GEOTECHNICAL ENGINEERING REPORT

PROPOSED SINGLE-FAMILY RESIDENTIAL ADDITIONS
8429 SE 33rd Place
Mercer Island, Washington 98040

Project No. 2537.01
28 January 2022

Prepared for:
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ZGA Project No. 2537.01
28 January 2022

Kevin & Suzette Piper
8429 SE 33rd Place
Mercer Island, Washington 98040

Attention: Mr. and Mrs. Piper

Subject: Geotechnical Engineering Report
Proposed Single-family Residential Additions
8429 SE 33rd Place
Mercer Island, Washington 98040

Dear Mr. and Mrs. Piper:

In accordance with your request and written authorization, Zipper Geo Associates, LLC (ZGA) has completed the subsurface exploration and geotechnical engineering evaluation for the proposed single-family residential additions at the above-referenced address. This report presents the findings of the subsurface exploration and geotechnical recommendations for the project. Our work was completed in general accordance with our *Scope of Geotechnical Engineering Services and Fee Estimate* (Proposal No. P21305) dated 7 December 2021. Written authorization to proceed was provided by you on 9 December 2021. We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further assistance, please contact us.

Respectfully submitted,
Zipper Geo Associates LLC

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Signed 1.28.22

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INTRODUCTION

This report documents the subsurface conditions encountered at the site and our geotechnical engineering recommendations for the proposed project. The project description, site conditions and our geotechnical conclusions and design recommendations are presented in the text of this report. Supporting data including detailed exploration logs and field exploration procedures, as well as results of laboratory testing are presented as appendices.

Our geotechnical engineering scope of services for the project included a site reconnaissance, subsurface evaluation, laboratory testing and preparation of this report. The subsurface evaluation consisted of completing three geotechnical hand auger borings (HA-1 to HA-3). The explorations extended to depths of approximately 3 ½ to 10 ½ feet below existing grade.

Figure 1, the Site and Exploration Plan, presents the approximate locations of our subsurface explorations. Appendix A contains a description of our field procedures and exploration logs. Appendix B contains a description of the laboratory testing procedures and the test results. Appendix C contains a geotechnical engineering report, dated 30 May 2007, prepared by Robert M. Pride, LLC concerning restoration of a landslide that impacted the property in 2007.

SITE DESCRIPTION

The property is located at the east end of a cul-de-sac that borders an undeveloped and wooded area on a steep slope extending downward to the east. The site is bordered to the north and south by developed single-family residential parcels. The dwelling has a daylight basement configuration with the two-story portion at the rear and facing east. Extensive concrete flatwork, stone walks, and irrigated and illuminated landscaping surround the dwelling. A level lawn extends east of the dwelling and borders a tied-back soldier pile retaining wall that was constructed circa 2007 following a landslide that impacted the rear of the property. The lawn is supported by fill material placed behind the retaining wall.

PROJECT UNDERSTANDING

It is our understanding that the proposed project includes constructing building, deck and stairway additions along the east (rear) of the dwelling, and a building addition and new deck in the area of the patio along the south side. Ground disturbance will be reduced by cantilevering additions to the extent feasible and supporting the proposed deck at the southeast portion of the dwelling on four pin piles that were installed in 2007 in anticipation of future expansion to the east. We understand that the additions are being planned to reduce ground disturbance to the extent feasible as part of efforts to reduce intrusion into mapped geologic hazards and their buffers.

SITE CONDITIONS

Surface Conditions
The site currently supports a single-family dwelling with a shed to the southwest as shown on Figure 1, the Site and Exploration Plan. The dwelling and driveway are in the upper west and middle portions of the site, while the year yard extends east of the dwelling at a grade consistent with the dwelling's daylight basement. The ground surface elevation adjacent to the dwelling's main floor is about 270 feet while the rear lawn is at about 262 feet. A maximum 6-foot tall rockery at the southeast dwelling corner effects the grade transition between the south patio and the rear yard, while a stone walkway does the same at the north.

The rear yard borders a soldier pile wall with a maximum height of about 11 feet and undeveloped slopes descending to the east. The wall was constructed, along with some grading of the yard, as part of restoration efforts completed in response to a landslide that impacted the area in 2007. The descending slopes to the east of the yard and wall are locally steep, with inclinations as great as approximately 98 percent as indicated on Figure 1. The east slope vegetation consists of mature evergreen and deciduous trees, vines, and sparce brush on the forest floor. The areas adjacent to the dwelling support extensive landscaping, including shrubs, small trees, landscaping stepping stones, light, irrigation, and a concrete patio slab on the south side of the dwelling. We observed that the south patio was in a serviceable condition and did not exhibit evidence of significant settlement.

Subsurface Conditions

Published Geologic Mapping
The *Geologic Map of Surficial Deposits in the Seattle 30' by 60' Quadrangle,* Washington (by Yount, JC, Minard, JP, and Dembroff, GR) published by US Geological Survey, indicates the site is underlain by Vashon glacial till deposited during the Fraser Glaciation in the late Pleistocene period. Vashon till is described as a light to dark gray, non-sorted, non-stratified mixture of clay, silt, sand and gravel up to boulder-size and being very stiff and impermeable. Older Vashon advance outwash deposits are also mapped in the area. Vashon advance outwash is described as slightly oxidized, light red-brown gravel and sand and light brown to gray silt and clay, moderately- to well-sorted, well stratified. Normally consolidated recessional outwash, typically sand and gravel with a variable silt and cobble content, has been mapped nearby as well.

Soil Conditions
Our subsurface evaluation consisted of excavating three hand auger borings (HA-1 through HA-3) on January 5, 2022. The approximate exploration locations are illustrated on Figure 1. Detailed descriptive logs presenting the subsurface conditions encountered and the procedures utilized in the subsurface

exploration program are presented in Appendix A. Generalized descriptions of subsurface soil conditions observed in specific areas of the site are presented below.

Variations in subsurface conditions exist between the exploration locations and the nature and extent of variations between the explorations may not become evident until construction. Stratification boundaries on the logs represent the approximate depth of changes in soil types, although the transition between materials may have been gradual. If variations become apparent during construction, it may be necessary to reevaluate the recommendations of this report.

At the location of boring HA-1, we observed about 8 inches topsoil over loose, wet, brown, silty sand, trace to with gravel, and trace broken glass fill to a depth of about 3 feet below existing grade. From 3 feet to 5 feet, we observed the soils to consist of loose, wet, brown, sand, with silt. From 5 feet to 10 feet, we observed the soils to consist of loose to medium dense, wet, brown, silty sand to sand with silt, with the soil grading with trace silt and to medium dense. We observed what we interpreted to be native medium dense sand with trace silt at approximately 9 feet, and the boring was terminated at about 10 feet below existing grade.

Hand auger HA-2 disclosed about 2 inches of fine mulch between landscaping shrubs over about 10 inches of topsoil. From about 1 foot to 3 feet we observed the soils to consist of loose, wet, brown, silty sand, trace gravel fill. Hand auger HA-2 was terminated at 3 feet below existing grade due to encountering a suspected utility at the bottom of the boring.

At the location of hand auger boring HA-3 we observed grass over about 6 inches of topsoil. From ½ foot to about 6 ½ feet we observed soft, wet, brown, silt with some sand, with the soil grading to native stiff to very stiff, moist to wet, brown, sandy SILT about 4 feet below existing grade. The boring was terminated at about 6 ½ feet below existing grade.

Groundwater

We did not observe groundwater at the time of drilling in any of the three hand augers. The presence of groundwater may vary depending on seasonal precipitation, site utilization, and other factors. Groundwater may tend to perch within the fill material above the dense to very dense unweathered glaciated soils, which typically has a low permeability, during the winter and spring and during extended periods of precipitation. Fluctuation of the groundwater level will likely occur due to seasonal variations in the amount of rainfall, runoff, lake level and other factors not evident at the time the explorations were performed. Therefore, groundwater levels during construction or at other times in the life of the dwellings may vary from the conditions we observed.

