Geotechnical Engineering Services

Aegis Mercer Island
Mercer Island, Washington

for
Aegis Senior Communities, LLC

October 26, 2015

GeoEngineers
Earth Science + Technology

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File No. 19811-009-00

October 26, 2015

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GEOENGINEERS

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INTRODUCTION

This report presents the results of GeoEngineers' geotechnical engineering services for the Aegis Mercer Island project, located at 7445 SE 24th Street in Mercer Island, Washington. The site is shown relative to surrounding physical features on the Vicinity Map (Figure 1) and the Site Plan (Figure 2).

The purpose of this report is to provide final geotechnical engineering conclusions and recommendations for the design of the new development. GeoEngineers' geotechnical engineering services have been completed in general accordance with our proposal executed on January 15, 2015 and our Request for Additional Services executed on August 7, 2015.

PROJECT DESCRIPTION

Presently, the site is occupied by a nursing home, consisting of a two-story masonry structure built in 1964. Based on discussions with the project team and preliminary architectural plans provided in an email dated October 20, 2015, the proposed redevelopment will comprise of a three-story building with one level of below-grade parking. The lowest finished floor will be at Elevation 101.25 feet (NAVD 88 datum). Based on the current plans we anticipate temporary shoring will be required on the north, west and south sides of the project in order to accomplish excavation to establish planned grades. Foundations will likely consist of a combination of shallow foundations bearing on glacially consolidated soils and shallow foundations bearing on structural fill over recent deposits. Additionally, site retaining walls will be required to make grades changes across the site, especially to the west due to the existing steep slope.

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

The subsurface conditions at the site were evaluated by drilling six borings, GEI-1 through GEI-6, to depths ranging from approximately 21½ to 41½ feet. The approximate locations of the explorations are shown in Figure 2. Descriptions of the field exploration program and the boring logs are presented in Appendix A.

Laboratory Testing

Soil samples were obtained during drilling and were taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for the determination of the percent fines content, moisture content and Atterberg Limits. Descriptions of the laboratory testing and the test results are presented in Appendix B.

PREVIOUS SITE EVALUATIONS

We reviewed, the logs of selected explorations from previous site evaluations by others completed in the project vicinity in addition to the explorations we performed as part of this evaluation. The reviewed geotechnical information includes:

- The logs of two borings and two test pits from the report titled "Geotechnical Engineering Study, Mercer Island Two Office Building, SE 24th Street and 76th Avenue SE, Mercer Island, Washington," completed in 1985 by Earth Consultants, Inc.

The approximate locations of the four explorations (B-2, B-5, TP-2, and TP-3) are presented on the Site Plan, Figure 2. The logs of the explorations completed as part of the referenced previous study are presented in Appendix C.

SITE CONDITIONS

Surface Conditions

Parcels to the north and south of the site are developed with several wood framed structures (King County parcel Nos. 531510-0445-00 and 531310-0448-07[North] and King County parcel Nos. 531510-0498-06, 531510-0495-09, and 531510-0496-08[South]). There are three multi-story buildings to the east, including an office and apartment buildings (King County parcel Nos. 531510-0546-08, 531510-0525-03, and 531510-0505-07). To the west there are several vacant and occupied (residential structures) parcels (King County parcels No. 531510-0460, 531510-0458, and 531510-004-056).

The site is 1.56 acres and is currently occupied by a two-story masonry building with adjacent parking area. The western portion of the site consists of a vegetated and wooded slope. The site generally slopes down from south and west to the north and east, with site grades ranging from approximately Elevation 160 feet along the western edge of the property along 74th Avenue SE to Elevation 110 feet in the southeast and northeast corners. Buried utilities within the site include sanitary sewer, power, stormwater, communication lines, and water. There are overhead power lines along the west central portion of the property.

Subsurface Soil Conditions

The materials encountered at the site include fill, reworked native soil, and recent deposits that overlie competent glacially consolidated soils. Interpreted subsurface conditions are presented in Cross Section A-A', Figure 3. A brief summary of select conditions observed is presented below:

- A 6-inch layer of asphalt concrete overlies an 18 inch layer of crushed rock base material in boring GEI-1. Crushed rock surfacing was also encountered at boring GEI-2.
- The fill and/or recent deposits at the boring locations generally consists of medium stiff to stiff silt or clay with variable sand content and loose to medium dense silty sand with variable gravel content. Most of the fill soils we observed appear to be reworked native material associated with excavation and backfill for the existing building. The thickness of fill and/or recent deposits encountered in the explorations completed for this study ranged up to approximately 22½ feet, with the thickest area of recent deposits found in boring GEI-2.
- The glacially consolidated soils were encountered below the fill and/or recent deposits. The depth to glacially consolidated soils was typically 10 to 15 feet, with the greatest depth of approximately 22½ feet observed in boring GEI-2. The glacially consolidated soils typically consisted of stiff to hard silt and clay with variable sand and gravel content.

Although not encountered during drilling, occasional boulders have been observed in glacially consolidated soils and may be present at the site.

Groundwater Conditions

Perched groundwater was encountered in the recent deposits near the interface with the glacially consolidated soils in borings GEI-2 and GEI-3. Based on our understanding of the proposed building, perched groundwater may be encountered within excavated soils above the base of the planned excavation. A regional groundwater table was not encountered to the depth explored in any of the borings and we anticipate the regional groundwater table will be located below the base of the planned excavation. Groundwater is likely to fluctuate as a function of location and season.

Geologic Reconnaissance

A geologist from GeoEngineers conducted a surface reconnaissance on January 22, 2015 to observe slope and drainage conditions at the site with regard to potential for slope stability considerations. Steep slopes surround the west, south, and northwest sides of the existing building on the site. The slopes are typically inclined at between 60 and 80 percent, and are locally inclined at over 100 percent [1 Horizontal (H):1 Vertical (V)]. Based on LiDAR hill shade imagery, we interpret that these slopes are likely associated with historic cutting and grading of the broader regional east-facing slopes during periods of past development of the area, rather than formed largely by natural processes prior to development.

Four discrete scarp-like features were observed on the slopes to the west and south of the existing building on site (Figure 4). Two of these features were observed north of the power line corridor that bisects the western portion of the site, and are roughly parallel and adjacent to the 74th Avenue SE right-of-way. The northernmost of these two features is approximately 12 feet tall and is inclined at up to 110 percent. No evidence of active or recent sloughing, or of groundwater seepage was observed along this feature at the time of our site reconnaissance. The southernmost of these two features is approximately 2 feet tall and has exposed bare soil. It is unclear if these scarp-like features are entirely related to past slope cuts during prior site development, or if they are features formed in part by natural slope processes.

We also observed a convergent, hollow-shaped, scarp-like feature in the west-central portion of the site near the southern edge of the power line corridor. No evidence of exposed soil, recent sloughing, slumping, groundwater seepage, or surface water flow was observed at the time of our reconnaissance. However, we interpret that this feature has likely formed as the result of episodic erosion caused by surface water and/or groundwater seepage/piping during past wet periods. However, we were unable to identify a direct source of water that may be contributing to erosion of the feature.

A fourth scarp-like feature was observed on the south central portion of the site. This feature is roughly linear and is oriented east-west along the top of a north-facing steep slope that is typically inclined at about 90 percent. This feature appears to be related to a cut-fill along an old road grade, which coincides with the crest of the slope. It is unclear whether this feature is related in part to slope movement. We observed no evidence of recent sloughing, creeping, or movement of slope soils, or of groundwater seepage, or of distressed vegetation along this slope that would suggest slope instability.

We observed a number of linear cracks in the asphalt concrete surfaces immediately east of the existing building on the site. The cracks are oriented roughly north-south, perpendicular to the regional slope aspect, and are up to 70 feet long. The cracks have no vertical offset, and are only slightly separated (less than an eighth of an inch).

CONCLUSIONS AND RECOMMENDATIONS

Summary

A summary of the primary 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 located within the following geological hazard areas: seismic, landslide, and erosion.
- The site is designated as Soil Profile Type D per the 2012 International Building Code (IBC).
- Cantilever soldier piles and conventional soldier piles walls with ground anchors are considered the best temporary shoring alternative for the planned excavation because of the presence of the weaker fill and colluvium soils (recent deposits) throughout much of the planned excavation depth. An easement for temporary ground anchors may be required for shoring walls adjacent to other private properties.
- Due to the steep slope along the western property line deep excavations at the site will require significant additional costs for retaining structures. Designing the structure to bear at or near existing grades will reduce the cost of slope stabilization.
- Given the presence of the fill and recent deposits at the site, the design lateral earth pressures for portions of the shoring wall will be higher than typical values.
- The permanent below-grade wall and the structural walls/foundation elements must be designed to resist the unbalanced lateral earth pressures resulting from the significant grade change across the site.
- We understand that the lowest finished floor will be at Elevation 101.25 feet (NAVD 88 datum) and anticipate that foundations will bear 2 to 3 feet below this elevation. Accordingly, it is our opinion the planned building can be supported on shallow foundations or mat foundations. Allowable soil bearing pressures are anticipated to vary throughout the site because of the variable soil conditions. Some areas will likely have foundations bearing on competent glacially consolidated soils or structural fill that is extended to glacially consolidated soil, where this is feasible. For this condition, an allowable soil bearing pressure of 8 kips per square foot (ksf) can be assumed. Based on the current building layout, assumed foundation bearing elevation, and the boring information we estimate an allowable soil bearing pressure of 8 ksf can be used over approximately the western third of the building footprint. In other areas of the site shallow foundation excavations located near existing grades will likely bear in less competent recent deposits. We recommend shallow foundations *NOT* bear directly on the recent deposits. We further recommend that recent deposits present at planned foundation grade be over excavated at least 2 feet and replaced with compacted structural fill. For this condition, an allowable soil bearing pressure of 4 ksf can be assumed. Specific and detailed foundation plans are not presently available at this time, in addition we expect variable soil conditions across the site. Accordingly, we recommend that GeoEngineers be provided the opportunity to review the foundation layout and elevations and recommend changes or modifications as necessary and appropriate to make sure the intent of our recommendations has been properly interpreted and included in the structural foundation plans.
- Conventional slabs-on-grade are appropriate for this site and should be underlain by a 6-inch-thick layer of clean, crushed rock. A separation geotextile should be placed between the prepared subgrade and

capillary rock layer. In areas where soft fill soils and/or recent deposits are present, localized removal of soft subgrade soils followed by replacement with properly compacted structural fill up to 1-foot-thick may be required. The need and extent of removal and replacement will be determined during subgrade preparation based on proof-rolling or probing.

