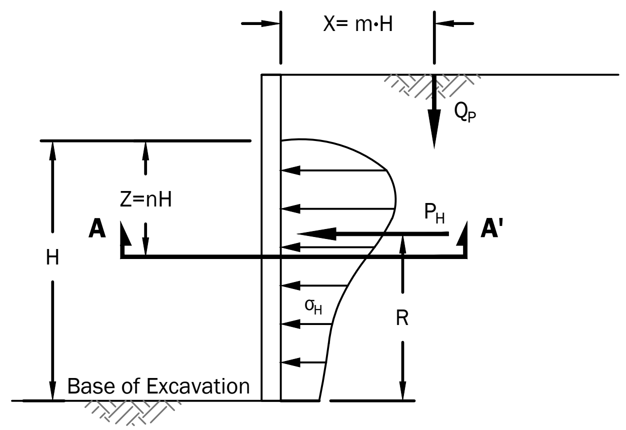
Figure 7

Lateral Earth Pressure from Point Load, Q_P (Spread Footing)



For $m \le 0.4$

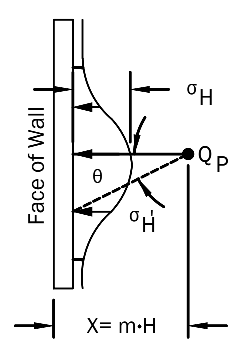
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2 (0.16+n^2)^3}$$

For $m > 0.4$

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

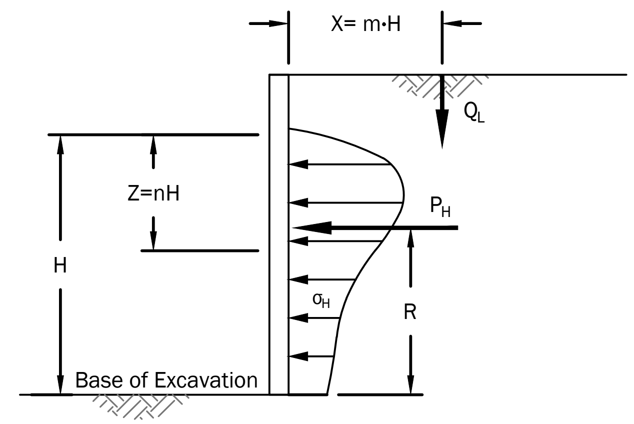
$$\sigma'_H = \sigma \, COS^2 (1.1\theta)$$

m	$P_H \left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



Section A-A'
Pressures from Point Load Q_P

Lateral Earth Pressure from Line Load, Q_L (Continuous Wall Footing)



For $m \le 0.4$

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

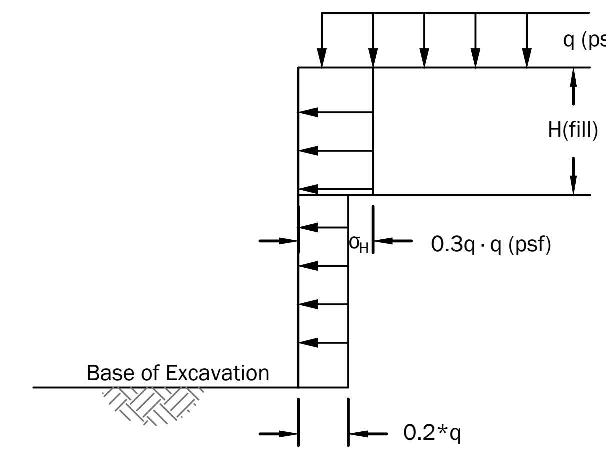
For $m > 0.4$

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

$$\text{Resultant } P_H = \frac{0.64 Q_L}{(m^2+1)}$$

m	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

Uniform Surcharges, q (Floor Loads, Large Foundation Elements or Traffic Loads)



σ_H = Lateral Surcharge Pressure from Uniform Surcharge

Definitions:

- Q_P = Point load in pounds
- Q_L = Line load in pounds/foot
- H = Excavation height below footing, feet
- H(fill) = Fill/recent deposits thickness behind wall
- σ_H = Lateral earth pressure from surcharge, psf
- q = Surcharge pressure in psf
- θ = Radians
- σ'_H = Distribution of σ_H in plan view
- P_H = Resultant lateral force acting on wall, pounds
- R = Distance from base of excavation to resultant lateral force, feet
- X = Resultant lateral force acting on wall, pounds
- Z = Depth of σ_H to be evaluated below the bottom of Q_P or Q_L
- m = Ratio of X to H
- n = Ratio of Z to H

Notes:
1. Procedures for estimating surcharge pressures shown above are based on Manual 7.02 Naval Facilities Engineering Command, September 1986 (NAVFAC DM 7.02).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figures 6 and 7.
3. See report text for where surcharge pressures are appropriate.