CONCLUSIONS AND RECOMMENDATIONS

General Considerations

In our opinion, construction of the proposed site additions appears feasible from the geotechnical perspective utilizing conventional, shallow, isolated, and continuous foundations or driven pin piles where the depth to bearing soils is sufficiently deep that construction of conventional shallow foundations is not practical.

The following sections of this report present specific geotechnical recommendations for the project. Our recommendations are based on the observed soil conditions at specific exploration locations. Differing soil conditions than those observed at the exploration locations may become evident during construction. The risk of such differing conditions is elevated on sites where uncontrolled fill was placed in association with prior development. Our recommendations are further based on the assumption that earthwork for site grading, utilities, foundations, floor slabs, and pavements will be monitored by a qualified geotechnical engineer.

Regulated Environmental Geologic Hazard Critical Areas

Chapter 19 of the Mercer Island City Code (MICC) regulates development activities in critical areas and their associated buffers. The property has been mapped by the City of Mercer Island as having potential landslide, steep slope, erosion, and seismic geologic hazard critical areas. The approximate extents of these regulated hazards on and near the property are shown on the Critical Areas Map, Figure 2. The City may allow alteration within a regulated geologic hazard area or buffer if the proposal effectively demonstrates that there is no impact on the regulated areas or that it adequately mitigates risks of the hazards.

The proposed improvements include very small additions along the eastern end of the south side of the dwelling and a small addition extending east of the south end of the east side of the building. Grading will consist only of temporary excavations associated with new foundation construction in level portions of the site; there will be no grade changes associated with the proposed improvements. Our conclusions regarding the nature of regulated geologic hazards and the potential impacts of the proposed site improvements are summarized below.

<u>Landslide/Steep Slope Hazard</u>

Portions of the site and adjacent areas to the east meet the criteria for landslide and steep slope hazards by virtue of having slope segments with 10 or more feet of relief and inclinations of 40 percent or greater. A landslide impacted the eastern portion of the site in 2007 due to a leaking landscape irrigation system, but not due to natural causes. Circumstances regarding the landslide and its subsequent restoration through the retaining wall construction, grading, and drainage improvements are described in the *Report on Geotechnical Investigation, Emergency Repair of Landslide Failure* prepared by Robert M. Pride, LLC (30 May 2007) which is attached in Appendix C. The report documents the circumstances resulting in the landslide, subsurface conditions, and the tied back driven pin pile and treated timber retaining wall that was construction in the back yard as mitigation of the landslide. The location of the retaining wall is shown on Figure 1. Figure 1 also illustrates the top of the regulated steep slope at the rear of the property.

Seismic Hazards

Seismic hazard areas are those subject to severe risk of damage as a result of earthquake-induced ground shaking, slope failure, settlement or subsidence, soil liquefaction, surface faulting, or tsunamis. City mapping as shown on Figure 2 includes a very small area of the driveway at the northwest corner of the site as within a seismic hazard area, and we suspect that this is likely due to mapping of non-glacially consolidated outwash soils at this location. The explorations completed for our evaluation disclosed fill material above medium dense to dense recessional outwash soils. The May 2007 Robert M. Pride, LLC report describes similar conditions, with the outwash underlain by glacially consolidated till or older fine-grained Olympia deposits. We did not observe groundwater at our exploration locations, nor did the Robert M. Pride, LLC report describe observing groundwater, including in the area of the 2007 landslide. Based on these conditions, it is our opinion that the risk of liquefaction and associated settlement is low. Given the site location, it would not be subject to tsunamis.

Our authorized scope of services did not include advancing borings and completing numerical stability analysis of the steeper slopes at the eastern portion of the site and beyond, but we point out that this area is not included within the City-mapped seismic hazard area. According to the US Geological Survey online Quaternary fault mapping website, splays of the Southern Whidbey Island Fault have been mapped about 600 feet south of the property. The splays are estimated to be less than 15,000 years old and are estimated to have a slip rate ranging from approximately 0.2mm to 1.0 mm per year. Given presently available mapping, the proximity of the splays is such that the risk of fault rupture at the site is low, in our opinion. Given the above, it is our opinion that the risk of a seismic event presenting a severe risk of damage is low, and as such, the site does not meet the MICC definition of a seismic hazard.

Erosion Hazard

Erosion hazards are generally described as areas containing soils which are at high risk from water erosion according to the mapped description units of the US Department of Agriculture NRCS. NRCS mapping for the site describes the Kitsap silt loam, 2 to 8 8 slopes (KpB) in the western to middle portions of the site, while the Kitsap silt loam, 15 to 30 percent slopes (KpD) soils have been mapped in the eastern portion of the site. The KpD soils are described as presenting a severe risk of erosion. Based on the published mapping, it appears that the portions of the site inclined at 15 percent or greater are consistent with erosion hazards as defined by the MICC.

It should be noted that the areas where extensions to the dwelling are proposed, the patio at the south side and the yard at the east side are level. Provided that construction is completed in accordance with TBMPs contained in a City-approved TESC plan, it is our opinion that the risk of sediment generation and off-site sediment transport will be low.

Minimal Risk Statement

The foundations for the proposed extensions and deck at the southeastern portion of the dwelling, which are expected to consist of both conventional shallow foundations and driven pin piles, are located as close as approximately 20 feet from the regulated steep slopes and the 2007 landslide repair retaining wall, and consequently, inside the buffers. As such, the foundations will not exert additional loading on the regulated steep slopes and the lack of grade changes will not alter existing surface water drainage. For these reasons, and consistent with the requirements of MICC19.07.160(B), we have concluded the following:

Based on the favorable geologic conditions observed at the stie, neither the proposed construction work or the completed project will subject people or property, including areas off site, to an increased risk of associated impacts, in our opinion. The proposed improvements have been designed so that the risk to the site and adjacent property are such that the site is determined to be safe. Construction practices as proposed for the alteration would render the development as safe as if it were not located in a geologically hazardous area and would not adversely impact adjacent properties.

Site Preparation

Existing Utility Removal: We recommend that all underground utilities within the proposed building addition footprints be completely removed if they are not going to be reused. Utility pipes outside the building envelope could be abandoned in place, provided they are fully grouted with controlled density fill (CDF) and the trench backfill is density tested to verify that it meets the compaction levels recommended herein. Localized excavations made for removal of utilities or existing unsuitable trench backfill should be backfilled with structural fill as recommended subsequently.

Erosion Control Measures: Stripped surfaces and soil stockpiles are typically a source of runoff sediments. We recommend that silt fences, berms, and/or swales be installed around the downslope side of stripped areas and stockpiles in order to capture runoff water and sediment. If earthwork occurs during wet weather, we recommend that all stripped surfaces be covered with straw to reduce runoff erosion, whereas soil stockpiles should be protected with anchored plastic sheeting.

Temporary Drainage: Stripping, excavation, grading, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and provide proper control of erosion. The site soils have a high fines (soil particles finer than the US No. 200 sieve) content and are highly susceptible to disturbance and erosion when wet. The site should be graded to prevent water from ponding in construction areas and/or flowing into and/or over excavations. Exposed grades should be crowned, sloped, and compacted to a smooth surface at the end of each day to facilitate drainage if inclement weather is forecasted. Accumulated water must be removed from subgrades and work areas immediately and prior to performing further work in the area. Equipment access may be limited and the amount of soil rendered unfit for use as structural fill may be greatly increased if drainage efforts are not accomplished in a timely manner. Successful drainage of saturated zones due to accumulations of surface water would be relatively slow due to the fines content of the soils. Instead, aeration or removal and replacement may be necessary.

Demolition and Stripping: Once surface runoff is controlled, the areas of the proposed additions should be stripped of topsoil and vegetation, along with portions of the south patio as necessary. Based on our observations, we estimate that organic material stripping depths may range from about from about 6 to 12 inches. However, deeper areas of organic-rich soil, such as in planters may be encountered and should be removed to the recommended depth determined in the field by the owner's geotechnical representative.

