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May 31, 2022

JN 22167

Coombes Development
4701 Southwest Admiral Way #385
Seattle, Washington 98116

Attention: Jon Coombes
via email: jon@coombesdev.com

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Residence
6221 – 83rd Place Southeast
Mercer Island, Washington

Dear Mr. Coombes

Attached to this transmittal letter is our geotechnical engineering report for the new residence to be constructed in Mercer Island, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design considerations for foundations, retaining walls, subsurface drainage, and temporary excavations. This work was authorized by your acceptance of our proposal, P-11163, dated April 15, 2022.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

James H. Strange, P.E.
Associate

cc: **JW Architects**– Julian Weber
via email: jw@jwaseattle.com

MKM/JHS:kg

GEOTECHNICAL ENGINEERING STUDY
Proposed Residence
6221 – 83rd Place Southeast
Mercer Island, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed new residence to be located in Mercer Island.

We were provided with site plans and a topographic map. JW Architects developed these plans, which are dated April 29, 2022, and Terrane developed the site survey, dated November 22, 2021. Based on these plans, we understand that a new residence is proposed to be constructed at the site. The new residence will be two stories in height and will contain a west facing daylight basement in its southern approximate half. The basement will step up to crawlspace areas both to the east for the main floor kitchen, and to the north for additional living space and a garage. A new concrete driveway will provide access to the garage space, and a small, additional parking space is proposed to be constructed off the northeastern corner of the garage between the residence and property line. Preliminary property line setbacks of 11 feet from the north, at least 20 feet from the east, as close as 7.5 feet from the south, and greater than 25 feet from the west are shown on the site plan. However, the new parking space northeast of the garage is shown to be set on the northern property line. Preliminary finish floor elevations of 308.27 and 318 feet are proposed for the basement and main floors, respectively. Based on these elevations, we anticipate that excavations as little as a few feet for the lower, western side of the basement and on-grade portions of the residence are being proposed, with excavations of up to approximately 9 to 10 feet for the taller basement cuts.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site on the south side of Mercer Island. The trapezoidal shaped site has approximate dimensions of 62.1 to 100 feet in the north-south direction, and 123.3 to 133.8 feet in the east-west direction. The site is bordered to the north, south, and east by single family parcels, and to the east by 83rd Place Southeast.

The grade across the site slopes gently downwards from east to west, with a total elevation change of 8 feet across the property bounds at an average inclination of 5 to 7 percent. From the eastern property line, the grade carries out flat to very gently sloped out across the existing asphalt driveway and grass yard, continuing to the existing one-story residence which set in the approximate center of the property. To the north of the garage, a short, 2- to 4-foot-tall rockery descends to the adjacent northern property. The grade descends gently across the residence footprint, with short rockeries providing a terraced grade drop in the northern and southern side yards, which facilitates the grade drop between the main and basement levels of the existing residence. An elevated deck with concrete patio space beneath extends off the northwestern corner of the residence. Past the western face of the existing residence, the grade again carries out gently, descending downward to the western property line. An approximately 6-foot-tall rockery slope lines the western property line on the neighboring western parcel. This rockery slope is steeply inclined based on the topographic survey.

The City of Mercer Island GIS maps the western extent of the site as a Potential Slide and Erosion Hazard Area. While no steep slope areas are present within the site, and within the direct vicinity, a larger Potential Slide Area is mapped starting approximately 2 lots west of the site. This larger feature is mapped to contain a mapped head scarp and a geologic contact, likely between the upper Esperance sands and underlying silt is mapped within this feature. This mapped area continues downward to West Mercer Way. As noted above, the grade across the site descends from east to west at an inclination of only 5 to 7 percent. Thile the survey shows the perimeter rockery as a small steep slope area, the Mercer Island GIS maps the topographic relief across this sloped area upwards of 12 feet before the slope gradient flattens again. Based on the GIS contours, this slope would meet Mercer Island's criteria for a Steep Slope and therefore a Landslide Hazard Area. No signs of recent instability were observed while onsite.

As stated above, residential parcels are set to the north, south, and west of the site. The adjacent northern lot contains a one-story residence set approximately 4.9 feet from the property line at its closest. This property is set at a lower elevation than the site along its eastern half, with its western half set at the approximate same elevation as the subject site. The southern property contains a one-story residence that overlies a partial footprint daylight basement. This residence and its decks and patios are set 5 to 10 feet from the property line. A rockery feature shared by both properties facilitates a terraced grade drop on both lots from the upper, eastern to lower, western yard areas. The western properties also contain two story residences that are set at a lower elevation than the site and are set greater than 10 feet from the property lines.

SUBSURFACE

The subsurface conditions were explored by excavating two test pits and two test holes at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The test pits were excavated on May 16, 2022 with a tracked excavator, and the test holes were excavated with hand tools on the same day where machine access was not possible at the site. A geotechnical engineer from our staff observed the excavation process, logged the test pits, and obtained representative samples of the soil encountered. "Grab" samples of selected subsurface soil were collected from the backhoe bucket. The Test Pit Logs are attached to this report as Plates 3 and 4.

Soil Conditions

Test Pits 1 and 2 were excavated on the eastern side of the property, near the southeastern corner of the residence, and east of the garage, respectively. Beneath the ground surface, the test pits encountered a layer of loose fill ranging in thickness from 3 to 5 feet. This fill deposit was deepest in Test Pit 1, which was excavated near the eastern basement wall of the existing residence. Beneath the fill, native, silty, and slightly silty sand was revealed. This native soil was initially loose, becoming medium-dense beneath a depth of 4.5 feet in Test Pit 2, and dense beneath a depth of 7 feet in Test Pit 1. Dense silt was revealed beneath depths of 5 to 7.5 feet in Test Pits 2 and 1, respectively, and continued to the base of Test Pit 2 at a depth of 6 feet. A layer of wet, dense silty sand was revealed beneath the silt in Test Pit 1, continuing to the base of the excavation at a depth of 9.5 feet, where the machine could not excavate any deeper.

