Preliminary Geotechnical Engineering Draft Report

Mercer Island Rowhouse
3003 77th Avenue SE
Mercer Island, Washington

for
Ryan Companies US, Inc.

May 18, 2021

GeoEngineers

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Table of Contents

LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of GeoEngineers, Inc.'s (GeoEngineers) geotechnical engineering services for the proposed Mercer Island Rowhouse project located at 3003 77th Avenue SE in Mercer Island, Washington. The site is shown relative to surrounding physical features in Figure 1 Vicinity Map, and Figure 2 Site Plan.

The purpose of this report is to provide preliminary geotechnical engineering conclusions and recommendations for the design and construction of the planned development. The site consists of one King County Parcel (parcel number 531510-1015) and covers approximately 5.88 acres. The proposed Mercer Rowhouse development will only utilize the northeast corner of the site. GeoEngineers' services have been completed in accordance with our consultant agreement with Ryan Companies US, Inc. executed on December 1, 2019 and approved addenda.

2.0 SCOPE OF SERVICES

GeoEngineers' scope of services includes:

- Review available reports and studies for the subject property and surrounding area available from our files.
- Drilling and sampling five borings using hollow-stem auger drilling equipment, advancing four cone penetration tests (CPTs), and installing monitoring wells in two of the five borings.
- Completing geotechnical laboratory testing on selected samples obtained from the borings. The laboratory testing included moisture content, sieve analysis, percent passing the U.S. No. 200 sieve, and plasticity index (Atterberg limits) tests.
- Providing recommendations for seismic design in accordance with the International Building Code (IBC).
- Providing foundation, temporary shoring, slab-on-grade and permanent below-grade wall recommendations.
- Providing recommendations for temporary dewatering and permanent below-grade drainage and groundwater seepage estimates.
- Providing consultation to the project team, as needed.
- Preparing this report.

3.0 PROJECT DESCRIPTION

GeoEngineers understands that Ryan Companies US, Inc. is interested in developing a mixed-use building with one below-grade parking level. The site is currently occupied by an office building and adjacent multi-level parking structure. The project will consist of renovating the office building, demolishing a portion of the existing parking structure and constructing an apartment building. The new apartment building is anticipated to have three wood framed levels. Excavation depths for the planned development are anticipated to range up to 10 feet below existing site grades.

Temporary shoring is anticipated to be required to complete the planned excavation. Based on explorations completed on the site, it is anticipated that the planned building can be supported on shallow foundations. Soft soil conditions were observed in the southeast portion of the site to depths of up to 13 feet deep. It may be necessary to remove and replace some non-bearing soils in the southeastern portion of the building to achieve adequate bearing, implement ground improvement, or deep foundations, a combination of these foundation options.

4.0 PREVIOUS SITE EVALUATIONS

In addition to the explorations completed as part of this evaluation, the logs of selected explorations from previous site evaluations in the project vicinity were reviewed and are presented in Appendix C, Boring Logs from Previous Studies. The approximate locations of these explorations are also shown on Figure 2.

5.0 SITE CONDITIONS

5.1. Surface Conditions

The Mercer Island Rowhouse project is bounded by SE 29th Street to the north, 77th Avenue SE to the east, Mercerdale Park to the south and private properties to the west. The site is currently occupied by a five-story office building, parking structure and surface parking lot. Existing site grades slope moderately down from west to east from approximately Elevation 120 to 85 feet.

Buried utilities consisting of sanitary sewer, storm drain, gas, water, electric and telecommunications fiber are anticipated in the right-of-way adjacent to the site.

5.2. Subsurface Soil Conditions

GeoEngineers' understanding of subsurface conditions is based on the review of existing geotechnical information and the results of four CPTs and five borings drilled as part of this study. The approximate locations of the previous and recent explorations are presented in Figure 2.

The soils encountered in the site vicinity consist of shallow fill and recent deposits overlying glacially consolidated soils.

The fill and recent deposits generally consisted of loose to dense silty sand with variable gravel content and soft to medium stiff silt deposits with variable sand content. Fill and recent deposits on site ranged up to 17 feet thick, with the thickest deposits in the southwestern portion of the site.

The glacially consolidated soils were encountered below the fill and recent deposits and extended to the depths explored. The glacially consolidated soils consist of stiff to hard clay and silt, and medium dense to very dense silty sand with gravel and occasional cobbles. Glacially consolidated soils were encountered at shallow depths in the north portion of the site and below the fill and recent deposits in the south portion of the site.

Although not encountered in explorations at this site, occasional cobbles and boulders are typical of glacially consolidated soils and may be present at the site and have been encountered in nearby construction projects.

5.3. Groundwater Conditions

The depth to groundwater was measured in the monitoring wells completed for this study (GEI-1 and GEI-4). Automated dataloggers were installed in these monitoring wells to observe the variability in groundwater levels seasonally and after significant rainfall events.

The following table provides a summary of the monitoring well measurements at the site.

Well ID	Ground Surface Elevation (feet)	Top of Casing Elevation (feet)	Approximate Well Screen Elevation (feet)	Measured Groundwater Elevation (feet)
GEI-1	85.0	84.74	45 to 55	78.64 (04/27/21)
GEI-4	84.0	83.72	54 to 64	78.77 (04/27/21)

Groundwater at the site is present in a deep, confined aquifer and in isolated perched groundwater layers. The groundwater levels present in GEI-1 and GEI-4 are interpreted to be consistent with the deep confined aquifer.

The water bearing soil layers (sand layers) were encountered below the planned base of excavation elevation. Based on groundwater levels measured in the monitoring wells screened in the deep confined aquifer, the phreatic surface will be above the bottom of excavation elevation, and close to the lowest finished floor elevation. Based on observations during drilling, the water bearing layers are below thick silt layers. As a result, groundwater encountered in the excavation will be associated with isolated perched groundwater layers and seepage flows into the excavation and into the foundation drainage system are anticipated to be small, on the order of 15 gallons per minute (gpm) or less. No active dewatering is anticipated for the majority of the excavation and groundwater control is anticipated to be completed using sumps and pumps. If deeper excavations are required (for instance, elevator pits), then depressurization of the confined groundwater layer with a depressurization well may be necessary. This condition should be reviewed once the foundation plan has been completed.

During shoring construction, observations should be made during soldier pile/vertical element drilling to confirm that groundwater conditions are consistent with the observations described above and to keep a record of water bearing zones, if encountered.

Groundwater levels are anticipated to vary as a function of location, precipitation, season, and other factors. Additional groundwater measurements will be taken leading up to construction to assess seasonal variations in groundwater elevations.

6.0 CONCLUSIONS AND RECOMMENDATIONS

A summary of the geotechnical considerations is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- The site is anticipated to be designated as Site Class D per the 2018 IBC.

- Perched groundwater is anticipated to be encountered during excavation. Dewatering by means of sumps and pumps is anticipated during construction. Depressurization may be required for deeper excavations and should be further assessed once the foundation plan has been finalized.
- Excavation support can be provided by soldier pile and tieback walls. Soldier pile and tieback walls will be required to be temporary because the ground anchors will extend into the public right-of-way. The permanent below-grade building walls will be required to resist the permanent lateral earth pressures. Additionally, the City of Mercer Island may require that ground anchors extending into the public right-of-way be de-stressed once the temporary shoring is no longer required. The permanent below-grade building walls should be designed and constructed to facilitate de-stressing of temporary ground anchors, where present.
- Current architectural concepts show foundations will bear approximately 5 to 10 feet below street grades. Due to the variable soils present at foundation subgrade elevation, shallow foundations are recommended at the northern portion of the project site. Either shallow foundations bearing on improved ground or deep foundations are recommended for the southern portion of the project site.
- GeoEngineers recommends that the project team review the foundation subgrade elevations relative to the bearing soil elevations presented on Figure 6, Estimated Elevation of Top of Bearing Soils, to select the preferred foundation support option in the southern portion of the site. Once the preferred foundation support option has been selected (ground improvement or deep foundations) and the procurement method selected (design-build or design-bid-build), then GeoEngineers will update this geotechnical report to include ground improvement recommendations/performance criteria and/or deep foundation recommendations.
- Shallow foundations may be used where undisturbed glacially consolidated soils are located at foundation subgrade elevation. For shallow foundations bearing directly on undisturbed dense to very dense glacially consolidated soils, an allowable soil bearing pressure of 6 kips per square foot (ksf) may be used.
- Ground improvement should be implemented to provide uniform foundation bearing across the variable soil conditions at the foundation elevation, and to limit static and seismic settlement to acceptable levels. Several options for ground improvement are available, including removal and replacement, rigid inclusions, and rammed aggregate piers. Stone columns may not be preferred due to vibration and soil disturbance related to compressed air.
- A variety of deep foundation options are available for the southern portion of the site. Given the relatively light building loads, suitable pile options may include driven pin piles or augercast piles.
- Conventional slabs-on-grade are considered appropriate for this site and should be underlain by a 6-inch-thick layer of clean crushed rock (for example, City of Seattle Mineral Aggregate Type 22). The underslab drainage system is anticipated to consist of a perimeter foundation drain and one or two longitudinal drains.

Our specific geotechnical recommendations are presented in the following sections of this report.

6.1. Earthquake Engineering

6.1.1. Liquefaction

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength. In general, soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands that are below the water table.

Groundwater levels at the site are within the cohesive recent deposit layers or dense to very dense/very stiff to hard glacially consolidated soils. Our analysis indicates that the soils that underlie the proposed building area have a low risk of liquefying because of the density and gradation of these soils.

6.1.2. Other Seismic Hazards

Due to the location of the site and the site's topography, the risk of adverse impacts resulting from seismically induced slope instability, differential settlement, or surface displacement due to faulting is considered to be low.

6.1.3. 2018 IBC Seismic Design Information

The 2018 IBC references the 2016 version of *Minimum Design Loads for Buildings and Other Structures* (American Society of Civil Engineers [ASCE] 7-16). Per ASCE 7-16 Section 11.4.8, a ground motion hazard analysis or site-specific response analysis is required to determine design ground motions for structures on Site Class D sites with S_1 greater than or equal to 0.2 g (where g represents gravitational acceleration). For this project, the site is classified as Site Class D with an S_1 value of 0.49g; therefore, this provision applies. Alternatively, the parameters listed in the table below may be used to determine the design ground motions provided Exception 2 of Section 11.4.8 of ASCE 7-16 is used. Using this exception, the seismic response coefficient (C_s) is determined by Equation (Eq.) (12.8-2) for values of $T \leq 1.15T_S$, and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$, where T represents the fundamental period of the structure and T_S=0.63 seconds (sec).

If requested, we can complete a site-specific seismic response analysis, which could provide reduced seismic demands from the parameters in the table below and the requirements of ASCE 7-16 Section 11.4.8 Exception 2 depending on the building configuration and site-specific subsurface conditions.

2018 IBC Parameter[1]	Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Period, S_s (g)	1.40
Mapped MCE_R Spectral Response Acceleration at 1-second period, S_1 (g)	0.49
Short Period Site Coefficient, F_a	1.00
Long Period Site Coefficient, F_v	1.81[2]
Design Spectral Acceleration at 0.2-second period, S_{DS} (g)	0.94

2018 IBC Parameter[1]	Value
Design Spectral Acceleration at 1.0-second period, S_{D1} (g)	0.59[2]
T_S (seconds)	0.63

Notes:

[1] Parameters developed based on latitude 47.5823 and longitude -122.2457 using the Applied Technology Council (ATC) Hazards online tool (https://hazards.atcouncil.org/).

[2] These values are only valid if the structural engineer utilizes Exception 2 of Section 11.4.8 (ASCE 7-16).

6.2. Temporary Dewatering

The regional groundwater table in the site vicinity is located below the base of the planned excavation. Perched groundwater is expected to be present at higher elevations. Active dewatering is not anticipated to be necessary to complete the excavation. Localized sumps and pumps should be anticipated for temporary dewatering and removal of surface water from precipitation.

Groundwater is present at depth in a confined aquifer with phreatic water levels near the planned finished floor elevation. Depressurization may be required for deeper excavations and should be further assessed once the foundation plan has been finalized.

For planning purposes, groundwater flow rates for temporary dewatering of up to 15 gpm can be assumed for the planned excavation. Surface water from rainfall will likely contribute significantly to the volume of water that needs to be removed from the excavation during construction and will vary as a function of season and precipitation.

6.3. Excavation Support

We understand that the planned building will have one partial below-grade level and that the excavation may extend up to 10 feet below site grades. For preliminary design, temporary shoring should be assumed to be consist of cantilever soldier piles or soldier piles and tiebacks.

Ground anchors should be designed to maintain an acceptable clearance from buried utilities in the right-of-way. The ground anchors will be required to be temporary because the ground anchors will extend into the City of Mercer Island right-of-way. The following section highlights specific considerations for each shoring wall.

We provide recommendations for conventional soldier pile and tieback walls below.

6.3.1. West Shoring Wall

The west shoring wall will be located adjacent to a portion of the existing parking garage that will remain in place.

6.3.2. South Shoring Wall

The south shoring wall will be located adjacent to the existing office building on the site.

6.3.3. North and East Shoring Walls

The north and east shoring walls will need to coordinate with existing utilities located within the right-of-way.

6.3.4. Excavation Considerations

Site soils may be excavated with conventional excavation equipment, such as trackhoes or dozers. It may be necessary to rip the glacially consolidated soils locally to facilitate excavation. The contractor should be prepared for occasional cobbles and boulders in the site soils. Likewise, surficial fill may contain foundation elements and/or utilities from previous site development, debris, rubble and/or cobbles and boulders. We recommend that project specifications identify procedures for measurement and payment of work associated with obstructions.

6.3.5. Soldier Pile and Tieback Walls

Soldier pile walls consist of steel beams that are concreted into drilled vertical holes located along the wall alignment, typically about 8 feet on center. After excavation to specified elevations, tiebacks are installed, if necessary. Once the tiebacks are installed, the pullout capacity of each tieback is tested, and the tieback is 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 tremied 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.

6.3.5.1. Soldier Piles

We recommend that soldier pile walls be designed using the earth pressure diagram presented in Figure 3, Earth Pressure Diagrams — Temporary Soldier Pile & Tieback Wall. The earth pressures presented in Figure 3 are for cantilever soldier pile walls or soldier pile walls with a single level of tiebacks, and the pressures represent the estimated loads that will be applied to the wall system for various wall heights.