Subgrade Preparation: Once site preparation is complete, all areas of exposed subgrade that do not require over-excavation and are at design subgrade elevation, such as the south patio, should be compacted to a firm and non-yielding condition. Some moisture conditioning of site soils may be required to achieve a moisture content appropriate for compaction. During periods of extended wet weather, this could entail aeration and drying, although it may not be feasible depending on weather conditions and space available to spread wet soils. During the drier summer months, blending moisture into dry of optimum soils may be necessary. A suitable moisture content is generally within ±2 percent of the soil's optimum moisture content.

Earthwork should be completed during drier periods of the year when the soil moisture content can be controlled by aeration and drying. If earthwork or construction activities take place during extended periods of wet weather, or if the *in situ* moisture conditions are elevated above the optimum moisture content, the soils could become unstable or not be compactable. In the event the exposed subgrade becomes unstable, yielding, or unable to be compacted due to high moisture conditions, we recommend that the materials be removed to a sufficient depth in order to develop stable subgrade soils that can be compacted to the minimum recommended levels. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the subgrade soils during wet weather and wet site conditions.

Once compacted, floor slab, and foundation subgrades should be evaluated through density testing to assess the subgrade adequacy and to detect soft and/or yielding soils. In the event that the soils are not firm and unyielding, the upper 12 inches of subgrade should be scarified and moisture conditioned as necessary to obtain at least 95 percent of the maximum laboratory density (per ASTM D 1557). Those soils which are soft, yielding, or unable to be compacted to the specified criteria should be over-excavated and replaced with suitable material as recommended in the Structural Fill section of this report. In the event that it is not feasible to proof roll the subgrades, we recommend that they be observed and evaluated by a qualified geotechnical consultant.

Freezing Conditions: If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be compacted prior to placing subsequent lifts of structural fill. Alternatively, the frozen material could be stripped from the subgrade to expose unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

Structural Fill Materials and Placement

Structural fill includes any material placed below foundations, slabs, within utility trenches, and behind retaining walls. Prior to the placement of structural fill, all surfaces to receive fill should be prepared as previously recommended in the Site Preparation section of this report.

Laboratory Testing: Representative samples of on-site and imported soils to be used as structural fill should be submitted for laboratory testing at least four days in advance of its intended use in order to complete the necessary Proctor tests.

Re-use of Site Soils as Structural Fill: It is our opinion that the non-organic native and fill soils encountered on the site are suitable for reuse as general structural fill from a compositional standpoint provided they are placed and compacted in accordance with the recommendations presented in this report. Some of the site soils may be wet of optimum at the time of construction and will require moisture conditioning (drying) prior to use as structural fill. The re-use of site soils as structural fill during wet weather will be difficult or impossible due to their moisture sensitivity. Re-using over-optimum soils during periods of wetter, cooler weather would likely require stabilization with Portland cement. We recommend that site soils used as structural fill have less than 4 percent organics by weight and have no woody debris greater than ½ inch in diameter. We recommend that all pieces of organic material greater than ½ inch in diameter be picked out of the fill before it is compacted. Organic-rich soil derived from earthwork activities should be used in landscaping areas or be wasted from the site.

Imported Structural Fill: Imported structural fill may be required due to weather, wet soil conditions, or other reasons. The appropriate type of imported structural fill will depend on the prevailing weather conditions. During extended periods of dry weather when soil moisture can be controlled, we recommend that imported fill meet the requirements of Common Borrow, Options 1 or 2, as specified in Section 9-03.14(3) of the Washington State Department of Transportation, *Standard Specifications for Road, Bridge, and Municipal Construction*. The non-organic on-site soils would be classified as Common Borrow. During wet weather, higher-quality (lower fines content) structural fill might be required, as Common Borrow may contain sufficient fines to be moisture sensitive. During wet weather we recommend that imported structural fill meet the general requirements of Gravel Borrow as specified in Section 9-03.14(1) of the WSDOT Standard Specifications although we recommend that the fines content be limited to 5 percent based on the soil fraction passing the ¾-inch sieve.

Moisture Content: The suitability of soil for use as structural fill will depend on the prevailing weather at the time of construction, the *in situ* moisture content of the soil, and the fines content of the soil. As the amount of fines increases, the soil becomes increasingly sensitive to small changes in moisture content. Soils containing more than about 5 percent fines (such as the on-site soils) cannot be consistently compacted to the appropriate levels when the moisture content is more than approximately 2 percent above or below the optimum moisture content (per ASTM D 1557). Optimum moisture content is that moisture content which results in the greatest compacted dry density with a specified compactive effort.

Fill Placement: Structural fill should be placed in horizontal lifts not exceeding 10 inches in loose thickness. Each lift of fill should be compacted using compaction equipment suitable for the soil type and lift thickness. Each lift of fill should be compacted to the minimum levels recommended below based on the maximum laboratory dry density as determined by the ASTM D 1557 Modified Proctor Compaction Test. Moisture content of fill at the time of placement should be within plus or minus 2 percent of optimum moisture content for compaction as determined by the ASTM D 1557 test method.

Compaction Criteria: We recommend compacting structural fill placed below new foundations or new patio slab to at least 95 percent of the modified Proctor maximum dry density per ASTM D 1557. We recommend that a geotechnical engineer be present during grading so that an adequate number of density tests may be conducted as structural fill placement occurs. In this way, the adequacy of the earthwork may be evaluated as it proceeds.

Temporary and Permanent Slopes

Temporary excavation slope stability is a function of many factors, including:

- The presence and abundance of groundwater;
- The type and density of the various soil strata;
- The depth of cut;
- Surcharge loadings adjacent to the excavation; and
- The length of time the excavation remains open.

As the cut is deepened, or as the length of time an excavation is open, the likelihood of bank failure increases; therefore, maintenance of safe slopes and worker safety should remain the responsibility of the contractor, who is present at the site, able to observe changes in the soil conditions, and monitor the performance of the excavation.

It is exceedingly difficult under the variable circumstances to pre-establish a safe and "maintenance-free" temporary cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe temporary slope configurations since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered. We recommend the contractor make a determination of excavation side slopes based on classification of soils encountered at the time of excavation in accordance with the guidelines presented in Section 296-155, Part N of the Washington State Administrative Code and applicable construction industry specific guidelines. Adjustments to the slope angles should be determined by the contractor at that time. Unsupported vertical slopes or cuts deeper than 4 feet are not recommended if worker access is necessary. The cuts should be adequately sloped, shored, or supported to prevent injury to personnel from local sloughing and spalling. Based on our observations, the soil likely to be exposed in excavations will be consistent with the Type C classification.

Seismic Design Criteria

The 2018 IBC indicates that the seismic site classification is based on the average soil and bedrock properties in the top 100 feet. To determine the Site Class, we used the data from boring HA-1 and HA-3. The current scope does not include a 100-foot soil profile determination. The seismic site class definition recommended in the following table considers that soils encountered at depth in our borings continue below the termination depth. Per the 2018 IBC seismic design procedures and ASCE 7-16, we recommend using Site Class D for seismic design.

Foundation Considerations

Hand auger boring HA-1 was advanced at the proposed location of the addition at the south side of the dwelling and disclosed loose fill material to a depth approaching 9 feet; we anticipate that this is near the likely base of the original excavation made to construct the daylight basement. The existing loose fill material is inadequate for support of new foundations given the likelihood of settlement. We recommend that the new foundations on the south side of the dwelling be designed and constructed so that they are supported on the following:

1. At least medium dense native soil below the existing loose fill,

2. Controlled Density Fill (CDF) placed above at least medium dense native soils, or

3. Pin piles driven to refusal in the native soils below the fill.

For cases 1 and 2 above, we also recommend that the new foundations be located below a line with a 2H:3V (Horizontal:Vertical) slope extending upward toward the ground surface from the basement finished floor elevation at the south side of the basement wall. This configuration will allow the new foundations to be constructed without imparting new lateral loads on the existing basement wall. This recommended foundation configuration is illustrated on Figure 3, the South Exterior Foundation Depth Schematic.

In the case where CDF is used to backfill excavations below footings, the excavation only needs to extend 6 inches beyond the limits of the foundation regardless of the depth of over-excavation. The resulting excavation should be backfilled with CDF having a minimum 28-day compressive strength of 100 psi.

Our recommendations for conventional shallow foundations and driven pin pile foundations are presented below.

