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Subsurface Exploration, Geologic Hazard,
and Geotechnical Engineering Report

MUNSON RESIDENCE
Mercer Island, Washington

Prepared For:
MARC AND TRACY MUNSON

Project No. 180490E001
November 9, 2018

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November 9, 2018
Project No. 180490E001

Marc and Tracy Munson
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Attention: Ms. Debbi Cleary, Cleary Design Studio, LLC

Subject: Subsurface Exploration, Geologic Hazard,
and Geotechnical Engineering Report
Munson Residence
Mercer Island, Washington

Dear Ms. Cleary:

We are pleased to present the enclosed copies of the above-referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and geotechnical engineering studies, and offers recommendations for the design and development of the proposed project. Our recommendations are preliminary in that construction details have not been finalized at the time of this report.

We have enjoyed working with you on this study and are confident the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions, or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Kurt D. Merriman, P.E.
Senior Principal Engineer

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SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND GEOTECHNICAL ENGINEERING REPORT

MUNSON RESIDENCE

Mercer Island, Washington

Prepared for:
Marc and Tracy Munson
4628 Forest Avenue SE
Mercer Island, Washington 98040

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I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and geotechnical engineering study for the subject project. Our recommendations are preliminary in that construction details have not been finalized at the time of this report. The location of the subject site is shown on the "Vicinity Map," Figure 1. The approximate location of the exploration accomplished for this study is presented on the "Existing Site and Exploration Plan," Figure 2. In the event that any changes in the nature or design of the proposed project are planned, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the design and development of the subject project. Our study included reviewing available geologic literature, drilling one exploration boring, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow groundwater conditions. Geotechnical engineering studies were also conducted to assess the type of suitable foundation, allowable foundation soil bearing pressures, anticipated foundation settlements, basement/retaining wall lateral pressures, floor support recommendations, and drainage considerations. This report summarizes our current fieldwork and development recommendations based on our present understanding of the project.

1.2 Authorization

Authorization to proceed with this study was granted by Ms. Tracy Munson. Our study was accomplished in general accordance with our scope of work letter, dated October 3, 2018. This report has been prepared for the exclusive use of Marc and Tracy Munson, and their agents, for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

2.0 PROJECT AND SITE DESCRIPTION

The subject site is the existing single-family residential property located at 4628 Forest Avenue SE in Mercer Island, Washington (King County Parcel No. 1324049031). Site topography is generally flat-lying to gently-sloping, with steeply sloping ground leading up to the property to the east of the subject site. Vegetation at the site consists chiefly of grass lawn areas, landscaping shrubbery and small- to medium-sized trees. We understand that the current plan, as part of a remodel to the existing residence, is to construct an addition to the front, western side of the existing residence at the subject site. The subject site lies within Erosion, Seismic, and Landslide Hazard Areas, as delineated in the City of Mercer Island “Geological Hazard Maps.” Therefore, the City of Mercer Island has required a geotechnical study for the proposed project.

3.0 SITE EXPLORATION

The site exploration was conducted on October 15, 2018, and consisted of one exploration boring and a geologic and geologic hazard reconnaissance to gain information about the site. The various types of materials and sediments encountered in the exploration, as well as the depths where characteristics of these materials changed, are indicated on the exploration boring log presented in the Appendix. The depths indicated on the log where conditions changed may represent gradational variations between sediment types in the field. If changes occurred between sample intervals in our boring, they were interpreted. The location of the exploration boring is shown on the “Existing Site and Exploration Plan,” Figure 2. The conclusions and recommendations presented in this report are based on the exploration boring completed for this study. The number, location, and depth of the exploration were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions beyond the field exploration is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations beyond the field exploration may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Boring

The exploration boring was completed on the property using a hand-portable drill rig advancing a 3.75-inch inside-diameter, hollow-stem auger. During the drilling process, samples were obtained at generally 2.5-foot intervals. The boring was continuously observed and logged by a geologist from our firm. The exploration log presented in the Appendix is based on the field log, drilling action, and observation of the samples secured.

Disturbed but representative samples were obtained by using the Standard Penetration Test (SPT) procedure in accordance with *American Society for Testing and Materials* (ASTM) D-1586. This test and sampling method consists of driving a standard, 2-inch outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance (“N”) or blow count. If a total of 50 blows are recorded at or before the end of one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. These values are plotted on the attached boring log.