Test Holes 1 and 2 were excavated along the lower, western side of the site, near the northwestern and southwestern corners of the proposed residence, respectively, beneath the ground surface, a layer of recent fill approximately 2 feet in thickness was revealed in both test holes. Slightly gravelly sand was revealed beneath the fill, which contained roots and trace organics. This sand layer was initially in a loose state, becoming medium-dense beneath depths of 5 to 5.5 feet. This sand layer was interpreted to be older fill soils that were placed to level out the lower, western yard and development area for the construction of the existing residence. This soil continued to the base of the test holes at depths of 7 to 7.5 feet.

No obstructions were revealed by our explorations. However, debris, buried utilities, and old foundation and slab elements are commonly encountered on sites that have had previous development.

Groundwater Conditions

Groundwater seepage was observed at a depth of 9 feet in Test Pit 1. The test pits and test holes were left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level.

It should be noted that groundwater levels vary seasonally with rainfall and other factors. We anticipate that groundwater could be found in more permeable soil layers and between the looser near-surface soil and the underlying denser soil.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. The relative densities and moisture descriptions indicated on the test pit and test hole logs are interpretive descriptions based on the conditions observed during excavation.

The compaction of test pit backfill was not in the scope of our services. The test pits were backfilled with excavated soil that was lightly tamped into place. Loose soil will therefore be found in the area of the test pits. If this presents a problem, the backfill will need to be removed and replaced with structural fill during construction.

CRITICAL AREA STUDY (MICC 19.07)

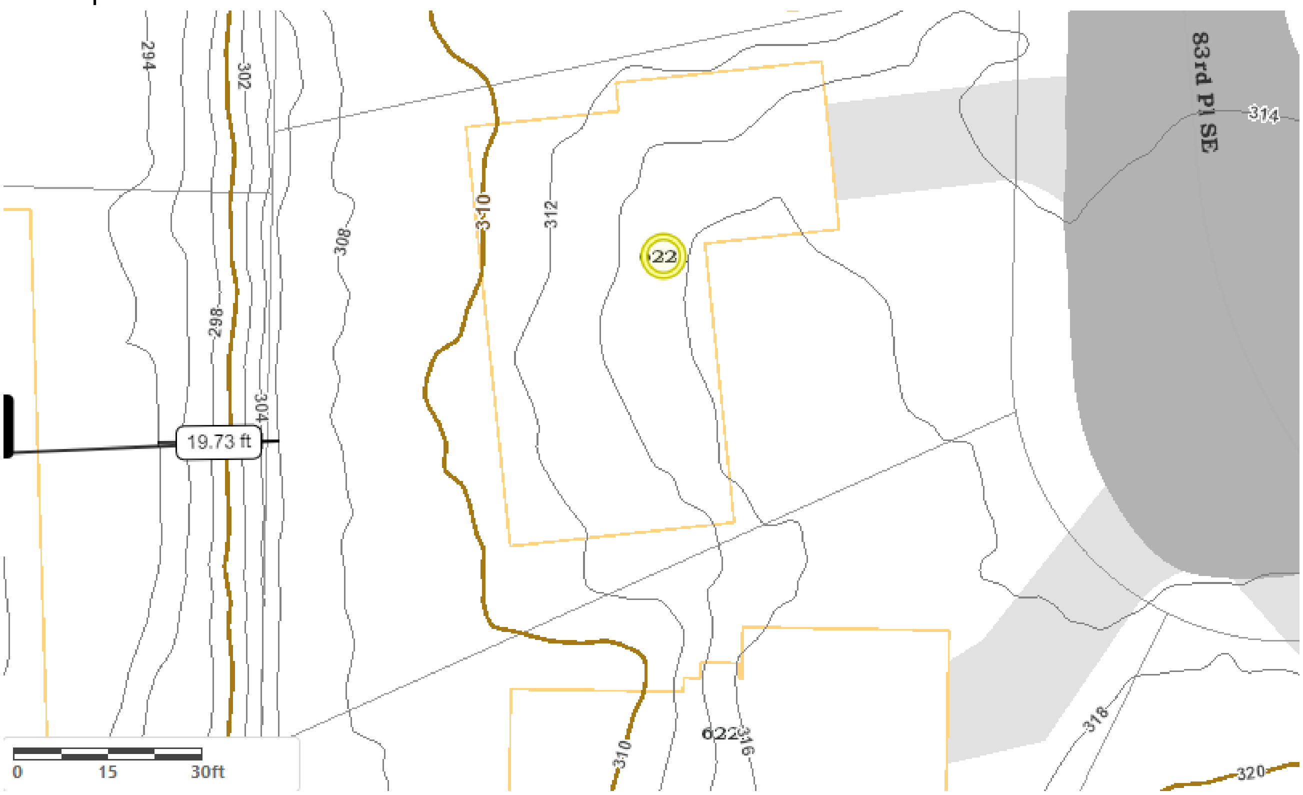
Seismic Hazard Areas: Not applicable. The site is not mapped within a Seismic Hazard Area.

Landslide Hazard Areas: There are several criteria for being a Landslide Hazard Area based on the MICC. The first of several criteria are as follow:

1. Areas of historic failures.
2. Areas with all three of the following characteristics:
   a. Slopes steeper than 15 percent; and
   b. Hillsides intersecting geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment or bedrock; and

   c. Springs or ground water seepage.
3. Areas that have shown evidence of past movement or that are underlain or covered by mass wastage debris from past movements.
4. Areas potentially unstable because of rapid stream incision and stream bank erosion.