- Below grade drainage must be provided between temporary and permanent walls and below the slabs-on-grade.
- Depending on the location of the planned building, new retaining walls may be required along the west side of the project to retain the existing slopes and provide catchment for potential surficial slope instability in steeply inclined areas. The soldier piles used for temporary shoring may be an appropriate wall type for this condition. The size and extent of such a retaining wall should be reviewed once the development plan is further defined.

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

Geologic Hazard Evaluation

GeoEngineers has reviewed the City of Mercer Island's geologic hazard area ordinance (Title 19.07.060 of the City of Mercer Island City Code) and maps available online through the King County and City of Mercer Island geographic information system (GIS) websites. Based on our review of the GIS maps, the site is located within seismic, landslide and erosion hazard zones. We also reviewed LiDAR hill shade imagery available online through King County. The LiDAR imagery was used as a tool to aid in our interpretation of local and regional geomorphic features.

Seismic Hazard

The site is mapped in a seismic hazard area, which is an area identified as subject to severe risk of damage from earthquake induced motion. We evaluated seismic hazards including liquefaction, lateral spreading, fault rupture and earthquake induced slope instability. Additional discussion is presented in the section titled "Earthquake Engineering". Our analysis indicates that the soils that underlie the site have a low risk of liquefying because of their fine grained character and the absence of a shallow groundwater table. Due to the lack of liquefiable soils, the site also has low risk of liquefaction-induced lateral spreading. Based on United States Geologic Survey (USGS) maps of active faults in the Puget Sound region, the site is located within the Seattle Fault Zone, which is thought to have a recurrence interval of greater than 1,000 years. In our opinion, there is a relatively low risk of surface fault rupture because of the thickness of glacially consolidated deposits overlying bedrock. The site does have a moderate risk of seismically-induced slope movement; however, the risk of seismic induced slope instability will be mitigated by the planned structure. Site retaining walls may be designed for some surficial sloughing on the adjacent slopes or for debris catchment.

Landslide Hazard

The site is mapped in an area potentially subject to landslide occurrence. Two identified landslides have historically occurred on the downslope side of the property near the northeast property corner. With improvements made to the adjacent properties at 7525 SE 24th Street and 2431 76th Avenue, the risk of a downslope failure is very low. No known landslides have occurred upslope (west and south sides of the site) within or on neighboring properties to the site; however, scarp features were identified, as described and discussed in the "Geologic Reconnaissance" section of this report.

The planned development will not be constructed on top of the slope, but excavated into the slope. The planned development will include shoring consisting of soldier pile walls with ground anchors for support, and the building will be designed to resist the lateral soil loads on a permanent basis. Both the shoring and the permanent below-grade building walls will be designed to result in an adequate factor of safety for slope stability. The proposed development will not negatively impact the stability of the slope, provided that the recommendations discussed in this geotechnical report are implemented.

Mitigation measures to control surface water runoff from adjacent upslope properties and SE 74th Street may be required.

Erosion Hazard

Critical areas maps indicate that the site is also within an erosion hazard area. These areas are underlain by soils that may be subject to severe erosion, if exposed. Clearing and grading is typically regulated in these areas, with requirements for erosion control measures. Specific restrictions depend on the size and nature of the project and grading. Provided our design recommendations are followed and our recommendations for best management practices (BMPs) to limit erosion are implemented during construction, it is our opinion that the proposed improvements will not adversely impact the erosion hazard in this area.

Earthquake Engineering

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.

Our analysis indicates that the soils that underlie the proposed building area have a low risk of liquefying because of their predominantly fine grained character and the absence of a shallow groundwater table.

Other Seismic Hazards

The site is located near the Seattle Fault Zone. Based on the location of the site and the site topography, the risk of adverse impacts resulting from differential settlement, surface displacement due to faulting, or lateral spreading is considered to be low. The risk of seismic induced slope instability will be mitigated by the planned structure. The static and seismic earth pressures required to be resisted by the planned structure are presented below.

2012 IBC Seismic Design Information

We recommend the use of the following 2012 IBC parameters for soil profile type, short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1) and seismic coefficients (F_A and F_V) for the project site.

2012 IBC Parameter	Recommended Value
Soil Profile Type	D
Short Period Spectral Response Acceleration, S_S (percent g)	137
One-second Period Spectral Response Acceleration, S_1 percent g)	53
Seismic Coefficient, F_A	1.0
Seismic Coefficient, F_V	1.5

Temporary Dewatering

The planned excavation will likely encounter zones of perched groundwater and temporary dewatering should be anticipated. Temporary dewatering may be accomplished using a variety of means; however, the use of submersible pumps located around the perimeter of the excavation is anticipated for this site.

Excavation Support

Based on current development concepts for a new one-level below grade building, we anticipate excavation depths could range up to approximately 20 feet on the western side of the property. Soldier pile and tieback shoring is the preferred excavation support system for the site because of the depth of the planned excavation, the extent of fill and recent deposits across the site, and better deflection control of this system for deep excavations. It is our opinion that soil nails are not a suitable option for this project.

The shoring walls will be partially within fill, recent deposits and glacially consolidated soils. Due to the presence of the fill and recent deposits, lateral earth pressures for temporary shoring and the permanent building walls will be higher than walls constructed fully in glacially consolidated soils.

The shoring system should be designed to limit lateral deflection to less than 1 inch in order to reduce the risk of damage to existing improvements. The City of Mercer Island may require that remedial measures be implemented if lateral deflections reach one inch.

Coordination will be required in those areas with overhead power located in the vicinity to allow for shoring construction. Portions of the shoring system will be required to be temporary if tiebacks extend into right-of-way and a street use permit will be required. Additionally, easements will be required for shoring extending onto adjacent properties to the north, west and south.

We provide preliminary geotechnical design and construction recommendations for soldier pile and tieback walls below. We recommend that GeoEngineers have an opportunity to review shoring design completed by others.

Excavation Considerations

The 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, the surficial fill may contain foundation elements and/or utilities from previous site development, debris, rubble and/or cobbles and boulders. We recommend that procedures be identified in the project specifications for measurement and payment for work associated with these potential obstacles.

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 eight 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 tremie or pressure grouted. Timber lagging is typically installed behind the flanges of the steel beams to retain the soil located between the soldier piles. Geotechnical design recommendations for each of these components of the soldier pile and tieback wall system are presented in the following sections.

Soldier Piles

We recommend that soldier pile walls be designed using the earth pressure diagram presented in Figure 5. The earth pressures presented in Figure 5 are for full-height cantilever soldier pile walls and soldier pile walls with single or multiple levels of tiebacks, and the pressures represent the estimated loads that will be applied to the wall system for various wall heights.

The earth pressures presented in Figure 5 include the loading from traffic surcharge. Recommended surcharge pressures for design of the shoring walls are presented in Figure 6. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be considered by GeoEngineers on a case-by-case basis. No seismic pressures have been included in Figure 5 because it is assumed that the shoring will be temporary. If permanent walls are planned, GeoEngineers will provide seismic pressures on a case-by-case basis.

We recommend that the embedded portion of the soldier piles be at least two feet in diameter and extend a minimum distance of 10 feet below the base of the excavation to resist "kick-out" for the north, south, and west walls. We further recommend that the soldier piles along the west wall be embedded a minimum of distance of 10 feet below the base of the excavation or 5 feet into the glacially consolidated soils, whichever is greater. 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 10 ksf for piles supported on the 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.

Lagging

We recommend that the temporary timber lagging be sized using the procedures outlined in the Federal Highway Administration's Geotechnical Engineering Circular No. 4. The site soils are best described as competent soils. The following table presents recommend 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
25 to 35	3 inches	3 inches	3 inches	4 inches	4 inches	5 inches

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

The space behind the lagging should be filled with soil as soon as practicable. GeoEngineers recommends that voids 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 located behind the wall.

Material used as backfill in voids located behind the lagging must not cause buildup of hydrostatic pressure behind the wall. Lean concrete is a suitable option for the use of backfill behind the walls. Lean concrete will reduce the volume of voids present behind the wall. Alternatively, lean concrete may be used for backfill behind the upper 10 to 15 feet of the excavation to limit caving and sloughing of the upper soils, with on-site soils used to backfill the voids for the remainder of the excavation. Based on our experience, the voids between each lean concrete lift are sufficient for preventing the buildup of hydrostatic pressure behind the wall.

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 5) and within a stable soil mass (glacially consolidated soils). The anchors should be inclined downward at 15 to 45 degrees below the horizontal. Corrosion protection will not be required for the temporary tiebacks. Permanent tiebacks will be required to have double corrosion protection.

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

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

Tieback anchors should develop anchorage in the glacially consolidated soils. We recommend that spacing between tiebacks be at least three times the diameter of the anchor hole to minimize group interaction. We recommend a preliminary design load transfer value between the anchor and soil of 2 kips per foot for glacially consolidated soils. Higher adhesion values may be developed, depending on the anchor installation technique. The contractor should be given the opportunity to use higher adhesion values by conducting performance tests prior to the start of installing the production tieback anchors.

The 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, Ground Anchor Load Tests and Shoring Monitoring Program.

The tieback layout and inclination should be checked to confirm that the tiebacks do not interfere with adjacent buried utilities. Minimum clearances between ground anchors and existing utilities should be maintained, which is typically about 3 feet.

Drainage

A suitable drainage system should be installed to prevent the buildup of hydrostatic groundwater pressures behind the soldier pile and lagging wall. It may be necessary to cut weep holes through the lagging in wet areas. Seepage flows at the bottom of the excavation should be contained and controlled. Drainage should be provided for permanent below-grade walls as described below in the "Below-Grade Walls" section of this report.

Construction Considerations

Temporary casing or drilling fluid may be required to install the soldier piles and possibly the 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.

Post-grouting of tieback anchors installed in clay soils present in the recent deposits and glacially consolidated soils may result in ground heave or increased deformation of the shoring wall. It may be necessary to reduce the volume of post-grouting or grouting pressures, or to use alternative grouting techniques if shoring deformations exceed tolerable limits.

We recommend GeoEngineers observe and document the installation and testing of the shoring to verify conformance with the design assumptions and recommendations.