Not To Scale

Recommended Surcharge Pressure

Lynnwood Auto Dealership
Lynnwood, Washington

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Figure 8

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APPENDICES

GEOENGINEERS

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface conditions at the site were evaluated by drilling 11 boring (GEI-01 through GEI-11) to depths ranging from approximately 21½- to 41½-feet below existing grades. The borings were completed by Boretec1, Inc. on November 30 through December 2, 2020.

The locations of the explorations were recorded in the field using a handheld global positioning system (GPS) unit. The approximate exploration locations are shown on Figures 2A and 2B.

Borings

The borings were completed using track-mounted, continuous-flight, hollow-stem auger (HSA) drilling equipment. The borings were continuously monitored by a geotechnical engineer from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration.

The soils encountered in the borings were generally sampled at 2½- and 5-foot vertical intervals with a 2-inch outside diameter (O.D.) split-barrel standard penetration test sampler. The disturbed samples were obtained by driving the sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows for each 6-inch increment of penetration was recorded. The blow count ("N-value") of the soil was calculated as the number of blows required for the second and third 6-inch intervals. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions precluded drive at least 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1, Key to Exploration Logs. A key to the boring log symbols is also presented in Figure-A-1. The logs of the borings are presented in Figures A-2 through A-12, which are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change may actually be gradual. If the change occurred between samples, it was interpreted. The densities noted on the boring logs are based on the blow count data obtained in the borings and judgement based on the conditions encountered.

Observations of groundwater conditions were made during drilling. The groundwater conditions encountered during drilling are presented on the boring logs. Groundwater conditions observed during drilling represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.

Dynamic cone penetrometer (DCP) soundings were performed by a staff geotechnical engineer from our office who recorded blow count versus cumulative penetration depth. This penetration resistance data was compared to the adjacent explorations where a detailed log of subsurface explorations was maintained, the soils encountered were visually classified and representative soil samples from the borings were obtained.

Laboratory Testing

Soil samples obtained from the explorations were transported to GeoEngineers' laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soil samples.

Representative samples were selected for laboratory testing to determine the moisture content, grain-size distribution (sieve analysis) and percent fines (material passing the U.S. No. 200 sieve). The tests were performed in general accordance with test methods of ASTM International (ASTM) or other applicable procedures.

Moisture Content (MC)

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs in Appendix A at the depths at which the samples were obtained.

Sieve Analysis (SA)

Sieve analyses were performed on selected samples in general accordance with ASTM D 422. The wet sieve analysis method was used to determine the percentage of soils greater than the US No. 200 mesh sieve. The results of the sieve analyses were plotted, were classified in general accordance with the Unified Classification System (USCS) and are presented in Figure A-13.

It should be noted that the sieve analyses were performed on soils obtained from samplers that have an opening size of 1½ inches, so larger sized particles cannot be obtained by the samplers. Therefore, the sieve results do not account for soil particles that are larger than 1½ inches. Soils with larger sized materials are described in this report qualitatively based on visual observations and experience on projects where excavations were made into similar formations.

Percent Passing US No. 200 Sieve (%F)

Selected samples were "washed" through the US No. 200 mesh sieve to estimate the relative percentages of course- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of sample finer than the US No. 200 sieve. These tests were conducted to verify field descriptions and to estimate the fines content for analysis purposes. The tests were conducted in accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