The samples obtained from the split-barrel sampler were classified in the field and representative portions placed in watertight containers. The samples were then transported to our laboratory for further visual classification and geotechnical laboratory testing, as necessary.

The various types of soil and groundwater elevations, as well as the depths where soil and groundwater characteristics changed, are indicated on the exploration boring log presented in the Appendix of this report. Our exploration and reconnaissance were approximately located by measuring from known site features.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field exploration accomplished for this study, visual reconnaissance of the site, and review of applicable geologic literature. As shown on the field log, the exploration boring generally encountered fill overlying old landslide deposits. The following section presents more detailed subsurface information organized from the youngest to the oldest sediment types.

4.1 Stratigraphy

Topsoil/Fill

Exploration boring EB-1 encountered a surficial topsoil/fill layer that extended to roughly 8 inches below the ground surface. The topsoil/fill encountered generally consisted of loose silty sand with organics and a trace amount of fine gravel. Fill is also expected in unexplored areas of the site, such as the area surrounding and under the existing structure foundations, in existing utility trenches, and at previously graded landscaped areas. Due to their variable density and content, the existing fill soils are not suitable for foundation support.

Landslide Deposits

Sediments encountered below the topsoil/fill in exploration boring EB-1 generally consisted of soft to stiff clay, with very stiff material present below roughly 20 feet below the ground surface. The upper clay was fractured, with a blocky texture, while horizontally oriented laminations were observed below 17.5 feet. This fine-grained deposit was interpreted to originally represent glaciomarine drift sediments placed prior to the Fraser Glaciation (likely Possession-age) and subsequently compacted by the weight of the overlying glacial ice. The observed fracturing is indicative of deposits derived from past earth movement (landslides). The underlying very stiff horizontally-laminated material can be used for foundation support.

4.2 Geologic Mapping

Review of the regional geologic map titled *Geologic Map of Mercer Island, Washington* (2006) by Kathy G. Troost and Aaron P. Wisher, indicates that the site is expected to be underlain at shallow depths by pre-Olympia-age deposits (e.g. Possession Drift). Scarps are mapped in the vicinity of the subject site, including upgradient to the east, congruent with the presence of landslide deposits at the site. Our interpretation of the sediments encountered at the subject site is in general agreement with the regional geologic map.

4.3 Hydrology

Groundwater was not encountered within exploration boring EB-1. We expect shallow groundwater seepage across much of the site to be limited to interflow. Interflow occurs when surface water percolates down through the surficial weathered or higher permeability sediments and becomes perched atop underlying, lower permeability sediments. It should be noted that the occurrence and level of groundwater seepage at the site may vary in response to such factors as changes in season, amount of precipitation, and site use.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow groundwater conditions, as observed and discussed herein.

5.0 SLOPE STABILITY ASSESSMENT

The City of Mercer Island geologic hazard maps indicate that the site is located in a landslide hazard area. Therefore, the hazard must be addressed in the design of the foundation of the addition. The following paragraphs discuss the stability of the slope and recommendations to mitigate risks to the public health, safety, or welfare. It must be understood that no recommendations or engineering design can yield a guarantee of stable slopes. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

During our site reconnaissance and our subsurface exploration, we found no visual evidence of tension cracks, emergent seepage, hummocky topography, or other indications of recent slope instability at the subject site. Based on the absence of visual indications of recent deep-seated slope instability, it is our opinion that the risk of damage to the proposed project by deep-seated landslides at the subject site, under either static or seismic conditions, is low. This opinion is dependent upon site grading and construction practices being completed in accordance with the geotechnical recommendations presented in this report.

As stated above, we encountered landslide deposits in our exploration. The sloping area to the east of the subject site is mapped as a scarp, with several other scarps mapped across the vicinity. Based on our review of the Light Detection and Ranging (LIDAR) image encompassing the subject site, the slopes leading upward from the area of the subject site to the upland to the east include several bowl-shaped slide features. Given the broad nature of the delineated landslide hazard area upslope of the subject site and neighboring parcels, the ability to mitigate risks associated with landslides occurring along these slopes, based on the relative size of the slope complex as compared to the subject site, is limited. In addition, based on our field observations and document review, it is our opinion that the area surrounding the property is likely underlain by landslide deposits. In our opinion, there is no economically feasible mitigation for this relatively small structure to fully resist all movement, and the recommendations presented in this report are intended to mitigate on-site soil conditions to provide foundation support for the proposed addition.