Shallow Foundations

It is our understanding that the lowest finished floor will be at Elevation 101.25 feet. We anticipate that foundations will bear 2 to 3 feet below this elevation, accordingly, we recommend that the planned building be supported on conventional spread footings or mat foundations. Variable bearing conditions are anticipated given the change in grades present on the site and the soils encountered in the borings. Some footing excavations will likely bottom in the recent deposits and will require some over excavation and replacement with structural fill. Other footings may bear directly on competent glacially consolidated soils, or on structural fill that extends to competent glacial soils. For design purposes, spread footings designed for variable bearing pressures ranging from 4 to 8 ksf can be assumed.

Allowable Soil Bearing Pressure

In some areas it may be possible for foundations to bear directly on competent glacially consolidated soil or on structural fill that extends to glacially consolidated soil. For this condition, an allowable soil bearing pressure of 8 ksf can be assumed. Based on the current building layout, assumed foundation bearing elevation, and the boring information we estimate an allowable soil bearing pressure of 8 ksf can be used over approximately the western third of the building footprint. In other areas of the site shallow foundation excavations located near existing grades will likely bottom in less competent recent deposits. In this situation we recommend over excavation to at least 2 feet below foundation grade and replacement with compacted structural fill. We further recommend that over excavation and replacement with structural fill extend horizontally for at least 2 feet beyond the footing perimeter, measured at the bottom of the over excavation. For this condition, an allowable soil bearing pressure of 4 ksf can be assumed. Because of the variability in soil conditions and because the depth of the foundations are unknown at this time we recommend that GeoEngineers be provided the opportunity to review the foundation layout and elevations and recommend modifications as necessary and appropriate to meet the intent of our recommendations. The allowable soil bearing pressures apply to the total of dead and long-term live loads and may be increased by up to one-third for wind and seismic loads.

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 shallow foundations will be about 1 inch or less. The settlements will occur rapidly, essentially as loads are applied. Differential settlements between footings could be half of the total settlement. Note that smaller settlements will result from lower applied loads.

Size and Embedment

We recommend that the exterior footings be founded a minimum of 18 inches below the lowest adjacent grade. Interior footings should be founded a minimum of 12 inches below top of slab. Continuous wall footings and individual column footings should have minimum widths of 24 inches.

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 glacially consolidated soils, the allowable frictional resistance may be computed using a coefficient of friction of 0.35 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 350 pounds per cubic foot (pcf) (triangular distribution). This value is 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.

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.

If foundation construction is completed during periods of wet weather, we recommend foundation bearing surfaces be protected with a rat slab consisting of 2 to 4 inches of lean or structural concrete.

If soft areas are present at the footing bearing surface elevation, the soft areas must be removed and replaced with lean concrete or structural fill at the direction of GeoEngineers. In such instances, the zone of structural fill must extend laterally beyond the footing edges a horizontal distance at least equal to the thickness of the fill. Where lean concrete is used, the zone of lean concrete may be limited to the foundation footprint.

Slab-on-Grade Floors

The following sections provide design recommendations for subgrade preparation, slab-on-grade design parameters, and below slab drainage.

Subgrade Preparation

The exposed subgrade should be evaluated after site grading is complete. Proof-rolling with heavy, rubber-tired construction equipment should be used for this purpose during dry weather and if access for this equipment is practical. Probing should be used to evaluate the subgrade during periods of wet weather or if access is not feasible for construction equipment. The exposed soil must be firm and unyielding, and without standing water. Loose or disturbed soil must be removed and replaced with compacted structural fill.

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 either undisturbed glacially consolidated soils or on structural fill placed over the undisturbed glacially consolidated soils. Where soft fill or recent deposit soils are present at the slab-on-grade subgrade elevation, we recommend the upper 12 inches of the fill/recent deposits be removed and replaced with structural fill. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 125 pounds per cubic inch (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 Section 9-03.1(4)C, grading No. 57 of the 2012 Washington State Department of Transportation (WSDOT) Standard Specifications. Additionally, we recommend that a separator geotextile, such as a Mirafi 140N or equivalent, be placed over the prepared subgrade prior to placement of the capillary break.

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

Below-Slab Drainage

We recommend installing an underslab drainage system to remove water from below the slab-on-grade in case perched water is encountered below the slab.

The underslab drainage system should include an interior perimeter drain. 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 WSDOT gravel backfill for drains Section 9-03.12(4), or an alternative approved by GeoEngineers. The granular drainage material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, WSDOT Standard Specification 9-33. The 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 10 gallons per minute (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.

Below-Grade Walls

Permanent Subsurface Walls

Permanent subsurface walls constructed adjacent to temporary shoring walls should be designed using the earth pressure diagram presented in Figure 5. These pressures are consistent with the pressures used for design of the temporary shoring system. In addition to the static earth pressures presented in Figure 5, a rectangular seismic earth pressure equal to 8H pounds per square foot (psf) should be included where glacially consolidated soils are present and a rectangular seismic earth pressure equal to 16H psf should be specified where fill and/or recent deposits are present.

Recommended surcharge pressures for design of below grade walls are presented in Figure 6. Other surcharge loads, such as from foundations, construction equipment or construction staging areas, should be considered on a case-by-case basis.

The soil pressures recommended in Figure 5 assume that wall drainage will be installed to prevent the potential buildup of hydrostatic pressure behind the permanent subsurface walls. Prefabricated drainage board should be installed between the temporary shoring wall and the permanent subsurface walls and should extend to the base of the wall.

Other Cast-in-Place Walls

Conventional cast-in-place walls may be necessary for small 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. Additionally, the owner may want to consider catchment walls where structures or courtyards are adjacent to steep slopes. GeoEngineers can provide recommendations for catchment walls upon request.

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), and that 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 8H psf, where H is the height of the wall, should be included where glacially consolidated soils are present and a rectangular seismic earth pressure equal to 16H psf should be specified where fill and/or recent deposits are present. 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 in accordance with the "Lateral Resistance" discussion earlier in this report.

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

Drainage

Drainage behind the permanent below-grade walls constructed adjacent to temporary shoring walls is recommended to consist of drainage placed between the temporary soldier pile shoring wall and the permanent below grade wall. The drainage material should be connected to weep pipes that extend through the permanent below grade building walls at the footing elevation. The weep pipes through the permanent below grade wall should be spaced no more than 16 feet on center and should be hydraulically connected to the sump. These weep pipes may be designed for a hard connection to the perimeter drains discussed above in the "Below-Slab Drainage" section of this report.

Prefabricated geocomposite drainage material, such as Mirafi G100™, should be used where drainage material is required as full coverage drainage panels located between the temporary shoring wall and the permanent below grade walls. The drainage material should be installed on the excavation side of the temporary shoring wall with the fabric adjacent to the temporary shoring wall.

In areas where temporary cut slopes are used and conventional cast-in-place techniques are used to build the below grade walls, a conventional footing drain should be located on the outside of the building. The footing drain should be constructed consistent to drains recommended for cast-in-place walls, below.

Positive drainage should also be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of WSDOT gravel backfill for walls Section 9-03.12(2), with the exception that the percent passing the U.S. No. 200 sieve is to be less than 3 percent. A perforated or slotted 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 WSDOT gravel backfill for drains Section 9-03.12(4), or an alternative approved by GeoEngineers. The granular drainage material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, WSDOT Standard Specification 9-33. The wall drainpipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger-diameter pipe will allow for easier maintenance of drainage systems.

Earthwork

Structural Fill

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

- If structural fill is necessary beneath building foundations, the fill should consist of controlled density fill (CDF), structural concrete, or fill meeting the requirements of WSDOT gravel backfill for foundations Section 9-03.12(1)B.
- Structural fill placed as capillary break material should meet the requirements of Section 9-03.1(4)C, grading No. 57 of the 2012 WSDOT Standard Specifications.
- Structural fill placed behind retaining walls should meet the requirements of WSDOT gravel backfill for walls Section 9-03.12(2).
- Structural fill placed around perimeter footing drains, underslab drains and cast-in-place wall drains should meet the requirements of WSDOT gravel backfill for drains Section 9-03.12(4).
- Structural fill placed within utility trenches and below pavement and sidewalk areas should meet the requirements of WSDOT common borrow as described in Section 9-03.14(3). Common borrow is only suitable for use during dry weather. If fill is placed during wet weather, WSDOT gravel borrow should be used, as described in Section 9-03.14(1).
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Section 9-03.9(3) of the 2012 WSDOT Standard Specifications.

On-site Soils

The on-site soils are highly moisture-sensitive and generally have natural moisture contents higher than the anticipated optimum moisture content for compaction. As a result, the on-site soils will likely require moisture conditioning in order to meet the required compaction criteria during dry weather conditions and will not be suitable for reuse during wet weather. Furthermore, most of the fill soils required for 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. Cement treatment of on-site soils is an option for foundation backfill material below conventional slabs-on-grade. GeoEngineers can provide further guidance for the use of cement treated soils, as necessary.

Fill Placement and Compaction Criteria

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

- Structural fill placed in building areas (around foundations or below slab-on-grade floors) and in 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 subgrade walls should be compacted to between 90 and 92 percent. Care must be taken when compacting fill against subsurface walls to avoid over-compaction which could overstress the walls.

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

Weather Considerations

During wet weather, some of the exposed soils could become muddy and unstable. If so affected, we recommend that:

- The ground surface in and around the work area be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop.
- Slopes with exposed soils be covered with plastic sheeting or similar means.
- The site soils 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 activities be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

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 and colluvium/recent soil deposits be inclined at 1½H:1V 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 the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

Recommended Additional Geotechnical Services

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

During construction, GeoEngineers should observe the installation of the shoring system, review/collect shoring monitoring data, evaluate the suitability of the foundation bearing surfaces and slab 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.

LIMITATIONS

We have prepared this report for the exclusive use of Aegis Senior Communities, LLC and their authorized agents for the Aegis Mercer Island 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. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments should be considered a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to the Appendix E, Report Limitations and Guidelines for Use, for additional information pertaining to use of this report.

REFERENCES

City of Mercer Island, Information and Geographic Services, http://pubmaps.mercergov.org/SilverlightViewer/Viewer.html?ViewerConfig=http://pubmaps.mercergov.org/Geocortex/Essentials/REST/sites/MercerIslandPublicViewer/viewers/MercerIslandPublicViewer/virtualdirectory/Config/Viewer.xml, accessed on March 2015.

King County, Interactive Mapping Project, http://www5.kingcounty.gov/iMAP/viewer.htm?mapset=kcproperty, accessed on March 2015.