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Figure A-1

Drilled	Start 11/30/2020	End 11/30/2020	Total Depth (ft) 31.5	Logged By CAH / Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	388 / NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Track-mounted drill rig EC95
Easting (X) / Northing (Y)	1277994 / 305935	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
388	0				TS	Vegetated surface (low grass, blackberry vines) with approximately 6 inches of topsoil			
					SP-SM	Brown fine to coarse sand with silt and gravel (very dense, moist) (weathered till)			
385		7	50/5"	1					
	5	9	50/5"	2					
					SM	Gray silty fine to coarse sand with gravel (very dense, moist) (glacial till)			
380		17	70/11"	3 %F			9	35	
	10	14.5	71	4	SP	Gray fine to medium sand (very dense, moist)			
					SM	Gray silty fine to coarse sand with gravel (very dense, moist)			
375		5	50/6"	5 MC			11		
	15	4.5	50/4"	6					
370		17.5	63	7					
	20	17	52	8	ML	Gray sandy silt (hard, wet) (advanced outwash)			
									Water bearing at 21 feet
365									Water bearing at 23 to 23½ feet
	25	18	77	9					Groundwater observed at approximately 25 feet below ground surface during drilling
360									
	30	11	50/3"	10		Grades to silt with sand, becomes moist			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-01

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Drilled	Start 11/30/2020	End 11/30/2020	Total Depth (ft) 41.5	Logged By CAH / Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum: 397 / NAVD88	Hammer Data: Rope & Cathead, 140 (lbs) / 30 (in) Drop	Drilling Equipment: Track-mounted drill rig EC95	
Easting (X) / Northing (Y): 1277817 / 305987	System Datum: WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
397	0							TS	Vegetated surface (low grass and bramble) with approximately 8 inches of topsoil			
								SM	Brown fine to coarse sand with organic matter (medium dense, moist) (fill material)			
	5		7	16		1			Organic matter grades out			
			11	8		2						
	10		13.5	35		3 %F		SM	Brown silty fine to coarse sand with gravel (dense, moist) (weathered till)	11	28	
			17	74		4		SP-SM	Brown fine to medium sand with silt (very dense, moist) (glacial till)			
	15		17	54		5 MC			Grades to fine sand, becomes gray and moist to wet	17		Increase in moisture at 16½ feet
			11	50/3"		6			Grades to fine to coarse sand			
	20		2	50/2"		7		SM	Gray silty fine to coarse sand (very dense, moist to wet)			
	25		2	50/4"		8						Groundwater observed at approximately 26½ feet below ground surface during drilling
								ML	Gray sandy silt (hard, wet) (advanced outwash)			
	30		11	50/6"		9						Fractured rock at approximately 30 feet
	35											

Elevation scale: 395, 390, 385, 380, 375, 370, 365

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-02

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Figure A-3
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		11	50/6"		10 / SA				22	68	Groundwater observed at approximately 35½ feet below ground surface during drilling
360												
								SP-SM	Gray silty fine to coarse sand (very dense, moist)			
	40		0	50/6"		11						

Log of Boring GEI-02 (continued)

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Drilled	Start 11/30/2020	End 11/30/2020	Total Depth (ft)	20.5	Logged By CAH Checked By CL	Driller Boretec 1, Inc.	Drilling Method	Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	409 NAVD88		Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop			Drilling Equipment	Track-mounted drill rig EC95
Easting (X) Northing (Y)	1277604 305963		System Datum	WA State Plane North NAD83 (feet)		Groundwater not observed at time of exploration		

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
409	0							TS	Vegetated with approximately 4 to 6 inches of topsoil			
								SP-SM	Brown fine to coarse sand with silt and gravel (fill material)			
405								SP-SM	Gray fine to coarse sand with silt and gravel (very dense, moist) (glacial till)			
	5		10.5	50/5"		1 %F				4		
			8	50/6"		2		SM	Gray silty fine to coarse sand with gravel (very dense, moist)			
400												
	10		9	50/4"		3						
395												
	15		4	50/3"		4						
390												
	20		4.5	50/3"		5						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Date:12/17/20 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\24\24789001\GINT\2478900100.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring GEI-03

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Drilled	Start 11/30/2020	End 11/30/2020	Total Depth (ft) 21.5	Logged By CAH / Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 416 / Vertical Datum NAVD88	Hammer Data: Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment: Track-mounted drill rig EC95
Easting (X) 1277626 / Northing (Y) 306197	System Datum: WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
415	0							SM	Brown silty fine to coarse sand with gravel (weathered till)			
410	5		12	75		1		SP-SM	Gray fine to coarse sand with silt and gravel (very dense, moist) (advance outwash)			
			13.5	85/10"		2						
405	10		2	50/6"		3						
400	15		12	50/5"		4						
395	20		5.5	50/4"		5 / MC				7		