Earth pressures presented in Figure 3 include the loading from traffic surcharge. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be applied to the shoring system as recommended in Figure 4, Recommended Surcharge Pressure. No seismic pressures have been included in Figure 3 because it is assumed that the shoring will be temporary.

We recommend that 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." The axial capacity of the soldier piles must resist the downward component of the anchor loads and other vertical loads, as appropriate. We recommend using an allowable end bearing value of 30 ksf for piles supported on glacially consolidated soils. The allowable end bearing value should be applied to the base area of the drilled hole into which the soldier pile is concreted. This value includes a factor of safety of about 2.5. The allowable end bearing value assumes that the shaft bottom is cleaned out immediately prior to concrete placement. If necessary, an allowable pile skin friction of 1.5 ksf may be used on the embedded portion of the soldier piles to resist the vertical loads.

6.3.5.2. Lagging

The following table presents GeoEngineers' recommended lagging thicknesses (roughcut) as a function of soldier pile clear span and depth.

Depth (feet)	Recommended Lagging Thickness (roughcut) 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 perched groundwater or 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 backfilled as soon as practicable. The voids should be backfilled immediately or within a single shift, depending on the selected method of backfill. Placement of this material will help reduce the risk of voids developing behind the wall and damage to existing improvements behind the wall.

Controlled density fill (CDF) is a suitable option for backfill behind the wall, as it will reduce the volume of voids. Full-depth CDF backfill is recommended for the walls located near adjacent buildings, for improved deflection control.

6.3.5.3. Tiebacks

Tieback anchors can be used for wall heights where cantilever soldier pile walls are not cost effective. Tieback anchors should extend far enough behind the wall to develop anchorage beyond the "no-load" zone (defined in Figure 3) and within a stable soil mass. The anchors should be inclined downward at 15 to 25 degrees below the horizontal. Corrosion protection will not be required for the temporary tiebacks.

Centralizers should be used to keep the tieback in the center of the hole during grouting, and 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.

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. Drilled tieback holes should be grouted/filled promptly to reduce potential ground loss.

Tieback anchors should develop anchorage in the glacially consolidated soils. We recommend that the spacing between tiebacks be at least three times the diameter of the anchor hole to minimize group interaction. We recommend a design load transfer value between the anchor and soil of 2.5 kips per foot for glacially consolidated soils and 1.5 kips per foot for fill deposits.

Tieback anchors should be verification- and proof-tested to confirm that the tiebacks have adequate pullout capacity. The pullout resistance of tiebacks should be designed using a factor of safety of 2. The pullout resistance should be verified by completing at least two successful verification tests in each soil type and a minimum of 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 D.

Tieback layout and inclination should be checked to confirm that the tiebacks do not interfere with adjacent buried utilities. The City of Mercer Island minimum clearances between ground anchors and existing utilities should be maintained.

6.3.5.4. Drainage

Drainage for soldier pile and lagging walls is achieved through seepage through the timber lagging. 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 provided for permanent below-grade walls as described below in the "Below-Grade Walls" section of this report.

6.3.5.5. Construction Considerations

Shoring construction shall be completed by a qualified shoring contractor. A shoring contractor is qualified if they have successfully completed at least 10 projects of similar size and complexity in the Seattle/ Bellevue area during the previous five years. Interested shoring contractors should prepare a submittal documenting their qualifications, unless this requirement is waived by GeoEngineers. The shoring contractor's superintendent shall have a minimum of three years' experience supervising soil nail/soldier pile and tieback shoring construction and the drill operators and on-site supervisors shall have a minimum of three years' experience installing soil nails/soldier piles and tiebacks. The personnel experience shall be included in the qualification's submittal.

Temporary casing or drilling fluid will be required to install the soldier piles and casing will 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
- Perched groundwater is present.

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

6.3.6. Shoring Wall Performance

Temporary shoring walls typically move on the order of 0.1 to 0.2 percent of H, where H is the vertical distance between the existing ground surface and the base of excavation.

Deflections and settlements are usually highest at the excavation face and decrease to negligible amounts beyond a distance behind the wall equal to the height of the excavation. Localized deflections may exceed the above estimates and may reflect local variations in soil conditions (such as around side sewers) or may be the result of the workmanship of the constructed shoring wall. Given that some movement is expected, existing improvements located adjacent to the temporary shoring system will also experience movement. The deformations discussed above are not likely to cause structural damage to structurally sound existing improvements; however, some cosmetic damage should be expected (for instance, cracks in drywall finishes; widening of existing cracks; minor cracking of slabs-on-grade/hardscapes; cracking of sidewalks, curbs/gutter, and pavements/pavement panels; etc.). For this reason, it is important to complete pre-construction survey and photo documentation of existing buildings and nearby improvements prior to shoring construction. Refer to Appendix D for more detailed recommendations for shoring monitoring and preconstruction surveying.

6.4. Foundation Support

Current architectural concepts show foundations will bear approximately 5 to 10 feet below street grades. The soils at the anticipated foundation elevation are expected to vary across the site. Soils at the foundation elevation in the northern portion of the site consist of competent glacially consolidated soils and shallow foundations are considered to be the preferred foundation option. Recent deposits that are not considered suitable for shallow foundations are present at the foundation subgrade elevation in the southern portion of the site. Ground improvement consisting of overexcavation and replacement with structural fill or CDF is considered a cost effective and feasible option for subgrades where the depth to bearing soils is relatively shallow (on the order of 4 feet below the foundation subgrade elevation). Where the depth to bearing soils is more than approximately 4 feet below the foundation subgrade elevation, ground improvement by means of rigid inclusions, or rammed aggregate piers may be used. Alternatively, deep foundations such as driven pin piles or augercast piles can be used.

GeoEngineers recommends that the project team review the foundation subgrade elevations relative to the bearing soil elevations presented on Figure 6, Estimated Elevation of Top of Bearing Soils to select the preferred foundation support option in the southern portion of the site. Once the preferred foundation support option has been selected (ground improvement or deep foundations) and the procurement method selected (design-build or design-bid-build), then GeoEngineers will update this geotechnical report to include ground improvement recommendations/performance criteria and/or deep foundation recommendations. We provide shallow foundation recommendations below for the northern portion of the site and where ground improvement is planned in the southern portion of the site.

6.4.1. Allowable Bearing Pressure

For foundations bearing directly on competent glacially consolidated soils or structural fill/CDF extending down to undisturbed glacially consolidated soils, a preliminary allowable bearing pressure of 6 ksf can be assumed. Where ground improvement consisting of rigid inclusions, or rammed aggregate piers is used, an allowable bearing pressure or 4 to 6 ksf can be used for preliminary design. The allowable soil bearing pressure applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. The allowable soil bearing pressures are net values.

6.4.2. Modulus of Subgrade Reaction

For mat foundations designed as a beam on an elastic foundation, a static modulus of subgrade reaction of 42 pounds per cubic inch (pci) may be used for structural mat foundation bearing on glacially consolidated soils. GeoEngineers should review the structural engineer’s estimated deformation and applied bearing pressures to confirm that this subgrade modulus is appropriate and is consistent with our foundation design.

6.4.3. Settlement

Provided that all loose soil is removed and that the subgrade is prepared as recommended under “Construction Considerations” below, we estimate that the total settlement of the core mat(s) will be about 1 inch or less. The settlements will occur rapidly, essentially as loads are applied. Differential settlements across the mat foundations could be half of the total settlement. Note that smaller settlements will result from lower applied loads.

6.4.4. Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the shallow foundations. For shallow foundations supported on native soils, CDF, or structural fill, the allowable frictional 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 400 pounds per cubic foot (pcf) (triangular distribution). These values are appropriate for foundation elements that are poured directly against undisturbed glacially consolidated soils or surrounded by structural fill.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

6.4.5. Construction Considerations

We recommend that 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 as anticipated.

During wet weather conditions or when wet weather is forecasted, the foundation subgrades are recommended to be protected with a rat slab consisting of 2 to 4 inches of lean or structural concrete in order to prevent deterioration of the subgrade during mat foundation steel and concrete placement.

If soft areas are present at the footing subgrade elevation, the soft areas should be removed and replaced with lean concrete or structural concrete at the direction of GeoEngineers.

We recommend that 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 is required for foundation reinforcement. Leaving subgrade high will help reduce damage to the subgrade resulting from construction traffic or other activities on site.

6.5. Slab-on-Grade Floors

6.5.1. Subgrade Preparation

The exposed subgrade should be evaluated after site grading is complete. Probing should be used to evaluate the subgrade. The exposed soil should be firm and unyielding, and without significant groundwater. Disturbed areas should be recompacted if possible or removed and replaced with compacted structural fill.

6.5.2. Design Parameters

Conventional slabs may be supported on-grade, provided the subgrade soils are prepared as recommended in the “Subgrade Preparation” section above. We recommend that the slab be founded on the native soils or structural fill extending down to the native soils. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 150 pci may be used for subgrade soils prepared as recommended.

We recommend that the slab-on-grade floors be underlain by a 6-inch-thick capillary break consisting of material meeting the requirements of Mineral Aggregate Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14.

Provided that loose soil is removed and the subgrade is prepared as recommended, we estimate that slabs-on-grade will not settle appreciably.

6.5.3. Below-Slab Drainage

We expect the static groundwater level to be located below the slab-on-grade level for the proposed building, and perched groundwater may be present above the slab subgrade elevation. We recommend installing an underslab drainage system to remove water from below the slab-on-grade. The underslab drainage system should include an interior perimeter drain and one to two longitudinal drains. The civil engineer should develop a conceptual foundation drainage plan for GeoEngineers to review. The drains should consist of perforated Schedule 40 polyvinyl chloride (PVC) pipes with a minimum diameter of 4 inches placed in a trench at least 12 inches deep. The top of the underslab drainage system trenches should coincide with the base of the capillary break layer. The underslab drainage system pipes should have adequate slope to allow positive drainage to the sump/gravity drain.

The drainage pipe should be perforated. Perforated pipe should have two rows of ½-inch holes spaced 120 degrees apart and at 4 inches on center. The underslab drainage system trenches should be backfilled with Mineral Aggregate Type 22 or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.14, or an alternative approved by GeoEngineers. The Type 22 or Type 5 material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, Washington State Department of Transportation (WSDOT) Standard Specification 9-33. The underslab drainage system pipes 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. The flow rate for the planned excavation in the below-slab drainage and below-grade wall drainage systems is anticipated to be less than 15 gpm.

If no special waterproofing measures are taken, leaks and/or seepage may occur in localized areas of the below-grade portion of the building, even if the recommended wall drainage and below-slab drainage provisions are constructed. If leaks or seepage is undesirable, below-grade waterproofing should be specified. A vapor barrier should be used below slab-on-grade floors located in occupied portions of the building. Specification of the vapor barrier requires consideration of the performance expectations of the occupied space, the type of flooring planned and other factors, and is typically completed by other members of the project team.

6.6. Below-Grade Walls

6.6.1. Permanent Subsurface Walls

Permanent below-grade walls constructed adjacent to temporary shoring walls should be designed for the earth pressures presented in Figure 5. 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 4.

The soil pressures recommended above assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as described above in the "Excavation Support" section of this report, and tied to permanent drains to remove water to suitable discharge points.

6.6.2. Other Cast-in-Place Walls

Conventional cast-in-place walls may be necessary for retaining structures located on site. The lateral soil pressures acting on conventional cast-in-place 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 horizontal 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). For seismic loading conditions, a rectangular earth pressure equal to 7H 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 frictional 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 400 pcf (triangular distribution). The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

The above soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed below.

6.6.3. Drainage

Drainage behind the permanent below-grade walls is typically provided using prefabricated drainage board attached to the temporary shoring walls. Weep pipes that extend through the permanent below-grade wall should be installed around the perimeter of the building at the foundation elevation. The weep pipes should have a minimum diameter of 4 inches. The weep pipes should be connected to the interior perimeter underslab drain and routed to a sump.

The earth pressures for permanent below-grade walls assume that adequate drainage is provided behind the wall. Prefabricated vertical geocomposite drainage material, such as Aquadrain 15X, should be installed vertically to the face of the timber lagging. The vertical drainage material should extend a minimum of 2 feet below the planned weep pipe locations. The weep pipes that penetrate the basement wall should be connected to the vertical drainage material with a drain grate. For soldier pile shoring walls, the drainage material should be installed on the excavation side of the timber lagging, with the fabric adjacent to the timber lagging.

Full wall face coverage is recommended to minimize seepage and/or wet areas at the face of the permanent wall. Full wall face coverage should extend from 2 feet below the weep pipe elevation up to about 3 to 5 feet below site grades to reduce the potential for surface water to enter the wall drainage system. 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, the potential still exists for these conditions to occur. If this is a concern, waterproofing should be specified.

Positive drainage should be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14, with the exception that the percent passing the U.S. No. 200 sieve is to be less than 3 percent. A perforated drainpipe should be placed near the base of the retaining wall to provide drainage. The drainpipe should be surrounded by a minimum of 6 inches of Mineral Aggregate Type 22 (¾-inch crushed gravel) or Type 5 (1-inch washed gravel), City of Seattle Standard Specification 9-03.14, or an alternative approved by GeoEngineers. The Type 22 or Type 5 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.

6.7. Earthwork

6.7.1. Subgrade Preparation

Exposed subgrade in structure and hardscape areas should be evaluated after site excavation is complete. Foundation subgrades should be prepared as recommended in "Shallow Foundations" above. Where hardscape subgrade soils consist of disturbed soils, it will likely be necessary to remove and replace the disturbed soil with approved structural fill unless the soil can be adequately moisture-conditioned and compacted.

6.7.2. Structural Fill

Fill placed to support structures or foundations, placed behind retaining structures, for foundation drainage, and/or placed below pavements and sidewalks shall consist of structural fill as specified below:

- If structural fill is necessary beneath building foundations or building slabs, the fill should consist of Mineral Aggregate Type 2 or Type 17 (1¼-inch minus crushed rock or bank run gravel), City of Seattle Standard Specification 9-03.14, or CDF.
- Structural fill placed as capillary break material should meet the requirements of Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed behind retaining walls should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.14.
- Structural fill placed around perimeter footing drains, underslab drains and cast-in-place wall drains should meet the requirements of Mineral Aggregate Type 5 (1-inch washed gravel) or Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.14, with the exception that the percent fines be less than 3 percent.
- Structural fill placed within utility trenches and below pavement and sidewalk areas should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9 03.14.

- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.14.

6.7.2.1. On-site Soils

On-site soils are moisture-sensitive and have natural moisture contents higher than the anticipated optimum moisture content for compaction. As a result, on-site soils will likely require moisture conditioning 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 the project have specific gradation requirements, and the on-site soils do not meet these gradation requirements. Therefore, imported structural fill meeting the requirements described above should be used where structural fill is necessary.

It may be feasible to reuse on-site soils with the addition of cement treatment. If cement treatment is considered, GeoEngineers can work with the contractor to determine the soil/cement ratio and placement procedures.

6.7.2.2. Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition and 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 meet the following criteria:

- Structural fill placed in building areas (including around foundations and supporting slab-on-grade floors), pavement and sidewalk areas (including utility trench backfill) should be compacted to at least 95 percent of the maximum dry density (MDD) estimated in general accordance with ASTM International (ASTM) D 1557.
- Structural fill placed against retaining walls should be compacted to between 90 and 92 percent. Care should be taken when compacting fill against retaining 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 to verify compliance with compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

6.7.2.3. Weather Considerations

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 percent above the optimum moisture content, these soils become muddy and unstable, and equipment operation becomes 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 should be sloped so that surface water is directed away from the work area. 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.
- 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 unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

6.7.3. Temporary Slopes

Temporary slopes may be used around the site to facilitate early installation of shoring or in the transition between levels at the base of the excavation. We recommend that temporary slopes constructed in the fill be inclined at 1½H:1V (horizontal to vertical) and that temporary slopes in the glacially consolidated soils be inclined at 1H:1V. 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 5 feet 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; and
- The general condition of the slopes be observed periodically by the geotechnical engineer to confirm adequate stability.

Because the contractor has control of 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.

7.0 RECOMMENDED ADDITIONAL GEOTECHNICAL SERVICES

GeoEngineers will prepare a final geotechnical report that reflects the final design of the planned development and the project team’s selected foundation type (ground improvement/deep foundations) in the southern portion of the site where unsuitable soils are present at the foundation subgrade elevation.

During construction, GeoEngineers should observe the installation of the shoring system; review/collect shoring monitoring data; evaluate the suitability of the foundation subgrades; observe installation of subsurface drainage measures; evaluate structural backfill; observe the condition of 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 described in Appendix E, Report Limitations and Guidelines for Use.

8.0 LIMITATIONS

We have prepared this report for the exclusive use of Ryan Companies US, Inc. and their authorized agents for the Mercer Island Rowhouse project in Mercer Island, 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 E, Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

9.0 REFERENCES

ASCE (2016), "SEI/ASCE 7-16, Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers.

Cascade Testing Laboratory, Inc., 1977, "Foundation Investigation Proposed Albertson's Store #450, Southeast 29th Street and 77th Avenue Southeast, Mercer Island, Washington."

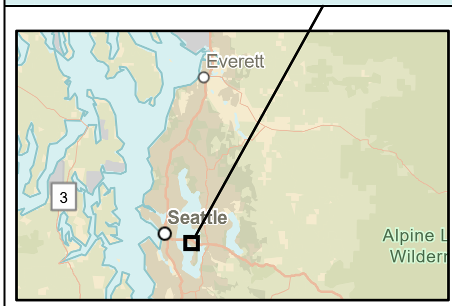
City of Seattle, 2020, "Standard Specifications for Road, Bridge and Municipal Construction."

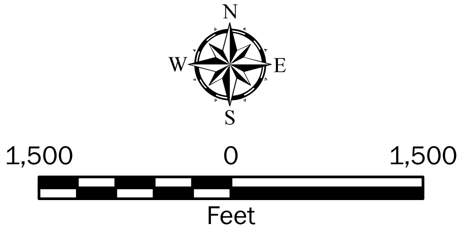
Earth Consultants, Inc., 1977, "Soil and Foundation Investigation, Proposed Apartment Complex, 76th Avenue S.E., Mercer Island, Washington."

International Code Council, 2018. "International Building Code."

United States Seismic Design Maps, United States Geological Survey - Earthquake Hazards Program, IBC 2015 (https://earthquake.usgs.gov/ws/designmaps/ibc-2015.html).

Washington State Department of Transportation, 2020, "Standard Specifications for Road, Bridge and Municipal Construction."





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: ESRI

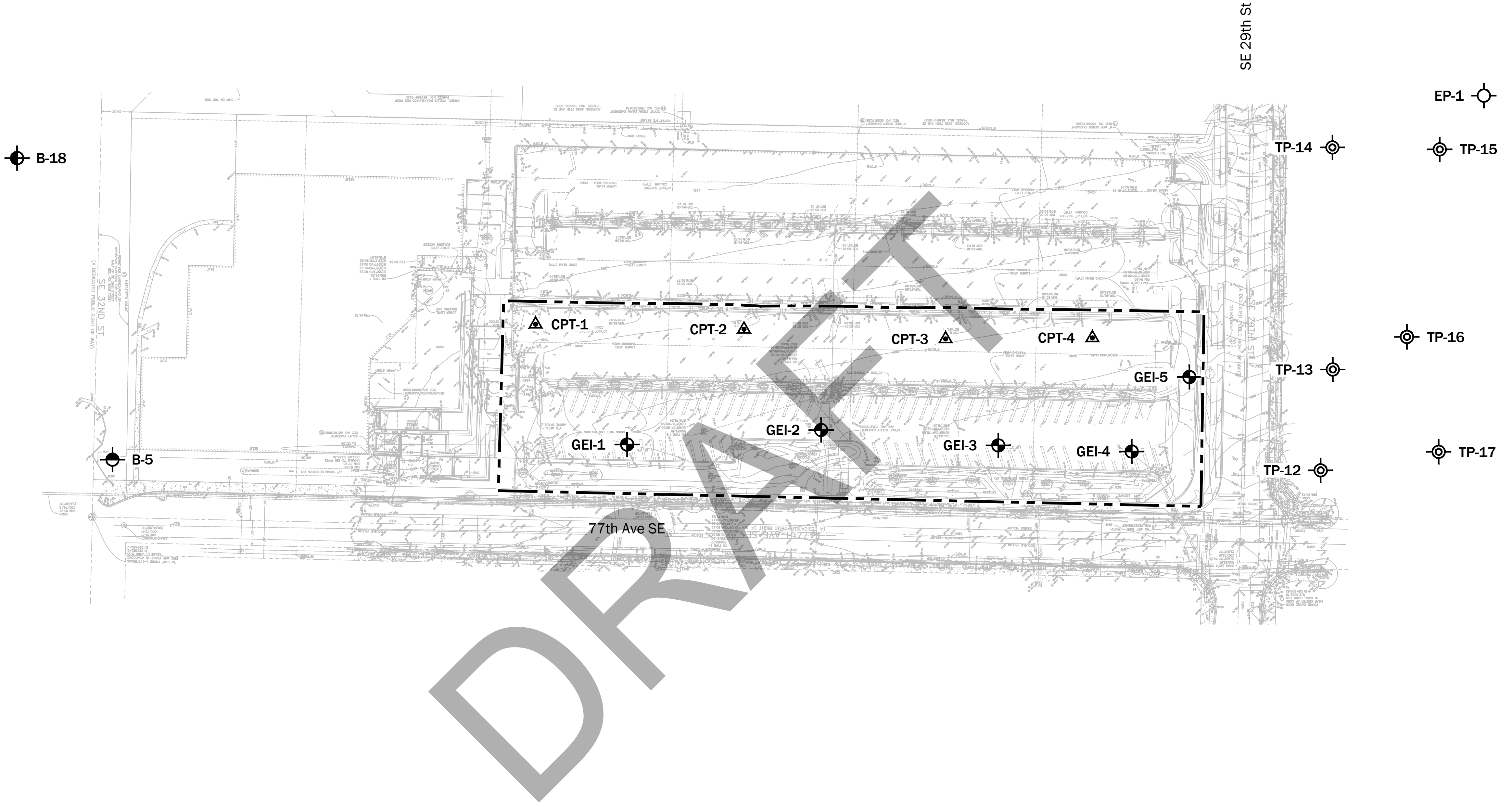
Projection: NAD 1983 StatePlane Washington North FIPS 4601 Feet

Vicinity Map

Mercer Island Rowhouse
Mercer Island, Washington

GEOENGINEERS

Figure 1



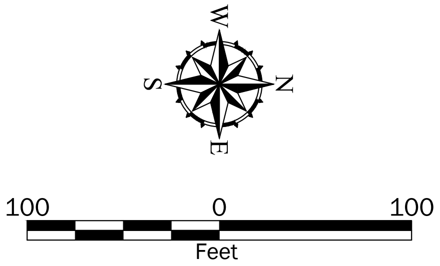
Legend

- Site Boundary
- **GEI-1** Boring by GeoEngineers, Inc., 2021
- **CPT-1** Cone Penetrometer Test by GeoEngineers, Inc., 2021
- **EP-1** Test Pit by Associated Earth Sciences, Inc., 1987
- **B-1** Boring by Hart-Crowser & Associates Co, 1979
- **TP-13** Boring by Cascade Testing Labratory, 1977
- **B-18** Boring by Earth Science Engineering, 1977

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 survey from Ryan Companies US, INC. dated 04/15/21.

Projection: NAD83 Washington State Planes, North Zone, US Foot



Site Plan

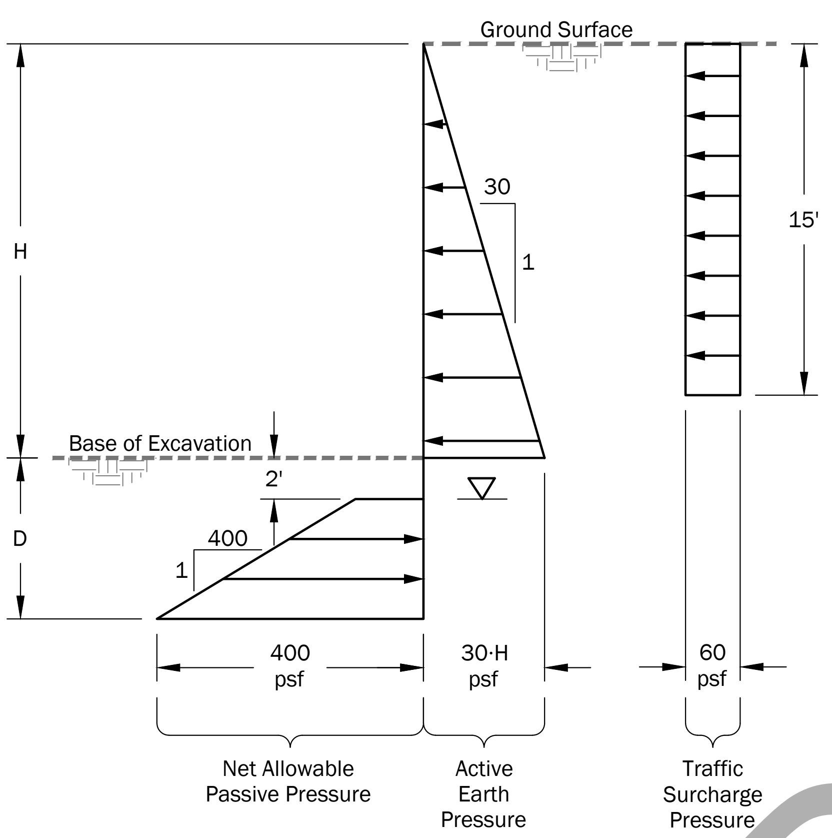
Mercer Rowhouse
Mercer Island, Washington

GeoEngineers

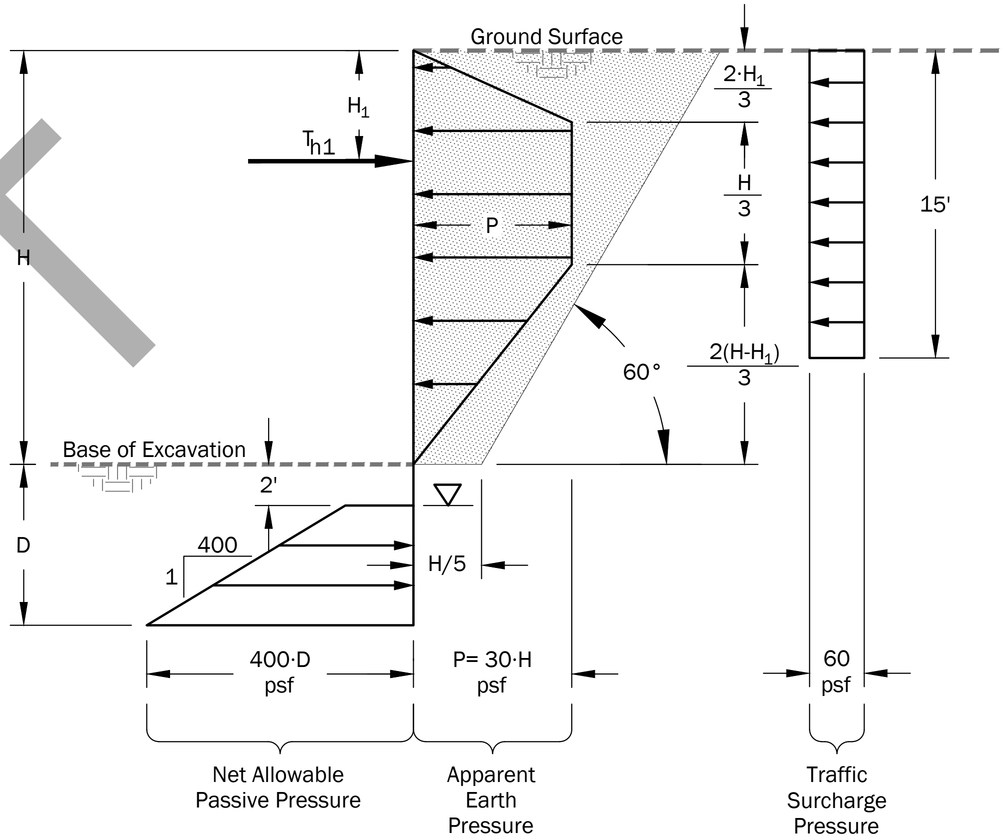
Figure 2

\\geoengineers.com\WAN\Projects\22\22512008\CAD\03\Geotech\2251200803_F02_Site Plan.dwg TAB:F02 Date Exported: 05/17/21 - 11:16 by tbyrd

Cantilever Soldier Pile



Conventional Soldier Pile Wall with One Level of Tiebacks



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

T_{h1} = Horizontal Load in Uppermost Ground Anchor

P = Maximum Apparent Earth Pressure Pounds per Square Foot

▽ Design Groundwater Elevation for Drained Walls/ Passive Resistance Design

Notes:

1. Active/apparent earth pressure and traffic surcharge pressure act over the pile spacing above the base of the excavation.
2. Passive earth pressure acts over 2.5 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
3. Passive pressure includes a factor of safety of 1.5
4. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 4.
5. 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.
6. Figure should be used in conjunction with discussion and qualifications in the report text.