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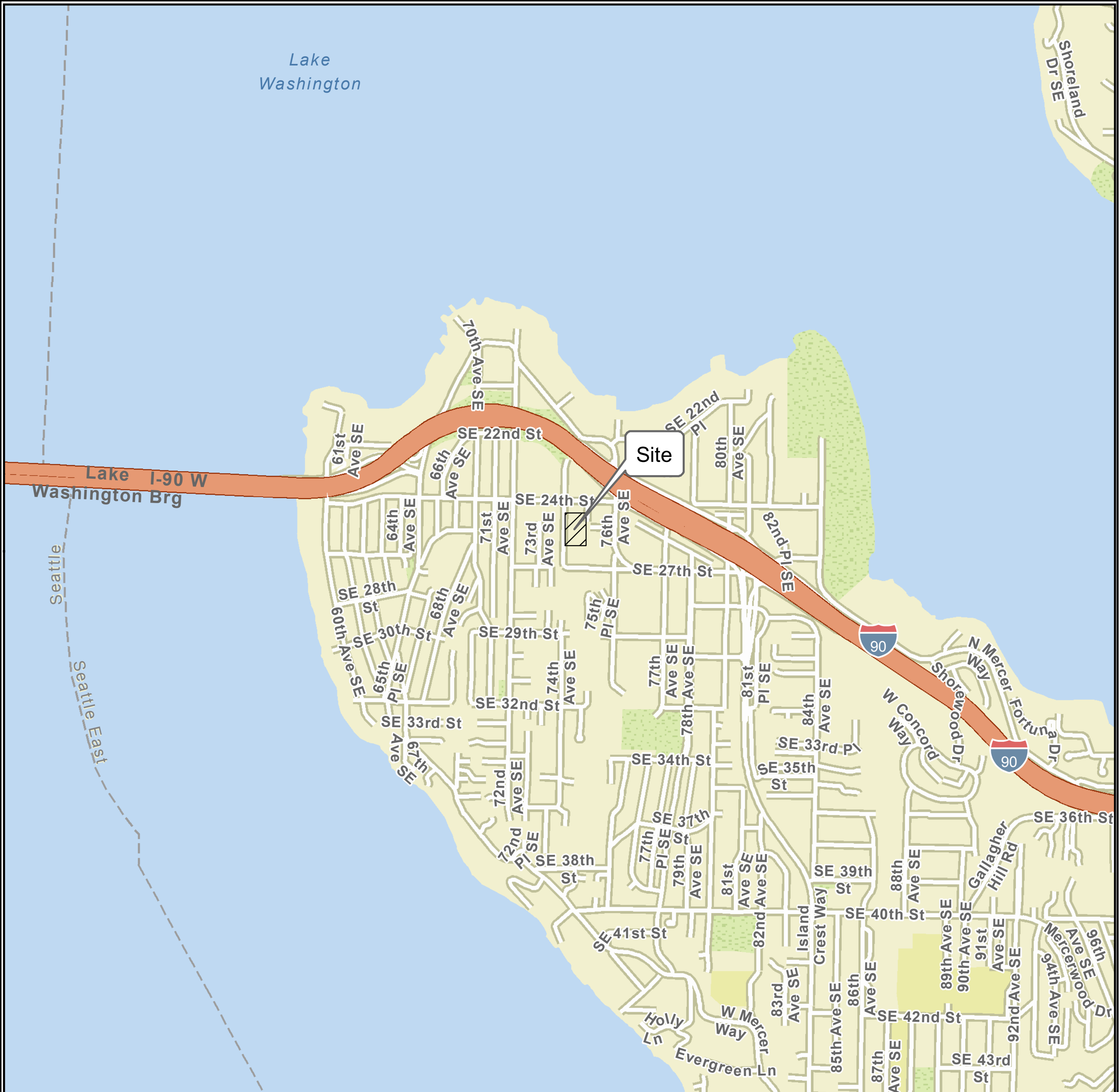
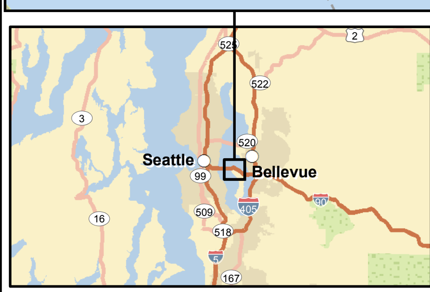
U.S. Department of Transportation, Federal Highways Administration, 1999, “Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems,” FHWA Report No. FHWA-IF-99-015.

U.S. Geological Survey – National Seismic hazard Mapping project Software “Earthquake Ground Motion Parameters, Version 5.0.9a,” 2002 data, 2009.

Washington State Department of Transportation, 2014, “Standard Specifications for Road, Bridge and Municipal Construction.”

FIGURES





N
W E
S

2,000 0 2,000

Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

Data Sources: ESRI Data & Maps, Street Maps 2005

Transverse Mercator, Zone 10 N North, North American Datum 1983
North arrow oriented to grid north

Vicinity Map

Aegis Mercer Island
Mercer Island, Washington

GeoEngineers

Figure 1

Office: Redmond Path: \\red\projects\19\19811009\GIS\1981100900_F1_VicinityMap.mxd Map Revised: 3/23/2015 EL

Legend

GEI-1 Approximate Location Boring by GeoEngineers (2015)

B-1 Approximate Location of Boring by Earth Consultants, Inc. (1985)

TP-1 Approximate Location of Test Pit by Earth Consultants, Inc. (1985)

A A' Cross-Section Location

N E S W

60 0 60

Feet

Notes

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

Reference: Base Topographic Site Survey by ALTA/ACSM Land Title dated 10/29/14.

Site Plan

Aegis Mercer Island
Mercer Island, Washington

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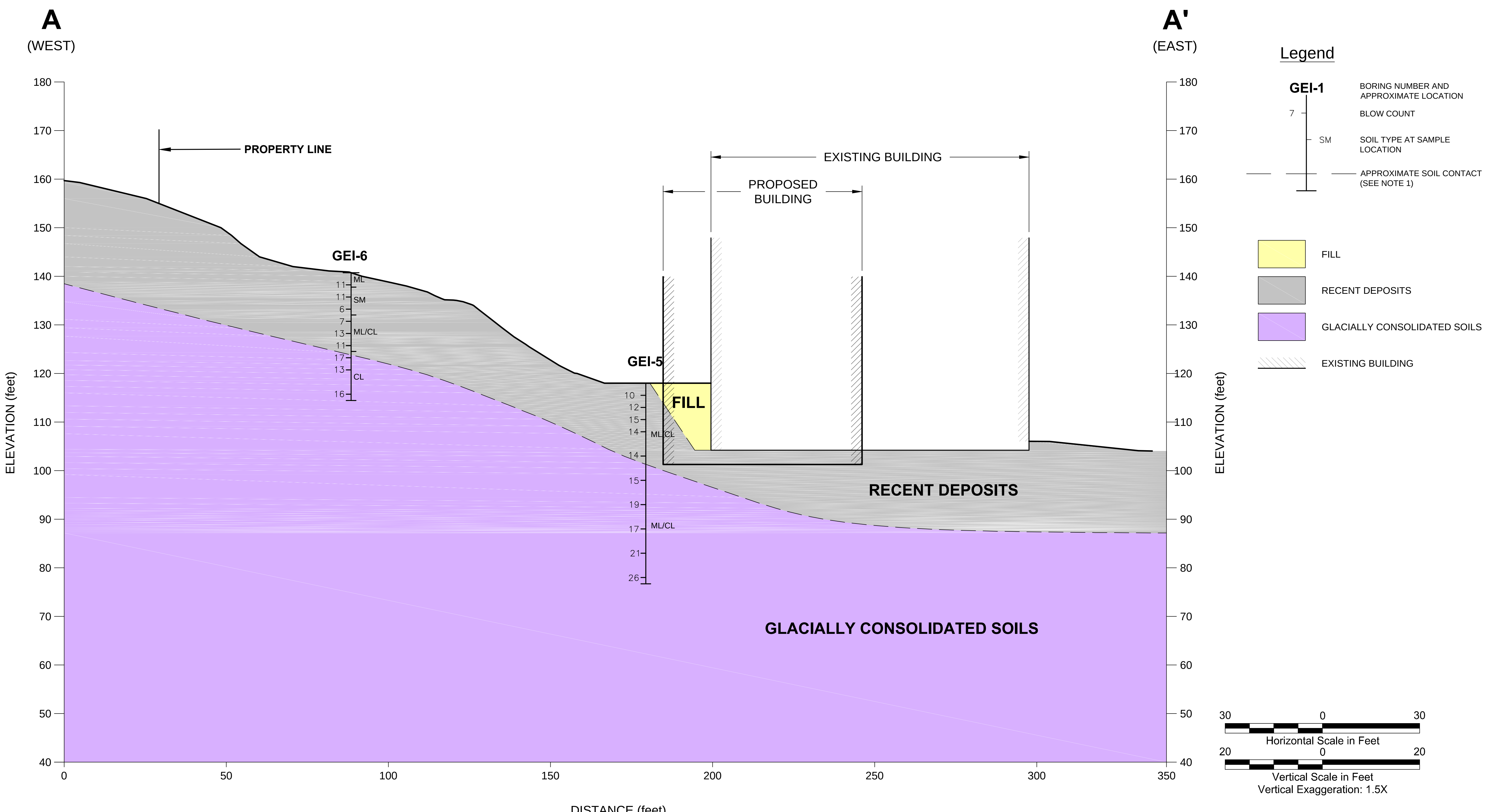
Figure 2