There is a fifth criteria with regards to Landslide Hazard areas: Any slope that is 40 percent or greater measured over a 30-foot horizontal run (Steep Slope). As noted earlier, the western perimeter slope would meet this fifth criteria based on its height and inclination 9see clip below). This slope appears to have been at least partially modified either by a cut made to grade the western property, or during a fill to level out the subject site's backyard. While this potential hazard exists, the grade across the remainder of the property generally slopes downward at a 5 to 7 percent declination from east to west, and the soils encountered at depth in our test pits would indicate that the core of the site is comprised of glacially compressed soil, which are not susceptible to deep seated landslides.



While a Potential Landslide Hazard Area exists due west of the site, it is our opinion that the mapping of this area should start at the top of the perimeter rockery slope located on the western property line, and not in the gently sloped yard area within the site bounds. Based on MICC 19.07.160, for Steep Slope Landslide Hazard Areas whose only potential is a shallow landslide, this prescriptive buffer is 25 feet. At this time, we are not aware that any alteration from the prescriptive buffers will be needed for the project, given the preliminary siting location placing the western edge of the new residence outside of the prescriptive buffer. If sitework close to the top of the steep slope is avoided, the planned project will not adversely impact the slope's stability from its current state.

The approximate prescriptive shallow landslide hazard buffer is shown on the attached Site Plan, Plate 2 for reference.

Erosion Hazard Area: The site also is mapped as an Erosion Hazard Area.

Proper erosion control implementation will be important to prevent adverse impacts to the site and neighboring properties, particularly if grading and construction occurs during the wet season. The temporary erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered during the site work. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil areas to prevent accumulated water or runoff from the work area from becoming silty in the first place. Silty water cannot be discharged off the site, so a temporary holding tank should be planned for wet weather earthwork. A wire-backed silt fence bedded in compost, not native soil, or sand, should be erected as close as possible to the planned work area, and the existing vegetation west of the silt fence be in place. Rocked construction access and staging areas should be established wherever trucks will have to drive off of pavement, in order reduce the amount of soil or mud carried off the property by trucks and equipment. Covering the base of the excavation with a layer of clean gravel or rock is also prudent to reduce the amount of mud and silty water generated. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. Silty water accumulating in the excavation must not be allowed to flow off the site, or into Lake Washington. In wet conditions, this can require the use of temporary holding tanks (aka Baker tanks). Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface.

Buffers and Mitigation: The attached Site Exploration Plan (Plate 2) denotes the extents of the critical areas that cover the site. Under MICC 19.07.160(C), the code-prescriptive buffer of 25 feet is required from all sides of a shallow landslide-hazard area. On our site plan, we show this buffer starting from the top of the western perimeter rockery slope. As noted above, the western extent of the site lies within a mapped Potential Landslide Hazard Area, and the prescriptive buffer would extend close to the western edge of the proposed residence. This buffer is for reference only, is not drawn to scale, and is measured from the top of the western perimeter steep slope, not from the upslope edge of the Mercer Island GIS mapping location. If development is proposed near the top of the steep slope, additional mitigation measures may need to be implemented to facilitate the proposed construction without adversely affecting the slope.

No buffer is required by the MICC for an Erosion Hazard Area.

Regardless of the hazard mapping, the recommendations presented in this geotechnical report are intended to allow the project to be constructed in the proposed configuration without adverse impacts to critical areas on the site or the neighboring properties. The geotechnical recommendations associated with foundations, excavation and shoring, and erosion control will mitigate any potential hazards to geologic critical areas on the site.

Statement of Risk: In order to satisfy the City of Mercer Island’s requirements, a statement of risk is needed. As such, we make the following statement:

> *Provided the recommendations in this report are followed, it is our professional opinion that the recommendations presented in this report for the planned alteration will render the development as safe as if it were not located in a geologically hazardous area and will not adversely impact critical areas on adjacent properties.*

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The test pits conducted for this study encountered dense silt and silty sand beneath a mantle of fill and weathered native soils ranging in thickness from 5 to 7 feet, increasing in depth to the west. This dense native silty sand and silt is glacially compressed and is suitable for support of new structural loads. However, the two test holes excavated along the lower, western perimeter of the development exposed loose and medium-dense apparent fill soils through their full depth, and the native, dense glacially compressed soils were not revealed.

Excavations of on the order of 9 feet are anticipated for the taller basement cuts, with shallower excavations for the on-grade crawlspace, and slab-on-grade features, as well as the western portion of the basement, where the residence footprint daylights into the lower yard area. Where the deeper excavations are proposed, or where overexcavations can be conducted without encroaching over the property lines, conventional foundations bearing upon the underlying dense native soils or adequately compacted rock fill placed atop the competent native soils can be used to support the new foundation loads. However, where shallower excavations are being proposed, and where overexcavations are not feasible to expose the underlying dense native soils, the most practical foundation support system would be to utilize deep foundations such as small diameter pipe piles to support the new residence loads. These pipe piles would be driven through the looser soils to refusal in the underlying glacially compressed soils. Recommendations for both foundation systems can be found in the ***Conventional Foundations*** and ***Pipe Piles*** sections of this report.

At this time, we recommend that the western approximate half of the basement, and western perimeter of the northern crawlspace be supported on pipe piles. This transition between shallow and deep foundations may exist as a “seam” that may be observable during the mass excavation. The project geotechnical engineer should be allowed to visit the site during the mass excavation to help determine the approximate extents of the use of either foundation system, and determine if piles need to be added, or can be removed. This can be evaluated early in the project’s excavation after the existing residence has been demolished by conducting several test pits with the onsite equipment.