Not To Scale

Earth Pressure Diagrams -
Temporary Soldier Pile & Tieback Wall

Mercer Rowhouse
Mercer Island, Washington

GeoEngineers

Figure 3

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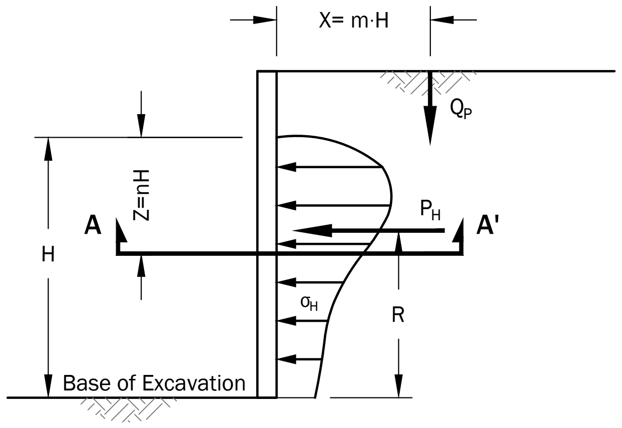
Lateral Earth Pressure from Point Load, Q_P (Spread Footing)



For m ≤ 0.4

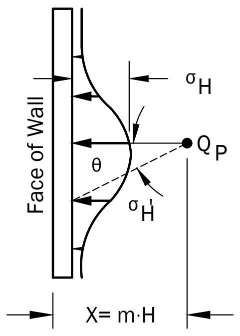
$$\sigma_H = \frac{0.28Q_P n^2 \cdot k}{H^2(0.16+n^2)^3}$$

For m > 0.4

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

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

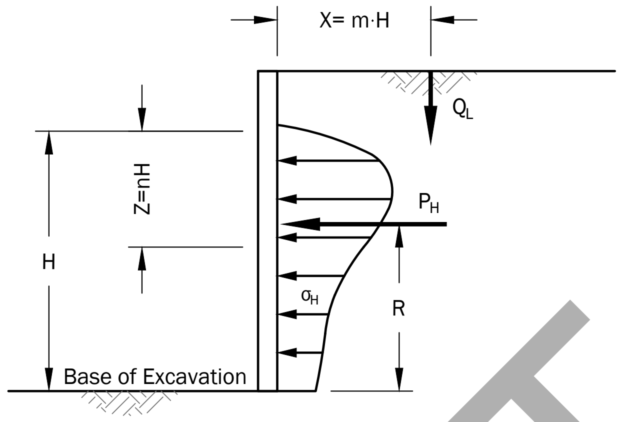
m	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



Section A-A'

Pressures from Point Load Q_P

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



For m ≤ 0.4

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

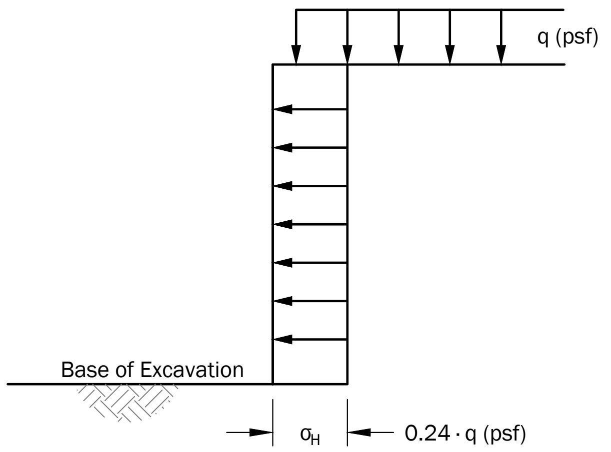
For m > 0.4

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

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

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

Wall Type	Factor Surcharge, k
Rigid	1.0
Flexible	0.5

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 Figure 3.
3. See report text for where surcharge pressures are appropriate.
4. Determination of surcharge factor (k). Flexible is for a system that allows small movements (temporary shoring, retaining walls, etc.) and rigid is for a system that does not allow small movements (permanent basement walls, below grade utility structures, etc.). If permanent basement walls are cast/poured directly against temporary shoring, then the lateral surcharge factor should be assumed as flexible when analyzing lateral surcharges.

Definitions:

Q_P = Point load in pounds
Q_L = Line load in pounds/foot
H = Excavation height below footing, feet
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

Recommended Surcharge Pressure

Mercer Rowhouse
Mercer Island, Washington

GeoEngineers

Figure 4

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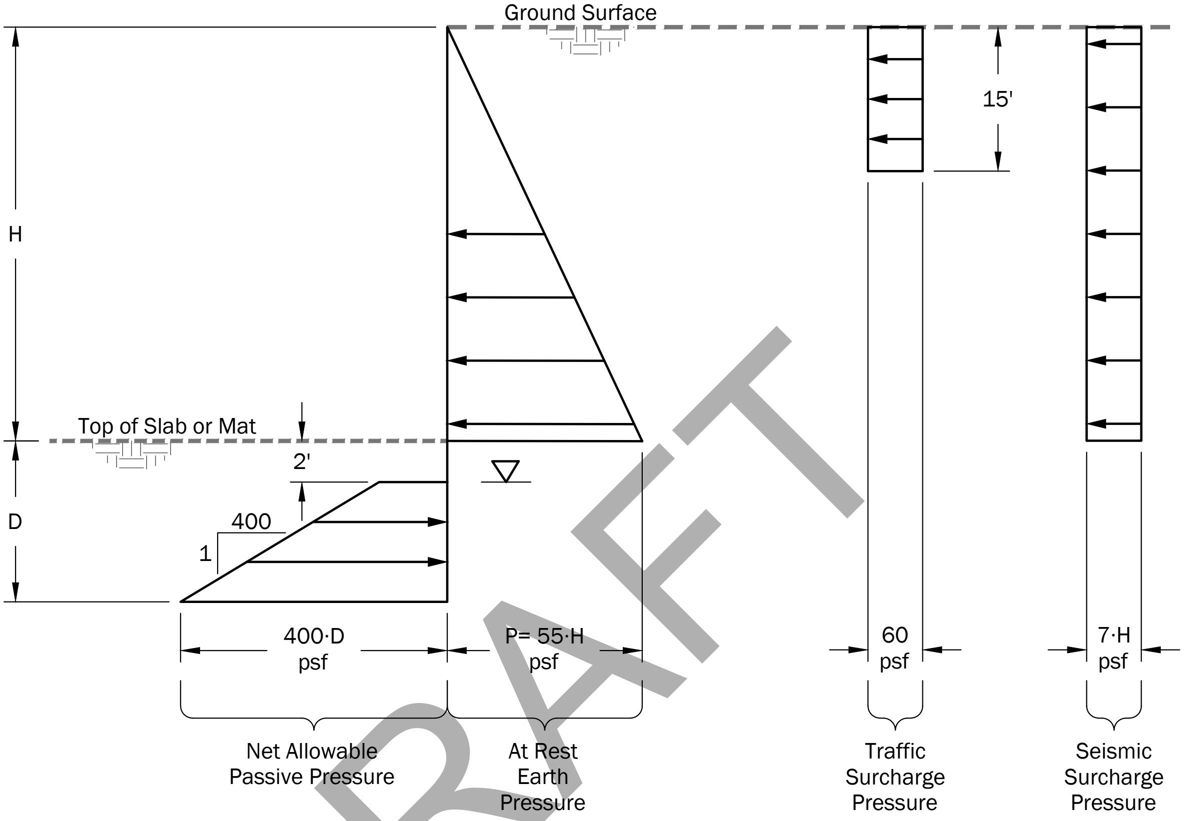
Permanent Basement Wall Against Temporary Shoring



Legend

H = Height of Basement Wall, Feet

D = Foundation Embedment Depth, Feet

P = Maximum At Rest 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 4.
3. This pressure diagram is appropriate for permanent basement walls constructed in front of temporary shoring 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.
4. The at-rest earth pressure does not include a factor of safety and represents the actual anticipated at-rest earth pressure.

Earth Pressure Diagram
Permanent Below Grade Walls

Mercer Rowhouse
Mercer Island, Washington

GeoEngineers

Figure 5

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APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

Subsurface conditions were explored at the site by drilling five borings (GEI-1 through GEI-5) with two equipped with monitoring wells (GEI-1 and GEI-4) and four cone penetration tests (CPTs). The explorations were completed to depths ranging from about 8.8 to 44 feet below the existing ground surface, respectively. The borings were completed by Holocene Drilling, Inc. on April 14, and April 15, 2021. The CPT explorations were completed by ConeTec on April 14, 2021.

The locations of the explorations and the monitoring well elevations were measured by surveying equipment or by taping in the field. The approximate exploration locations are shown on the Site Plan, Figure 2.

Borings

The borings were completed using a truck-mounted or track-mounted, continuous-flight, hollow-stem auger drilling equipment. The borings were continuously monitored by a geotechnical technician 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 split-barrel standard penetration test (SPT) sampler. The disturbed samples were obtained by driving the sampler 18 or 24 inches into the soil with a 140-pound automatic hammer free-falling 30 inches. The number of blows required for each 6 inches 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 driving 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-6, 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 judgment 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.

Monitoring Wells

A representative of GeoEngineers observed the installation of a monitoring well in borings GEI-1 and GEI-4. The monitoring wells were constructed using 2-inch-diameter polyvinyl chloride (PVC) casing. The depth to which the casings were installed was selected based on our understanding of subsurface soil and groundwater conditions in the project area. The lower portion of the casings are slotted to allow entry of water into the casing. Medium sand was placed in the borehole annulus surrounding the slotted portion of the casing. A bentonite seal was placed above and below the slotted portion of the casing. The monitoring wells are protected by installing a flush-mount steel monument set in concrete. Completion details for the monitoring wells are shown on the logs presented in Figures A-2 and A-5.

The monitoring wells completed at the project site will require decommissioning by a licensed well driller prior to excavation for the planned development. The decommissioning of the wells includes backfilling the monitoring wells and providing documentation of the decommissioning to the Washington State Department of Ecology (Ecology). The well installation log and Ecology registry information required for decommissioning and documentation are included on the boring logs attached in Appendix A.

Cone Penetration Tests

The CPT is a subsurface exploration technique in which a probe with small-diameter steel cone tip is continuously advanced with hydraulically operated equipment. Measurements of tip and sleeve resistance and porewater pressure allow interpretation of the soil profile and the consistency of the strata penetrated. The tip resistance, sleeve friction ratio and pore water pressure are recorded on the CPT logs. The logs of the CPT probes are presented as Figures A-7 through A-10. The CPT soundings were backfilled in general accordance with procedures outlined by the Washington State Department of Ecology.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
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

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS		TYPICAL DESCRIPTIONS
GRAPH	LETTER	
■	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

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.

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

GEOENGINEERS

Figure A-1

Drilled	Start 4/15/2021	End 4/15/2021	Total Depth (ft) 41.5	Logged By JSO Checked By JDB	Driller Holocene Drilling, Inc.
Drilling Method	Hollow-stem Auger				
Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	B-120 Truck Rig		
Surface Elevation (ft) Vertical Datum	85 NAVD88	Top of Casing Elevation (ft)	84.74		
Easting (X) Northing (Y)	1294427 215812	Horizontal Datum	WA State Plane North NAD83 (feet)		

DOE Well I.D.: BMR 916
A 2-in well was installed on 4/15/2021 to a depth of 41.5 ft.

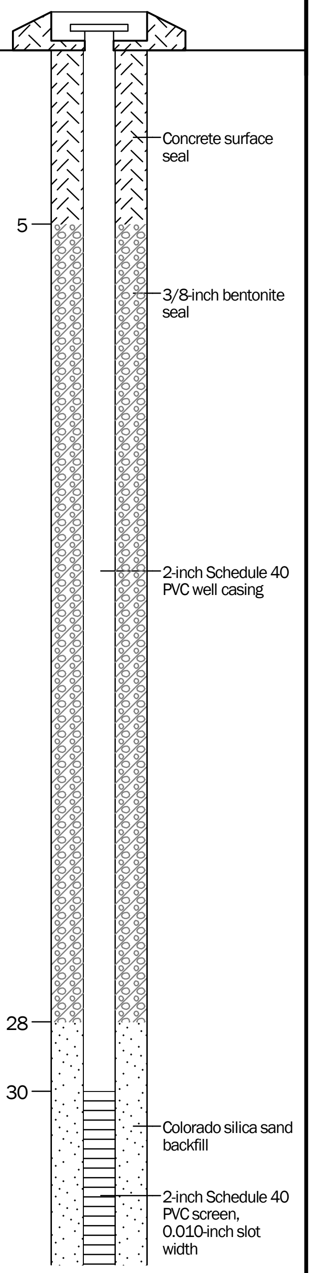
Groundwater

Date Measured	Depth to Water (ft)	Elevation (ft)
4/27/2021	6.10	78.90

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0					AC	2 inches of asphalt		
							5 inches crushed rock base course		
						SM	Brown silty fine to medium sand with gravel and occasional cobble (medium dense to dense, moist) (fill)		
		10	33	1					
80	5	9	26	2					
					▼				
		13	6	3 %F		ML	Gray silt with sand (medium stiff, moist) (recent deposits)	29	79
75	10	10	27	4 %F		GP-GM	Gray gravel with silt and fine to coarse sand (medium dense, wet) (Perched water encountered at time of drilling)	7	8
						ML	Gray silt with sand seams (very stiff to hard, moist) (glacially consolidated soils)		
70	15	18	18	5					
							Grades to sandy silt		
65	20	18	26	6 %F				24	61
							With occasional gravel		
60	25	18	23	7			(Groundwater encountered at 25 feet at time of drilling)		
							Without gravel, becomes wet		
55	30	12	42	8 %F				18	60
50	35								

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Aerial Imagery. Vertical approximated based on Topographic Survey.