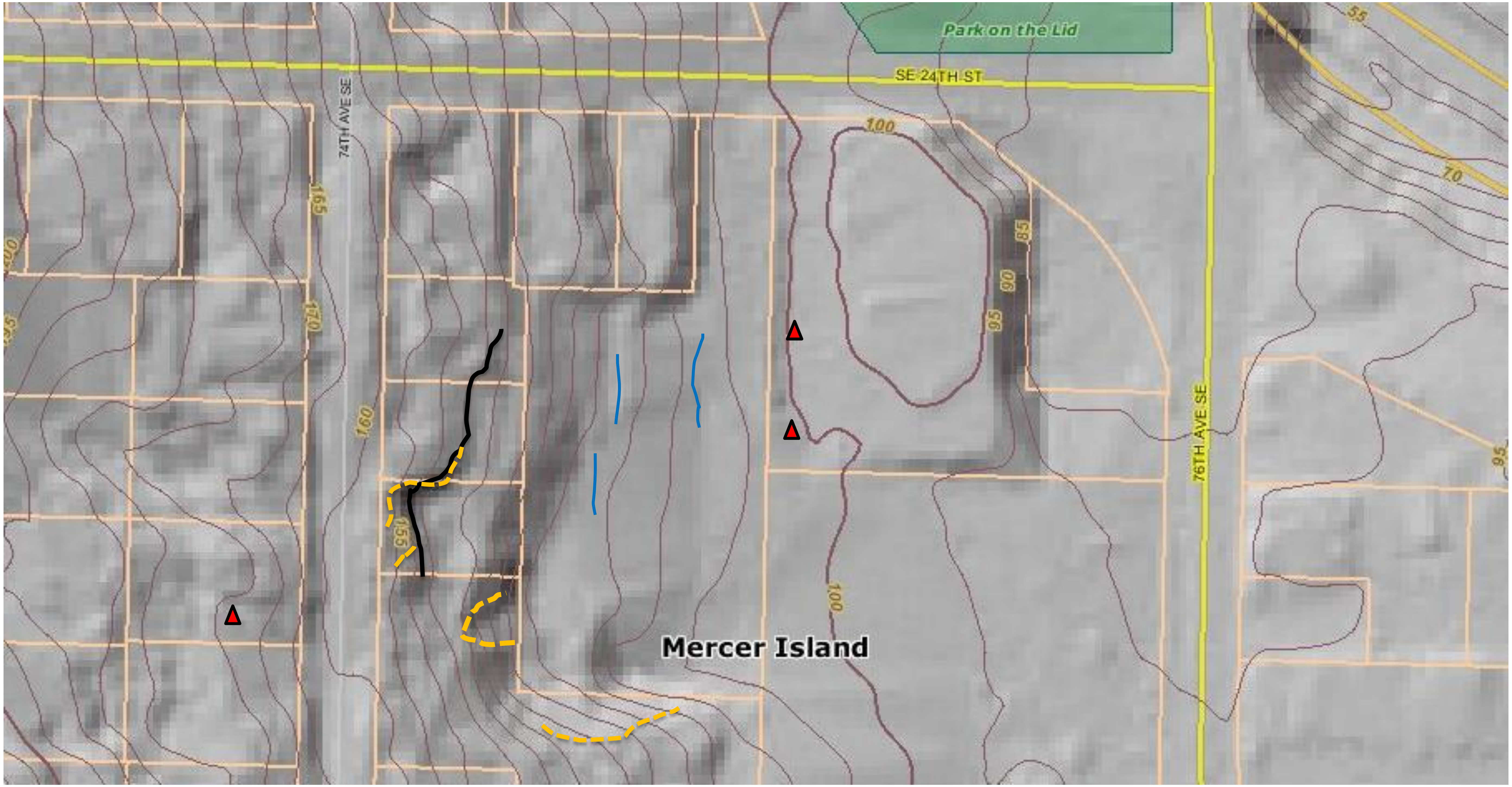
Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 2 for location of Cross Section.
3. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The master hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Cross-Section A-A'

Aegis Mercer Island
Mercer Island, Washington

GEOENGINEERS

Figure 3

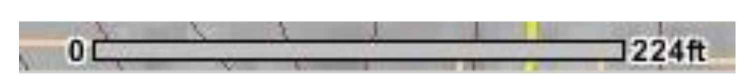


Notes:

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 the Geological Hazards section of this report. 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.

Legend:

- Mercer Island identified landslide
- Mercer Island mapped scarp
- Potential scarp (GeoEngineers 2015)
- Cracked pavement (GeoEngineers 2015)



Geologic Site Features

Aegis Mercer Island
Mercer Island, Washington

GEOENGINEERS

Figure 4

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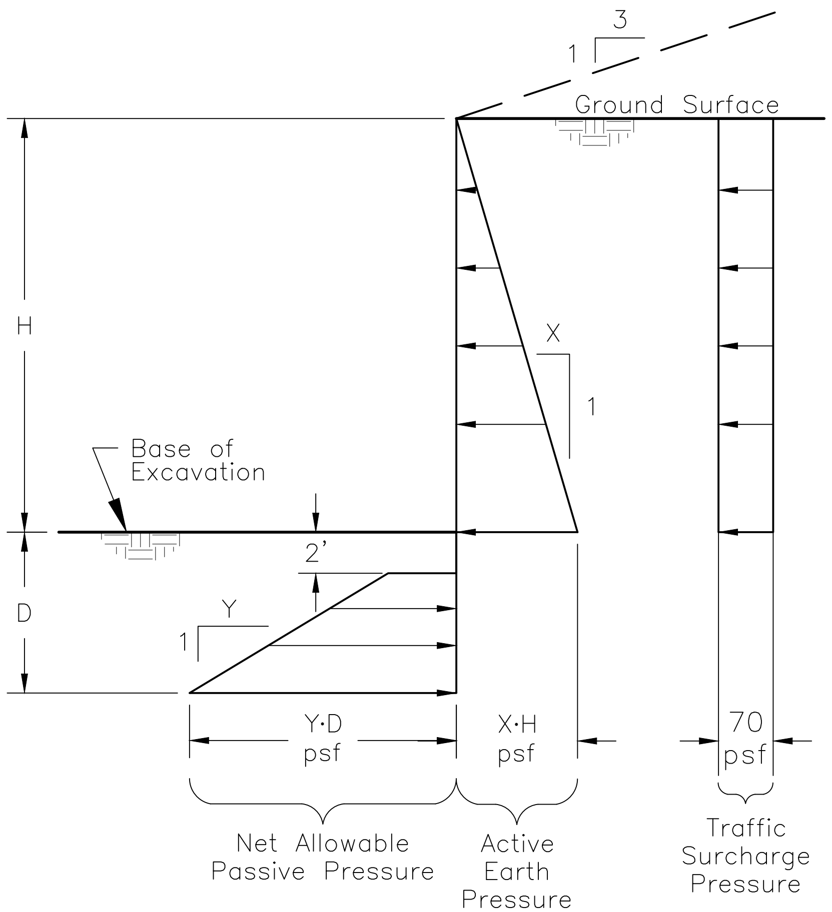
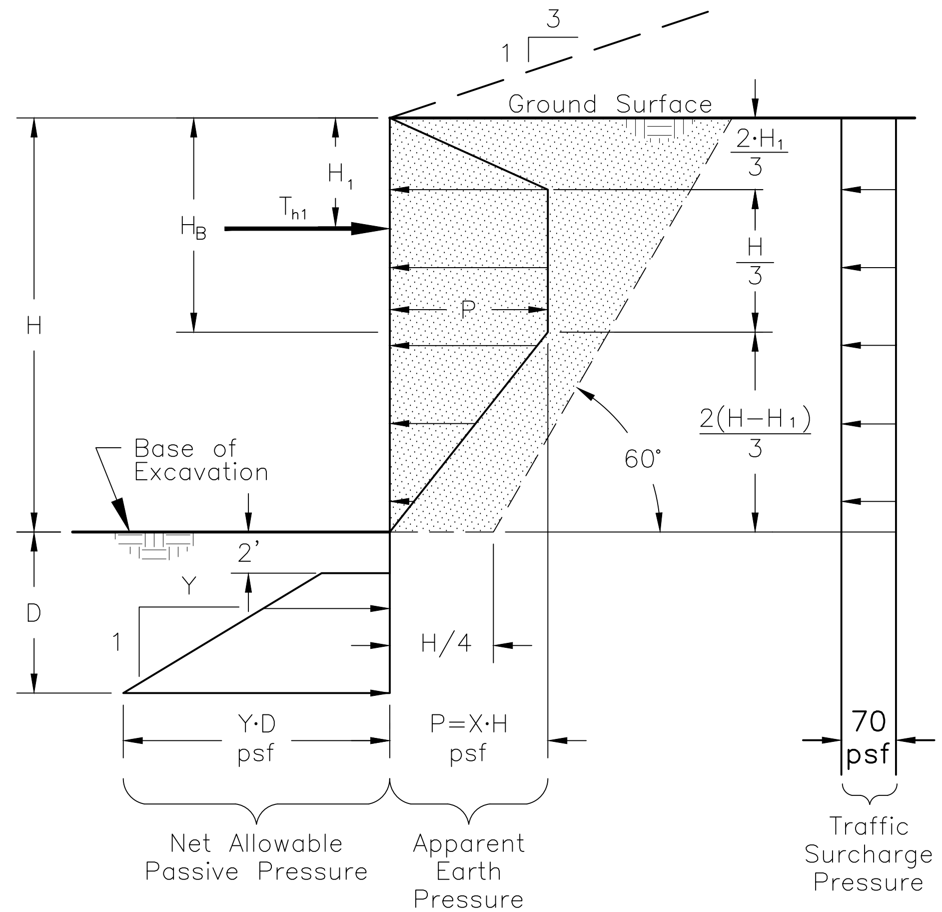
CANTILEVER SOLDIER PILE



West Wall*		
Soil Type	X	Y
Fill / Recent Deposits	60	200
Glacially Consolidated Soil	40	350

*Assumes 3H:1V Back Slope

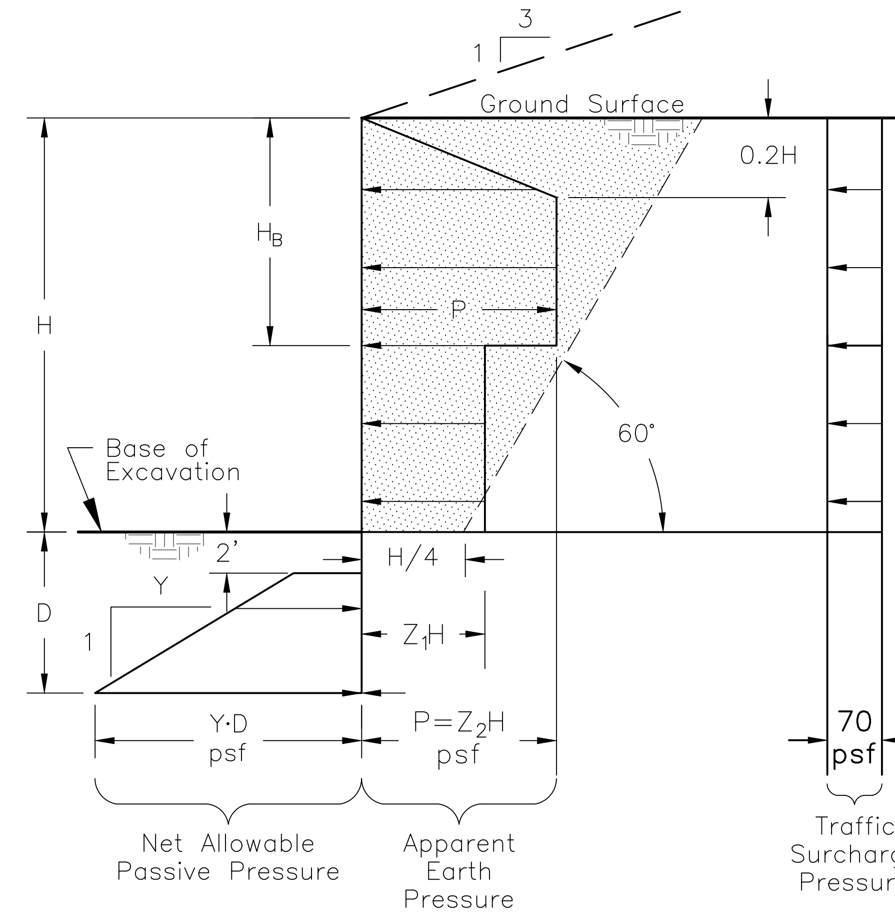
SOLDIER PILE WALL WITH ONE LEVEL OF GROUND ANCHORS



North, South, and East Wall*		
Soil Type	X	Y
Fill / Recent Deposits	46	200
Glacially Consolidated Soil	35	350

