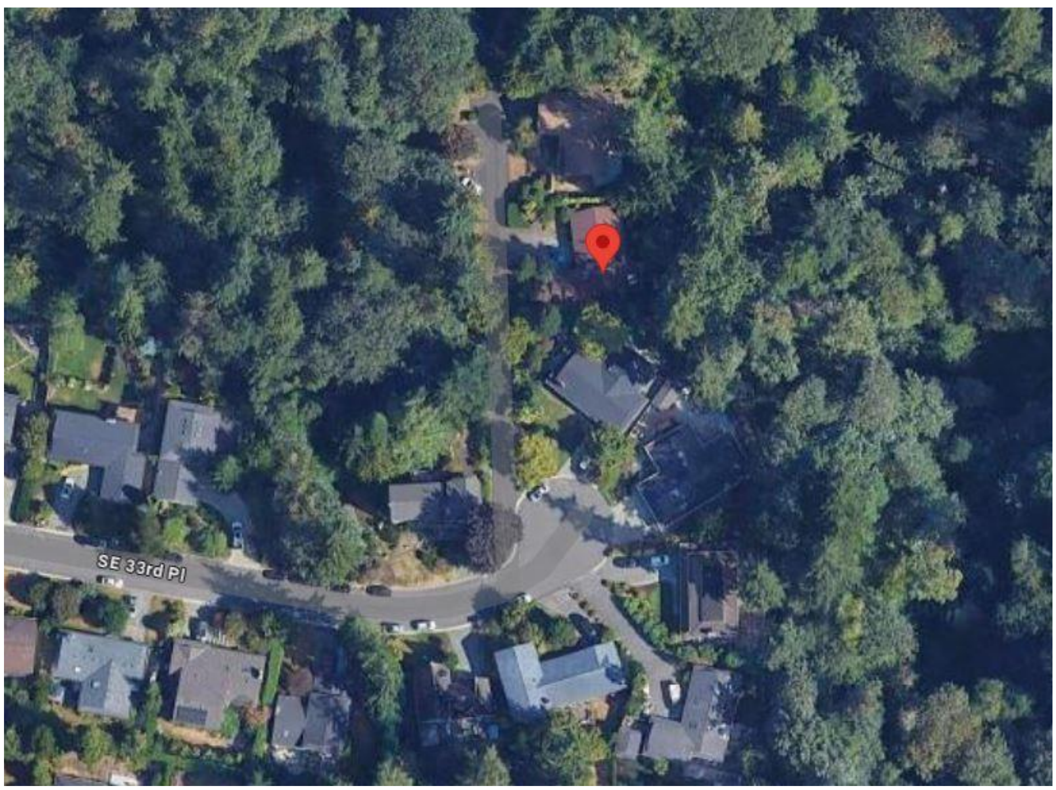
GEOTECHNICAL ENGINEERING REPORT

PROPOSED SINGLE-FAMILY RESIDENTIAL REMODEL
8441 SE 33rd Place
Mercer Island, Washington 98040

Project No. 2727.01
28 November 2023

Prepared for:
Mr. Nathan Korpela



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ZGA Project No. 2727.01
28 November 2023

Mr. Nathan Korpela
8407 Linden Avenue North
Seattle, Washington 98103

Subject: Geotechnical Engineering Report
Proposed Single-family Residential Remodel
8441 SE 33rd Place
Mercer Island, Washington 98040

Dear Mr. Korpela:

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 remodel 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. P22162) dated 31 May 2023. Written authorization to proceed was provided by you on 9 June 2023. 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

Sincerely,
Zipper Geo Associates LLC

11.28.23

David C. Williams, LG, LEG
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11.28.23

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MERCER ISLAND, WASHINGTON

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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 four geotechnical hand auger borings (HA-1 to HA-4). The explorations extended to depths of approximately 2 ½ to 4 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.

SITE DESCRIPTION

The property is located along the east side of a private drive that extends north from the east end of the SE 33rd Place cul-de-sac that borders an undeveloped and wooded area on a steep slope extending downward to the east. The 0.27-acre 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. Concrete flatwork, gravel-surfaced paths, a wood deck and steps, and landscaping surround the dwelling. A concrete driveway accesses the existing garage from SE 33rd Place at the west. The dwelling was constructed in 1966.

PROJECT UNDERSTANDING

It is our understanding that the proposed remodeling includes adding a new garage and living space to the northwest portion of the dwelling, relocating much of the north foundation wall to the south, removing the steps near the southeast portion of the dwelling, and establishing a path along the north side of the dwelling. No expansion of the dwelling toward the steep slope at the east is planned.

SITE CONDITIONS

Surface Conditions

The site currently supports a single-family dwelling as shown on Figure 1, the Site and Exploration Plan. The dwelling and driveway are in the middle to western portions of the site, while a narrow gravel-surfaced trail and small at-grade wood deck section at the daylight basement grade extend to the east and adjacent to the steep east slope. The ground surface elevation adjacent to the dwelling's main floor is about 258 feet while the path at the rear is at about 250 feet. A small, approximately 6-foot tall, rockery at the southeast corner of the dwelling effects the grade transition between the rear path and a small concrete slab and at-grade wood deck adjoining a wood stairway at the southeast. A second rockery, approximately 5 to 5 ½ feet tall extends from the dwelling's northeast foundation wall corner to the adjacent property boundary to the north.