As with all steep slopes, surface drainage should be properly controlled and directed away from sloping areas. At no time should loose fill be pushed over the top of the slope or soil excavated from the toe area without support by an engineered retaining structure. Uncontrolled fill on slopes or toe excavation may promote landslides or debris flow activity. Associated Earth Sciences, Inc. (AESI) should review grading plans if grading is desired at the top of, on, or near the toe of the steep slope.

6.0 SEISMIC HAZARDS AND MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these events are small, and are usually not felt by people. However, large earthquakes do occur, as evidenced by the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this region during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates indicates that an earthquake of the magnitude between 5.5 and 6.0 is likely within a given 20- to 40-year period.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

6.1 Surficial Ground Rupture

The subject site is located within the mapped limits of the Seattle Fault Zone. Recent studies by the U.S. Geological Survey (USGS) (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington*, Geology, v. 22, p.71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington - Implications for Earthquake Hazards,* Geological Society of America Bulletin, July 1999, v. 111, n. 7, p. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. The recognition of this fault is relatively new, and data pertaining to it are limited, with the studies still ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years. Due to the suspected long recurrence interval, the potential for surficial ground rupture is considered to be low during the expected life of the structure, and no mitigation efforts beyond complying with the current (2015) *International Building Code* (IBC) are recommended.

6.2 Seismically Induced Landslides

Due to the landslide deposits found during our exploration, the field and subsurface observations noted in Section 5.0, and the very stiff nature of the soils underlying the landslide deposits, it is our opinion that the risk of seismically induced landslides lies predominantly within the upper soil sequence. Therefore, we recommend the use of a deep foundation placed at an elevation below the encountered fractured slide debris to mitigate the potential risk. As noted previously, this opinion is dependent upon site grading and construction practices being completed in accordance with the geotechnical recommendations presented in this report.

6.3 Liquefaction

Liquefaction is a condition where loose, saturated, typically sandy soils lose shear strength when subjected to high-intensity cyclic loads, such as occur during earthquakes. The resulting reduction in strength can cause differential foundation settlements and slope failures. Loose, saturated, fine-grained sands that cannot dissipate the buildup of pore water pressure are the predominant type of sediments subject to liquefaction. It is our opinion that the encountered stratigraphy has a low potential for liquefaction due to its fine-grained texture and lack of significant groundwater.

6.4 Ground Motion

Structural design of the addition should follow 2015 IBC standards using Site Class “D” as defined in Table 20.3-1 of *American Society of Civil Engineers* (ASCE) *7 – Minimum Design Loads for Buildings and Other Structures*.

7.0 EROSION HAZARDS AND MITIGATION

A properly developed, constructed, and maintained erosion control plan consistent with local standards and best management erosion control practices will be required for this project. It will be necessary to make adjustments and provide additional measures to the Temporary Erosion and Sedimentation Control (TESC) plan in order to improve its effectiveness. Ultimately, the success of the TESC plan depends on a proactive approach to project planning and contractor implementation and maintenance.

The erosion hazard of the site soils is low to moderate, depending primarily on slope and runoff velocity. Maintaining cover measures atop disturbed ground provides the greatest reduction to the potential generation of turbid runoff and sediment transport. During the local wet season (typically October through April), exposed soil should not remain uncovered for more than 2 days, unless it is actively being worked. Ground-cover measures can include erosion control matting, plastic sheeting, straw mulch, crushed rock or recycled concrete, or mature hydroseed.

7.1 Erosion Hazard Mitigation

To mitigate the erosion hazards and potential for off-site sediment transport, we recommend the following:

1. All TESC measures for the work area should be installed prior to any activity.

2. Construction access points should be surfaced to mitigate sediment track out onto adjacent streets. If practical, existing paved surfaces may be used. Any sediment that is tracked onto adjacent streets should be promptly swept up.