The proposed residence will be set at least 25 feet away from the western property line, where the previously mentioned steeply inclined rockery slope is located. Based on our test holes in the back yard, it would appear that loose fill soils exist behind this rockery. The western perimeter foundations for the new residence will be supported on deep foundations that will be embedded into the underlying dense core of the site, which will transfer the structural loading beneath the fill soils. This will remove the potential for the weight of the new residence to cause adverse impacts to this rockery.

The underlying native silty sand and silt soils are moisture sensitive, and easily disturbed by foot and machine traffic as well as wet weather. Where conventional foundations are able to be used, we recommend that, at a minimum, a thin layer of clean, crushed rock be placed atop the prepared bearing surfaces to protect the moisture sensitive subgrade soils from disturbance.

The fill and native soils are fine-grained, silty, and variable in composition. These qualities make the soil poorly drained, and exceedingly difficult to adequately compact for use as structural fill. Considering this, we do not recommend that the onsite soils be reused as structural fill beneath the new foundations. Imported, clean, angular rock fill such as quarry spalls, or ballast rock should be used if structural fill is needed beneath the foundations. This open-graded rock fill can be easily placed and compacted with minimal effort from typical excavation equipment and does not require the use of vibratory compactors for shallower fills.

The dense, fine-grained, silty nature of the native soils will essentially stop the downward percolation of stormwater. Considering the qualities of the native soils, and the presence of uncontrolled fill on the lower side of the site, we do not recommend that onsite infiltration or dispersion system be used for this project. Any attempt to infiltrate or disperse large quantities of stormwater will form a perched layer atop the dense silty sand and silt and could cause adverse drainage impacts to the proposed basement space, as well as the downslope developments. Infiltrating into the lower fill soils could cause the fill soils to destabilize and settle. All collected stormwater runoff should be tightlined offsite to the appropriate facilities.

The excavations for the residence will vary from as little as a few feet for the shallower, on-grade portions of the residence, to upwards of 9 feet to reach the basement level foundations. Based on the soils encountered in our explorations, a temporary excavation inclination of no steeper than a 1:1 (Horizontal:Vertical) is appropriate for this site. Vertical excavations should not be made on, or near the shared property lines, or near any adjacent settlement sensitive structure. Unshored excavations should not extend beneath a 1.5:1 (H:V) line drawn descending from any adjacent foundation, right-of-way, or settlement sensitive structure. Based on the preliminary layout, it would appear that much of the main body of the excavation will be able to be maintained within the property lines. However, the southern basement excavation will be set in relatively close proximity to the south property line, and sloped excavations, including a typical several foot over-dig for the concrete subcontractors, may extend over the southern property boundary. If this is the case, and excavation easements from the southern neighbor cannot be obtained, then temporary shoring will be needed. We can provide temporary shoring recommendations as the design progresses.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered. We anticipate that a silt fence will be needed around the downslope sides of any cleared areas. Existing pavements, ground cover, and landscaping should be left in place wherever possible to minimize the amount of exposed soil. Rocked staging areas and construction access roads should be provided to reduce the amount of soil or mud carried off the property by trucks and equipment. Trucks should not be allowed to drive off of the rock-covered areas. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Following clearing or rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface. On most construction projects, it is necessary to periodically maintain or modify temporary erosion control measures to address specific site and weather conditions.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking, cleaning, and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

As with any project that involves demolition of existing site buildings and/or extensive excavation and shoring, there is a potential risk of movement on surrounding properties. This can potentially translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. However, the demolition, shoring, and/or excavation work could just translate into *perceived* damage on adjacent properties. Unfortunately, it is becoming more and more common for adjacent property owners to make unsubstantiated damage claims on new projects that occur close to their developed lots. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring, and/or commencing with the excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners. Additionally, any adjacent structures should be monitored during demolition and construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

SEISMIC CONSIDERATIONS

In accordance with the International Building Code (IBC), the site class within 100 feet of the ground surface is best represented by Site Class Type D (Stiff Soil). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.46g and 0.51g, respectively.

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) during an earthquake be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period). The MCE peak ground acceleration adjusted for site class effects (F_{PGA}) equals 0.69g. The soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense nature.

Sections 1803.5 of the IBC and 11.8 of ASCE 7 require that other seismic-related geotechnical design parameters (seismic surcharge for retaining wall design and slope stability) include the potential effects of the Design Earthquake. The peak ground acceleration for the Design Earthquake is defined in Section 11.2 of ASCE 7 as two-thirds (2/3) of the MCE peak ground acceleration, or 0.46g.

CONVENTIONAL FOUNDATIONS

Where deeper excavations, or overexcavations will be able to be made, conventional continuous and spread footings bearing on undisturbed, dense, native soil, or on structural fill placed above this competent native soil could be used to support new structural loads. Prior to placing structural fill beneath foundations, the excavation should be observed by the geotechnical engineer to document that adequate bearing soils have been exposed.

We recommend that continuous and individual spread footings have minimum widths of 16 and 24 inches, respectively. Exterior footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface for protection against frost and erosion. The local building codes should be reviewed to determine if different footing widths or embedment depths are required. Footing subgrades must be cleaned of loose or disturbed soil prior to pouring concrete. Depending upon site and equipment constraints, this may require removing the disturbed soil by hand.

Depending on the final site grades, overexcavation may be required below the footings to expose competent native soil. Unless lean concrete is used to fill an overexcavated hole, the overexcavation must be at least as wide at the bottom as the sum of the depth of the overexcavation and the footing width. For example, an overexcavation extending 2 feet below the bottom of a 2-foot-wide footing must be at least 4 feet wide at the base of the excavation. If lean concrete is used, the overexcavation need only extend 6 inches beyond the edges of the footing. A typical detail for overexcavation beneath footings is attached as Plate 5.