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-04

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Drilled	Start 12/1/2020	End 12/1/2020	Total Depth (ft) 36.5	Logged By CAH / Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 409 Vertical Datum NAVD88	Hammer Data: Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment: Track-mounted drill rig EC95
Easting (X) 1277789 Northing (Y) 306183	System Datum: WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
409	0				AC	Approximately 1½ inches of asphalt concrete pavement			
					SM	Brown silty fine to coarse sand with gravel (fill material)			
					SM	Gray silty fine to coarse sand with gravel (dense, moist) (glacial till)			
405	5	11	49	1 / MC			8		
		4	50/6"	2		Becomes very dense			
400	10	4	50/5"*	3					*Blow counts potentially overstated Rock at 10 feet fractured in sampler
395		13	50/5"	4 / %F			8	36	
	15	8	50/6"	5					
		7	50/4"	6 / MC	SP-SM	Gray fine to coarse sand with silt (very dense, moist)	7		
390	20	7	50/5"	7	SM	Gray silty fine to coarse sand with gravel (very dense, moist)			
385					SP	Gray fine to medium sand (very dense, moist) (advanced outwash)			
	25	18	87	8					
380					ML	Gray sandy silt (hard, moist to wet)			
	30	11	50/4"	9					
375	35								

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-05

GEOENGINEERS

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		12	50/5"		10 %F			Grades to silt with sand, becomes moist to wet	20	83	Groundwater observed at approximately 35 feet below ground surface during drilling

Drilled Start 12/1/2020 End 12/1/2020	Total Depth (ft) 36.5	Logged By CAH Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) 403 Vertical Datum NAVD88		Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop		Drilling Equipment Track-mounted drill rig EC95
Easting (X) 1277955 Northing (Y) 306113		System Datum WA State Plane North NAD83 (feet)		See "Remarks" section for groundwater observed

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
403	0				SM	Brown silty fine to coarse sand with gravel (weathered till)			
	5	10.5	50/6"	1	SM	Gray silty fine to coarse sand with gravel (very dense, moist) (glacial till)			
		18	50/3"	2	SP	Gray fine to medium sand with gravel (very dense, moist)			
					SP-SM	Gray fine to coarse sand with silt and gravel (very dense, moist)			
	10	0	50/4"	3					
		0	50/2"	4	SM	Gray silty fine to coarse sand (very dense, moist)			
	15	10	50/3"	5					
		9.5	50/5"	6					
	20	13	79	7	SP-SM	Gray fine to coarse sand with silt (very dense, moist) (advanced outwash)			
	25	7	50/6"	8		Grades to with gravel			Increase in moisture, generally coarse
					ML	Gray sandy silt (hard, moist to wet)			
	30	18	72	9 MC			22		Groundwater observed at approximately 30 feet below ground surface during drilling
	35								

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-06

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	77		10 / SA			Becomes wet			Water bearing at 35 feet

Drilled	Start 12/1/2020	End 12/1/2020	Total Depth (ft) 36.5	Logged By CAH / Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	400 / NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Track-mounted drill rig EC95
Easting (X) / Northing (Y)	1278047 / 306163	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
400	0							SM	Gray silty fine to coarse sand with gravel (dense, moist) (weathered till)			
395	5		4	48*		1						Gravel piece in sampler tip; *blow counts potentially overstated
			13.5	80/11"		2 / MC		SM	Gray silty fine to coarse sand with gravel (very dense, moist) (glacial till)	9		
390												Driller noted gravel
	10		5	50/6"		3		GP-GM	Gravel with silt and sand (very dense, moist)			
			8	50/4"		4		SM	Gray silty fine to coarse sand with gravel (very dense, moist)			
385	15		7	50/6"		5 / %F				6	23	
			8	50/2"		6			Grades to fine to medium sand with occasional gravel			Gradation change
380	20		6	50/5"		7			Grades to fine to coarse sand with gravel			
375								ML	Gray sandy silt (hard, moist to wet) (advanced outwash)			
	25		17	80		8 / MC				19		
												Smooth drilling action
370	30		16	83		9 / SA				20	60	Groundwater observed at approximately 30 feet below ground surface during drilling
365	35								Becomes dark gray			

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-07

GEOENGINEERS

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	85		10		SP	Dark gray fine to medium sand (very dense, moist to wet)			

Log of Boring GEI-07 (continued)