The above-referenced improvements at the rear of the dwelling border an undeveloped slope descending to the east. The descending slope has ground surface elevations ranging from about 250 feet down to 198 feet at the east property line. The east slope vegetation consists of mature evergreen and deciduous trees, vines, and thick brush on the forest floor. A wood-chip surface trail with treated wood/pin pile steps extended part of the way from the rear path toward the east property line. We observed an arcuate and steeply sloped feature in the northeastern portion of the property and near the slope toe that we have interpreted as a relic landslide scarp, the approximate location of which is shown on Figure 1.

We observed that the majority of the mature trees on the east slope, which included some up to 53 inches diameter breast height (dbh) were characterized by straight trunks; i.e., the trees generally did not exhibit curved trunks or leaning orientations, characteristics that frequently indicate significant downslope soil displacement. This includes the area immediately below the relic landslide scarp which supported a straight trunked, 24-inch dbh mature maple tree.

We did not observe flowing surface water or groundwater seepage on the steep eastern slope during our site visits. We did observe some slight puddling of water on the gravel path at the rear of the dwelling during significant rain events. Much of the water at the puddle location was derived as runoff dripping from the main level wood deck that extends above the southern portion of the gravel path.

We observed that the visible portions of the perimeter foundation, and the dwelling's exterior brickwork, were in a serviceable condition and did not exhibit evidence of significant settlement. With the exception of the relic landslide scarp referenced above, we did not observe tension cracks, lobate mounds, or terrace features that may be considered representative of recent or ongoing slope instability.

Subsurface Conditions

Published Geologic Mapping

The October 2006 *Geologic Map of Mercer Island, Washington* (by Troost, KG and Wisher, AP) indicates that 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 immediate 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. Based on our observations, advance outwash is the prevalent soil type at the site that is likely to be encountered in excavations associated with the planned remodeling. Please note that given the developed nature of the site, fill material may be expected in the form of foundation wall and underground utility trench backfill.

Soil Conditions

Our subsurface evaluation consisted of excavating four hand auger borings (HA-1 through HA-4) on 20 June 2023. 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.

Hand auger boring HA-1, advanced in a landscaping area in the footprint of the proposed new garage, disclosed about 2 inches of organic-rich sod above loose silty sand with trace gravel, roots, and some fine and fibrous organic material that extended to about 10 inches below grade. Native advance outwash consisting of dense silty sand with trace gravel and scattered cobbles extended to the boring's 2.6-foot termination depth. The boring was terminated in nested gravel.

The remaining hand auger borings disclosed fill material consisting of pea gravel, some plastic sheeting, crushed rock, and waste concrete to depths of approximately 3 to 16 inches. The fill was underlain by native advance outwash consisting of medium dense to dense sand with a variable silt and gravel content to the borings' termination depths.

The moisture content of samples of the native sand that we tested ranged from about 8 to 20 percent. The wetter soils were observed at the boring HA-4 location in the gravel path along the east side of the dwelling where water from the deck above was ponding. Native soils exposed on the east slope below the dwelling were consistent with advance outwash.

Groundwater

We did not observe groundwater at the time of drilling. Groundwater may tend to perch within fill material above the denser native soils during the winter and spring and during extended periods of precipitation. Fluctuation of the groundwater levels will likely occur due to seasonal variations in the amount of rainfall, runoff, 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 dwelling may vary from the conditions we observed.

CONCLUSIONS AND RECOMMENDATIONS

General Considerations

In our opinion, the proposed remodel appears feasible from the geotechnical perspective utilizing conventional, shallow foundations. 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 observed 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 being within the following regulated geologic hazard areas or their buffers: potential landslide, steep slope, erosion, and seismic. The approximate extents of these regulated hazards/buffers on and near the property as mapped by the City are shown on the Critical Areas Map, Figure 2. The steep slope, potential slide, and erosion hazards are all related to the eastern slope. The City may allow alteration within a regulated geologic hazard area or buffer if the alteration proposal effectively demonstrates that there is no impact on the regulated areas or that it adequately mitigates risks of the hazards.

The proposed remodel will include the addition of a new garage on the west side of the dwelling and about 25 feet from the regulated steep slope at the rear of the dwelling. The plans provided for our review do not show expansion of the dwelling footprint toward or into the eastern steep slope. Grading will consist of temporary excavations associated with partial removal of the north foundation wall and construction of a new one a few feet south of the existing one, construction of the garage, some excavation along the north side of the dwelling to establish a path, and likely some excavation/backfilling associated with relocation of some of the utilities in the front yard. Otherwise, there will be no significant 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.

Landslide/Steep Slope Hazard

The slope to the east of the dwelling meets 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. We have interpreted the landslide scarp we identified near the toe of the slope and located approximately as shown on Figure 1 as an old dormant feature. There has been no significant downslope displacement of the soil below the scarp for decades as evidenced by the straight-trunked 24-inch diameter maple tree growing at the base of the scarp.

As previously described, the site lacks tension cracks, lobate mounds, or terraces, features that are oftentimes associated with slope instability. The dwelling lacks evidence of settlement, further attesting to the site's overall stability.