Shallow Foundations

Allowable Bearing Pressure: We recommend supporting conventional spread footing foundations on at least medium dense/stiff native soils or above properly compacted structural fill or CDF with a 100 psi compressive strength placed above adequate native soils. Continuous and column footings bearing as described may be designed for a maximum allowable, net, bearing capacity of 2,000 psf. A one-third increase of the bearing pressure may be used for short-term dynamic loads such as wind and seismic forces.

Shallow Foundation Depth and Width: For frost protection, the bottom of all exterior footings should bear at least 18 inches below the lowest adjacent outside grade, whereas the bottoms of interior footings should bear at least 12 inches below the surrounding slab surface level. We recommend that all continuous wall and isolated column footings be at least 12 and 24 inches wide, respectively.

Lateral Resistance: We recommend considering ultimate base friction and passive earth values of 0.5 and 400 pcf equivalent fluid pressure, respectively. Appropriate safety factors should be used when evaluating lateral resistance. We recommend that passive resistance be neglected in the upper 18 inches of embedment.

Estimated Settlement: Assuming the foundation subgrade soils are prepared in accordance with recommendations presented herein, we estimate that total and differential settlements will be less than 1 inch and 1/2 inch, respectively.

Pin Pile Foundation Recommendations

We recommend that the use of driven pin piles be considered for support of the new addition to the south, and if the locations are appropriate, using the existing four driven 4-inch inside-diameter piles for the proposed deck on the east of the dwelling. The purpose of installing driven pin piles at the south side of the dwelling would be to transmit building loads through the load-sensitive loose fill material behind the basement wall to the underlying denser native soils. The use of driven pin piles for support of foundations is feasible from the geotechnical perspective.

Pin piles comprise relatively small diameter steel pipes which are driven into the ground with a pneumatic or hydraulic driver to a designated “refusal” criterion. Pile sections of about 5 to 11 feet are commonly used. Successive pile sections are either compression coupled or welded. Once the piles are installed, they are cut off to a pre-determined elevation, and lengths of reinforcing steel or top plates are generally welded to the top. The tops of the piles are then incorporated into a new foundation. The installation of pin piles generates ground vibrations. These ground vibrations are not typically sufficient to cause direct vibration structural damage, but can be sufficient to cause densification of loose soils and therefore settlement of buildings supported nearby on such loose soils. Additionally, the ground vibrations can be sufficient to result in cosmetic damage such as cracked interior finishes.

Pin piles in common use locally range from 2-inch to 6-inch inside-diameter. Based on our experience with other projects of a similar nature, we anticipate that the use of 2-inch inside-diameter Schedule 80, or 3-inch inside-diameter, Schedule 40 piles would be feasible.

Axial Compressive Capacity

Allowable axial compressive capacities for piles driven to "refusal" in the native soils are listed in the table below. Applied loads should include static and dynamic loads. These values incorporate a factor of safety of at least 2. A lateral capacity should not be assigned to piles driven plumb. Lateral loads should be accommodated by batter piles or passive earth pressures on the below-grade portion of the foundation or grade beam.

DESCRIPTION	PILE CRITERIA
Allowable axial compressive capacity[1]	2-inch ID Schedule 80 pile: 6 kips 3-inch ID Schedule 40 pile: 12 kips
Anticipated settlement	Less than 1 inch
Minimum percussion driver weight (pounds)	2-inch ID pile: 90 3-inch ID pile: 650
"Refusal" [2]	2-inch ID pile: Maximum 1 inch penetration over 1 minute of sustained driving 3-inch ID pile: Maximum 1 inch penetration over 15 seconds of sustained driving

1. The recommended allowable axial pile capacity incorporates a minimum Factor of Safety of 2.
2. "Refusal" recommendation is based upon the specific listed minimum driver weight. Heavier drivers will necessitate load testing in order to determine an appropriate refusal time.

It is our understanding that the proposed deck that will extend east from the south side of the east side of the dwelling may rely on the existing 4-inch pin piles that were installed in 2007 and described in the Robert M. Pride, LLC report dated May 30, 2007 (attached as Appendix C). It is our understanding that these test piles from the soldier pile wall were driven to refusal and embedded about 20 feet below existing grade. However, the pile installation summary does not describe the type of hammer used to install the piles, nor does it indicate the allowable axial compressive capacity of the piles as installed. We recommend evaluating the axial capacity of one of the piles via a load test completed in accordance with the *Quick Load Test Method for Individual Piles* described in *Standard Test Method for Piles Under Static Axial Compressive Load* per ASTM D-1143. We recommend that ZGA observe the load test as part of determining the piles' allowable axial compressive load.

We do not anticipate that the proposed deck will not impose particularly high axial compressive loads on the existing 4-inch pin piles. In the event that load testing of the existing 4-inch pin piles is determined to be infeasible because of cost or logistics, it would be feasible to install either 2-inch or 3-inch inside-diameter pin piles inside the 4-inch piles per the recommendations above.

Pin Pile Foundation Construction Considerations

Each of the piles should be driven to "refusal" per the criteria listed in the Axial Compressive Capacity section above. We recommend engaging the services of a contractor experienced with the installation of pin piles. The piles should be installed with a driver large enough that the allowable axial compressive capacity described previously can be developed. The axial compressive capacity for hammers larger than those listed in the table above should be evaluated following the completion of a load test conducted on an installed pile per the method described above. We recommend that a ZGA representative observe the pile installation and refusal criteria achievement as well as any load tests.

Please note that the presence of obstructions may make it difficult or impossible to install piles at the design location. If the obstructions cannot be removed, it may be necessary to relocate piles on an as-needed basis.

On-Grade Concrete Slabs

Subgrade Preparation: We understand that some of the south side patio will be removed and a new one constructed as part of the proposed improvements. We observed loose fill soils at the locations of borings HA-1 and HA-2. We recommend compacting these soils to at least 95 percent density to a depth of at least 12 inches prior to constructing the new slab.

Drainage Considerations

Final site grades should be sloped to carry surface water away from the additions and other drainage-sensitive areas. Additionally, site grades should be designed such that concentrated runoff on softscape surfaces is avoided.

CLOSURE

The analysis and recommendations presented in this report are based, in part, on the explorations completed for this study. The number, location, and depth of the explorations were completed within the constraints of budget and site access so as to yield the information to formulate our recommendations. Project plans were in the preliminary stage at the time this report was prepared. We therefore recommend we be provided an opportunity to review the final plans and specifications when they become available in order to assess that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design.

The performance of earthwork, structural fill, foundations, and pavements depend greatly on proper site preparation and construction procedures. We recommend that Zipper Geo Associates, LLC be retained to provide geotechnical engineering services during the earthwork-related construction phases of the project. If variations in subsurface conditions are observed at that time, a qualified geotechnical engineer could provide additional geotechnical recommendations to the contractor and design team in a timely manner as the project construction progresses.

REFERENCE: TOPOGRAPHIC SURVEY, BY APEX ENGINEERING, DATED 12/20/2021, SHEET 1 OF 1

Piper Residence Additions
8429 SE 33rd Place
Mercer Island, Washington

Site and Exploration Plan

Date: January 2022	Job No. 2537.01
Zipper Geo Associates, LLC 19019 36th Ave. W.,Suite E Lynnwood, WA	FIGURE 1 SHT.1 of 1