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	12/1/2020	12/1/2020	36.5	CAH	CL	Boretec 1, Inc.	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	406 / NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Track-mounted drill rig EC95
Easting (X) / Northing (Y)	1277996 / 306246	System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
405	0						TS	Vegetation, topsoil and gravel			
							SM	Brown silty fine to coarse sand with gravel (dense, moist) (weathered till)			
400	5	X	14.5	47		1		Grades to with occasional gravel			
		X	16	78		2	SM	Gray silty fine to coarse sand with gravel (very dense, moist) (glacial till)			
395	10	X	16	85/10"		3		Grades to fine to medium sand with occasional gravel			
		X	6	50/5"		4 / %F		Grades to fine to coarse sand	29	22	
390	15	X	5.5	50/4"		5		Grades to with gravel			
		X	0			6					
385	20	X	6	50/6"		7					
											Smoother drilling; chatter subsides
							SP	Gray fine to coarse sand (very dense, moist to wet) (advanced outwash)			
380	25	X	16	91/10"		8 / MC			15		
							ML	Gray sandy silt (hard, moist to wet)			
375	30	X	18	50/6"		9					Wet sampler
	35										

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-08

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Figure A-9
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
370	35		14	50/6"		10		SP	Gray fine sand (very dense, wet)			Lower 9 inches water bearing

Log of Boring GEI-08 (continued)

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Drilled	Start 12/2/2020	End 12/2/2020	Total Depth (ft) 41.5	Logged By CAH Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) 409 Vertical Datum NAVD88	Hammer Data: Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment: Track-mounted drill rig EC95
Easting (X) 1277989 Northing (Y) 306382	System Datum: WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0						TS	Brown silty fine to coarse sand with organic matter (roots) (topsoil)			
405							SM	Light gray silty fine to coarse sand with gravel (very dense, moist) (weathered till)			
	5	X	10.3	58		1					
400		X	8.5	38		2 %F			7	33	
	10	X	8	34		3					
395		X	9.5	50/3"		4	SM	Gray silty fine to coarse sand with gravel (very dense, moist) (glacial till)			
	15	X	6	50/4"		5 MC			7		
390		X	4.5	50/3"		6	SP-SM	Gray fine to coarse sand with silt (very dense, moist)			
							SM	Gray silty fine to medium sand (very dense, moist)			
	20	X	13.5	50/4"		7					
385							SP-SM	Gray fine to coarse sand with silt and gravel (very dense, moist) (advance outwash)			
	25	X	4.5	50/6"		8 MC			3		
380	30	X	15.5	27		9					
							ML	Gray sandy silt (hard, wet)			Water bearing at 31½ feet Driller noted denser at 32 feet
375		X	16	50/6"		10	SP-SM	Gray fine to coarse sand with silt and gravel (very dense, moist to wet)			
	35										

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-09

GEOENGINEERS

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Figure A-10
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		15.5	50/5"		11						
								GP	Gravel with silt and sand (very dense, moist to wet)			
								ML	Gray sandy silt (hard, moist to wet)			
370												
	40		16	50/6"		12						

Drilled	Start 12/2/2020	End 12/2/2020	Total Depth (ft) 21.5	Logged By CAH / Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	377 / NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Track-mounted drill rig EC95
Easting (X) / Northing (Y)	1278168 / 306285	System Datum	WA State Plane North NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				SM	Brown silty sand with organic matter (roots and wood) and gravel (fill material)			
375					SM	Brown silty sand with gravel (very dense, moist) (weathered till)			
					GM	Silty gravel with sand (very dense, moist) (glacial till)			
	5	6	73	1					
370		7.5	50/1"	2					Driller notes feels like concrete but able to advance auger
	10	14	50/6"	3A / 3B / %F	ML	Gray sandy silt (hard, moist) (advanced outwash)	11	63	Drilling action smooths out at 10 feet. Gravel grades out 10½ to 11½ feet
365		11.5	50/6"	4					
	15	16	50/2"	5					
360		6	50/5"	6 / MC	SM	Gray silty fine to medium sand with gravel (very dense, moist)	8		
	20	10	50/5"	7					

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-10

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00

Drilled	Start 12/12/2020	End 12/12/2020	Total Depth (ft) 21.5	Logged By CAH / Checked By CL	Driller Boretec 1, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	372 / NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Track-mounted drill rig EC95
Easting (X) / Northing (Y)	1278259 / 306225	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