3. During the wetter months of the year (typically October through April), or when large storm events are predicted during the summer months, the work area should be stabilized so that if showers occur, the work area can receive the rainfall without excessive erosion or sediment transport.

4. All disturbed areas should be revegetated as soon as possible. If it is outside of the growing season, the disturbed areas should be covered with mulch.

5. Under no circumstances should concentrated discharges be allowed to flow over the top of steep slopes.

6. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not limited to, covering with plastic sheeting, the use of low stockpiles in flat areas, or the use of straw bales/silt fences around pile perimeters.

8.0 STATEMENT OF RISK

For Section 19.07.060(D) of the Mercer Island Unified Land Development Code (ULDC), the City of Mercer Island requires a statement of risk by the geotechnical engineer. It is AESI’s opinion that the development practices proposed for the alteration would render the proposed addition as safe as if it were not located in a geologic hazard area provided the recommendations in this report are followed.

III. DESIGN RECOMMENDATIONS

9.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the property is suitable for the proposed development, provided the risks discussed are accepted and the recommendations contained herein are properly followed. The foundation bearing stratum suitable for foundation support was encountered in our exploration at a depth of approximately 20 feet or more below present surface grade. Due to the depth of the bearing soils relative to the existing ground surface, a driven pipe pile foundation is recommended for the proposed addition.

10.0 SITE PREPARATION

10.1 Clearing and Stripping

Site preparation of the planned building area should include removal of all trees, brush, debris, and any other deleterious materials. These unsuitable materials should be properly disposed of off-site. Additionally, any areas of organic topsoil should be removed and the remaining roots grubbed. Areas where loose surficial soils exist due to grubbing operations should be considered as fill to the depth of disturbance and treated as subsequently recommended for structural fill placement. Any buried utilities should be removed or relocated if they are under building areas. The resulting depressions should be backfilled with structural fill, as discussed under the "Structural Fill" section of this report.

10.2 Temporary and Permanent Cut Slopes

In our opinion, stable, temporary construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, we anticipate that temporary, unsupported cut slopes, or utility trenches greater than 4 feet in height or depth, completed within the unsaturated, existing fill or landslide deposits can be planned at a maximum slope of 1.5H:1V (Horizontal:Vertical). As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times. In the presence of groundwater seepage, flatter slopes or shoring may be required. Permanent cut and structural fill slopes should not exceed an inclination of 2H:1V. Permanent non-structural landscape fill should not exceed a 3H:1V inclination.

10.3 Site Disturbance

The existing fill and natural sediments contain a high percentage of fine-grained material that makes them moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill.

Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock or asphalt treated base (ATB). If crushed rock is considered for the access and staging areas, it should be underlain by engineering stabilization fabric to reduce the potential of fine-grained materials pumping up through the rock during wet weather and turning the area to mud. The fabric will also aid in supporting construction equipment, thus reducing the amount of crushed rock required. We recommend that at least 10 inches of rock be placed over the fabric.

11.0 STRUCTURAL FILL

Structural fill may be necessary to establish desired grades or to backfill around foundations and utilities. All references to structural fill in this report refer to subgrade preparation, fill type, placement, and compaction of materials, as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

After overexcavation/stripping has been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of exposed ground should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After stripping and subgrade preparation of the exposed ground is approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to 95 percent of the modified Proctor maximum density using ASTM D-1557 as the standard.

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material at least 3 business days in advance to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soils in structural fills should be limited to favorable dry weather conditions. The on-site soils are predominantly fine-grained and are considered moisture-sensitive, and we expect that this material may be difficult to compact to structural fill specifications, particularly during and following wet weather. Therefore, we recommend that a select, import material consisting of a clean, free-draining gravel and/or sand be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction.

A representative from our firm should observe the stripped subgrade and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill. As such, we are available to aid the owner in developing a suitable monitoring and testing frequency.

12.0 FOUNDATIONS

We recommend the use of steel pipe piles for the planned addition. Recommendations for pipe pile foundations are included in this section. We recommend that, for preliminary estimating purposes, pile lengths in the approximate 25- to 35-foot range be assumed. Actual pile lengths may differ significantly from the estimate depending on local variations in soil conditions, pile size, and driving equipment used. Pile lengths can best be determined by driving a series of test piles.