City of Mercer Island Property Hazard Report

Site Address: 8429 SE 33RD PL8429 SE 33RD PL #A

Parcel #: 6666800250 **Report Generated on** October 26, 2021

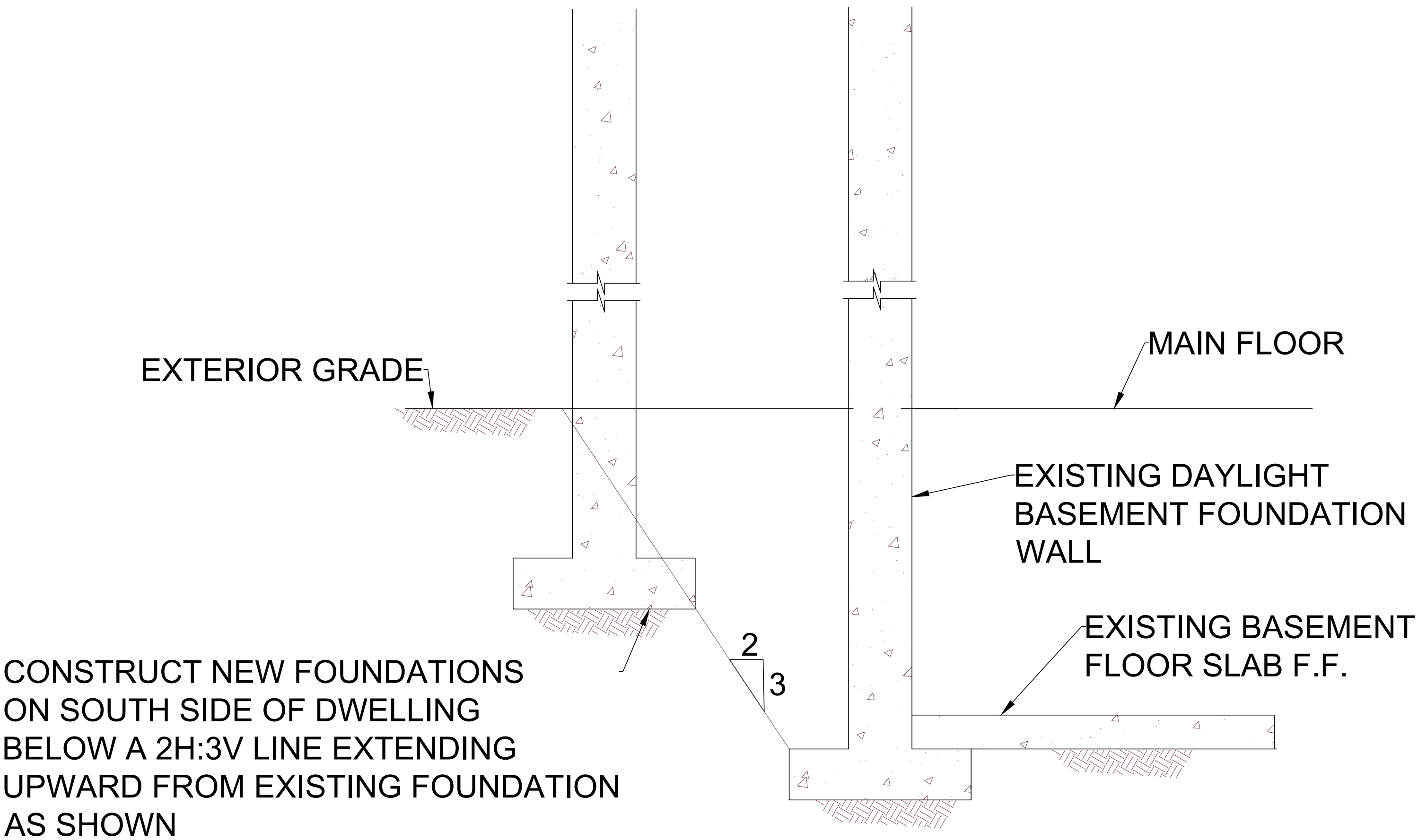
Potential Slide:

Steep Slope:

Erosion:

Seismic:

Piper Residence Additions 8429 SE 33rd Place Mercer Island, Washington 98040	
CRITICAL AREAS MAP	
Date: January 2022	Job No. 2514.01
Zipper Geo Associates, LLC 19019 36th Ave. W.,Suite E Lynnwood, WA	FIGURE 2 SHT. 1 of 1



NOT TO SCALE

Piper Residence Additions
8429 SE 33rd Place
Mercer Island, Washington 98040

SOUTH EXTERIOR FOUNDATION DEPTH SCHEMATIC

Date: January 2022 | Job No. 2514.01

Zipper Geo Associates, LLC
19019 36th Ave. W.,Suite E
Lynnwood, WA

FIGURE 3
SHT. 1 of 1

APPENDIX A

FIELD EXPLORATION PROCEDURES AND LOGS

FIELD EXPLORATION PROCEDURES AND LOGS

Field Exploration Description

Our field exploration for this project included advancing three hand auger explorations completed on January 5, 2022, the approximate locations of which are shown on the enclosed Site and Exploration Plan, Figure 1. Exploration locations were determined in the field by measuring distances from existing site features with a fiberglass tape relative to a 20 December 2021 *Topographic Survey* prepared by Apex Engineering. Ground surface elevations at the explorations interpolated from topographic lines presented on the referend survey. As such, the exploration locations and elevations should be considered accurate only to the degree implied by the means and methods to establish them. The following sections describe our procedures associated with the explorations. Descriptive logs of the explorations are enclosed in this appendix.

Hand Auger Procedures

A staff engineer from our firm advanced a 3.5-inch diameter auger by hand, continuously observing the soil cuttings as they were retrieved. Representative portions of the soils retrieved were placed in moisture tight containers and returned to our laboratory for further visual classification and testing.

Granular soil density and cohesive soil consistency were determined through the use of the Dynamic Cone Penetrometer in general accordance with ASTM Special Technical Publication No. 399. This procedure entails driving a steel rod equipped with a specifically sized conical tip into the ground with a 15-pound drop hammer free falling 20 inches. After seating the penetrometer into the soil a depth of 2 inches, the number of blows required to drive the penetrometer over three successive 1.75-inch intervals is recorded and averaged. This numerical value, "N_C", has been shown to correlate with the Standard Penetration Test (ASTM: D-1586) "N" value. The N_C values for specific depths are indicated on the logs in this appendix.

The enclosed hand auger logs indicate the vertical sequence of soils and materials encountered in each exploration, based primarily on our field classifications and supported by our subsequent laboratory testing. Where a soil contact was observed to be gradational or undulating, our logs indicate the average contact depth. Our logs also indicate the approximate depths of any sidewall caving or groundwater seepage observed in the explorations, as well as all sample numbers and sampling locations.

ZIPPER GEO ASSOCIATES, LLC

19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

Hand Auger Boring HA-1

Location: See Site and Exploration Plan, Figure 1
Approx. Ground Surface Elevation: 270 feet

Project: Piper Residence Additions
Project No: 2537.01
Date Excavated: 01/05/22

Depth (ft)	Material Description	Sample	N_C	%M	Testing
1	Very loose, wet, dark brown, sandy SILT, trace gravel (Topsoil)	S-1		28	MC
2	Loose, wet, brown, silty SAND, trace to with gravel, trace broken glass (Fill).	S-2		21	MC
3		S-3		20	MC
4	Loose, wet, brown, SAND, with silt.				
5		S-4		24	MC
6	Soft to medium stiff, wet, brown, sand SILT to SAND with silt.	S-5	5	24	GSA
7		S-6	4.7	25	MC
8					
9	Medium dense to dense, moist to wet, brown, SAND, trace silt				
10		S-7	64	18	MC
11	Boring terminated at approximately 10 ft. Groundwater was not observed at time of drilling.				
12					
13					
14					
15					
16					
17					
18					
	Note: N_C is the Dynamic Cone Penetrometer blow count per 1.75 inch interval measured in accordance with ASTM Special Technical Publication #399.				

ZIPPER GEO ASSOCIATES, LLC

19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

Hand Auger Boring HA-2

Location: See Site and Exploration Plan, Figure 1
Approx. Ground Surface Elevation: 269 feet

Project: Piper Residence Additions
Project No: 2537.01
Date Excavated: 01/05/22

Depth (ft)	Material Description	Sample	N_C	%M	Testing
1	2 inches of fine mulch between landscaping shrubs over loose, wet, dark brown, sandy SILT, with roots and organics (Topsoil).	S-1		30	MC
2					
3	Loose, wet, brown, silty SAND, trace gravel (Fill).	S-2		27	MC
4	Boring terminated at approximately 3 ½ ft due to suspected utilities. Groundwater was not observed at time of drilling.				
5					
6					
7					
8					
9					
10					
11					
12					
13					
14					
15					
16					
17					
18					
	Note: N_C is the Dynamic Cone Penetrometer blow count per 1.75 inch interval measured in accordance with ASTM Special Technical Publication #399.				