*Assumes Level Back

SOLDIER PILE WALL WITH MULTIPLE LEVELS OF GROUND ANCHORS



	Level Back Slope	3H:1V
Z_1	25	31
Z_2	35	45

Legend

No Load Zone

- H = Height of Excavation, Feet
- D = Soldier Pile Embedment Depth, Feet
- H_1 = Distance From Ground Surface to Uppermost Ground Anchor, Feet
- H_B = Depth to Glacially Consolidated Soils
- T_{h1} = Horizontal Load in Uppermost Ground Anchor
- P = Maximum Apparent Earth Pressure Pounds per Square Foot

Notes:

1. Apparent earth pressure and surcharge act over the pile spacing above the base of the excavation.
2. Passive earth pressure acts over 2 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, passive pressure assumes a level foreslope.
4. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 6.
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.

NOT TO SCALE

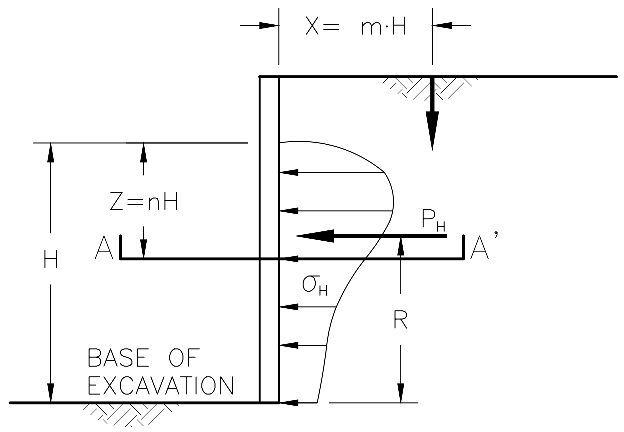
Earth Pressure Diagrams
Temporary Soldier Pile

Aegis Mercer Island
Mercer Island, Washington

GEOENGINEERS

Figure 5

LATERAL EARTH PRESSURE FROM POINT LOAD, Q_P (SPREAD FOOTING)



FOR m ≤ 0.4

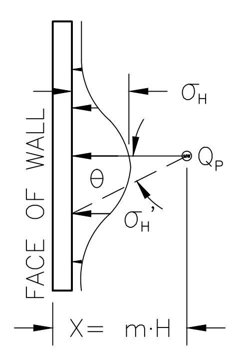
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2 (0.16+n^2)^3}$$

FOR m > 0.4

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

$$\sigma_H' = \sigma_H \cos^2(1.1\theta)$$

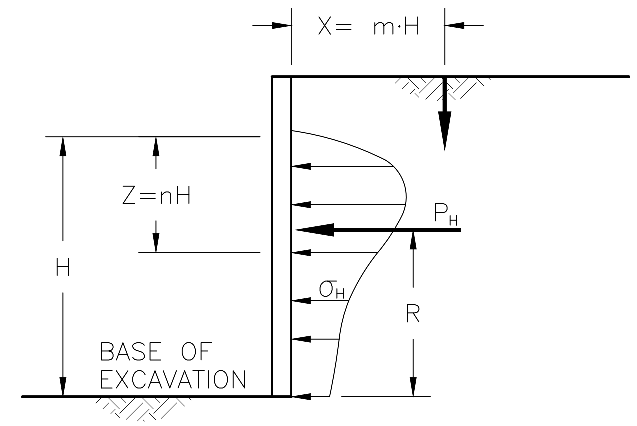
M	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



SECTION A–A'

Pressures from Point Load Q_P

LATERAL EARTH PRESSURE FROM LINE LOAD, Q_L (CONTINUOUS WALL FOOTING)



FOR m ≤ 0.4

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

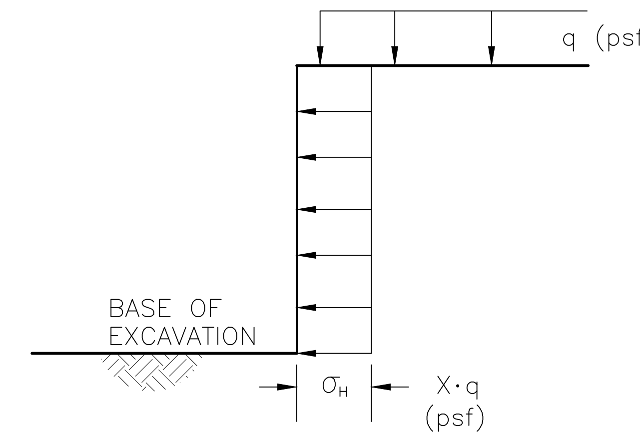
FOR m > 0.4

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

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

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

UNIFORM SURCHARGES, q (FLOOR LOADS, LARGE FOUNDATION ELEMENTS OR TRAFFIC LOADS)



σ_H = LATERAL SURCHARGE PRESSURE FROM UNIFORM SURCHARGE

X = 0.28 FOR GLACIALLY CONSOLIDATED SOIL OR 0.39 FOR FILL / RECENT DEPOSITS; ASSUMES LEVEL BACKSLOPE

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

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 5.
3. See report text for where surcharge pressures are appropriate.

Recommended Surcharge Pressure

Aegis Mercer Island
Mercer Island, Washington

GEOENGINEERS

Figure 6

APPENDICES

GeoEngineers

APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

General

Subsurface conditions were explored at the site by drilling six borings (GEI-1 through GEI-6). The borings were completed to depths ranging from about 21.5 to 41.5 feet below the existing ground surface. Borings GEI-1 through GEI-6 were completed by Geologic Drill Exploration, Inc. between January 22 and January 23, 2015.

The locations of the explorations were estimated by taping/pacing from existing site features. The approximate location of each exploration is shown on the Site Plan, Figure 2.

Borings

Borings were completed using trailer-mounted, continuous-flight, hollow-stem auger drilling equipment. The borings were continuously monitored by a geologist 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 inches into the soil with a 140-pound rope and cathead 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 final 12 inches of penetration. 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 the full 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.

Soil encountered in each boring was visually classified in general accordance with the classification system described in Figure A-1. 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 to A-7. The boring logs 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. 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.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING 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 GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

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 or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

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.

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

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PP	Pocket penetrometer
PPM	Parts per million
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
NT	Not Tested

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 1/22/2015	End 1/22/2015	Total Depth (ft)	21.5	Logged By BHC / Checked By DPC	Driller	Geologic Drill, Inc.	Drilling Method	Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	106.5 / NAVD88	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop
Drilling Equipment	Bobcat		
Easting (X) / Northing (Y)		System / Datum	
Groundwater: Date Measured	Depth to Water (ft)	Elevation (ft)	
	Not encountered		

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Interval / Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	6 inches asphalt concrete pavement			
105								CR	18 inches crushed rock base course			
		18	12		1			CH	Brown fat clay with sand (stiff, moist) (recent deposits)			Undisturbed bedding
100	5	12	9		2							
		18	16		3							
95	10	18	12		4 / AL					41		AL (LL = 90; PI = 51) Dark oxidation staining along shear lines
90	15	18	21		5			MH/CH	Gray silt to clay with sand (very stiff, moist) (glacially consolidated soils)			Massive fabric
85	20	18	21		6				Grades to silt with trace sand			Massive fabric

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

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Log of Boring GEI-1

GEOENGINEERS

Project: Aegis Mercer Island
Project Location: Mercer Island, Washington
Project Number: 19811-009-00

Figure A-2
Sheet 1 of 1

Drilled	Start 1/22/2015	End 1/22/2015	Total Depth (ft) 26.5	Logged By BHC / Checked By DPC	Driller Geologic Drill, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) 110 / Vertical Datum NAVD88	Hammer Data: Rope & Cathead, 140 (lbs) / 30 (in) Drop	Drilling Equipment: Bobcat	
Easting (X) / Northing (Y)	System Datum	Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	See remarks
Notes:			

Elevation (feet)	Depth (feet)	Interval / Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							CR	Crushed rock surfacing			Hard/rough drilling from surafce Obstruction at 1 foot
		18	5		1			MH	Light brownish gray silt (medium stiff, moist) (recent deposits)			Smoother drilling
												Disturbed fabric
105	5	18	5		2							
		18	13		3				Grades to very stiff			
									Grades to brown with sand, stiff			
100	10	18	10		4							
									Grades to moist to wet			
												Driller indicates layer of gravel at 13.5 feet
95	15	18	11		5 %F				Grades to wet	25	72	Perched groundwater encountered at 16 feet at time of drilling Oxidation staining
90	20	18	14		6							Disturbed soil fabric
								MH/CH	Brown silt or clay (very stiff, moist) (glacially consolidated soils)			
85	25	18	25		7							Massive fabric Fine sand partings

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

Log of Boring GEI-2

GeoEngineers

Project: Aegis Mercer Island
Project Location: Mercer Island, Washington
Project Number: 19811-009-00

Drilled	Start 1/23/2015	End 1/23/2015	Total Depth (ft) 21.5	Logged By ERH, Checked By DPC	Driller Geologic Drill, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) 118, Vertical Datum NAVD88	Hammer Data: Rope & Cathead, 140 (lbs) / 30 (in) Drop	Drilling Equipment: Bobcat	
Easting (X), Northing (Y)	System Datum	Groundwater — Date Measured / Depth to Water (ft) / Elevation (ft)	See remarks
Notes:			