A landslide impacted the steep east-facing slope of the residential property at 8429 SE 3rd Place (about 300 feet south of the project site) in 2007. However, that landslide was the result of a leaking landscape irrigation system, and 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 included in Appendix C.

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 in the lower eastern portion 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 some fill material above dense native advance outwash soils. We did not observe groundwater at our exploration locations, nor did we observe groundwater seepage on the eastern slope below the dwelling. 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 steep east slope 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 900 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 0.2 mm 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 percent 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 when disturbed. 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 some grading will occur, including for the new garage and the new north foundation wall, are relatively level. Provided that construction is completed in accordance with BMPs 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 north foundation wall is about 5 feet from the regulated east slope, and consequently, inside the critical area buffers. The new north foundation wall and new interior foundations will be within the footprint of the existing building. The closest portion of the new garage will be about 25 feet from the east slope. Consequently, the new foundations will not exert new loading on the regulated slope and the proposed grade change will not significantly alter existing surface water drainage. We understand that the functional result of the remodel, which will include reconfiguration of the roof, is that less stormwater is likely to flow to the east compared to current conditions. 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 site, neither the proposed construction work nor 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.

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. Based upon our review of the plans received to date, it is our opinion that mitigation of potential landslide hazard impacts associated with the proposed remodel has been achieved by having no disturbance of the landslide hazard area itself and by limiting the proposed building alterations to a small footprint within an area that was graded during initial site construction. Limiting the proposed alterations to level portions of the site will also effectively mitigate potential adverse impacts to the portions of the site inclined at 15 percent or greater which are erosion hazard areas per the MICC.

Site Preparation

Existing Utility Removal

We recommend that all underground utilities within the proposed building addition footprint 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's sand soils have a moderate fines (soil particles finer than the US No. 200 sieve) content and will be 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.

Demolition and Stripping

Once surface runoff is controlled, the proposed garage addition area should be stripped of topsoil and vegetation, along with the existing driveway as necessary. Based on our observations, we estimate that organic material stripping depths may range from about 4 to 6 inches. However, deeper areas of organic-rich soil may be encountered and should be removed to the recommended depth determined in the field by the owner's geotechnical representative. The north foundation wall and the adjacent portions of the east and west walls should be removed as necessary, along with the existing basement floor slab in the area of demolition.

Subgrade Preparation

Once site preparation is complete, all areas of exposed subgrade that do not require over-excavation and are at design subgrade elevation 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 hand probing to assess the subgrade adequacy and to detect loose 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 loose, 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.

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 re-use 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. 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 (which includes some of 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 concrete flatwork to at least 95 percent of the modified Proctor maximum dry density per ASTM D 1557. We recommend compacting structural fill placed in landscaping areas to between 88 and 90 percent. 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 existing fill soil likely to be exposed in excavations will be consistent with the Type C classification, while the undisturbed dense native soils is consistent with the Type B classification, in our opinion.

Seismic Design Criteria

IBC Seismic Design Parameters**:** 2018 IBC Seismic Design parameters are summarized in the table below.

Criteria	Factor
2018 International Building Code (IBC) [1]	C [2]
S_S Spectral Acceleration for a Short Period	1.399g (Site Class B)
S_1 Spectral Acceleration for a 1-Second Period	0.487g (Site Class B)
F_a Site Coefficient for a Short Period	1.2
F_v Site Coefficient for a 1-Second Period	1.5
S_{MS} Maximum considered spectral response acceleration for a Short Period	1.679g (Site Class C)
S_{M1} Maximum considered spectral response acceleration for a 1-Second Period	0.73g (Site Class C)
S_{DS} Five-percent damped design spectral response acceleration for a Short Period	1.119g

Criteria	Factor
S_{D1} Five-percent damped design spectral response acceleration for a 1-Second Period	0.487g

1. In general accordance with ASCE 7-16
2. The 2018 International Building Code, and by reference ASCE 7-16, considers a site soil profile determination extending a depth of 100 feet for seismic site classification. The current authorized scope did not include the required 100-foot soil profile determination. The hand auger borings advanced as part of our evaluation extended to a maximum depth of approximately 4-1/2 feet and this seismic site class definition considers that dense to very dense soils as noted on the published geologic mapping exist below the maximum depth of the subsurface exploration. Additional exploration to greater depths could be considered to confirm the conditions below the current depth of exploration, if necessary.

Foundation Considerations

The explorations disclosed a variable depth of fill material mantling native advance outwash sand which is adequate for support of conventional shallow foundations. Our recommendations for conventional shallow foundations are presented below.

Allowable Bearing Pressure

We recommend supporting conventional spread and continuous foundations on at least medium dense 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 2,500 psf. A one-third increase of the bearing pressure may be used for short-term dynamic loads such as wind and seismic forces. We recommend providing ZGA the opportunity to observe the foundation excavation subgrades prior to placement of forms and reinforcing steel.

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. Settlements should occur relatively soon after loading given the granular nature of the soils.