An allowable bearing pressure of 2,000 pounds per square foot (psf) is appropriate for footings supported on competent native soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil, or on structural fill up to 5 feet in thickness, will be about one-half-inch, with differential settlements on the order of one-half-inch in a distance of 30 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level, well-compacted fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.40
Passive Earth Pressure	300 pcf

Where: pcf is Pounds per Cubic Foot, and Passive Earth Pressure is computed using the Equivalent Fluid Density.

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. The above ultimate values for passive earth pressure and coefficient of friction do not include a safety factor.

PIPE PILES

Three- or 4-inch-diameter pipe piles driven with an 850- or 1,100- or 2,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (850-pound hammer)	FINAL DRIVING RATE (1,100-pound hammer)	FINAL DRIVING RATE (2,000-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
3 inches	10 sec/inch	6 sec/inch	2 sec/inch	6 tons
4 inches	16 sec/inch	10 sec/inch	4 sec/inch	10 tons

Note: The refusal criteria indicated in the above table are valid only for pipe piles that are installed using a hydraulic impact hammer carried on leads that allow the hammer to sit on the top of the pile during driving. If the piles are installed by alternative methods, such as a vibratory hammer or a hammer that is hard mounted to the installation machine, numerous load tests to 200 percent of the design capacity would be necessary to substantiate the allowable pile load. The appropriate number of load tests would need to be determined at the time the contractor and installation method are chosen.

As a minimum, Schedule 40 pipe should be used. The site soils are not highly organic and are not located near salt water. As a result, they do not have an elevated corrosion potential. Considering this, it is our opinion that standard “black” pipe can be used, and corrosion protection, such as galvanizing, is not necessary for the pipe piles.

We expect that the City of Mercer Island will require geotechnical observation of the pile installation. Considering this, the recommendations we have made above for minimum refusal criteria, and our previous experience with pile projects in close proximity to the site, it is our professional opinion that the recommended capacities do not need to be verified by load testing.

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads being applied to the piles. Subsequent sections of pipe can be connected with slip or threaded couplers, or they can be welded together. If slip couplers are used, they should fit snugly into the pipe sections. This may require that shims be used or that beads of welding flux be applied to the outside of the coupler.

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level compacted fill. We recommend using an ultimate (no safety factor included) passive earth pressure of 300 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate.

FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Lateral Earth Pressure *	40 pcf
Passive Earth Pressure	300 pcf
Coefficient of Friction**	0.40
Soil Unit Weight	125 pcf

Where: pcf is Pounds per Cubic Foot, and Lateral and Passive Earth Pressures are computed using the Equivalent Fluid Pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above lateral equivalent fluid pressure. This applies only to walls with level backfill.**

**** For use only in designing conventional foundations.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above lateral fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired.

The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized the wall and reinforcing design for a distance of 1.5 times the wall height from corners or bends in the walls, or from other points of restraint. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

Per IBC Section 1803.5.12, a seismic surcharge load need only be considered in the design of walls over 6 feet in height. A seismic surcharge load would be imposed by adding a uniform lateral pressure to the above-recommended lateral pressure. The recommended seismic surcharge pressure for this project is 9**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

<u>Retaining Wall Backfill and Waterproofing</u>

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. A minimum 12-inch width of free-draining gravel or a drainage composite similar to Miradrain 6000 should be placed against the backfilled retaining walls. The gravel or drainage composites should be hydraulically connected to the foundation drain system. Free draining backfill should be used for the entire width of the backfill where seepage is encountered. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls at one to 2 percent to reduce the potential for surface water to percolate into the backfill.

Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. Foundation drainage and waterproofing systems are not intended to handle large volumes of infiltrated water. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The recommended wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew, or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a buildup of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design or minimizing the potential for infestations of mold and mildew are desired.

SLABS-ON-GRADE

The building floors can be constructed as slabs-on-grade atop competent native soil, or on structural fill. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. All interior slabs-on-grade should be underlain by a capillary break drainage layer consisting of a minimum 4-inch thickness of clean gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. Pea gravel or crushed rock are typically used for this layer.

As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI recommends a minimum 10-mil thickness vapor retarder for better durability and long-term performance than is provided by 6-mil plastic sheeting that has historically been used. A vapor retarder is defined as a material with a permeance of less than 0.3 perms, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where vapor retarders are used under slabs, their edges should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection.

If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

We recommend that the contractor, the project materials engineer, and the owner discuss these issues and review recent ACI literature and ASTM E-1643 for installation guidelines and guidance on the use of the protection/blotter material.

EXCAVATIONS AND SLOPES

Temporary excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Also, temporary cuts should be planned to provide a minimum 2 to 3 feet of space for construction of foundations, walls, and drainage. Temporary cuts to a maximum overall depth of about 4 feet may be attempted vertically in unsaturated soil if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type B. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut.

The above-recommended temporary slope inclination is based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface runoff be directed away from the top of temporary slope cuts. Cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger. These recommendations may need to be modified if the area near the potential cuts has been disturbed in the past by utility installation, or if settlement-sensitive utilities are located nearby.

All permanent cuts or fills into native soil should be inclined no steeper than 2.5:1 (H:V). To reduce the potential for shallow sloughing, fill must be compacted to the face of these slopes. This can be accomplished by overbuilding the compacted fill and then trimming it back to its final inclination. Adequate compaction of the slope face is important for long-term stability and is necessary to prevent excessive settlement of patios, slabs, foundations, or other improvements that may be placed near the edge of the slope.