Date:5/18/21 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\22\22512008\GINT\2251200803.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_WELL_%F

Log of Monitoring Well GEI-1

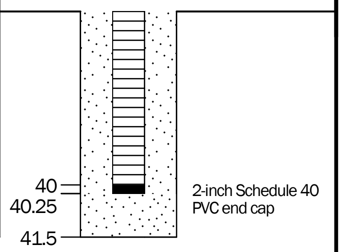
GEOENGINEERS

Project: Mercer Island Rowhouse
Project Location: Mercer Island, Washington
Project Number: 22512-008-03

Figure A-2
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA Interval	FIELD DATA Recovered (in)	FIELD DATA Blows/foot	FIELD DATA Collected Sample	FIELD DATA Sample Name Testing	FIELD DATA Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	35	☒	18	47		9						
45	40	☒	18	36		10						

WELL LOG



Log of Monitoring Well GEI-1 (continued)

GeoEngineers

Project: Mercer Island Rowhouse
Project Location: Mercer Island, Washington
Project Number: 22512-008-03

Drilled	Start 4/15/2021	End 4/15/2021	Total Depth (ft) 36.5	Logged By JSO Checked By JDB	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) 85 Vertical Datum NAVD88			Hammer Data Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment B-120 Truck Rig	
Easting (X) 1294416 Northing (Y) 215968			System Datum WA State Plane North NAD83 (feet)		See "Remarks" section for groundwater observed	
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
	0							AC	3 inches of asphalt			
								SM	5 inches of crushed rock base course			
									Brown silty fine to medium sand with gravel (medium dense to dense, moist) (fill)			
		☒	18	44		1 SA				12	49	
80	5	☒	18	30		2						
								ML	Gray silt (stiff to very stiff, moist) (glacially consolidated soils)			
		☒	18	19		3						
75	10	☒	18	23		4			With lenses of fine sand			
								SM	Gray silty fine sand (medium dense, wet)			Groundwater observed at approximately 13¾ feet during drilling
70	15	☒	18	19		5A %F 5B				26	15	
								ML	Gray silt (stiff, moist to wet)			
65	20	☒	18	14		6 %F			With interbeds of fine sand, becomes moist to wet	27	53	
								SM	Gray silty fine sand (dense, wet)			
60	25	☒	18	41		7A 7B						
								ML	Gray silt (hard, moist)			
55	30	☒	18	37		8			With occasional sand lenses			
50	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Aerial Imagery. Vertical approximated based on Topographic Survey.

Log of Boring GEI-2

GEOENGINEERS

Project: Mercer Island Rowhouse
Project Location: Mercer Island, Washington
Project Number: 22512-008-03

		FIELD DATA										
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	51		9			Without sand lenses			

Log of Boring GEI-2 (continued)

GEOENGINEERS

Project: Mercer Island Rowhouse
Project Location: Mercer Island, Washington
Project Number: 22512-008-03

Figure A-3
Sheet 2 of 2

Drilled	Start 4/14/2021	End 4/14/2021	Total Depth (ft) 31.5	Logged By JSO Checked By JDB	Driller Holocene Drilling, Inc.	Drilling Method	Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	84 NAVD88		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	B-120 Truck Rig
Easting (X) Northing (Y)	1294428 216110		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
	0							AC	2 inches of asphalt			
									5 inches of crushed rock base course			
								ML	Brown-gray silt with sand (very stiff, moist) (fill)			
		☒	12	16		1						
80												
	5	☒	18	25		2 %F				19	74	
								SM	Brown-gray silty fine to medium sand with occasional gravel (dense, moist) (glacially consolidated soils)			
		☒	15	32		3						
75												
								ML	Gray silt with occasional sand (stiff to very stiff, moist)			
	10	☒	15	21		4						
70												
	15	☒	18	14		5 AL				18		AL (LL = 24, PI = 3)
									Without sand			
65												
	20	☒	18	18		6			With occasional gravel			
60												
	25	☒	18	20		7			Without gravel			
55												
	30	☒	13	28		8						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Aerial Imagery. Vertical approximated based on Topographic Survey.

Log of Boring GEI-3

GEOENGINEERS

Project: Mercer Island Rowhouse
Project Location: Mercer Island, Washington
Project Number: 22512-008-03

Drilled	Start 4/14/2021	End 4/14/2021	Total Depth (ft) 31.5	Logged By JSO Checked By JDB	Driller Holocene Drilling, Inc.	Drilling Method Hollow-stem Auger	

Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop

Drilling Equipment: B-120 Truck Rig

Surface Elevation (ft): 84

Vertical Datum: NAVD88

Top of Casing Elevation (ft): 83.72

Easting (X): 1294433

Northing (Y): 216217

Horizontal Datum: WA State Plane North NAD83 (feet)

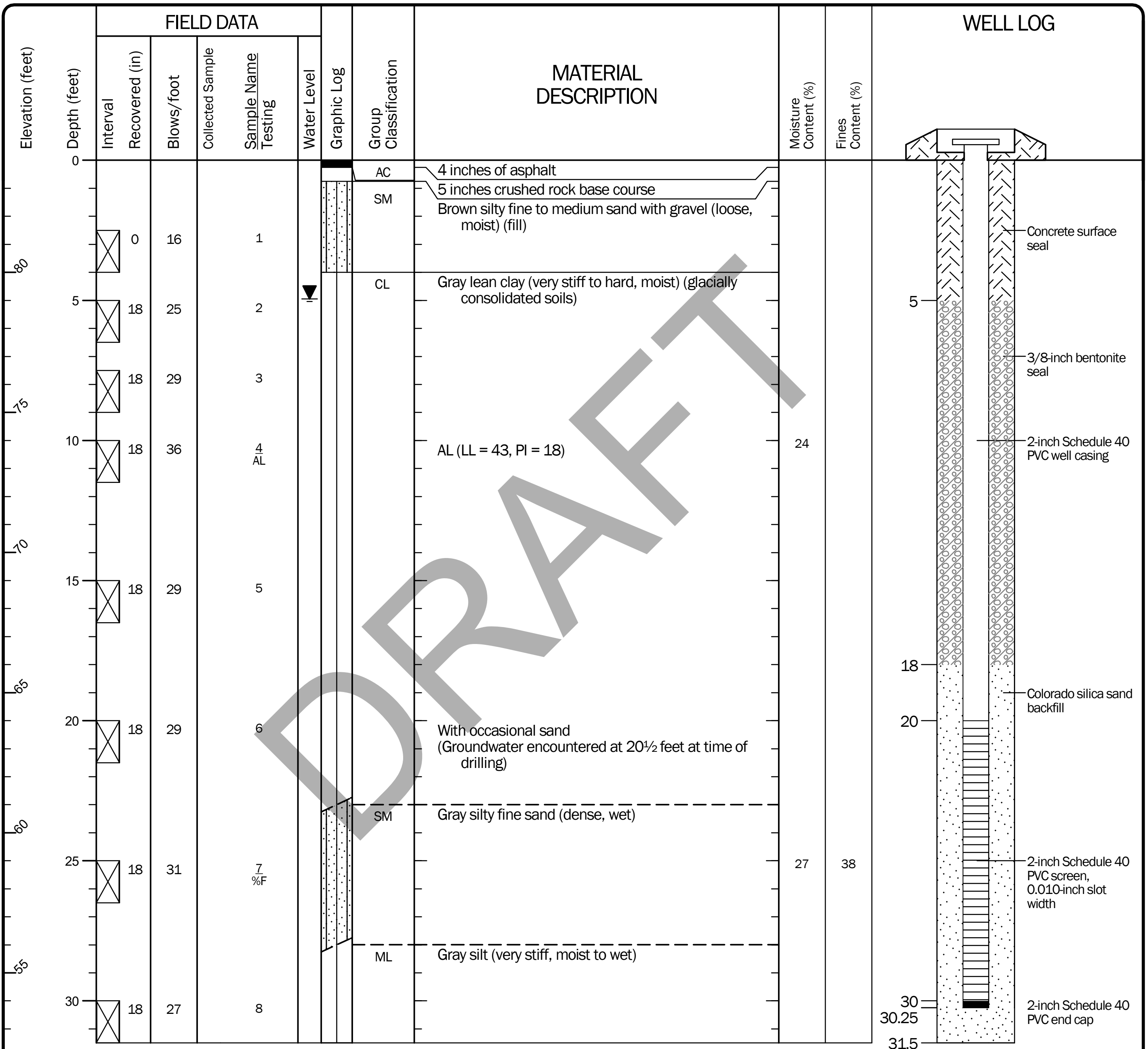
DOE Well I.D.: BMR 915

A 2-in well was installed on 4/14/2021 to a depth of 31.5 ft.

Groundwater

Date Measured	Depth to Water (ft)	Elevation (ft)
4/27/2021	4.95	78.77

Notes:



Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Moisture Content (%)	Fines Content (%)
2.5	0	16	1		
5	18	25	2		
7.5	18	29	3		
10	18	36	4 / AL	24	
15	18	29	5		
20	18	29	6		
25	18	31	7 / %F	27	38
30	18	27	8		

Group Classification	MATERIAL DESCRIPTION
AC	4 inches of asphalt
	5 inches crushed rock base course
SM	Brown silty fine to medium sand with gravel (loose, moist) (fill)
CL	Gray lean clay (very stiff to hard, moist) (glacially consolidated soils)
	AL (LL = 43, PI = 18)
	With occasional sand (Groundwater encountered at 20½ feet at time of drilling)
SM	Gray silty fine sand (dense, wet)
ML	Gray silt (very stiff, moist to wet)

Note: See Figure A-1 for explanation of symbols.

Coordinates Data Source: Horizontal approximated based on Aerial Imagery. Vertical approximated based on Topographic Survey.

Log of Monitoring Well GEI-4

GeoEngineers

Project: Mercer Island Rowhouse

Project Location: Mercer Island, Washington

Project Number: 22512-008-03

Figure A-5

Sheet 1 of 1

Drilled	Start 4/14/2021	End 4/14/2021	Total Depth (ft)	44	
Logged By	JSO	Checked By	JDB	Driller	Holocene Drilling, Inc.
Drilling Method	Hollow-stem Auger				
Surface Elevation (ft) Vertical Datum	80 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	B-120 Truck Rig
Easting (X) Northing (Y)	1294373 216263	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
	0							CC	5 inches concrete			
								SM	3 inches crushed rock base course			
									Gray silty fine to medium sand with gravel (very dense, moist) (glacially consolidated soils)			
		☒	18	70		1						
								ML	Gray silt with lenses of fine to medium sand (stiff to very stiff, moist)			
75	5	☒	18	30		2						
		☒	18	25		3			Without sand lenses			
70	10	☒	18	26		4						
		☒	18	22		5 %F				24	86	
65	15											
		☒	18	25		6						
60	20											
		☒	18	12		7			With occasional lenses of fine to medium sand			Driller added water at 22½ feet to prevent heave
55	25											
								SM	Gray silty fine to medium sand (very dense, wet)			
		☒	8	50/6"		8 %F				25	25	
50	30											
		☒	18	56		9 %F				24	28	
45	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Aerial Imagery. Vertical approximated based on Topographic Survey.

Log of Boring GEI-5

GeoEngineers

Project: Mercer Island Rowhouse
Project Location: Mercer Island, Washington
Project Number: 22512-008-03

Figure A-6
Sheet 1 of 2

		FIELD DATA										
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											
								ML	Gray silt (hard, wet)			
		☒	18	34		10						
40	40											
		☒		49		11						

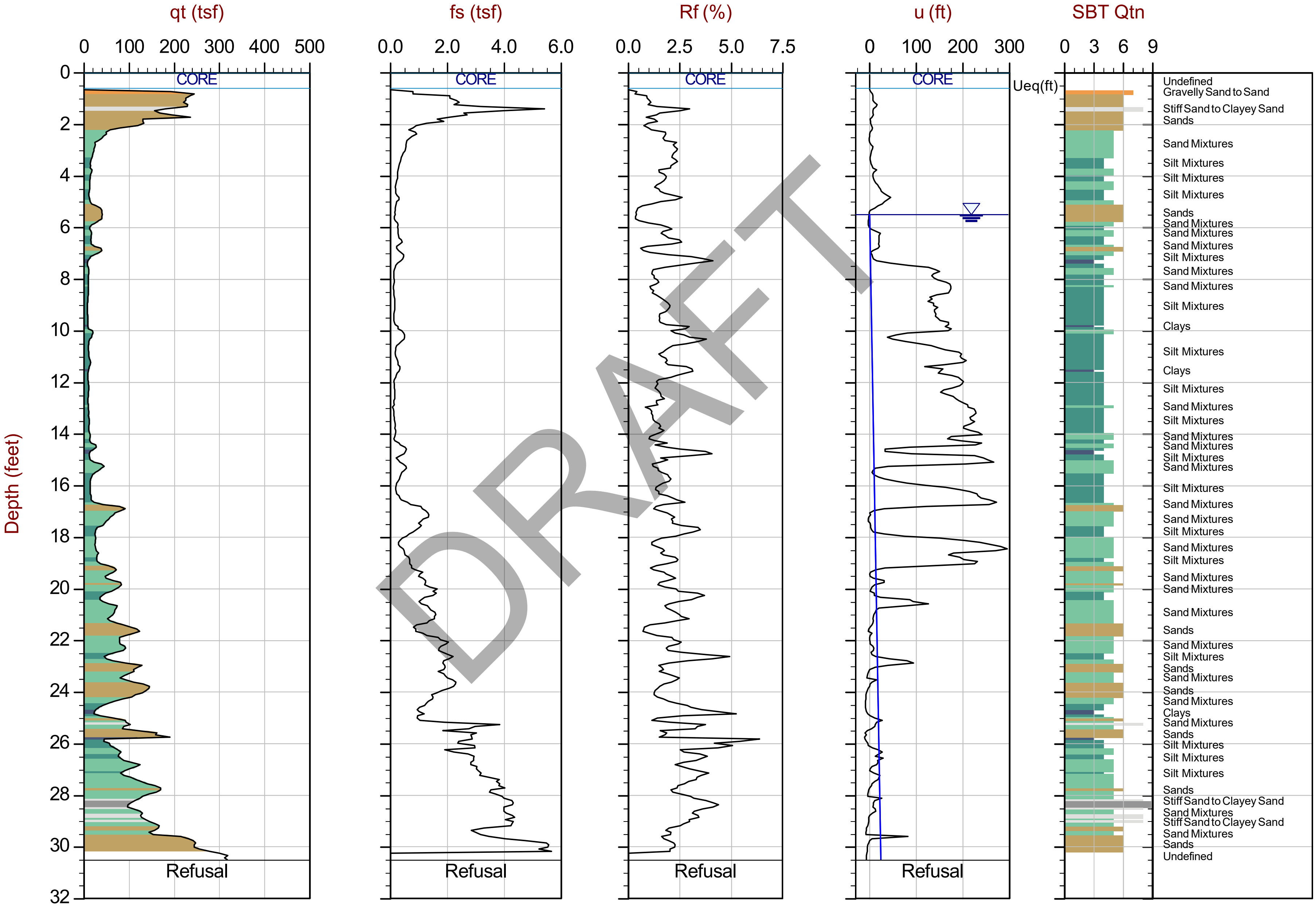
Log of Boring GEI-5 (continued)