Backfilled Permanent Retaining Walls

Lateral Earth Pressures

The lateral soil pressures acting on backfilled retaining walls will depend on the nature and density of the soil behind the wall, and the ability of the wall to yield in response to the earth loads. Yielding walls (i.e. walls that are free to translate or rotate) that are able to displace laterally at least 0.001H, where H is the height of the wall, may be designed for active earth pressures. Non-yielding walls (i.e. walls that are not free to translate or rotate) should be designed for at-rest earth pressures. Non-yielding walls include walls that are braced to another wall or structure, and wall corners.

Assuming that walls are backfilled and drained as described in the following paragraphs, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (active earth pressure). Non-yielding walls should be designed using an equivalent fluid density of 50 pcf (at-rest earth pressure). Surcharge pressures due to sloping backfill, adjacent footings, vehicles, construction equipment, etc. must be added to these lateral earth pressure values.

For yielding walls with level backfill conditions, we recommend that a uniformly distributed seismic pressure of 4.5H psf for the active case and 9.0H psf for the at-rest case, where H is the height of the wall, be applied to the walls.

The above equivalent fluid pressures are based on the assumption of no buildup of hydrostatic pressure behind the wall. If groundwater is allowed to saturate the backfill soils, hydrostatic pressures will act against a retaining wall; however, if the recommended drainage system is included with each retaining wall, we do *not* expect that hydrostatic pressures will develop.

Drainage

Adequate drainage measures must be installed to collect and direct subsurface water away from subgrade walls. All backfilled walls should include a drainage aggregate zone extending at least 12 inches from the back of wall for the full height of the wall. The drainage aggregate should consist of material meeting the requirements of WSDOT 9-03.12(2) Gravel Backfill for Walls. A minimum 4-inch diameter, perforated PVC drainpipe should be provided at the base of backfilled walls to collect and direct subsurface water to an appropriate discharge point. Drainpipe perforations should be protected using a non-woven geotextile such as Mirafi 140N. Wall drainage systems should be independent of other drainage systems such as roof drains.

On-Grade Concrete Slabs

Subgrade Preparation: In the event that the existing driveway slab at the proposed addition is removed and a new slab is constructed along with the proposed garage extension, the existing subgrade and any new fill placed beneath the floor slab should be compacted to a minimum of 95 percent of the modified Proctor maximum dry density. It is possible that some previously placed fill material or disturbed native soil below the driveway slab will not meet this recommended level of compaction. As such, the actual amount of over-excavation and replacement will be dependent on the horizontal and vertical extent of loose soil and its relative compaction. Existing fill material that is not compacted to a minimum of 95 percent of the modified Proctor maximum dry density should be reworked and compacted in order to achieve the minimum recommended compaction levels.

Capillary Break: We recommend the on-grade interior slabs be underlain by a 6-inch thick layer of compacted granular fill consisting of coarse sand and fine gravel containing less than 5 percent fines, based on that soil fraction passing the US No. 4 sieve. Alternatively, a clean angular gravel such as No. 7 aggregate per WSDOT: 9-03.1(4)C could be used for this purpose. Alternative capillary break materials should be submitted to ZGA for review and approval before use.

Vapor Barrier: The use of a vapor retarder should be considered beneath concrete slabs-on-grade that will be covered with wood, tile, carpet or other moisture-sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture or is otherwise considered moisture-sensitive. When conditions warrant the use of a vapor retarder, the slab designer and contractor should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Subgrade Modulus: We recommend a vertical modulus of subgrade reaction of 225 pounds per cubic inch (pci) be used for design of a new garage slab if one is constructed. This value is suitable for soil compacted to at least 95 percent density per ASTM D 1557.