Water should not be allowed to flow uncontrolled over the top of any temporary or permanent slope. All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil.

DRAINAGE CONSIDERATIONS

Footing drains should be used where: (1) crawl spaces or basements will be below a structure; (2) a slab is below the outside grade; or (3) the outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. These drains should be surrounded by at least 6 inches of 1-inch-minus, washed rock that is encircled with non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space. The discharge pipe for subsurface drains should be sloped for flow to the outlet point. Roof and surface water drains must not discharge into the foundation drain system. A typical footing drain detail is attached to this report as Plate 6. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains. Clean-outs should be provided for potential future flushing or cleaning of footing drains.

Underslab drainage should also be provided where (1) a crawl space or slab will slope or be lower than the surrounding ground surface, (2) an excavation encounters significant seepage, or (3) an excavation for a building will be close to the expected high groundwater elevations. We can provide recommendations for interior drains, should they become necessary, during excavation and foundation construction.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing a few inches of free draining gravel underneath the vapor retarder is also prudent to limit the potential for seepage to build up on top of the vapor retarder.

Groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to the residence should slope away at least one to 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the ***Foundation and Retaining Walls*** section.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. It is important that existing foundations be removed before site development. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, or in other areas where the underlying soil needs to support loads. All structural fills should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

Fills placed on sloping ground should be keyed into the native, dense soils. This is typically accomplished by placing and compacting the structural fill on level benches that are cut into the competent soils. The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches, but should be thinner if small, hand-operated compactors are used. We recommend testing structural fill as it is placed. If the fill is not sufficiently compacted, it should be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended levels of relative compaction for compacted fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the test pits and test holes are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test pits and test holes. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Jon Coombes his representatives, for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with our understanding of current local standards of practice, and within the scope of our services. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew, and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

In addition to reviewing the final plans, Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services when requested by you or your representatives. Please be aware that we can only document site work we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 4	Test Pit and Test Hole Logs
Plate 5	Typical Footing Overexcavation
Plate 6	Typical Footing Drain Detail

We appreciate the opportunity to be of service on this project. Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

5/31/22

James H. Strange, Jr., P.E.
Associate

MKM/JHS:kg

NORTH

(Source: King County iMap)

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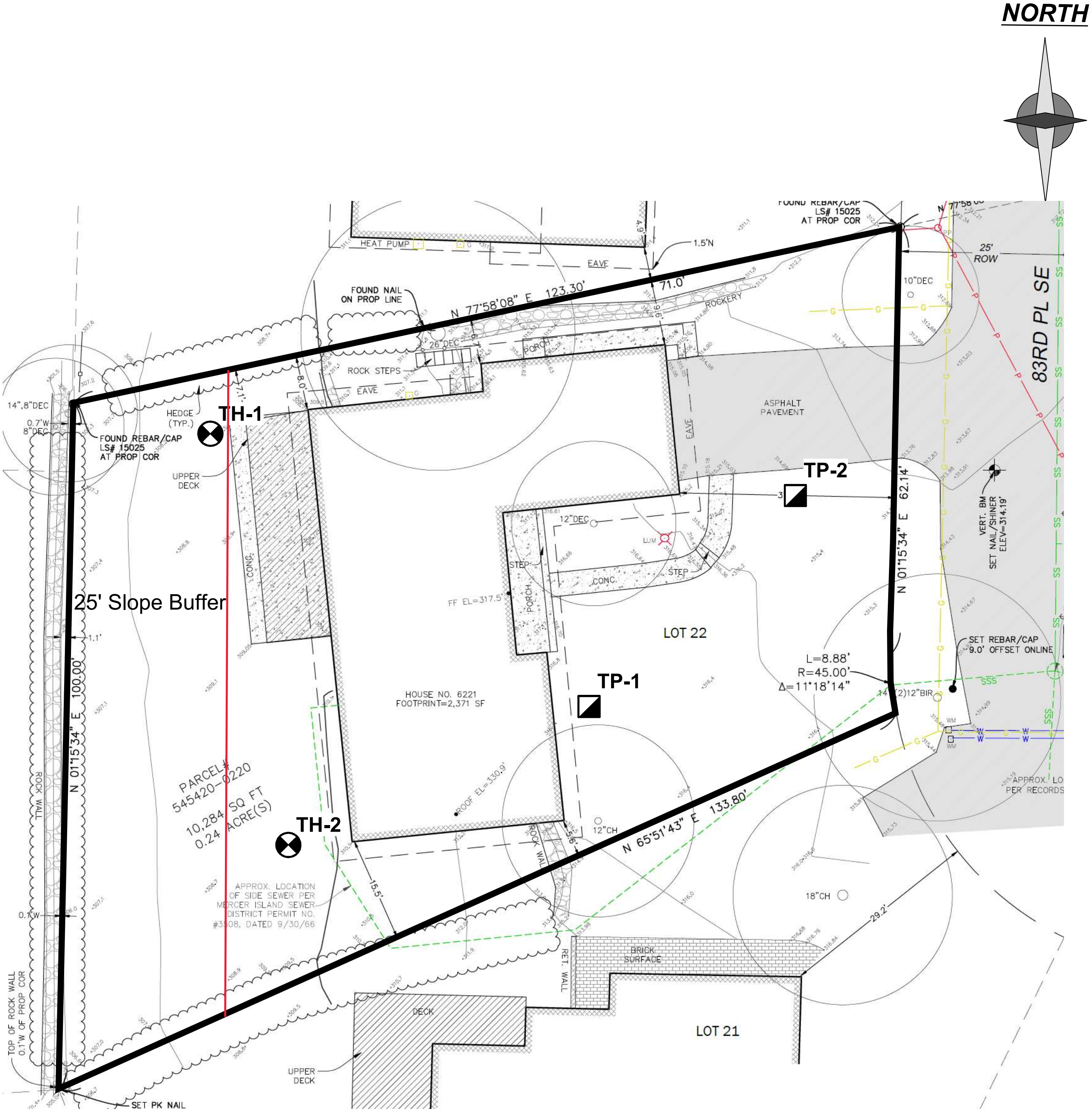
VICINITY MAP