ZIPPER GEO ASSOCIATES, LLC
19019 36th Avenue West, Suite E, Lynnwood, Washington 98036

	Hand Auger Boring HA-3 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: 262 feet	**Project:** Piper Residence Additions **Project No:** 2537.01 **Date Excavated:** 01/05/22			
Depth (ft)	**Material Description**	**Sample**	**N_C**	**%M**	**Testing**
1	Grass over about 6 inches of loose, wet, dark brown, sandy SILT, with roots and organics (Topsoil).	S-1		29	MC
2	Soft, wet, brown, SILT, some sand (Probable fill)	S-2		31	MC
3					
4	Stiff to very stiff, moist to wet, brown, SILT, some sand				
5		S-3	18	27	GSA
6					
7		S-4	56	25	MC
8	Boring terminated at approximately 6 ½ ft. Groundwater was not observed at time of drilling.				
9					
10					
11					
12					
13					
14					
15					
16					
17					
18					
	Note: N_C is the Dynamic Cone Penetrometer blow count per 1.75 inch interval measured in accordance with ASTM Special Technical Publication #399.				

APPENDIX B
LABORATORY TESTING PROCEDURES AND RESULTS

LABORATORY TESTING PROCEDURES AND RESULTS

A series of laboratory tests were performed during the course of this study to evaluate the index and geotechnical engineering properties of the subsurface soils. Descriptions of the types of tests performed are given below.

Visual Classification

Samples recovered from the exploration locations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in moisture tight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was generally done in accordance with ASTM D 2488. Visual soil classification includes evaluation of color, relative moisture content, soil type based upon grain size, and accessory soil types included in the sample. Soil classifications are presented on the exploration logs in Appendix A.

Moisture Content Determinations

Moisture content determinations were performed on representative samples obtained from the explorations in order to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM D 2216. Moisture contents are presented on the exploration logs in Appendix A.

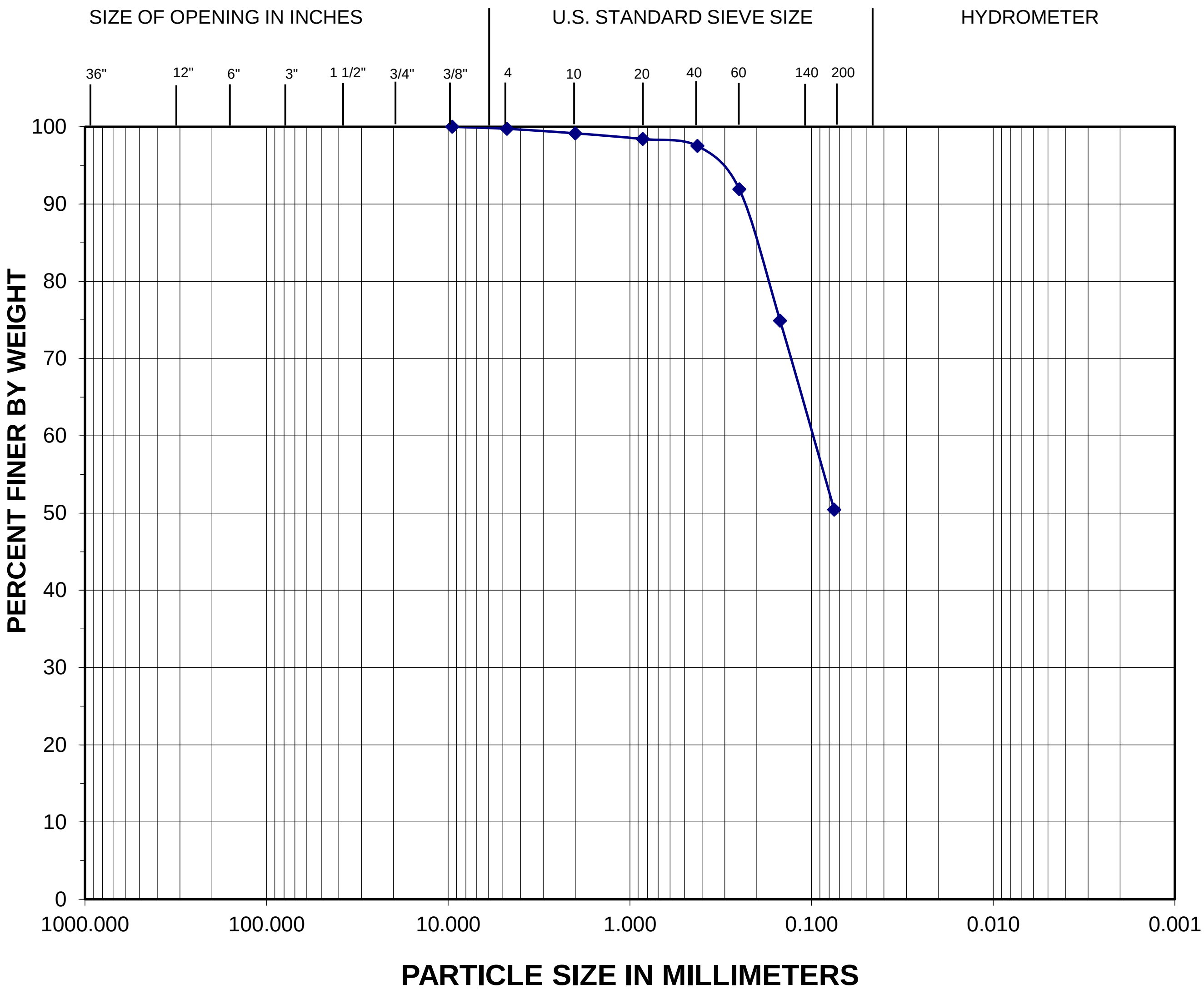
Grain Size Analysis

A grain size analysis indicates the range in diameter of soil particles included in a particular sample. Grain size analyses were performed on representative samples in general accordance with ASTM D 6913. The results of the grain size determinations for the samples were used in classification of the soils, and are presented in this appendix.

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Comments: Trace to some organics in sample