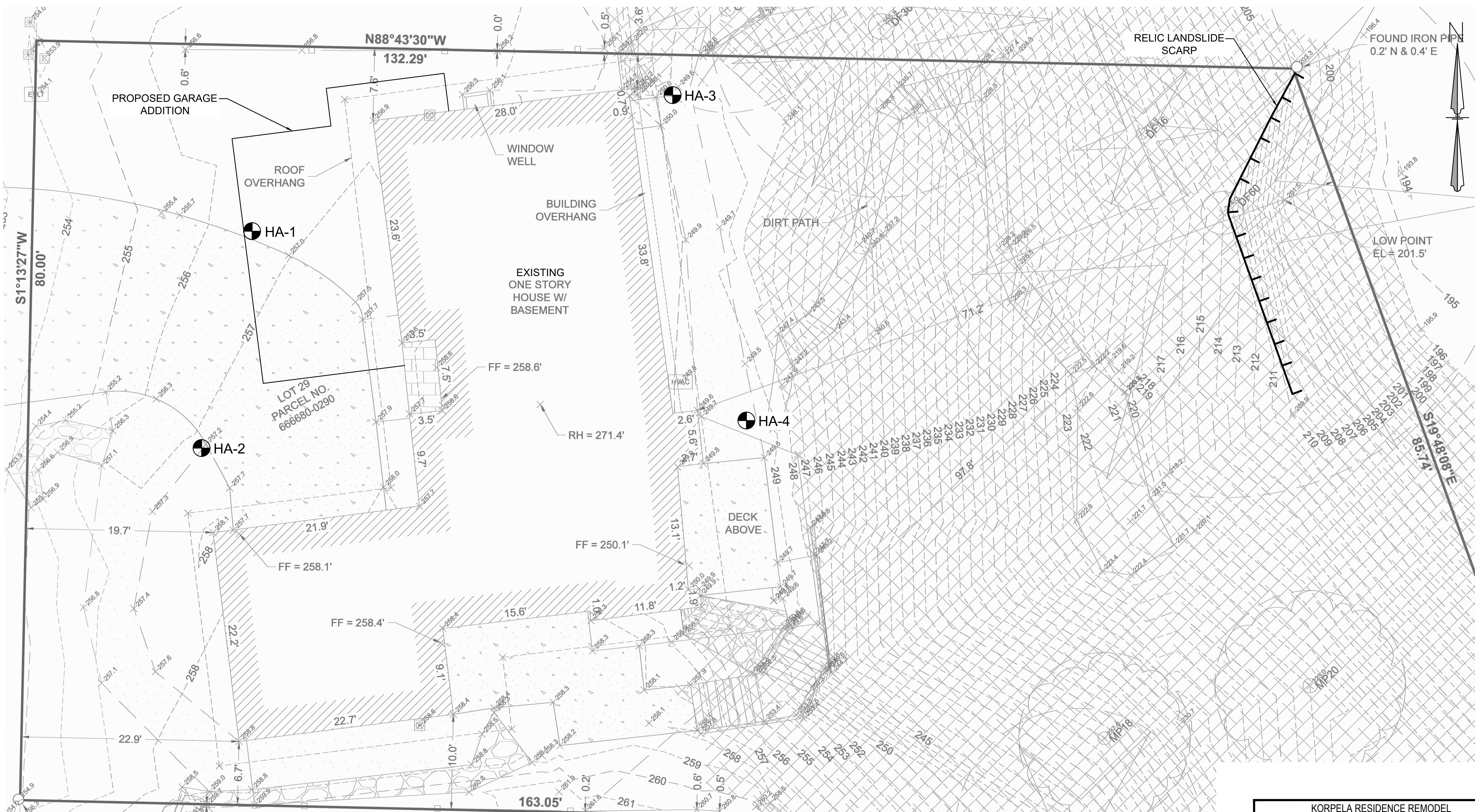
Drainage Considerations

Final site grades should be sloped to carry surface water away from the dwelling and other drainage-sensitive areas. Additionally, site grades should be designed such that concentrated runoff on softscape surfaces is avoided. We observed that some of the downspouts terminated in plastic pipes that extended below grade. We recommend determining the discharge locations for these pipes and their condition if additional water is directed to them as a result of the remodel.

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.



LEGEND

HA-1 HAND AUGER BORING NUMBER AND APPROXIMATE LOCATION

10 0 5 10

SCALE IN FEET

REFERENCE: TOPOGRAPHIC SURVEY, SHEET 1 OF 2, SITE SURVEYING INC., AUGUST 8, 2022

KORPELA RESIDENCE REMODEL
8441 SE 33RD PLACE
MERCER ISLAND, WASHINGTON

SITE AND EXPLORATION PLAN

DATE: NOVEMBER 2023 Job No. 2727.01

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

FIGURE 1

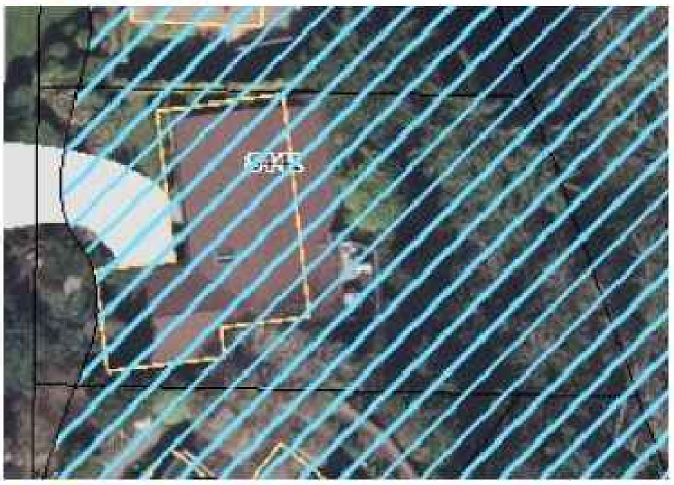
SHT. 1 of 1

City of Mercer Island Property Hazard Report

Site Address: 8441 SE 33RD PL

Parcel #: 6666800290 **Report Generated on** April 19, 2023

Potential Slide:



Steep Slope:

Erosion:

Seismic:

KORPELA RESIDENCE REMODEL 8441 SE 33RD PLACE MERCER ISLAND, WASHINGTON		
CRITICAL AREAS MAP		
DATE: NOVEMBER 2023	Job No.	2727.01
Zipper Geo Associates, LLC 19019 36th Ave. W.,Suite E Lynnwood, WA	FIGURE SHT. 1 of 1	2

APPENDIX A
FIELD EXPLORATION PROCEDURES AND LOGS

FIELD EXPLORATION PROCEDURES AND LOGS

Field Exploration Description

Our field exploration for this project included completing a reconnaissance of surface conditions and advancing four hand auger explorations on 20 June 2023, 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 and steel tapes relative to a *Topographic Survey* (dated 8 August 2022) prepared by Site Surveying, Inc. 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 ZGA engineering geologist 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 evaluated subjectively through the use of a 0.5-inch diameter steel hand probe.