6221 - 83rd Place Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
22167	May 2022		1



Legend:

- Test Pit Location
- Test Hole Location

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SITE EXPLORATION PLAN

6221 - 83rd Place Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
22167	May 2022	No Scale	2

TEST PIT 1

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			FILL	Brown and dark-brown silty SAND with roots and trace organics, fine-grained, moist, jumbled, loose **(FILL)** -becomes black with abundant organics and roots, very loose
5			SM	Gray-brown mottled orange, silty SAND with roots, fine-grained, very moist, loose to medium-dense -becomes bluish-gray with trace mottling and rusting, dense
			ML	Gray to gray-brown with rusting, sandy SILT, non-plastic, moist, dense
			SM	Gray silty SAND, fine-grained, wet, dense
10				

*** Test Pit terminated at 9.5 feet on May 16, 2022.**
*** Groundwater seepage was observed at 9 feet during excavation.**
*** Caving was observed from 4 to 5 feet during excavation.**

TEST PIT 2

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			FILL	Brown and dark-brown silty SAND with roots and organics, fine-grained, moist, loose **(FILL)**
			SP SM	Gray-brown slightly silty to silty SAND with trace roots, fine-grained, moist, loose to medium-dense -grades to slightly gravelly SAND, very moist to wet, medium-dense
5			ML	Gray-brown mottled orange, very sandy SILT, non-plastic, moist, dense
10				

*** Test Pit terminated at 6 feet on May 16, 2022.**
*** No groundwater seepage was observed during excavation.**
*** No caving observed during excavation.**

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TEST HOLE LOG

6221 - 83rd Place Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
22167	May 2022	MKM	3

TEST HOLE 1

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			FILL	Brown silty SAND with roots, fine-grained, moist to dry, loose **(FILL)**
			FILL?	Brown slightly gravelly SAND, fine to medium-grained, very moist, loose **(FILL?)** -becomes grayish-brown very gravelly, with roots -becomes medium-dense
5				
10				

*** Test Hole terminated at 7.5 feet on May 16, 2022.**
*** No groundwater seepage was observed during excavation.**
*** No caving observed during excavation.**

TEST HOLE 2

Depth (ft.)	Moisture Content (%)	Water Table	USCS	Description
			FILL	Brown silty SAND with roots, fine-grained, moist, loose **(FILL)**
			FILL?	Gray and brown slightly gravelly, slightly silty SAND with trace organics, fine-grained, very moist, loose to medium-dense **(FILL?)** -becomes mottled, gravelly, medium-dense
5				
10				

*** Test Hole terminated at 7 feet on May 16, 2022.**
*** No groundwater seepage was observed during excavation.**
*** No caving observed during excavation.**

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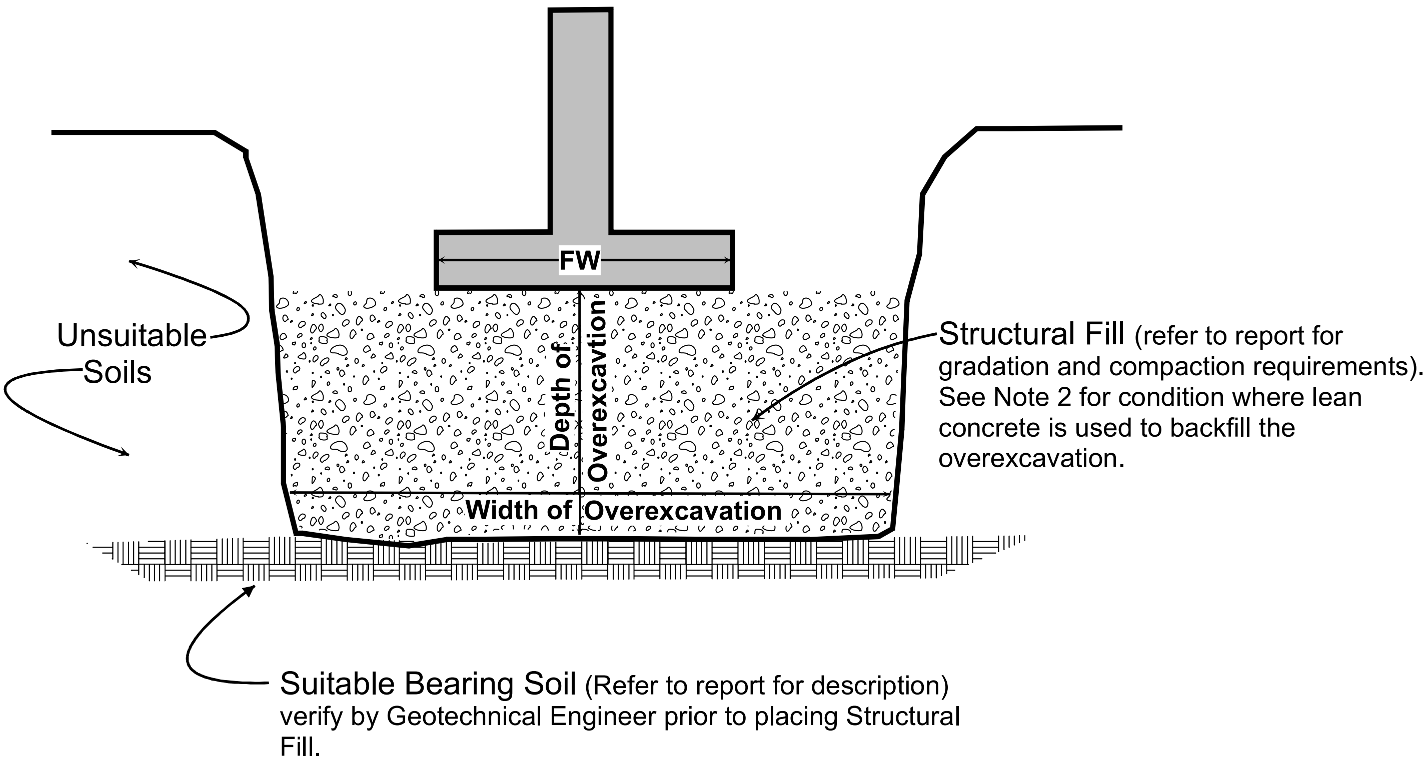
TEST HOLE LOG