12.1 Pipe Pile Foundations

Pipe piles for the addition should consist of 3-, 4-, or 6-inch-diameter pipe, depending on the required structural loads and equipment access constraints. Two-inch-diameter piles may be considered if the installation of larger piles is precluded due to access constraints. The piles should be galvanized steel pipe, driven with a suitable hammer to the refusal criteria shown in Table 1. The following table provides required minimum hammer weights, refusal criteria, and allowable loads for pipe piles.

Table 1
Pipe Pile Design Parameters

Pipe Diameter (inches)	Wall Thickness	Minimum Hammer Size (pounds)	Refusal Criteria* (seconds)	Allowable Axial Compressive Load** (kips)
2	Schedule 80	90	60	4
3	Schedule 40	400	25	10
4	Schedule 40	650	20	20
6	Schedule 40	1,500	15	20–30

* Refusal is defined as less than 1 inch of penetration in “X” seconds under constant driving.

** Allowable load to be verified by load tests in accordance with *American Society for Testing and Materials* (ASTM) D-1143 “quick load test.”

Anticipated settlement of pile-supported foundations should be less than ½ inch. Pile installation must be observed by AESI to verify that the design bearing capacity of the piles has been attained and that construction conforms to the recommendations contained herein. The City of Mercer Island may also require such inspections.

Lateral resistance can be derived from passive soil resistance against the buried portion of the foundation (i.e., the grade beam) or from the installation of batter piles. A passive equivalent fluid of 200 pounds per cubic foot (pcf) can be used to account for lateral resistance. Lateral resistance for batter piles should be taken as the horizontal component of the axial pile load. Batter piles are typically installed at 1H:4V inclination.

Pile Inspections

The actual total length of each pile may be adjusted in the field based on required capacity and conditions encountered during driving. Since completion of the pile takes place below ground, the judgment and experience of the geotechnical engineer or their field representative must be used as a basis for determining the required penetration and acceptability of each pile. Consequently, use of the presented pile capacities in the design requires that the installation of all piles be observed by a qualified geotechnical engineer or engineering geologist from our firm, who can interpret and collect the installation data and examine the contractor’s operations. AESI, acting as the owner’s field representative, would determine the required lengths of the piles and keep records of pertinent installation data. A final summary report would then be distributed following completion of pile installation.

Load testing should be performed to verify that the design bearing capacity of the piles has been attained. Because of the variation in the soil types and their densities, we recommend that AESI monitor the load testing program. A common pile load testing program would consist of one or more 200-percent verification tests of the design bearing capacity of the pile in the soil. Verification test piles are usually loaded in 25-percent increments that are held for 2 minutes up to the final load of 200-percent design load. The 200-percent load is commonly held for 20 minutes and creep-measured. The load is then reduced by 25-percent increments to evaluate the effect of elasticity in the pile to overall displacement.

13.0 LATERAL WALL PRESSURES

All backfill behind retaining walls or around foundation units should be placed as per our recommendations for structural fill and as described in this section of the report. Horizontally backfilled retaining walls that are free to yield laterally at least 0.1 percent of their height may be designed using an equivalent fluid equal to 35 pcf. Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for an equivalent fluid of 50 pcf. If roadways, parking areas, or other areas subject to vehicular traffic are adjacent to retaining walls, a surcharge equivalent to 2 feet of soil should be added to the wall height in determining lateral design forces. Retaining walls that retain sloping backfill at a maximum angle of 2H:1V should be designed using an equivalent fluid pressure of 55 pcf for yielding conditions or 75 pcf for fully restrained conditions.

In accordance with the 2015 IBC, retaining wall design should include seismic design parameters. Based on the site soils and assumed wall backfill materials, we recommend a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. A rectangular pressure distribution of 5H and 10H psf (where H is the height of the wall in feet) should be included in design for “active” and “at-rest” loading conditions, respectively. The resultant of the rectangular seismic surcharge should be applied at the midpoint of the walls.

The lateral pressures presented above are based on the conditions of a uniform horizontal backfill consisting of the on-site, natural, glacial sediments or imported sand and gravel compacted to 90 percent of ASTM D-1557. A higher degree of compaction is not recommended, as this will increase the pressure acting on the wall.