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 HA-1 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: 256 Feet	**Project:** Korpela Residence Remodel **Project No:** 2727.01 **Date Excavated:** 20 June 2023			
Depth (ft)	**Material Description**	**Sample**	**PID**	**%M**	**Testing**
	2 inches fine sod root matt above loose, wet, dark brown, silty SAND, trace gravel, fine roots, fine and fibrous organics; roots and organics decrease to 10 inches				
1	Dense, moist to wet, mottled gray and brown grading to gray at 2 feet, silty SAND with trace gravel, scattered cobbles (Qva)	S-1 @ 1 foot		15	
2		S-2 @ 2.5 feet		12	
3	Boring completed at approximately 2.6 feet – refusal in nested gravel. Groundwater not observed while excavating.				

ZIPPER GEO ASSOCIATES, LLC

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

	Hand Auger HA-2 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: 257 Feet	**Project:** Korpela Residence Remodel **Project No:** 2727.01 **Date Excavated:** 20 June 2023			
Depth (ft)	**Material Description**	**Sample**	**PID**	**%M**	**Testing**
	3 inches very loose, wet, gray fine GRAVEL (pea gravel Fill) above plastic sheeting above dense, moist, brown SAND with silt, trace gravel (Qva)				
1					
		S-1 @ 1 foot		10	
2					
3					
4					
		S-2 @ 2.5 feet		11	
	Boring completed at approximately 4.2 feet. Groundwater not observed while excavating.				

ZIPPER GEO ASSOCIATES, LLC

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

	Hand Auger HA-3 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: 249 Feet	**Project:** Korpela Residence Remodel **Project No:** 2727.01 **Date Excavated:** 20 June 2023			
Depth (ft)	**Material Description**	**Sample**	**PID**	**%M**	**Testing**
	1 inch loose, wet, gray fine GRAVEL (pea gravel Fill) above loose, damp, brown, silty SAND with gravel, cobble-size concrete clast (Fill)				
1					
	Medium dense to dense, moist, brown, SAND, trace silt and gravel (Qva)				
2					
		S-1 @ 2.5 feet		8	
3					
4					
		S-2 @ 2.5 feet		12	
	Boring completed at approximately 4.5 feet. Groundwater not observed while excavating.				

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

	Hand Auger HA-4 **Location:** See Site and Exploration Plan, Figure 1 **Approx. Ground Surface Elevation**: 249 Feet	**Project:** Korpela Residence Remodel **Project No:** 2727.01 **Date Excavated:** 20 June 2023			
Depth (ft)	**Material Description**	**Sample**	**PID**	**%M**	**Testing**
	3 inches loose, wet, gray fine GRAVEL (pea gravel and crushed rock Fill) above medium dense, wet, brown, fine SAND, trace silt (Qva)				
1					
		S-1 @ 1.5 feet		17	
2					
	Grades to dense and silty				
3					
		S-2 @ 3.5 feet		20	
	Boring completed at approximately 3.5 feet. Groundwater not observed while excavating.				
4					

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.