6221 - 83rd Place Southeast
Mercer Island, Washington

Job	Date:	Logged by:	Plate:
22167	May 2022	MKM	4



Width of Overexcavation = Footing Width (FW) + Depth of Overexcavation

NOTES:

1. Refer to report text for additional overexcavation, foundation, and structural fill considerations.
2. Where lean concrete (minimum 1-1/2 sacks of cement per cubic yard) is used to backfill the overexcavation, the overexcavation must extend only 6 inches beyond the edges of the footing.

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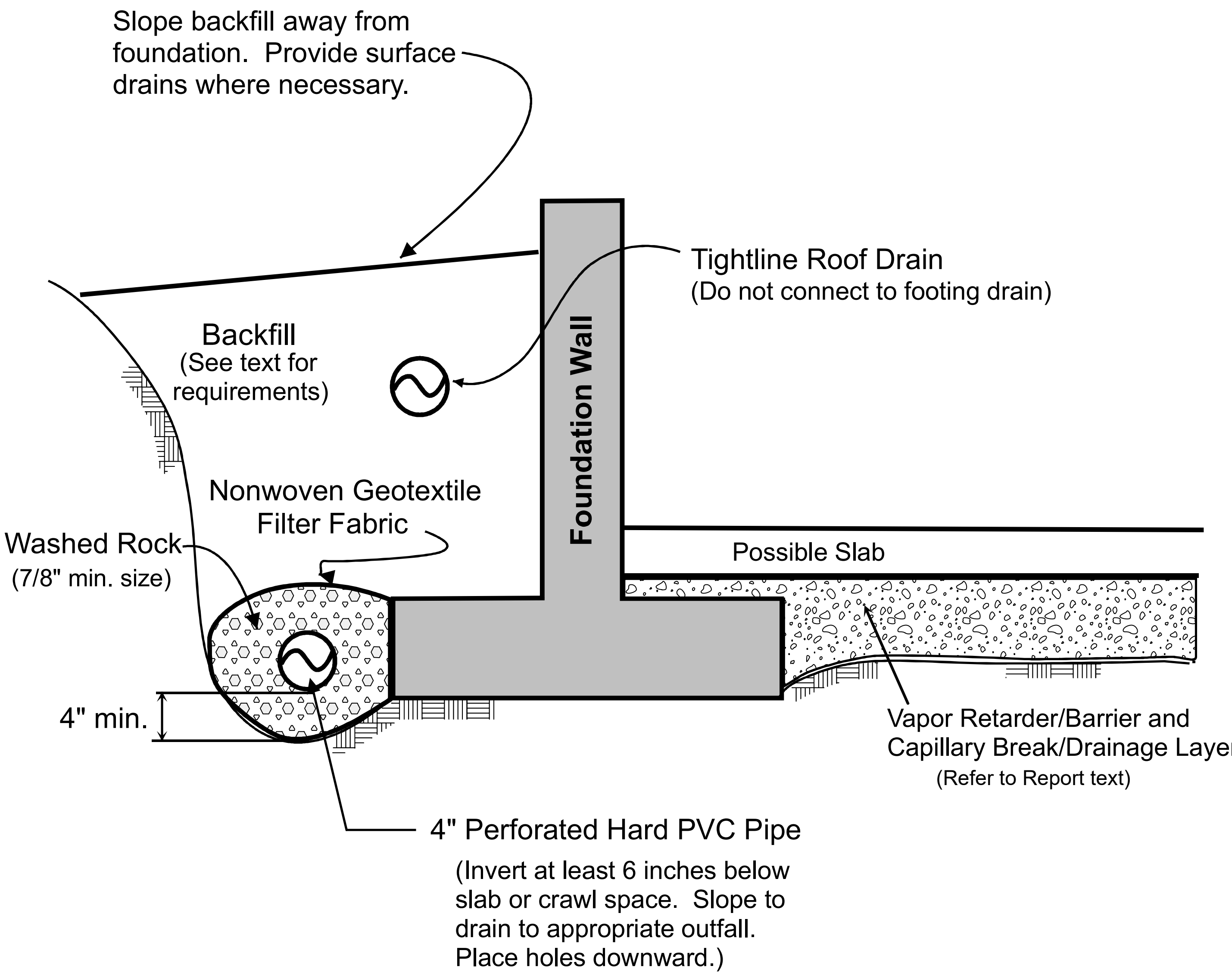
TYPICAL FOOTING OVEREXCAVATION

6221 - 83rd Place Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
22167	May 2022		5



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage, waterproofing, and slab considerations.

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FOOTING DRAIN DETAIL

6221 - 83rd Place Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
22167	May 2022		6