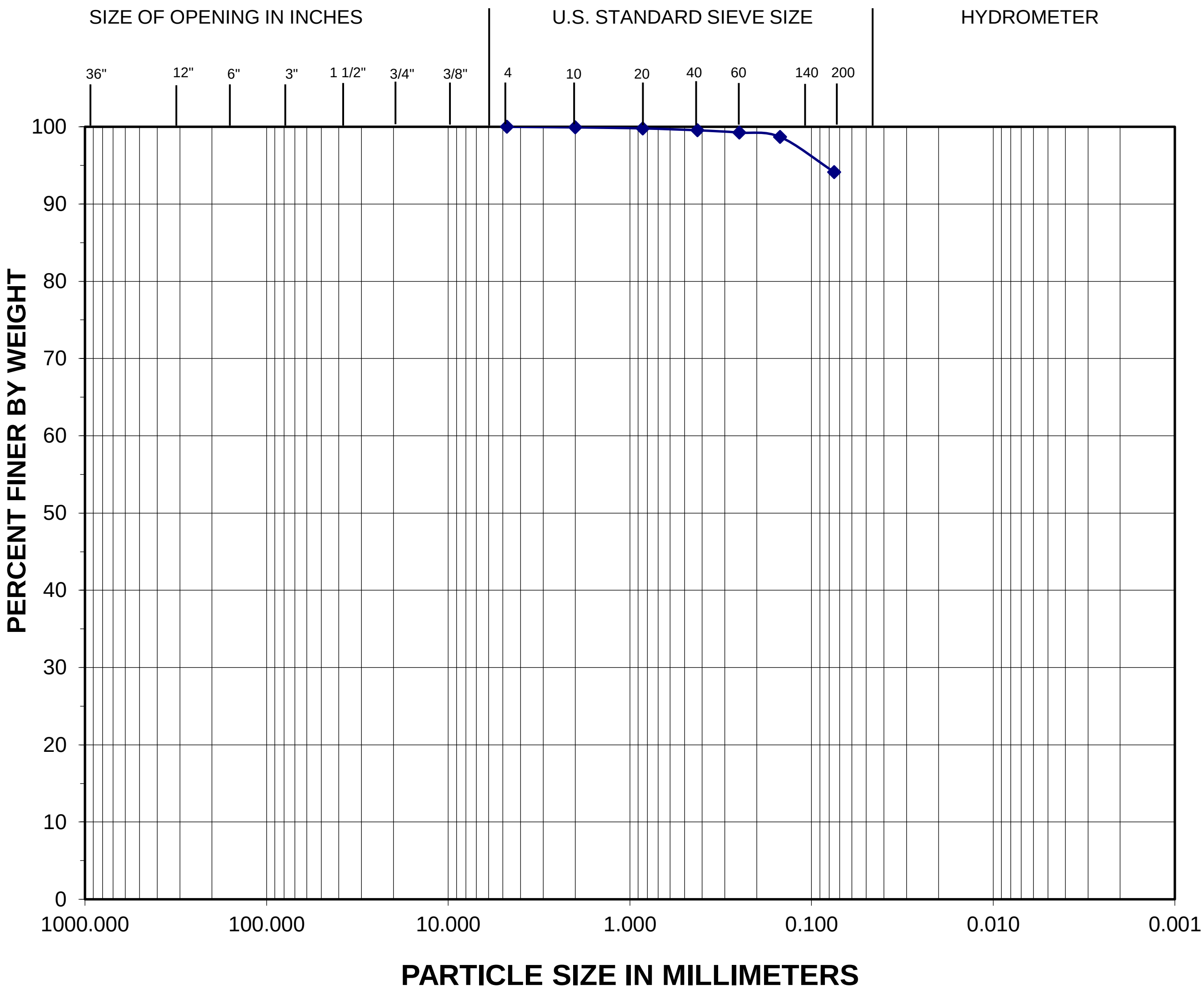
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
HA-1	S-5	5-5 1/2 feet	24.1	50.4	sandy SILT

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2537.01 DATE OF TESTING: 1/6/2022	PROJECT NAME: Piper Residence Additions

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



BOULDERS	COBBLES	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
		GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
HA-3	S-3	4-4 1/2 feet	26.5	94.1	SILT some sand

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2537.01 DATE OF TESTING: 1/6/2022	PROJECT NAME: Piper Residence Additions

APPENDIX C

ROBERT M. PRIDE, LLC MAY 2007 GEOTECHNICAL REPORT

Consulting Engineer

Robert M. Pride, LLC

May 30, 2007

Mr. Kevin Piper
8429 SE 33rd Place
Mercer Island, WA 98040

Re: **Report on Geotechnical Investigation**
Emergency Repair of Landslide Failure
8429 SE 33rd Place
Mercer Island, Washington
Project No. 07-145-01

Dear Mr. Piper,

This report presents the results of our geologic and geotechnical investigation of the slope failure at the rear of your residence at8429 SE 33rd Place Mercer Island. It is understood that this recent failure occurred along the easterly side of the house as a result of an irrigation system malfunction over the past two weeks.

The purpose of this report is to document subsurface soil and groundwater conditions, and to provide recommendations for the design of a new slope retention system. Structural calculations and design drawings are being prepared by Lucia Engineering that we will be submitted to the City of Mercer Island for permitting. Reference materials include recent geologic mapping (Troost – 2006) and the site topography provided by Touma Surveying.

Site Conditions

The two story residence is located at the easterly end of 33rd Place, and is situated at the top of a deep drainage ravine. A level building pad was created for the house and some fill soils were pushed out to create the easterly yard area. Observation of the slope failure showed that most of the fill that created this rear yard area was lost in the slide that carried much of these soils down into the drainage ravine.

Geologic exploration was performed by an engineering geologist to identify the subsoil profile extending below the rear yard. Several test pits were dug into the exposed steep slope to classify the native soils below the upper rear yard fill. Refer to the attached cross section on Drawing No 2 that shows the geologic profile based on our geologic site reconnaissance mapping.

Since the slide removed most of the loose fill at the top of the slope, there is only a minimal amount of fill remaining. A well-defined topsoil zone was seen under the fill, and below the organic topsoil layer were medium dense to dense silty sands that were classified as recessional outwash deposits (Qvr). At some depth below the recessional silty sands is glacial till (Qvt) and/or pre-Olympia deposits. Test pits dug into the steep slide area encountered about 12 to 18 inches of the native silty sands that led into the dense recessional soils that underlie the entire slide area. No groundwater seepage was observed over the exposed slide area.

Geotechnical Recommendations

At the present time the east foundation wall of the house ranges from 5 to more than 10 feet to the top of the slide scarp. While there is no immediate danger to the house, it is imperative that slope repairs be implemented as soon as possible to avoid additional loss of slope that could endanger the east foundation wall.

Our recommendation for repair of the existing slide and to protect the house foundation is to install a pipe pile wall as indicated on Drawing 1 attached to this report. Installation of the wall will require partial removal of landscaping along the north side of the property to allow access for equipment to install the pipe piles. Once the wall is constructed, the rear yard area will be brought back to grade with compacted backfill soil. A new subsurface drain line will be installed along the base of the wall and there will be area drains in the grass area of the rear yard. The new drains will help control surface runoff and aid in the protection of the lower portion of the slope.

Pipe Pile Wall Installation

Driven pipe pile walls are commonly used to repair slope failures and to provide essential lateral retention of adjacent foundation walls. Treated wood lagging is set between adjacent pipe piles to retain the soil backfill that will be placed up to the original yard grade. Steel walers will need to be installed below the top of the new wall, and tieback anchors then drilled into the native soils under the existing residence to provide for lateral restraint for the new pipe pile wall.

A tieback anchor consists of an augured hole at an inclination below horizontal behind the face of the planned wall. The tieback extends beyond the retained soils into competent sediments beyond the active zone. A steel cable or rod is then inserted into the augured hole and the back of the anchor is grouted in place to resist lateral movement. The front of the cable or rod is then loaded to pre-tension the anchor and locked into place on the front of the soldier pile wall. Alternatively helical anchors can be used with the same capacity.

Due to the steep descent of the slope below the planned pile locations, we recommend that a single row of tieback anchors be installed at a depth of about four feet below the top of the wall to resist the lateral loading on the wall. This pipe pile wall may be designed using an equivalent fluid pressure of 40 pounds per cubic foot (pcf) for the active earth pressure acting over the full height of the lagged wall. The wall should be designed to the height necessary to allow restoration of the previous grades. All pipe piles should be driven at least 8 feet into the underlying dense silty sands to be verified by field inspection during their installation.

The vertical load of the pile (including the pile weight and vertical component of the tieback anchor) may be resisted by end bearing for pipe piles driven to refusal. An allowable end bearing capacity of 10 kips for a 3 inch diameter pipe pile may be used. This value includes a factor of safety of 2.

Tieback anchors must be embedded into the dense recessional soils to a length of at least 20 feet for a 10 to 12 foot high wall, and 15 feet for a 5 to 8 foot high wall. The tieback anchors may be designed to utilize an allowable shaft friction of 500 psf acting beyond the active earth zone behind the wall. The active pressure zone at the back of the wall may be assumed to be a line extending up at a 60 degree inclination from the horizontal from the base of the piles.

All of the anchors should be performance tested prior to locking off to the design load. The performance testing should include at least one load test to 200 percent of the design load and all of the anchors should be loaded to 130 percent of design load. These tests should conform to the recommendations of the Post-Tensioning Institute for verification testing and proof-loading of production anchors. All anchor testing should be verified by a representative of this office.

The pipe pile spacing, pile embedment lengths and finish wall heights should be determined by the structural engineer. We recommend that two qualified pipe pile contractors experienced in this type of wall construction be contacted to discuss access conditions, pile installation requirements, and estimated costs for this work. All of the shoring installation operations should be observed and documented by a representative of this office.