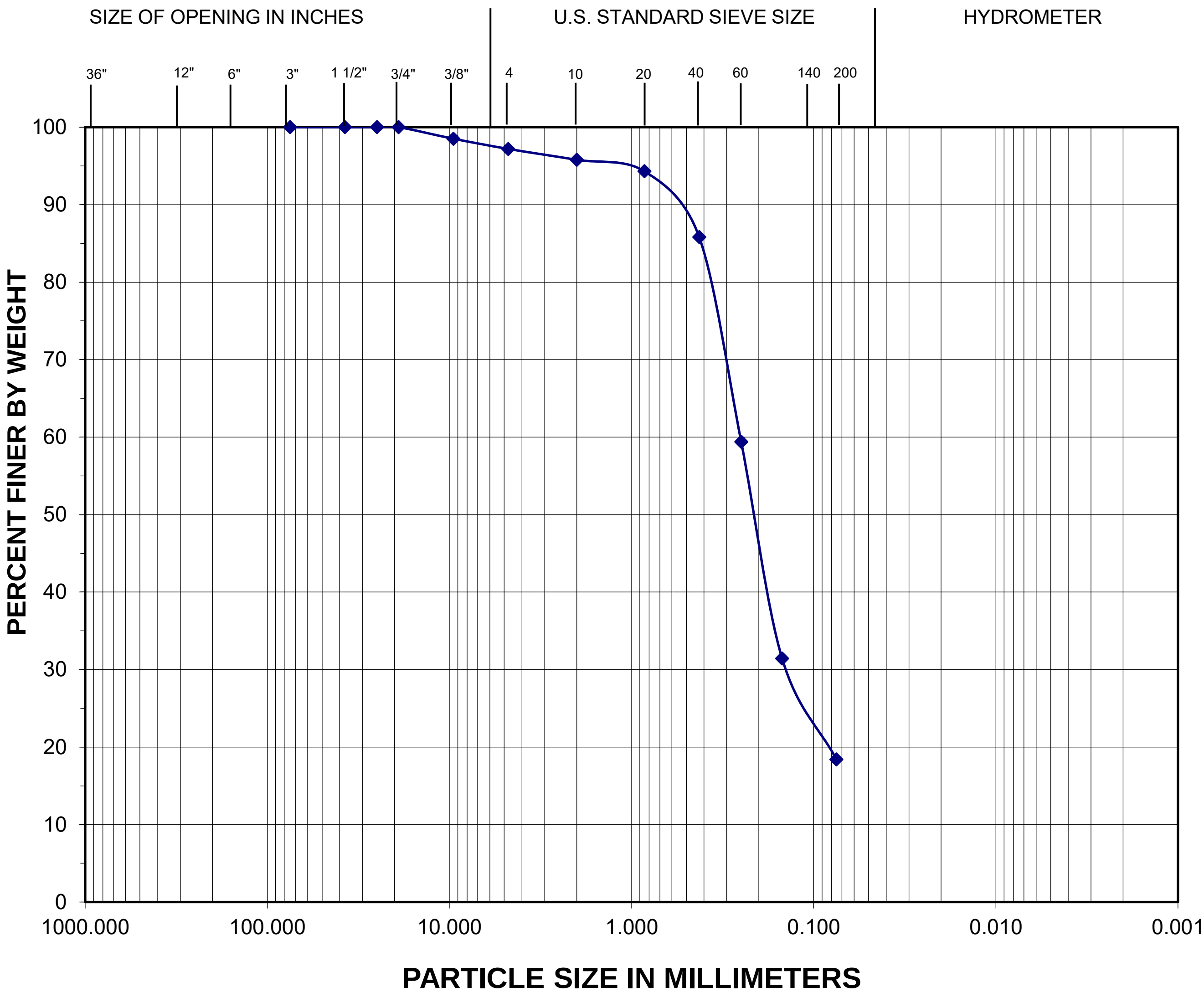
Grain Size Analysis

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:

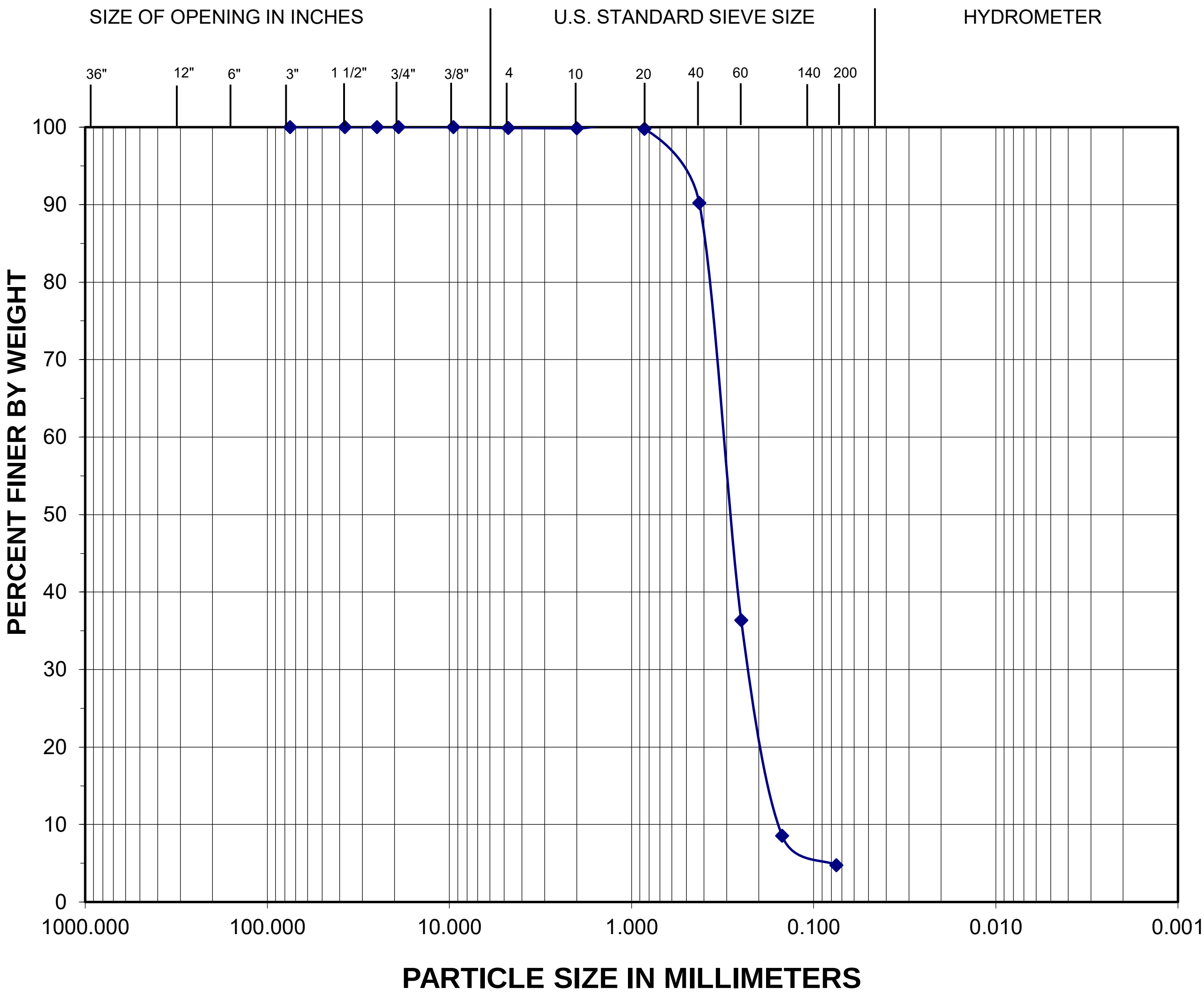
Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
HA-2	S-2	4.0	10.6	18.4	SAND with silt, and trace gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2727.01 DATE OF TESTING: 6/20/2023	PROJECT NAME: Korpela Residence Remodel