Figure A-7

GeoEngineers

Job No: 21-59-22241
Date: 2021-04-14 12:48
Site: Mercer Island Garage CPT

Sounding: CPT-01
Cone: EC725



Max Depth: 9.300 m / 30.51 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-59-22241_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58282 Long: -122.23574

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

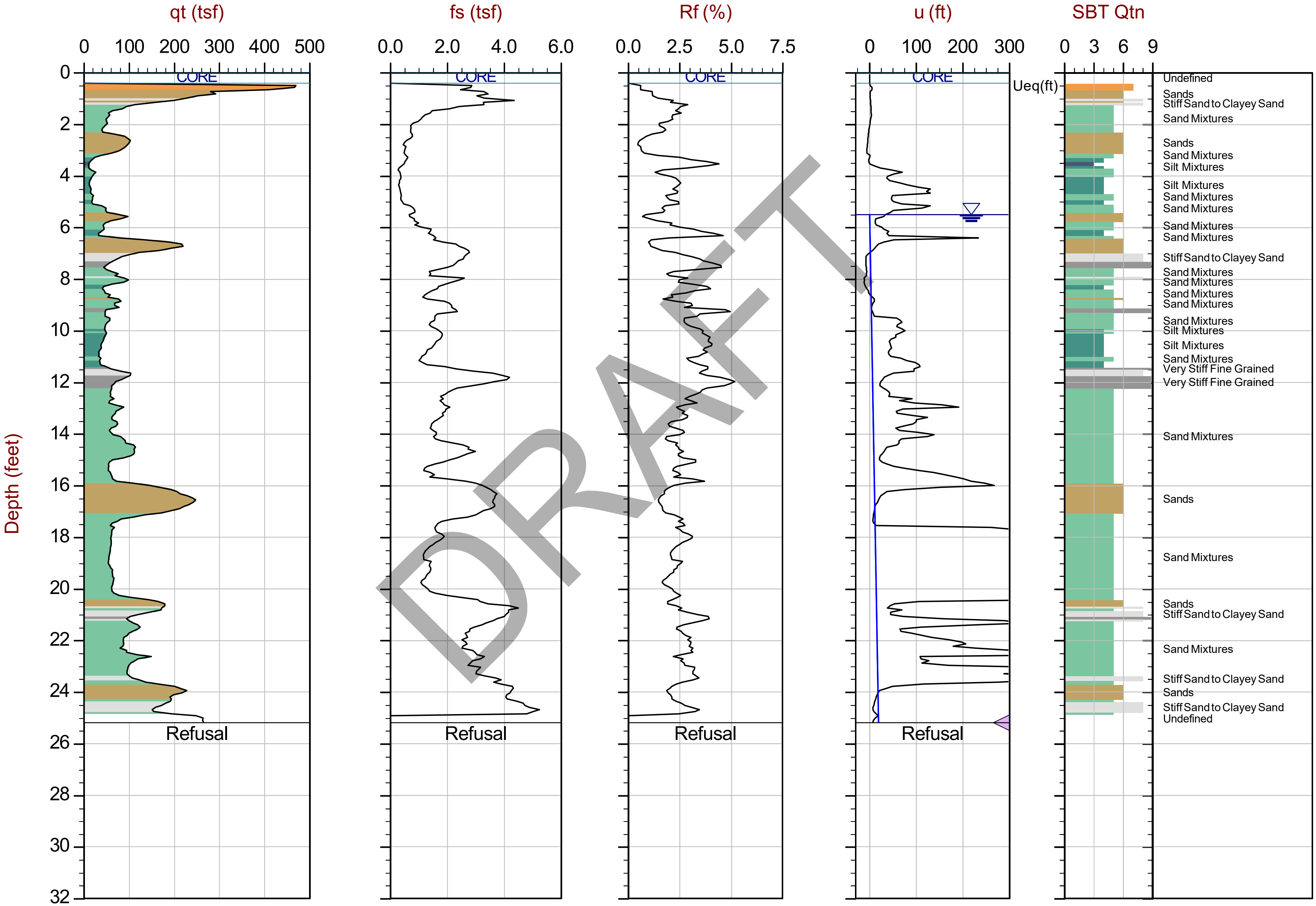
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Figure A-8

GeoEngineers

Job No: 21-59-22241
Date: 2021-04-14 11:20
Site: Mercer Island Garage CPT

Sounding: CPT-02
Cone: EC725



Max Depth: 7.675 m / 25.18 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-59-22241_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58313 Long: -122.23574

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

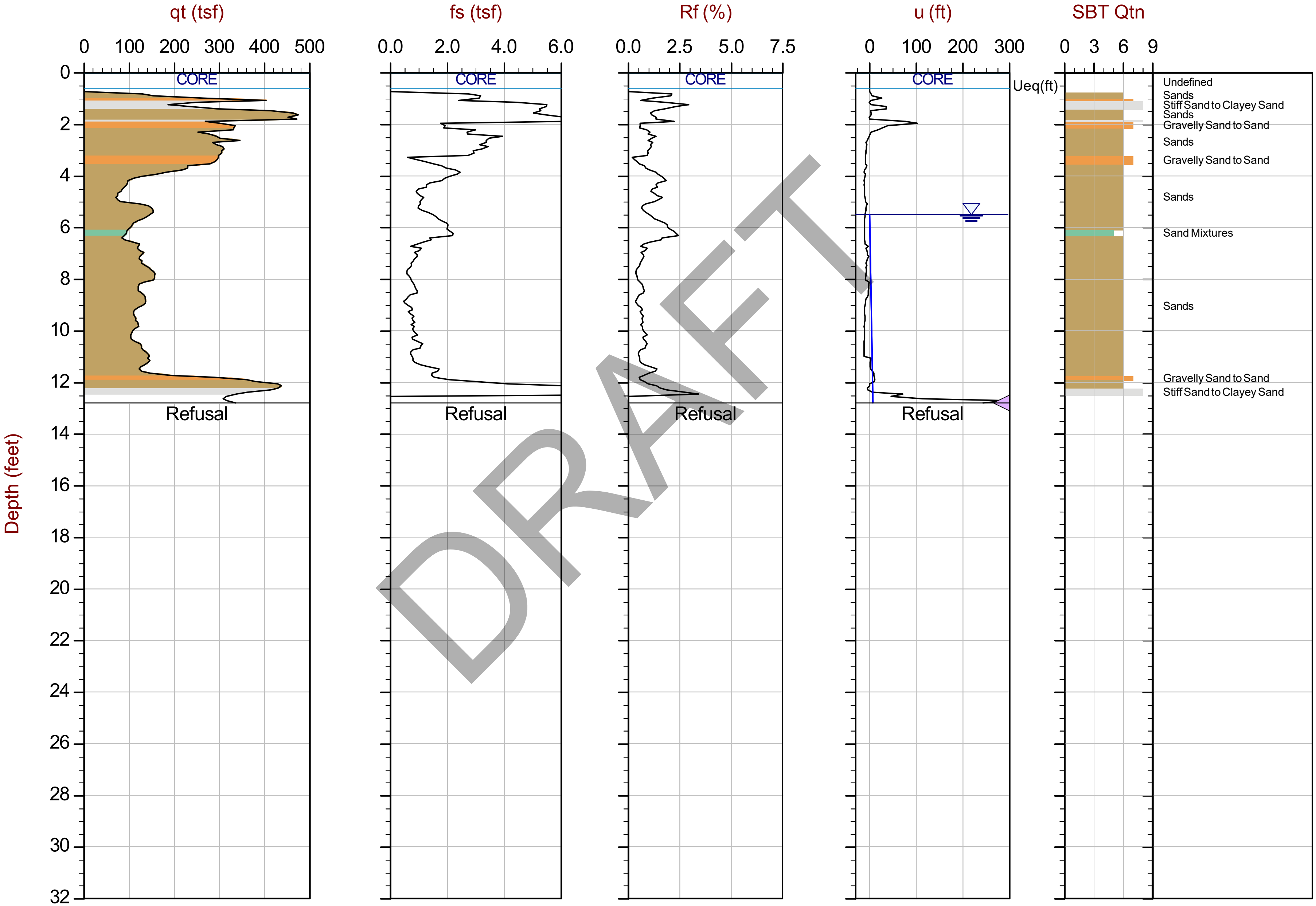
Figure A-9

ConeTec

GeoEngineers

Job No: 21-59-22241
Date: 2021-04-14 09:46
Site: Mercer Island Garage CPT

Sounding: CPT-03
Cone: EC725



Max Depth: 3.900 m / 12.80 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-59-22241_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58349 Long: -122.23573

● Equilibrium Pore Pressure (Ueq) ○ Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

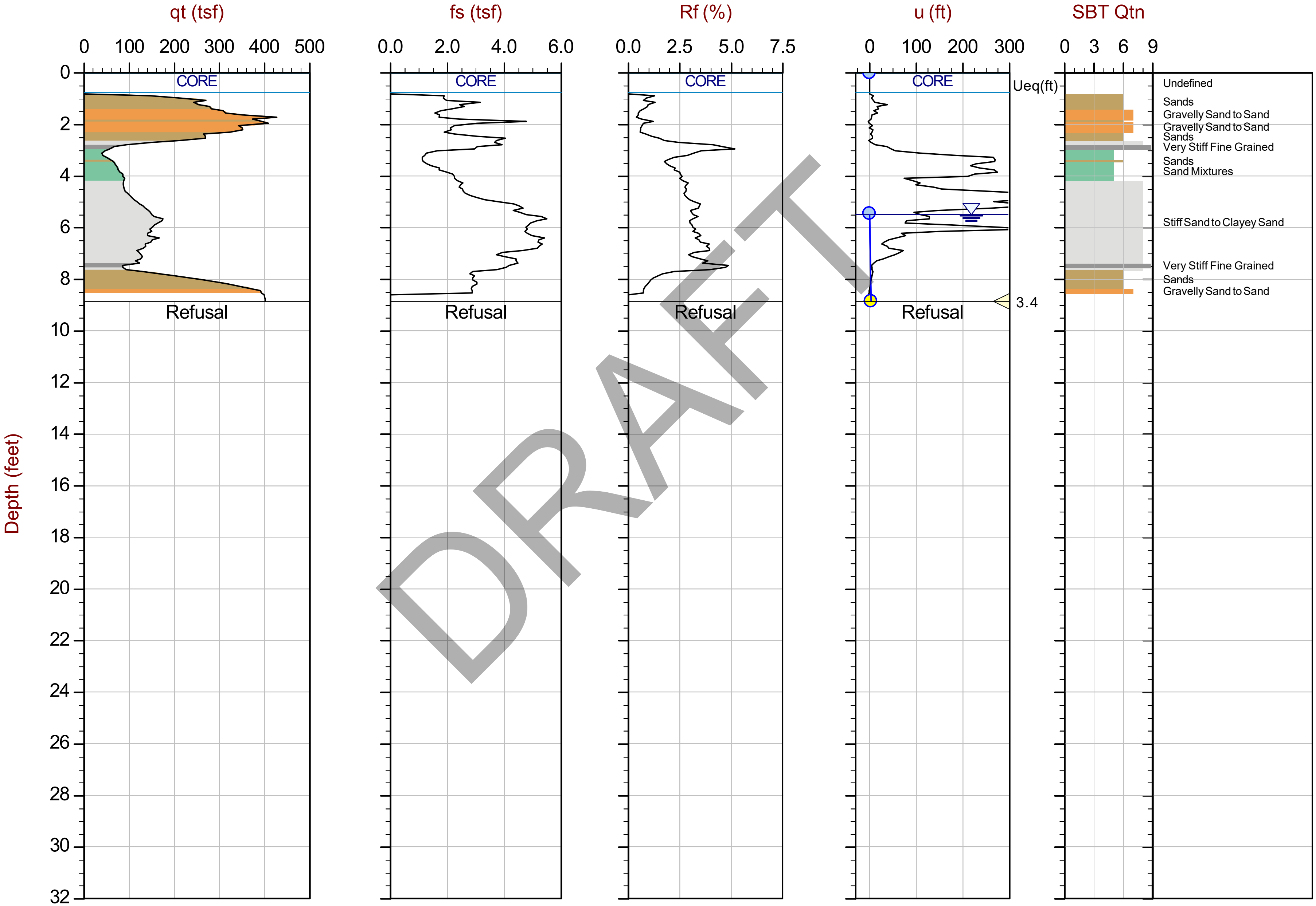
Figure A-10

CONETEC

GeoEngineers

Job No: 21-59-22241
Date: 2021-04-14 09:14
Site: Mercer Island Garage CPT

Sounding: CPT-04
Cone: EC725



Max Depth: 2.700 m / 8.86 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-59-22241_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.58389 Long: -122.23573

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

APPENDIX B

Laboratory Testing

APPENDIX B
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, percent fines (material passing the U.S. No. 200 sieve), and grain size distribution (sieve analysis). The tests were performed in general accordance with test methods of ASTM International (ASTM) or other applicable procedures.