Footing drains must be provided for all retaining walls, as discussed under the “Drainage Considerations” section of this report. It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. This would involve installation of a minimum, 1-foot-wide blanket drain to within 1 foot of the ground surface using imported, washed gravel against the walls placed to be continuous with the footing drain.

13.1 Passive Resistance and Friction Factors

Retaining wall grade beams/keyways cast directly against undisturbed dense soils in a trench may be designed for passive resistance against lateral translation using an allowable equivalent fluid equal to 200 pcf. The passive equivalent fluid pressure diagram begins at the top of the grade beam; however, total lateral resistance should be summed only over the depth of the actual key. Since the structure will be pile-supported, we do not recommend using base friction for resistance to lateral loads.

14.0 FLOOR SUPPORT

Due to the loose nature of the subgrade soils, we recommend that structural support be provided for settlement-sensitive slab-on-grade floors. If moisture intrusion through slab-on-grade floors is to be limited, the floors should be constructed atop a capillary break consisting of a minimum thickness of 4 inches of washed pea gravel, washed crushed rock, or other suitable material approved by the geotechnical engineer. The capillary breaks should be overlain by a 10-mil (minimum thickness) plastic vapor retarder. If pea gravel is used for the capillary break, a geotextile, such as Mirafi 500X or approved equivalent, should be placed between the pea gravel and underlying crushed rock.

15.0 DRAINAGE CONSIDERATIONS

All retaining and perimeter foundation walls should be provided with a drain at the base of the footing elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel. The level of the perforations in the pipe should be set at or slightly below the bottom of the footing grade beam, and the drains should be constructed with sufficient gradient to allow gravity discharge away from the building. In addition, all retaining walls should be lined with a minimum, 12-inch-thick, washed gravel blanket that extends to within 1 foot of the surface and is continuous with the foundation drain. Roof and surface runoff should not discharge into the foundation drain system, but should be handled by a separate, rigid, tightline drain. In planning, exterior grades adjacent to walls should be sloped downward away from the structure to achieve surface drainage. All collected runoff must be tightlined to a City-approved location.

16.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

Our recommendations are preliminary in that definite building locations and construction details have not been finalized at the time of this report. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. If significant changes in grading are made, we recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of this current scope of work. If these services are desired, please let us know, and we will prepare a proposal.

We have enjoyed working with you on this study and are confident these recommendations will aid in the successful completion of your project. If you should have any questions or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

23580
11-9-18

Jeffrey P. Laub, L.G., L.E.G.
Senior Engineering Geologist

Kurt D. Merriman, P.E.
Senior Principal Engineer

Attachments:	Figure 1:	Vicinity Map
	Figure 2:	Existing Site and Exploration Plan
	Appendix:	Exploration Log

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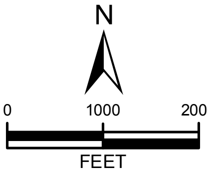

DATA SOURCES / REFERENCES:
USGS: 7.5' SERIES TOPOGRAPHIC MAPS
KING CO: STREETS, CITY LIMITS 1/18, PARCELS 8/18

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

VICINITY MAP

MUNSON RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
180490E001	10/18	1

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LEGEND:

- ● EXPLORATION BORING
- ▭ SITE
- ▭ PARCEL

DATA SOURCES / REFERENCES:
WA STATE PLANE NORTH (FIPS 4601), NAD83(HARN)
NAVD88 GEOID03 (GEOID03), US SURVEY FEET.
KING CO: STREETS, 1/18, PARCELS 9/18, AERIAL PICTOMETRY INT. 2015

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

N

0 20 40
FEET

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

EXISTING SITE AND EXPLORATION PLAN

MUNSON RESIDENCE
MERCER ISLAND, WASHINGTON

PROJ NO.	DATE:	FIGURE:
180490E001	10/18	2

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APPENDIX

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]	▨	**GW**	Well-graded gravel and gravel with sand, little to no fines
			▨	**GP**	Poorly-graded gravel and gravel with sand, little to no fines
		≥12% Fines[5]	▨	**GM**	Silty gravel and silty gravel with sand
			▨	**GC**	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]	▨	**SW**	Well-graded sand and sand with gravel, little to no fines
			▨	**SP**	Poorly-graded sand and sand with gravel, little to no fines
		≥12% Fines[5]	▨	**SM**	Silty sand and silty sand with gravel
			▨	**SC**	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		▨	**ML**	Silt, sandy silt, gravelly silt, silt with sand or gravel
			▨	**CL**	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			▨	**OL**	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		▨	**MH**	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			▨	**CH**	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			▨	**OH**	Organic clay or silt of medium to high plasticity
Highly Organic Soils			▨	**PT**	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT[2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50