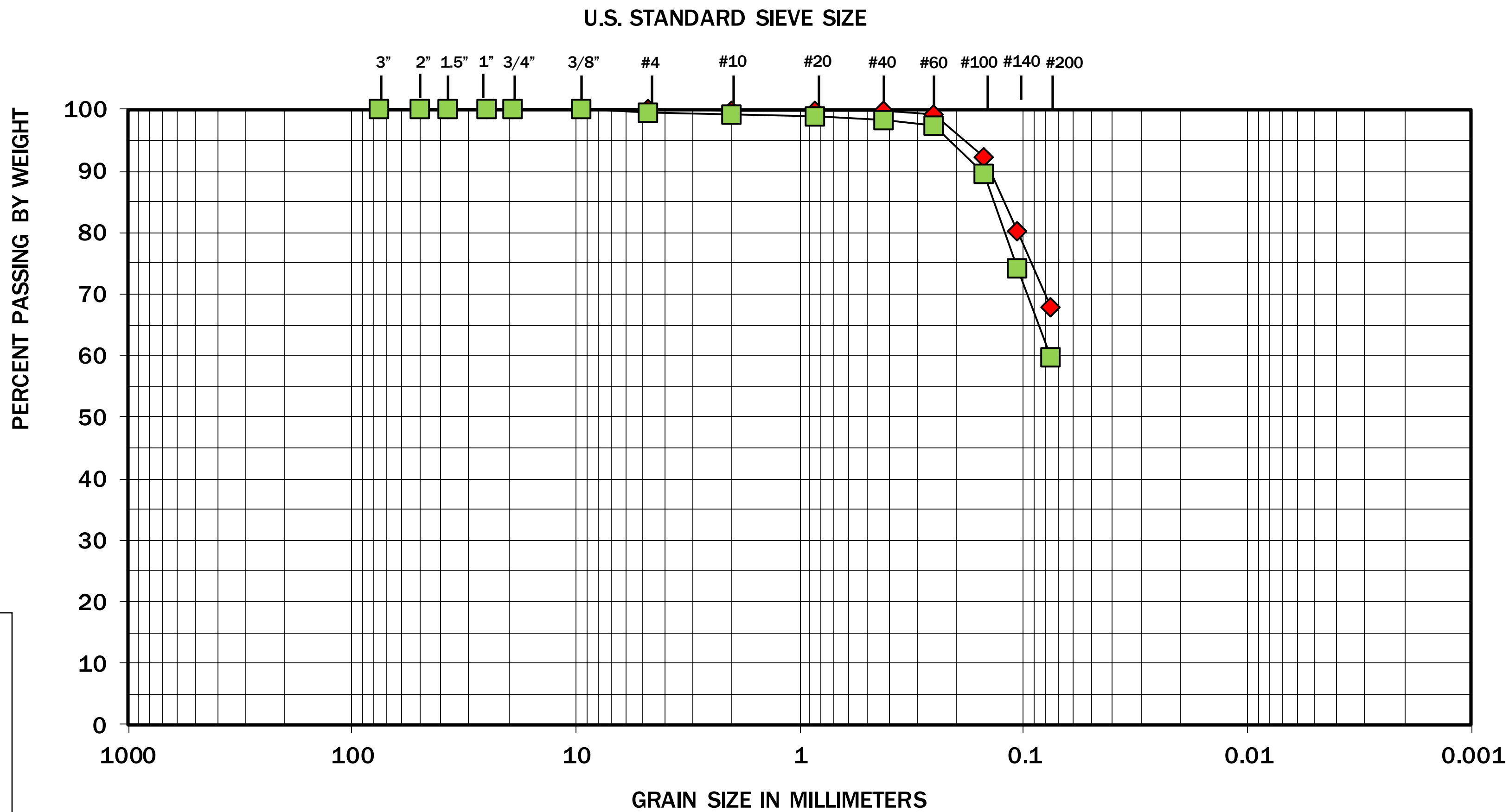
Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SP-SM	Brown fine to coarse sand with silt and gravel (fill material)			
370								SM	Brown silty fine to coarse sand with gravel (dense, moist to wet) (weathered till)			
	5		12	45		1						
365			18	25		2		SP-SM	Brown fine to coarse sand with silt (medium dense, moist to wet) (glacial till)			Groundwater observed at approximately 8 feet below ground surface during drilling
	10		18	25		3 / MC		SM	Brown silty fine to coarse sand with gravel (medium dense, moist to wet)	11		
360			5	50/5"		4		SM	Gray silty fine to coarse sand with gravel (very dense, moist)			
	15		10	50/5"		5 / %F		ML	Gray sandy silt (hard, moist to wet) (advanced outwash)	13	55	
355			4	50/4"		6		SM	Gray silty fine to coarse sand with occasional gravel (very dense, moist to wet)			
	20		4	50/4"		7						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Smartphone/Tablet (NA). Vertical approximated based on Topographic Survey.