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0					ML	Dark brown organic sandy silt (medium stiff) (fill)			
115		12	6	1						
						SM	Brown silty fine to coarse sand with gravel (loose to medium dense, moist)			Driller notes gravel
	5	8	12	2						Driller notes increased gravel Thin oxidized lens
110		10	18	3 %F				8	10	Thin oxidized lens
	10	12	13	4			Grades to fine to medium sand (wet)			Perched groundwater encountered at 10 feet at time of drilling
105		18	24	5		MH/CH	Gray silt or clay (very stiff to hard, moist) (glacially consolidated soils)			Driller notes silt sludge on rods
	15	18	26	6			Grades to sandy silt (moist to wet)			
100			29	7			Grades to silt with sand (moist)			Massive fabric
	20	18	27	8						Possible perched zone Massive fabric

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

Log of Boring GEI-3

GeoEngineers

Project: Aegis Mercer Island
Project Location: Mercer Island, Washington
Project Number: 19811-009-00

Figure A-4
Sheet 1 of 1

Drilled	Start	End	Total Depth (ft)	21.5	Logged By	ERH	Driller	Geologic Drill, Inc.	Drilling Method	Hollow-Stem Auger
	1/23/2015	1/23/2015			Checked By	DPC				

Surface Elevation (ft)	117.5	Hammer Data	Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment	Bobcat
Vertical Datum	NAVD88				
Easting (X)		System		Groundwater	
Northing (Y)		Datum		Date Measured / Depth to Water (ft) / Elevation (ft)	Not encountered
Notes:					

FIELD DATA

Elevation (feet)	Depth (feet)	Interval / Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							Forrest duff	Forest duff			
								SM	Dark brown silty fine to medium sand with organics (fill)			
115		18	18		1			ML/CL	Brown to dark brown silt and clay			
								ML	Dark brown sandy silt (medium stiff to stiff, moist)			
	5	18	8		2							Driller notes gravel
110		18	9		3 %F			ML	Orange-brown sandy silt with occasional gravel (stiff, moist to wet) (recent deposits)	23	64	
	10	18	33		4			CH	Orange-brown fat clay with occasional sand (very hard) (glacially consolidated soils)			
105		18	31		5							
	15	18	38		6				Grades to with sand			
100		8	11		7							
	20	18	10		8 AL					37		AL (LL = 60; PI = 29)

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

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Log of Boring GEI-4

GeoEngineers

Project: Aegis Mercer Island
Project Location: Mercer Island, Washington
Project Number: 19811-009-00

Figure A-5
Sheet 1 of 1

Drilled	Start 1/23/2015	End 1/23/2015	Total Depth (ft) 41.5	Logged By ERH Checked By DPC	Driller Geologic Drill, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) 117.5 Vertical Datum NAVD88	Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment Bobcat
Easting (X) Northing (Y)	System Datum	Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft) — Not encountered
Notes:		

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
117.5	0					MH/CH	Gray to blue-gray silt or clay (stiff to very stiff moist) (recent deposits)			
115		18	10	1						
	5	18	12	2						Smooth drilling Disturbed soil fabric
110			15	3						Smooth drilling
	10	24	14	4 AL			Grades to fat clay	37		AL (LL = 78; PI = 43) Disturbed soil fabric
105										
						CH	Gray fat clay (very stiff, moist) (glacially consolidated soils)			
	15	24	14	5						Smooth drilling Massive soil fabric
100										
	20	24	15	6						Smooth drilling Massive soil fabric
95										
	25	24	19	7 AL				33		AL (LL = 79; PI = 45) Smooth drilling
90										
	30	24	17	8						Slicken sides
85										
	35									

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

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Log of Boring GEI-5

GeoEngineers

Project: Aegis Mercer Island
Project Location: Mercer Island, Washington
Project Number: 19811-009-00

Figure A-6
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		24	21		9							Massive soil fabric
80													
	40		18	26		10							

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

Log of Boring GEI-5 (continued)

GeoEngineers

Project: Aegis Mercer Island
Project Location: Mercer Island, Washington
Project Number: 19811-009-00

Drilled	Start 1/23/2015	End 1/23/2015	Total Depth (ft) 26.5	Logged By ERH Checked By DPC	Driller Geologic Drill, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) 140.5 Vertical Datum NAVD88	Hammer Data Rope & Cathead 140 (lbs) / 30 (in) Drop	Drilling Equipment Bobcat
Easting (X) Northing (Y)	System Datum	Groundwater Date Measured / Depth to Water (ft) / Elevation (ft) Not encountered
Notes:		

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
140	0								ML	Dark brown silt with roots (forest duff)			
			12	11		1							
									SM	Orange-brown silty fine to medium sand with occasional gravel (loose to medium dense, moist) (fill)			
135	5		5	11		2							Gravel stuck in sampler shoe
			12	6		3							
									MH/CH	Orange-brown silt or fat clay with woody organics (medium stiff, moist) (recent deposits)			
130	10		18	7		4							Smooth drilling
			18	13		5							Disturbed fabric
125	15		18	11		6							
									CH	Gray silty fat clay (very stiff, moist) (glacially consolidated soils)			
			18	17		7							
120	20		18	13		8							Massive fabric
115	25		18	16		9							

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

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Log of Boring GEI-6

GEOENGINEERS

Project: Aegis Mercer Island
Project Location: Mercer Island, Washington
Project Number: 19811-009-00

Figure A-7
Sheet 1 of 1

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), grain size distributions (sieve analyses), and Atterberg Limits. The tests were performed in general accordance with test methods of ASTM International (ASTM) or other applicable procedures.

The Atterberg Limit test result is presented in Figure B-1. 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

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.

Atterberg Limits

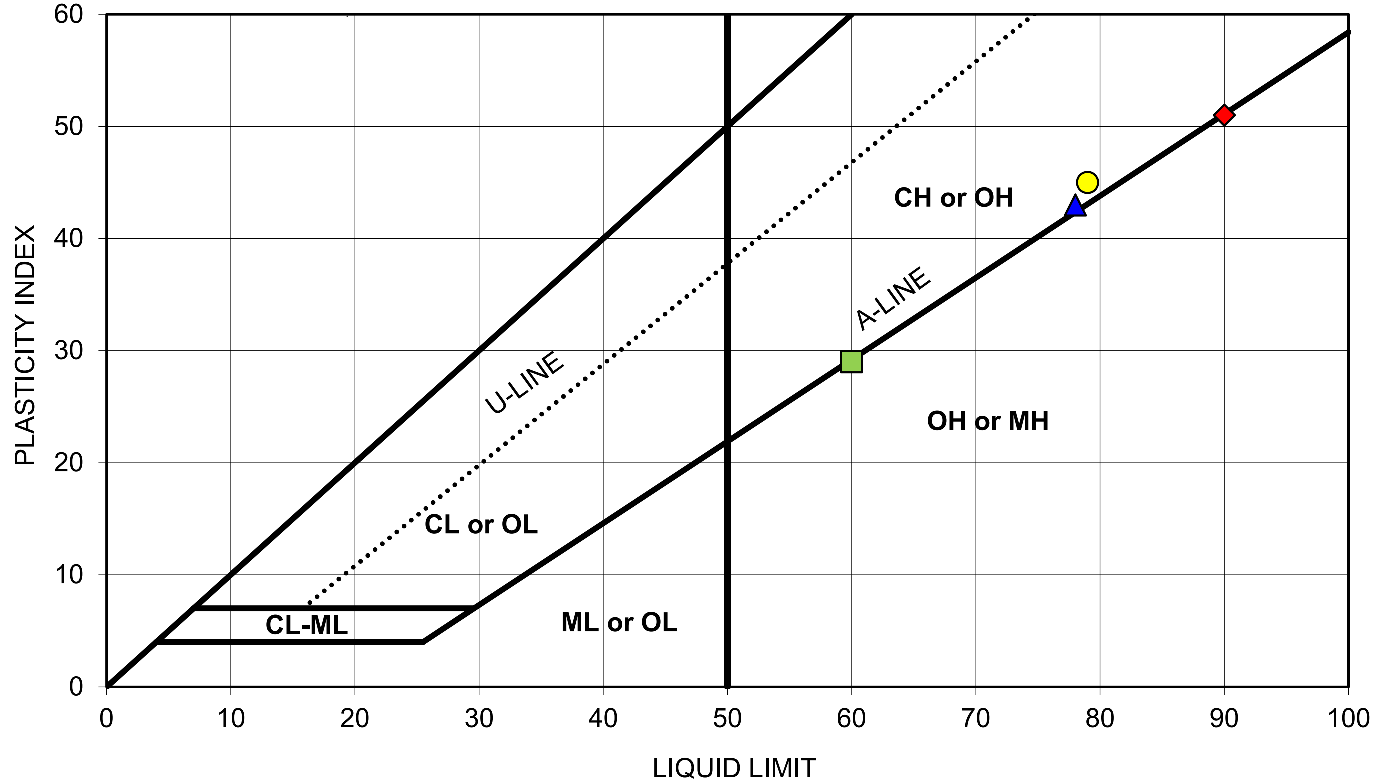
Atterberg limits testing was performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits testing are summarized in Figure B-1.

GEOENGINEERS

ATTERBERG LIMITS TEST RESULTS

FIGURE B-1

PLASTICITY CHART



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◆ (red)	GEI-1	10	41	90	51	Fat clay (CH)
■ (green)	GEI-4	20	37	60	29	Fat clay (CH)
▲ (blue)	GEI-5	10	37	78	43	Fat clay (CH)
● (yellow)	GEI-5	25	33	79	45	Fat clay (CH)

Appendix C
Boring Logs from Previous Studies

APPENDIX C
BORING LOGS FROM PREVIOUS STUDIES

Included in this section are logs from the following previous study completed near the project site

- The logs of two borings and two test pits from the report titled "Geotechnical Engineering Study, Mercer Island Two Office Building, SE 24th Street and 76th Avenue SE, Mercer Island, Washington" completed in 1985 by Earth Consultants Inc.