The sieve analysis results are presented in Figure B-1. The Atterberg limits results are presented in Figure B-2. The results of the moisture content and percent fines determinations are presented at the respective sample depths on the exploration logs in Appendix A.

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.

Percent Passing U.S. No. 200 Sieve (%F)

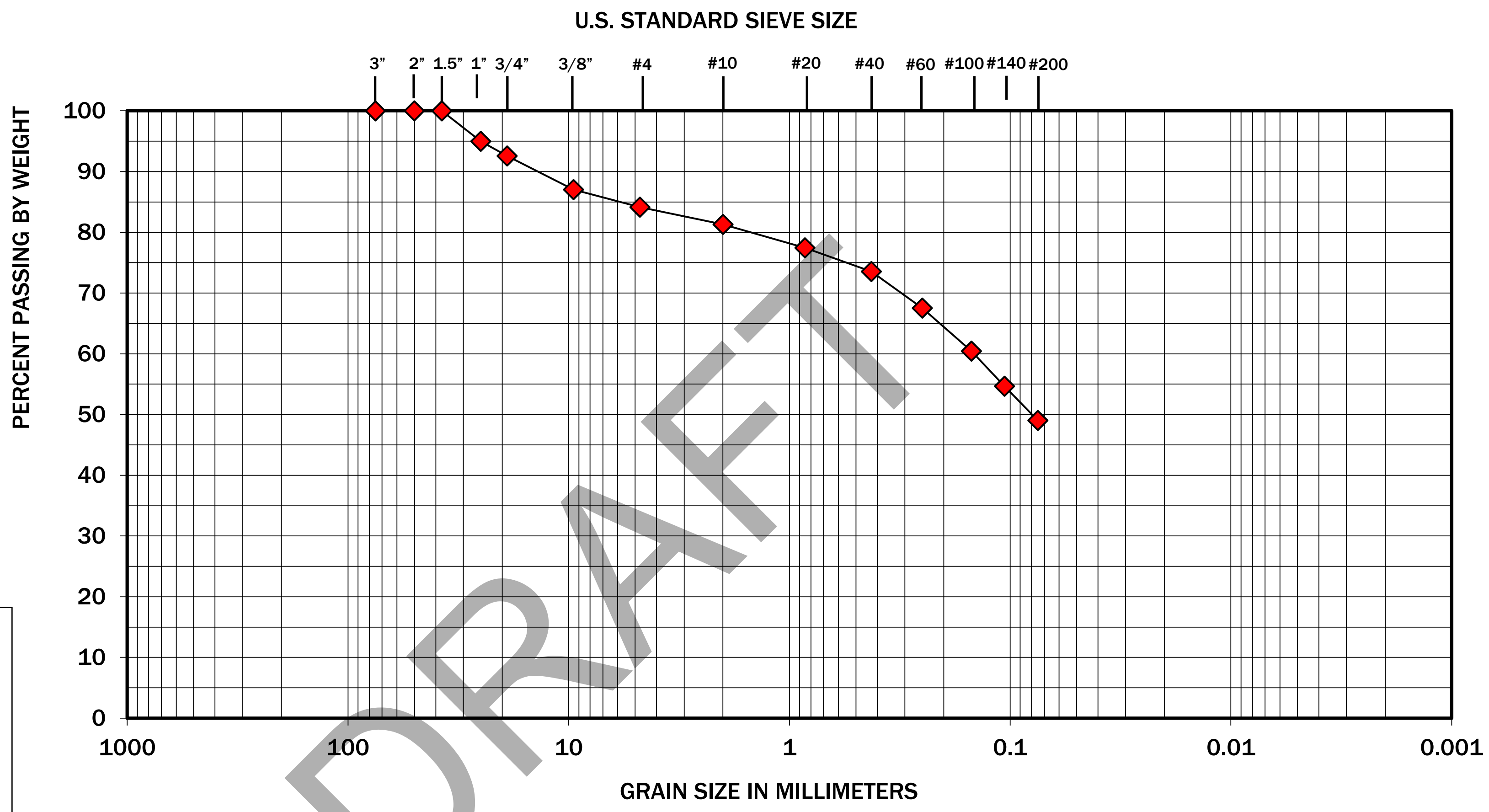
Selected samples were "washed" through the U.S. No. 200 mesh sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify 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.

Sieve Analysis (SA)

Sieve analysis testing was performed on selected samples in general accordance with ASTM D 422. The wet sieve analysis method was used to determine the percentage of soil passing the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified in general accordance with the Unified Soil Classification System (USCS), and are presented in Figure B-1.

Atterberg Limits Testing

Atterberg limits testing was performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits testing are presented in Figure B-2.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	GEI-2	2.5	12	Silty sand with gravel (SM)

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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

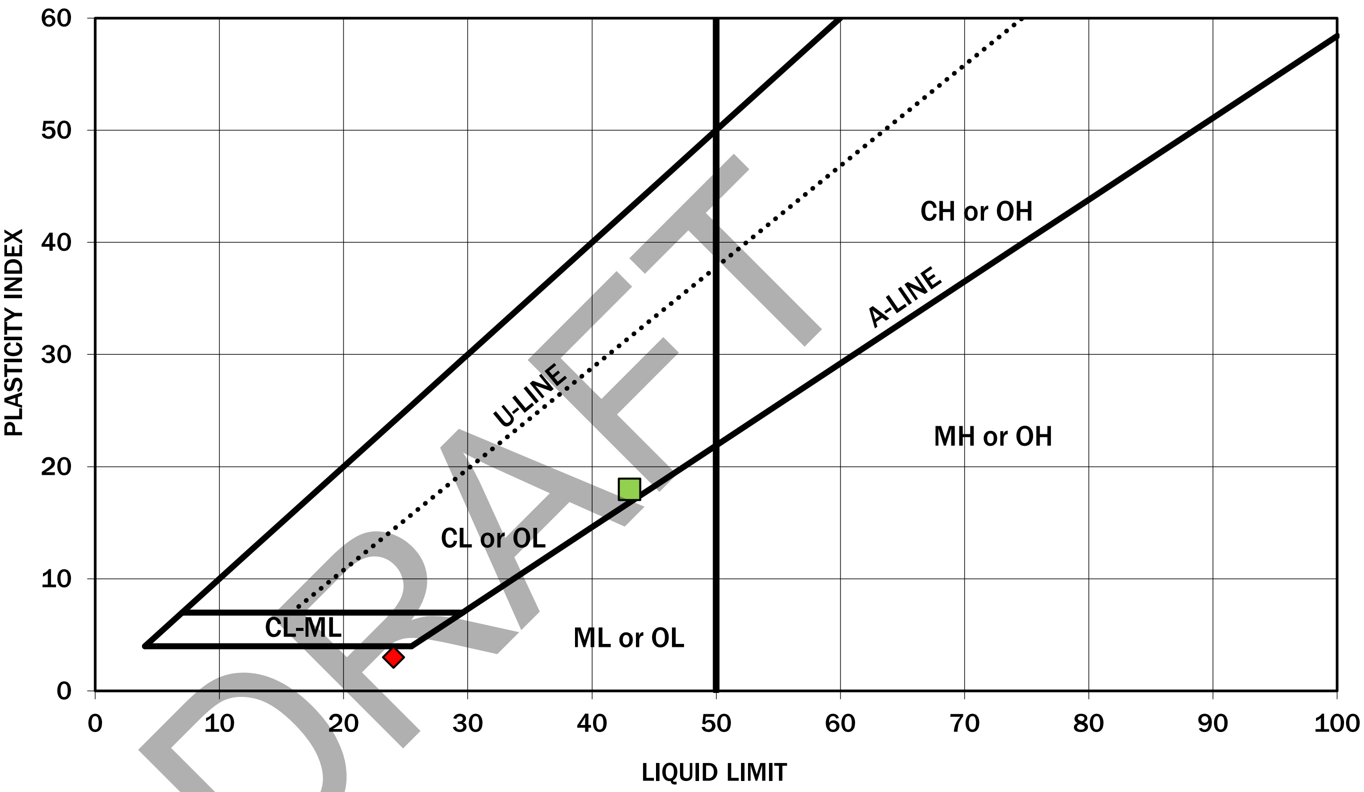
GEOENGINEERS

Mercer Island Rowhouse
Mercer Island, Washington

Sieve Analysis Results

Figure B-1

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	GEI-3	15	18	24	3	Silt (ML)
🟩	GEI-4	10	24	43	18	Lean clay (CL)

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Atterberg Limits Test Results

Mercer Island Rowhouse
Mercer Island, Washington

Figure B-2

APPENDIX C

Boring Logs from Previous Studies

APPENDIX C
BORING LOGS FROM PREVIOUS STUDIES

Included in this section are logs from previous studies completed in the immediate vicinity of the project site.

- The log of six test pits (TP-12 through TP-17) by Cascade Testing Laboratory, Inc., in 1977 for the Albertson's Store #450 Project;
- The log of one boring (B-18) completed by Earth Science Engineering, in 1977 for the City of Mercer Island;
- The log of one boring (B-5) by Hart-Crowser & Associates, Inc., in 1979 for the Farmers Insurance Group;
- The log of one test pit (EP-1) by Associate Earth Sciences, Inc., in 1987 for the Deavuille Apartments Project;

CASCADE TESTING LABORATORY, INC.
TESTING & INSPECTION - ENGINEERS
14120 N.E. 21st STREET
BELLEVUE, WASHINGTON 98007

TEST PIT SOILS LO

Date 5-29,30-77

Cert No. 773-60

Test Pit No. 12

Page.

Project ALBERTSON'S FOOD CENTER
MERCER ISLAND, WASHINGTON

Location SEE LOCATION MAP.

Depth Below Grade	Strata Log	Soil Bearing	Soil Description	Depth Test Sample	Depth Water Table
			BLACK HIGHLY ORGANIC TOPSOIL. LIGHT SEEPAGE AT BASE OF LAYER.		
			MODERATELY FIRM GREY AND BROWN MOTTLED CLAYEY SILT TO SILTY CLAY WITH THIN SAND SEAMS. HEAVY SEEPAGE AT 3½-4' FROM WEST SIDE OF PIT.		* * *
5			SAME AS ABOVE. BECOMES VERY STIFF TO HARD.		
	T.D. - 7.0'				
10					
15					
20					

Notes: * SEEPAGE AT 1'3".
** FLOW AT 3.5' - 4.0'

CASCADE TESTING LABORATORY, INC.
TESTING & INSPECTION - ENGINEERS
14120 N.E. 21st STREET
BELLEVUE, WASHINGTON 98007

TEST PIT SOILS LO

Date 3-29,30-77

Cert No. 773-60

Test Pit No. 13

Page.

Project ALBERTSON'S FOOD CENTER
MERCER ISLAND, WASHINGTON

Location SEE LOCATION MAP.

Depth Below Grade	Strata Log	Soil Bearing	Soil Description	Depth Test Sample	Depth Water Table
	■		BROWN HIGHLY ORGANIC TOPSOIL.		
	▨		FIRM GREY AND BROWN SLIGHTLY MOTTLED SILTY CLAY. WET.		*
5	▨ T.O.-5.0'		SAME AS ABOVE. BECOMES VERY HARD.		
10					
15					
20					

Notes: * SLIGHT SEEPAGE AT 3.0'- 3.5'.

CASCADE TESTING LABORATORY, INC.
TESTING & INSPECTION - ENGINEERS
14120 N.E. 21st STREET
BELLEVUE, WASHINGTON 98007

TEST PIT SOILS LC

Date 3 29,30-77

Cert No. 773-60

Test Pit No. 14

Page.

Project ALBERTSON'S FOOD CENTER
MERCER ISLAND, WASHINGTON

Location SEE LOCATION MAP.

Depth Below Grade	Strata Log	Soil Bearing	Soil Description	Depth Test Sample	Deptl Wate Table
	■		BLACK, HIGHLY ORGANIC TOPSOIL.		
5	▨		MODERATELY FIRM BECOMING FIRM AT DEPTH, GREY-BROWN SLIGHTLY MOTTLED SILTY CLAY TO CLAYEY SILT. HEAVY SEEPAGE AT 3.5'. VERY HEAVY SEEPAGE AT 5.5'.		* **
	▨		SAME AS ABOVE. BECOMES VERY HARD.		
	T.O. -7.0'				
10					
15					
20					

Notes: * HEAVY SEEPAGE AT 3.5'

** VERY HEAVY SEEPAGE AT 5.5'

CASCADE TESTING LABORATORY, INC.
TESTING & INSPECTION - ENGINEERS
14120 N.E. 21st STREET
BELLEVUE, WASHINGTON 98007

TEST PIT SOILS LO

Date 3-29,30-77

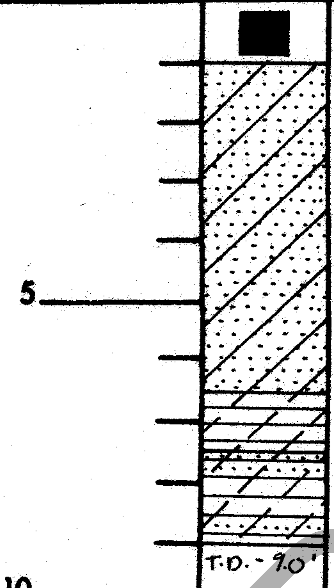
Project ALBERTSON'S FOOD CENTER
MERCER ISLAND, WASHINGTON

Cert No. 77360

Test Pit No. 15

Page.

Location SEE LOCATION MAP.



Depth Below Grade	Strata Log	Soil Bearing	Soil Description	Depth Test Sample	Depth Water Table
			DARK BROWN HIGHLY ORGANIC TOPSOIL.		
			FIRM GREY-BROWN LIGHTLY MOTTLED, SUPERFINE SAND TO SILT, WELL BEDDED. SATURATED. VERY HEAVY SEEPAGE 2'-6.5'.		*
			FIRM, GREY-BROWN MOTTLED SILTY CLAY.		
			FIRM BLUE GREY MOTTLED SILTY CLAY WITH INTERBEDDED THIN SAND LENSES (EXTRA FINE SAND - SATURATED).		
	T.D. - 9.0'				
10					
15					
20					

Notes: * SATURATED 2.0' - 6.5'. HEAVY CAVING IN SUPERFINE SANDS. VERY HEAVY SEEPAGE 2.0' - 6.5'.