Log of Boring GEI-11

GeoEngineers

Project: Lynnwood Auto Dealership
Project Location: Lynnwood, Washington
Project Number: 24789-001-00



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	GEI-02	35	22	Sandy silt (ML)
■	GEI-07	30	20	Sandy silt (ML)

The grain size analysis results were obtained in general accordance with ASTM C 136. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

AASHTO ACCREDITED

GeoEngineers

Lynwood Auto Dealership
Lynwood, WA

Sieve Analysis Results

Figure A-13

APPENDIX B
Ground Anchor Load Testing Program

APPENDIX B
GROUND ANCHOR LOAD TESTING PROGRAM

Ground Anchor Load Testing

The locations of the load tests shall be approved by the Engineer and shall be representative of the field conditions. Load tests shall not be performed until the tieback grout and shotcrete wall facing, where present, have attained at least 50 percent of the specified 28-day compressive strengths.

Where temporary casing of the unbonded length of test tiebacks is provided, the casing shall be installed to prevent interaction between the bonded length of the nail/tieback and the casing/testing apparatus.

The testing equipment shall include two dial gauges accurate to 0.001 inch, a dial gauge support, a calibrated jack and pressure gauge, a pump and the load test reaction frame. The dial gauge should be aligned within 5 degrees of the longitudinal tieback axis and shall be supported independently from the load frame/jack and the shoring wall. The hydraulic jack, pressure gauge and pump shall be used to apply and measure the test loads.

The jack and pressure gauge shall be calibrated by an independent testing laboratory as a unit. The pressure gauge shall be graduated in 100 pounds per square inch (psi) increments or less and shall have a range not exceeding twice the anticipated maximum pressure during testing unless approved by the Engineer. The ram travel of the jack shall be sufficient to enable the test to be performed without repositioning the jack.

The jack shall be supported independently and centered over the nail/tieback so that the nail/tieback does not carry the weight of the jack. The jack, bearing plates and stressing anchorage shall be aligned with the nail/tieback. The initial position of the jack shall be such that repositioning of the jack is not necessary during the load test.

The reaction frame should be designed/sized such that excessive deflection of the test apparatus does not occur and that the testing apparatus does not need to be repositioned during the load test. If the reaction frame bears directly on the shoring wall facing, the reaction frame should be designed so as not to damage the facing.

Verification Tests

Prior to production tieback installation, at least two tiebacks for each soil type shall be tested to validate the design pullout value. All test tiebacks shall be installed by the same methods, personnel, material and equipment as the production anchors. Changes in methods, personnel, material or equipment may require additional verification testing as determined by the Engineer. At least two successful verification tests shall be performed for each installation method and each soil type. The tiebacks used for the verification tests may be used as production tiebacks if approved by the Engineer.

Tieback design test loads should be the design load specified on the shoring drawings. Verification test tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL	1 minute
1.5DL	60 minutes
1.75DL	1 minute
1.0DL	10 minutes

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Nail/tieback deflections during the 1.5DL test load shall be recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes.

Proof Tests

Proof tests should be completed on each production tieback.

The allowable tieback load should not exceed 80 percent of the steel ultimate strength.

Tieback design test loads should be the design load specified on the shoring drawings. Proof test tiebacks should be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL	1 minute
1.33DL	10 minutes

The alignment load should be the minimum load required to align the testing apparatus and should exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied.

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of Holman Automotive Group, Inc. c/o 3J Consulting and other project team members for the Project specifically identified in the report. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the Lynnwood Auto Dealership project in Lynnwood, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;
- Elevation, configuration, location, orientation or weight of the proposed structure;
- Composition of the design team; or
- Project ownership.

1 Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns Are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions.

If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized.

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