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTION
Coarse Grained Soils	Gravel And Gravelly Soils	Clean Gravels (little or no fines)		GW gw	Well-Graded Gravels, Gravel-Sand Mixtures, Little Or No Fines
				GP gp	Poorly-Graded Gravels, Gravel-Sand Mixtures, Little Or No Fines
	More Than 50% Coarse Fraction Retained On No. 4 Sieve	Gravels With Fines (appreciable amount of fines)		GM gm	Silty Gravels, Gravel-Sand-Silt Mixtures
				GC gc	Clayey Gravels, Gravel-Sand-Clay Mixtures
More Than 50% Material Larger Than No. 200 Sieve Size	Sand And Sandy Soils	Clean Sand (little or no fines)		SW sw	Well-Graded Sands, Gravelly Sands, Little Or No Fines
				SP sp	Poorly-Graded Sands, Gravelly Sands, Little Or No Fines
	More Than 50% Coarse Fraction Passing No. 4 Sieve	Sands With Fines (appreciable amount of fines)		SM sm	Silty Sands, Sand-Silt Mixtures
				SC sc	Clayey Sands, Sand-Clay Mixtures
Fine Grained Soils	Silts And Clays	Liquid Limit Less Than 50		ML ml	Inorganic Silts & Very Fine Sands, Rock Flour, Silty-Clayey Fine Sands; Clayey Silts w/ Slight Plasticity
				CL cl	Inorganic Clays Of Low To Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean
				OL ol	Organic Silts And Organic Silty Clays Of Low Plasticity
More Than 50% Material Smaller Than No. 200 Sieve Size	Silts And Clays	Liquid Limit Greater Than 50		MH mh	Inorganic Silts, Micaceous Or Diatomaceous Fine Sand Or Silty Soils
				CH ch	Inorganic Clays Of High Plasticity, Fat Clays
				OH oh	Organic Clays Of Medium To High Plasticity, Organic Silts
Highly Organic Soils				PT pt	Peat, Humus, Swamp Soils With High Organic Contents

Topsoil		Humus And Duff Layer
Fill		Highly Variable Constituents

The Discussion In The Text Of This Report Is Necessary For A Proper Understanding Of The Nature Of The Material Presented In The Attached Logs

Notes :

Dual symbols are used to indicate borderline soil classification. Upper case letter symbols designate sample classifications based upon laboratory testing; lower case letter symbols designate classifications not verified by laboratory testing.

I	2" O.D. SPLIT SPOON SAMPLER	C	TORVANE READING, tsf
II	2.4" I.D. RING SAMPLER OR SHELBY TUBE SAMPLER	qu	PENETROMETER READING, tsf
P	SAMPLER PUSHED	W	MOISTURE, percent of dry weight
*	SAMPLE NOT RECOVERED	pcf	DRY DENSITY, pounds per cubic ft.
∇	WATER LEVEL (DATE)	LL	LIQUID LIMIT, percent
	WATER OBSERVATION WELL	PI	PLASTIC INDEX

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LEGEND

Proj. No. 2749 | Date Sept. '85 | Plate 6

BORING NO. 2

Logged By RWB

Date 9/12/85

ELEV. 80.5'

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	sm ml	Brownish, sandy SILT to silty sand w/ gravel (FILL) dry, medium dense		I	12	13%	
	cl	Olive mottled silty CLAY w/lenses and layers of fine sand, very stiff, moist, low to moderate plasticity	5	I	14	35%	q_u=2.5tsf
			9/25/85 ▽	I	8	42%	q_u=2.3tsf
	ml	Olive, SILT w/fine sand and gravel layers, wet, medium dense	10	I	23	26%	
				II		19%	
			15				
	CH	Gray silty CLAY, moist, hard, high plasticity		II		39%	LL=66 Pl-38
			20	I	46	39%	q_u=4.2tsf
		Very stiff to hard with sand lenses	25	I	34	31%	q_u=4.2tsf 3.5tsf
		Becoming hard	30	I	22	34%	q_u=4.1tsf
			35	I	33	32%	

Boring terminated @ 39 feet beneath existing grade. 3/4 inch PVC standpipe installed to bottom of boring. Bottom 15 feet slotted. Boring backfilled with drill cuttings. No groundwater encountered during drill or on 9/25/85.

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BORING LOG

MERCER ISLAND TWO
MERCER ISLAND, WASHINGTON

Proj. No.	Date	Plate
2749	Sept. '85	8

BORING NO. 5

Logged By RWB

Date 9/16/85 ELEV. 85'

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)	
	ml	Tan SILT with clay, sand and gravel (possible fill)					
	ch	Tan, olive, silty CLAY with organics moist, stiff, high plasticity	5	I	8	45%	
		Color to gray and very stiff		I	18	39%	q_u=2.8tsf
			10	I	26	36%	q_u=3.2tsf
				I		42%	q_u=3.0tsf
			15	I	30	40%	q_u=3.2tsf
				I	24	38%	q_u=3.7tsf
			20				
				I	27	32%	q_u=3.7tsf
			25				
				I	23	36%	q_u=4.2tsf
			30				
				I	32	33%	q_u- 4.7tsf

Boring terminated @ 34 feet beneath the existing grade. Boring backfilled with drill cuttings. No groundwater encountered during drill.

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BORING LOG

MERCER ISLAND TWO
MERCER ISLAND, WASHINGTON

Proj. No. 2749	Date Sept. '85	Plate 11

TEST PIT NO. 2

Logged By JWJ

Date 9/12/85

Elev. 94'±

Depth (ft.)	USCS	Soil Description	W (%)	
0–8	ch	Olive silty CLAY (FILL), moist, very stiff to stiff, heavily fractured (possible fill from 5 to 9 feet)	40%	
8–17	CH	Olive, silty CLAY, moist, very stiff to stiff high plasticity		C=1.0tsf
		Color to gray and hard	38%	
				LL=74 Pl=44
17–20	BOP			

Test Pit terminated at 17 feet below existing grade. No groundwater seepage encountered during excavation.

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TEST PIT LOGS
MERCER ISLAND TWO
MERCER ISLAND, WASHINGTON

Proj. No. 2749 | **Date** Sept.'85 | **Plate** 13

TEST PIT NO. 3

Logged By JWJ

Date 9/12/85

Elev. 93'±

Depth (ft.)	USCS	Soil Description	W (%)	
0–6	ch	Tan, silty CLAY (possible fill), moist (hard chunks, heavily fractured)	15%	
6–14	CH	(Appears to be weathered native zone from 6 to 8 feet) Olive, silty CLAY, moist, hard	37%	LL=75 Pl=45
		Color to gray	39%	
14	BOP			

Test Pit terminated at 14 feet below existing grade. No groundwater seepage encountered during excavation.

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TEST PIT LOGS
MERCER ISLAND TWO
MERCER ISLAND, WASHINGTON

Proj. No. 2749 | Date Sept. '85 | Plate 14

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 shall be approved by the Engineer and shall be representative of the field conditions. Load tests shall not be performed until the nail/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 nails/tiebacks is provided, the casing shall be installed to prevent interaction between the bonded length of the nail/tieback and the casing/testing apparatus.

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

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

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

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

Verification Tests

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

For soil nails, the unbonded length of the test nails shall be at least 3 feet unless approved otherwise by the Engineer. The bond length of the test nails shall not be less than 10 feet and shall not be longer than the bond length that would prevent testing to 200 percent of the design load while not exceeding the allowable bar load. The allowable bar load during testing shall not exceed 80 percent of the steel ultimate strength for Grade 150 bars or 90 percent of the steel ultimate strength for Grade 60 and 75 bars. The allowable tieback load should not exceed 80 percent of the steel ultimate strength.

For soil nails, the design test load shall be determined by multiplying the bond length of the nail times the design load pullout resistance (load transfer). Tieback design test loads should be the design load specified on the shoring drawings. Verification test nails/tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL	1 minute
1.5DL	60 minutes
1.75DL	1 minute
2.0DL	10 minutes

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

Proof Tests

Proof tests shall be completed on approximately 5 percent of the production nails at locations selected by the owner's representative. Additional testing may be required where nail installation methods are substandard. Proof tests shall be completed on each production tieback.

For soil nails, the unbonded length of the test nails shall be at least 3 feet unless approved otherwise by the Engineer. The bond length of the test nails shall not be less than 10 feet and shall not be longer than the bond length that would prevent testing to 200 percent of the design load while not exceeding the allowable bar load. The allowable bar load during testing shall not exceed 80 percent of the steel ultimate strength for Grade 150 bars or 90 percent of the steel ultimate strength for Grade 60 and 75 bars. The allowable tieback load should not exceed 80 percent of the steel ultimate strength.

For soil nails, the design test load shall be determined by multiplying the bond length of the nail times the design load pullout resistance (load transfer). Tieback design test loads should be the design load specified on the shoring drawings. Proof test nails/tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL (soil nails)	1 minute
1.33DL (tiebacks)	10 minutes
1.5DL (soil nails)	

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Nail/tieback deflections during the 1.33DL and 1.5DL test loads shall be recorded at 1, 2, 3, 5, 6 and 10 minutes.

Depending upon the nail/tieback deflection performance, the load hold period at 1.33DL (tiebacks) or 1.5DL (soil nails) may be increased to 60 minutes. Nail/tieback movement shall be recorded at 1, 2, 3, 5, 6 and 10 minutes. If the nail/tieback deflection between 1 and 10 minutes is greater than 0.04 inches, the 1.33DL/1.5DL load shall be continued to be held for a total of 60 minutes and deflections recorded at 20, 30, 50 and 60 minutes.

Test Nail/Tieback Acceptance

A test nail/tieback shall be considered acceptable when:

1. For verification tests, a nail/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 nail/tieback is considered acceptable if the creep rate is less than 0.04 inches per log cycle of time between 1 and 10 minutes or 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.
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 nail/tieback.

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

The Engineer shall evaluate the verification test results. Nail/tieback installation techniques that do not satisfy the nail/tieback testing requirements shall be considered inadequate. In this case, the contractor shall propose alternative methods and install replacement verification test nails/tiebacks.

The Engineer may require that the contractor replace or install additional production nails/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	Three times per week
After excavation is complete and wall movements have stabilized, and before the floors of the building reach 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 excavation. 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; and (2) on existing buildings located within a horizontal distance of the shoring walls equal to the height of the wall. The survey points should be located on every other soldier pile along the wall face for soldier pile and tieback shoring, 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 Aegis Senior Communities, LLC and other project team members for the proposed Aegis Mercer Island 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 Aegis Mercer Island 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.

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

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

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

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 purposes 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.

Have we delivered World Class Client Service?

Please let us know by visiting **www.geoengineers.com/feedback**.

GeoEngineers