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Cert No. 773-60

Test Pit No. 16

Page

Project ALBERTSON'S FOOD CENTER
MERCER ISLAND, WASHINGTON

Location SEE LOCATION MAP.

Depth Below Grade	Strata Log	Soil Bearing	Soil Description	Depth Test Sample	Depth Water Table
	■		BLACK ORGANIC TOPSOIL - SATURATED.		
5	▨		FIRM GREY-BROWN THINLY INTERBEDDED SILT, EXTRAFINE SAND AND CLAY LENSES. IRREGULAR SEQUENCE, THIN BEDDING. SANDS SATURATED.		*
	T.O.-8.0'				
10					
15					
20					

Notes: * SEEPAGE 1.5' - 8.0'.

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14120 N.E. 21st STREET
BELLEVUE, WASHINGTON 98007

TEST PIT SOILS LC

Date 3-29,30-77

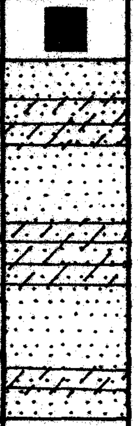
Project ALBERTSON'S FOOD CENTER
MERCER ISLAND, WASHINGTON

Cert No. 773-60

Test Pit No. 17

Page.

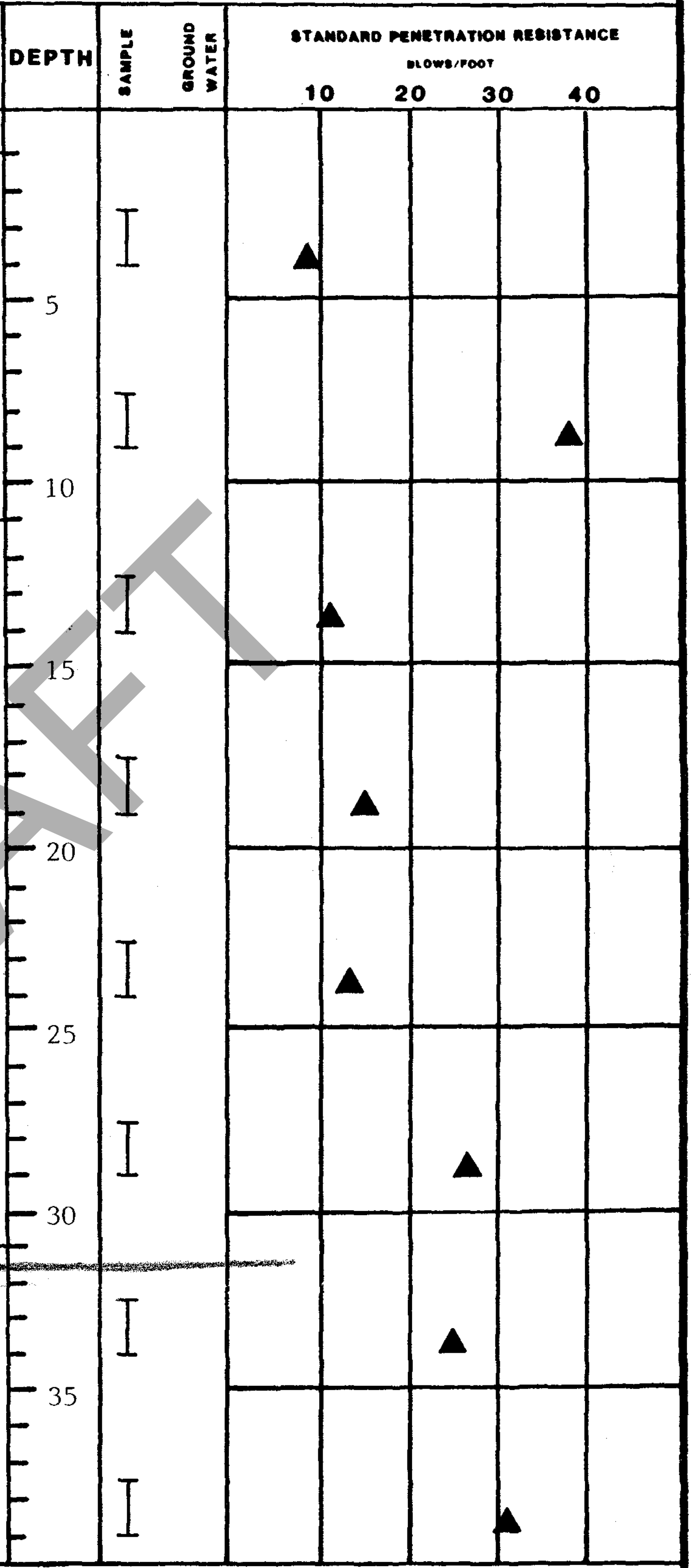
Location SEE LOCATION MAP.

Depth Below Grade	Strata Log	Soil Bearing	Soil Description	Depth Test Sample	Dept Wate Table
			BLACK, HIGHLY ORGANIC TOPSOIL.		
5			FIRM, GREY-BROWN LIGHTLY MOTTLED INTERBEDDED EXTRA FINE SAND AND SILTY CLAY. SEEPAGE FROM 3'8" - 5'8".		*
	T.O. - 7.0'				
10					
15					
20					

Notes: * SEEPAGE FROM 3'8" - 5'8".

EXPLORATION BORING LOG EB-1

GRAPH	USCS	SEDIMENT DESCRIPTION
		6" ACP over crushed rock.
		Stiff to hard, moist. tan silt with brown oxidation.
		Stiff to very stiff, damp, gray clay with some silt.
		Very stiff, damp, gray silt.



8707-24

August 1987

Deauville Apts.
2800 Block of 76th Ave. S.E.
Mercer Island

ASSOCIATED EARTH SCIENCES, INC

EXPLORATION BORING LOG EB-1

GRAPH

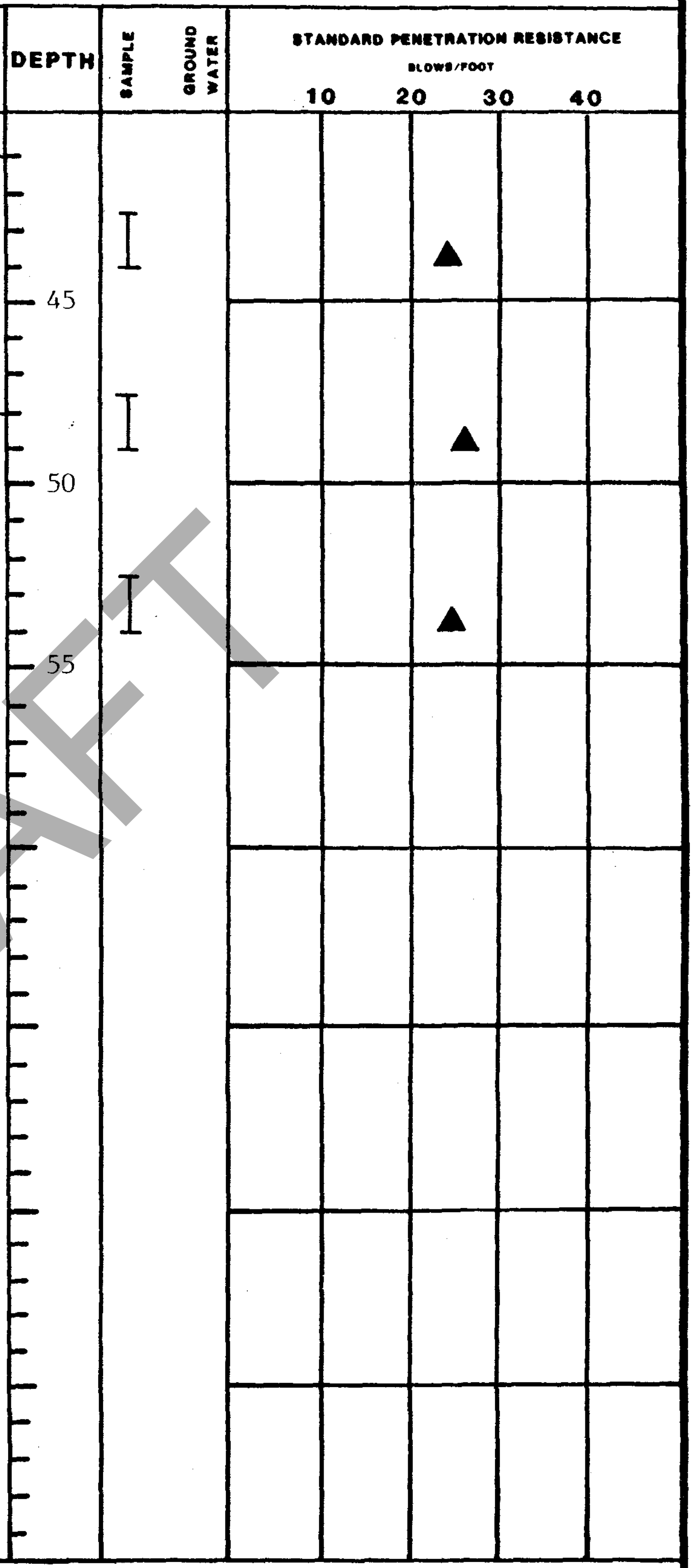
USCS

SEDIMENT DESCRIPTION

Very stiff, damp, gray silt with trace very fine sand grading into silt with some sand.

Very stiff, damp, gray clay with lt. gray silt partings.

BOH



8707-24

August 1987

Deauville Apts.
2800 Block of 76th Ave. S.E.
Mercer Island

ASSOCIATED EARTH SCIENCES, INC

APPENDIX D

Ground Anchor Load Tests and Shoring Monitoring Program

APPENDIX D
GROUND ANCHOR LOAD TESTS AND SHORING MONITORING PROGRAM

Ground Anchor Load Testing

The locations of the load tests should be approved by the engineer and should be representative of field conditions. Load tests should 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 should be installed to prevent interaction between the bonded length of the tieback and the casing/testing apparatus.

The testing equipment should 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 should be independently supported from the load frame/jack and the shoring wall. The hydraulic jack, pressure gauge and pump should be used to apply and measure the test loads.

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

The jack should be independently supported and centered over the tieback so that the tieback does not carry the weight of the jack. The jack, bearing plates and stressing anchorage should be aligned with the tieback. The initial position of the jack should 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 should be tested to validate the design pullout value. All test tiebacks should 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 should 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.

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. Verification 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
1.25 DL	1 minute
1.5DL	60 minutes
1.75DL	1 minute
2.0DL	10 minutes

The alignment load should 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. Tieback deflections during the 1.5DL test load should 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 not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied.

Test Tieback Acceptance

A test tieback should be considered acceptable when:

1. For verification tests, a tieback is considered acceptable if the creep rate is less than 0.08 inches per log cycle of time between 6 and 60 minutes and the creep rate is linear or decreasing throughout the creep test load hold period.
2. For proof tests, a tieback is considered acceptable if the creep rated is less than 0.04 inches per log cycle of time between the 1 and 10 minutes or a creep rate less than 0.08 inches per log cycle of time between 6 and 60 minutes and the creep rate is linear or decreasing throughout the creep test load hold period.
3. The total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.
4. Pullout failure does not occur. Pullout failure is defined as the load at which continued attempts to increase the test load result in continued pullout of the test tieback.

Acceptable proof test tiebacks may be incorporated as production tiebacks provided that the unbonded test length of the tieback hole has not collapsed and the test tieback length and bar size/number of strands are equal to or greater than the scheduled production tieback at the test location. Test tiebacks meeting these criteria should be completed by grouting the unbonded length. Maintenance of the temporary unbonded length for subsequent grouting is the contractor's responsibility.

The engineer should evaluate the verification test results. Tieback installation techniques that do not satisfy the tieback testing requirements should be considered inadequate. In this case, the contractor should propose alternative methods and install replacement verification test tiebacks.

The engineer may require that the contractor replace or install additional production tiebacks in areas represented by inadequate proof tests.

Shoring Monitoring

Preconstruction Survey

A shoring monitoring program should be established to monitor the performance of the temporary shoring walls and to provide early detection of deflections that could potentially damage nearby improvements. We recommend that a preconstruction survey of adjacent improvements, such as streets, utilities and buildings, be performed prior to commencing construction. The preconstruction survey should include a video or photographic survey of the condition of existing improvements to establish the preconstruction condition, with special attention to existing cracks in streets or buildings.

Optical Survey

The shoring monitoring program should include an optical survey monitoring program. The recommended frequency of monitoring should vary as a function of the stage of construction as presented in the following table.

Construction Stage	Monitoring Frequency
During excavation and until wall movements have stabilized	Twice weekly
During excavation if lateral wall movements exceed 1 inch and until wall movements have stabilized	Daily
After excavation is complete/wall movements have stabilized and prior to the floors of the building reaching the top of the excavation	Twice monthly

Monitoring should include vertical and horizontal survey measurements accurate to at least 0.01 feet. A baseline reading of the monitoring points should be completed prior to beginning shoring installation. The survey data should be provided to GeoEngineers for review within 24 hours.

For shoring walls, we recommend that optical survey points be established: (1) along the top of the shoring walls; (2) at the curb on the west side of 77th Avenue SE; (3) at the curb on the south side of SE 29th Street; and (4) on existing buildings located within 100 feet of the site. The survey points should be located at an approximate spacing of 25 feet along the wall face, and the points along the curb line/existing buildings should be located at an approximate spacing of 25 feet. If lateral wall movements are observed to be in excess of ½ inch between successive readings or if total wall movements exceed 1 inch, construction of the shoring walls should be stopped to determine the cause of the movement and to establish the type and extent of remedial measures required.

APPENDIX E

Report Limitations and Guidelines for Use

APPENDIX E
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 Ryan Companies US, Inc. and other project team members for the Mercer Island Rowhouse project. 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 Mercer Island Rowhouse project in Mercer Island, 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, do not 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;

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

- Composition of the design team; or
- Project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

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 manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most 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 subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by 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. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purpose of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

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 to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear 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.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.