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



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

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
HA-3	S-2	4.5	12.1	4.7	SAND, and trace silt and gravel

Zipper Geo Associates, LLC Geotechnical and Environmental Consultants	PROJECT NO: 2727.01 DATE OF TESTING: 6/20/2023	PROJECT NAME: Korpela Residence Remodel

APPENDIX C

ROBERT M. PRIDE, LLC MAY 2007 GEOTECHNICAL REPORT

Consulting
Robert M. Pride, LLC Engineer

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

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

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.

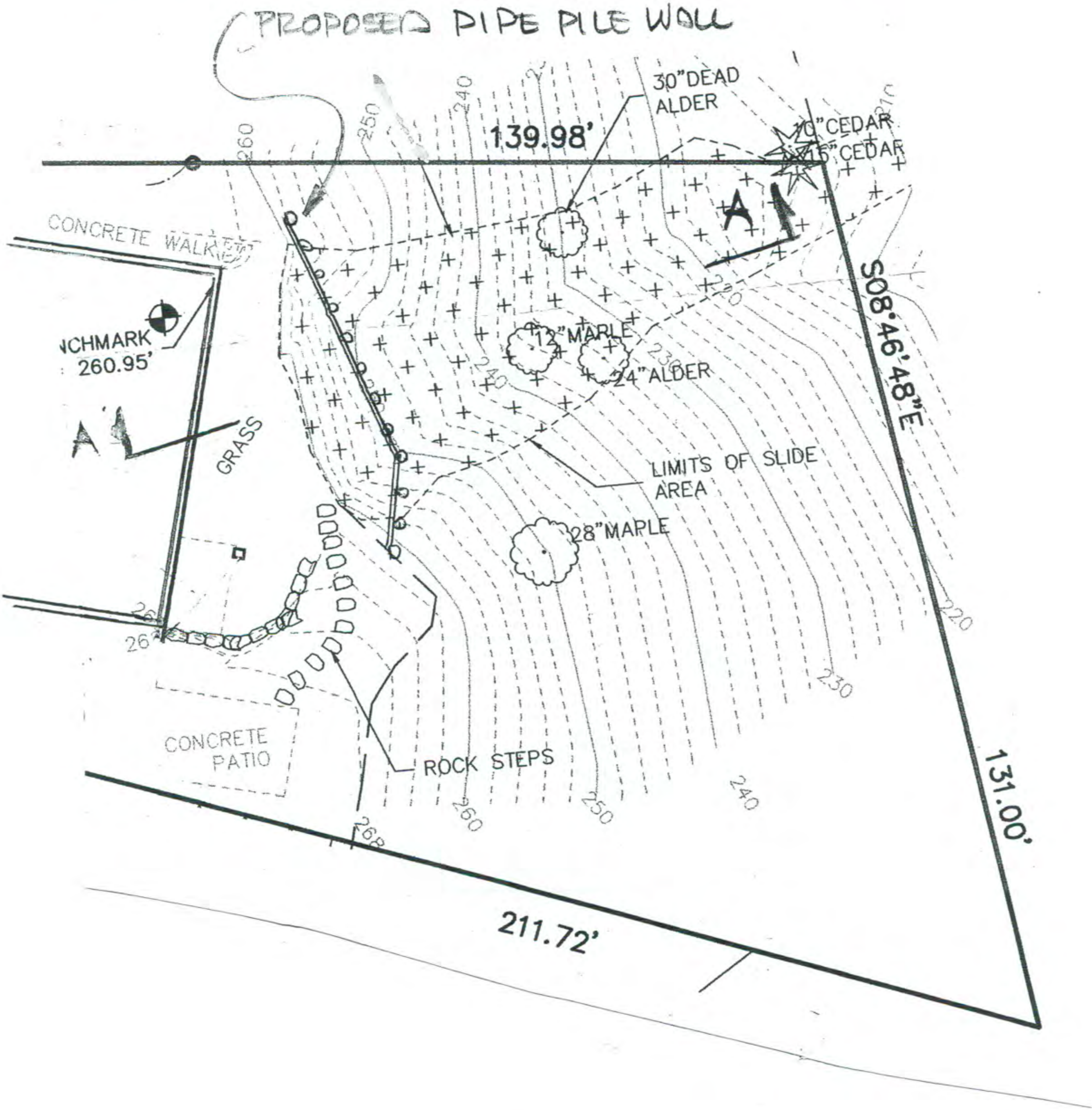
Respectfully submitted,

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

ROBERT M. PRIDE
STATE OF WASHINGTON
16271
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 7-20-08

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

rmp: PiperSlope1



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;

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PROFESSIONAL ENGINEER

EXPIRES