	Consistency	SPT[2] blows/foot
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

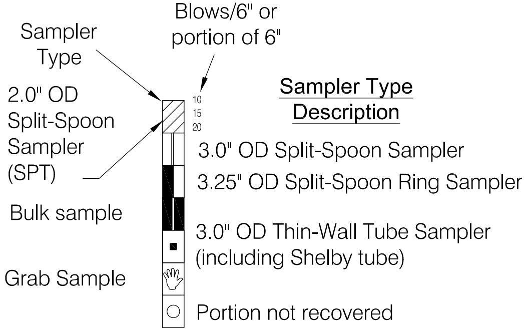
(3) Estimated Percentage

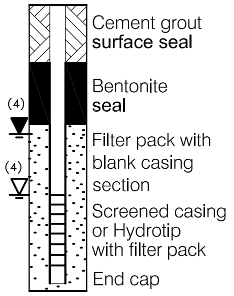
Component	Percentage by Weight
Trace	<5
Some	5 to <12
Modifier (silty, sandy, gravelly)	12 to <30
Very *modifier* (silty, sandy, gravelly)	30 to <50

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch
Slightly Moist - Perceptible moisture
Moist - Damp but no visible water
Very Moist - Water visible but not free draining
Wet - Visible free water, usually from below water table

Symbols





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(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of ground water
▼ ATD = At time of drilling
▽ Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 12%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

associated
earth sciences
incorporated

EXPLORATION LOG KEY

FIGURE A1

blocks \ dwg \ log_key.dwg LAYOUT: Layout 4 -2014 Qty Chng associated earth sciences incorporated

Exploration Log

Project Number	Exploration Number	Sheet
180490E001	EB-1	1 of 1

Project Name: Munson Residence
Location: Mercer Island, Wa
Driller/Equipment: CN Drilling / Portable HSA
Hammer Weight/Drop: 140# / 30"
Ground Surface Elevation (ft): 122
Datum: NAVD 88
Date Start/Finish: 10/15/18,10/15/18
Hole Diameter (in): 6 inches

Depth (ft)	S T	Samples	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot	Other Tests
		S-1	**Topsoil / Fill - 8 inches** Moist, dark brown, organic rich, silty, fine SAND, trace fine gravel, trace rootlets, trace fertilizer pellets; massive (SM). **Landslide Deposits of Possession Glaciomarine Sediments**			3 3 4	7	
		S-2	Moist, brownish gray to dark brown, CLAY, trace fine gravel, trace organics; massive (CH). Moist, brownish gray with mottling in some areas, CLAY, trace organics; massive (CH).			2 3 4	7	
5		S-3	As above.			2 2 4	6	
		S-4	As above; more gray and stickier than above.			1 1 2	3	
10		S-5	Moist, gray, CLAY; blocky texture; reacts with hydrochloric acid (CH).			1 2 4	6	
		S-6	As above.			3 4 7	11	
15		S-7	As above.			2 4 9	13	
		S-8	Moist, gray, CLAY; thin laminations oriented horizontally; reacts with hydrochloric acid (CH).			2 4 8	12	
20		S-9	As above; driller notes adding water starting at 20 feet.			4 8 11	19	
		S-10	As above.			3 9 11	20	
25		S-11	As above.			2 10 16	26	
			Bottom of exploration boring at 26.5 feet No groundwater encountered.					
30								
35								

Blows/Foot scale: 10 20 30 40

Sampler Type (ST):

▯ 2" OD Split Spoon Sampler (SPT)	☐ No Recovery	M - Moisture
▯ 3" OD Split Spoon Sampler (D & M)	◧ Ring Sample	▽ Water Level ()
✋ Grab Sample	▨ Shelby Tube Sample	▼ Water Level at time of drilling (ATD)

Logged by: KB
Approved by: CJK