Drainage Considerations

As previously stated, prior to backfilling behind the new wall, a drain should be installed at the back of the base of the temporary excavation. The drain should consist of 4 inch diameter perforated corrugated pipe surrounded by a minimum of 12 inches of washed drain rock or pea gravel. A "chimney drain" of pea gravel or drain rock should extend from the drain pipe up the back of the lagged wall to within 1 foot of the finish grade to collect potential ground water seepage emanating from the slope.

If free-draining backfill is used to backfill the upper portion of the wall, the chimney drain portion does not need to be installed in this area. However, the upper 1 foot of the backfill should not be free draining material as surface water should not be allowed to infiltrate behind the wall. Final rear yard grades should be designed with a slight gradient for surface water flow to new area drains. All drains should be installed to allow gravity discharge to the drainage ravine below the east side of the property.

Summary

We recommend that we review the final structural plans to determine that they are consistent with the recommendations of this report. Construction monitoring and consultation services should also be provided to verify that subsurface conditions are similar to those reported in the field explorations. Should conditions be revealed during construction that differ from the anticipated subsurface profile, we will evaluate those conditions and provide alternative recommendations where appropriate.

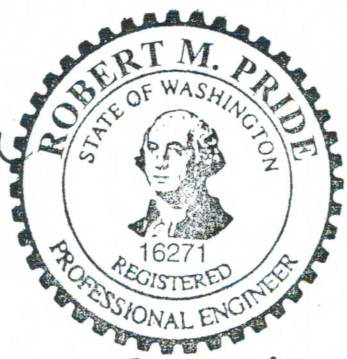
Field construction services should be considered an extension of this initial geotechnical investigation, and are essential to the determination of compliance with the project drawings and specifications. Such activities would include site and temporary slope excavations; subgrade preparation for floor slabs and pavement; subdrain installations; foundation excavations; and fill placement and compaction.

The conclusions and recommendations presented in this report are based on 1) our interpretation and evaluation of soil conditions between and beyond exploration locations, 2) confirmation of the actual subsurface conditions encountered during construction, and 3) the assumption that sufficient observation and testing will be provided during appropriate phases of the work.

Our findings and recommendations of this report were prepared in accordance with generally accepted principles of geotechnical engineering as practiced in the Puget Sound area at the time this report was submitted. We make no warranty, either express or implied.

Please call me if you have any questions, or you wish to discuss this report in greater detail.

Respectfully submitted,



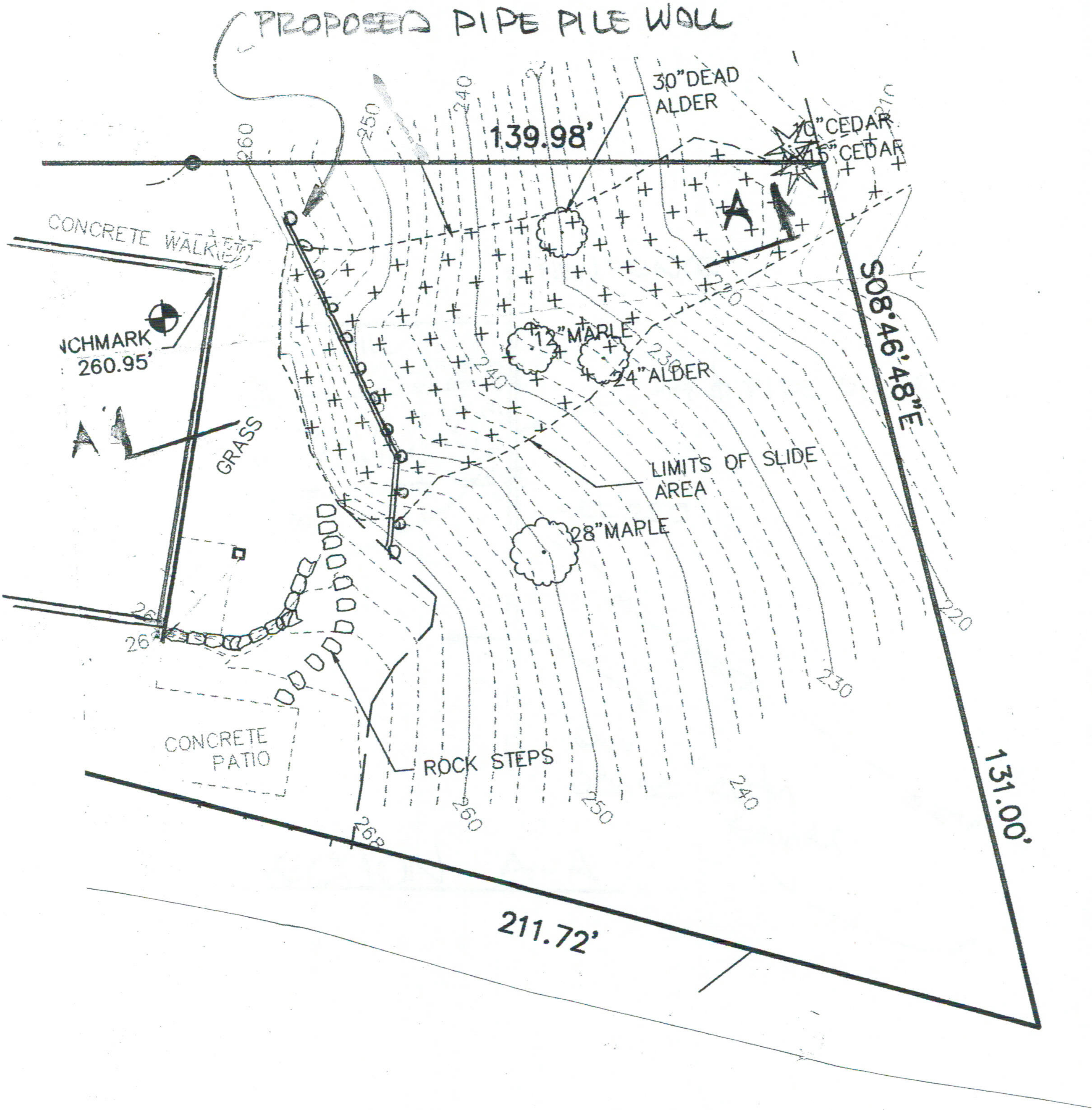
EXPIRES 7-20-08

Robert M. Pride, P. E.
Principal Geotechnical Engineer

dist: (3) Addressee
encl: Drawing No. 1 and 2

rmp: PiperSlope1

13203 Holmes Point Drive NE Kirkland, WA 98034
Phone: 425-814-3970 Fax: 425-814-5672



SITE PLAN

Proposed Slope Repair
8429 SE 33rd Place
Mercer Island, Washington

Project No. 07-145-01

Drawing No. 1

Robert M. Pride, LLC

Consulting Engineer

Yonemitsu Geological Services (YGS) Daily Report Applied Earth Sciences

Date: August 14, 2007 Project: Piper Residence Slide Repair

Equipment In use: Pipe Pile Hammer McDowell Weather: Sunny and cool

Report Number 2 07-1219-012 Perm # 0706-147

Piper Residence Slide Repair
8427 33rd Place
Mercer Island, Washington.

We were onsite at the request of Robert M. Pride to observe the installation of 4-inch diameter pipe piles for foundation support for addition of to the existing residence. Four scheduled 4-inch pipe piles were driven yesterday. Based on observation of the driving resistance, the piles meet refusal. Pipe pile hammer was set up on the two northern most piles (3 and 4) and were test driven for specifications of refusal. Both piles were driven for 16 seconds with no movement downward on the pipe pile. This exceeds the requirement of 1-inch per 6 seconds of driving. The stick up above ground is presented below:

- Pile 1 - Southern most 3.33 ft above ground 21 foot pipe pile
- Pile 2 - 2.2 feet above ground 21 foot pipe pile
- Pile 3 - 3.5 feet above ground 21 foot pipe pile
- Pile 4 - 2.5 feet above ground 21 foot pipe pile

Conclusions

The installation of the 4-inch pipe piles were installed in accordance with the design specifications presented in construction drawings. All piles were approved;

ROBERT M. PRIDE
STATE OF WASHINGTON
16271
REGISTERED
PROFESSIONAL ENGINEER

EXPIRES