GEOTECH
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JN 23440

Jonathan Spare
8265 Southeast 61st Street
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via email: jdspare@gmail.com

Subject: **Foundation and Critical Area Considerations**
Proposed Remodel of Existing Residence
8265 Southeast 61st Street
Mercer Island, Washington

Greetings:

This report presents our geotechnical engineering report related to the planned work associated with the remodel of your existing home. The scope of our services consisted of assessing the site surface and subsurface conditions, and then developing this summary report.

Based on the preliminary plans prepared by Ectypos Architecture, we understand that the existing residence will undergo a substantial remodel. As a part of this work, additional loads will be applied to the western foundation of the house from changes to the main floor. A new foundation is expected through this area to carry a moment frame. An addition will be constructed between the west side of the house and the existing detached garage. An expanded auto court and entry court will be created in the northeastern portion of the site, north of the garage and east of the residence. This will involve the construction of engineered retaining walls to backfill and stabilize an existing rockery-protected fill slope located along the west side of the existing driveway.

The City of Mercer Island GIS maps your entire lot to lie within both a Potential Landslide Hazard and Erosion Hazard area. The very western edge of the site, which will not be involved in the redevelopment project, is also mapped as a potential Seismic Hazard. There are no steep slopes mapped on, or around, your property.

SITE CONDITIONS

We visited the subject property on October 23, 2023 to meet with you and your design team, and to observe the existing site conditions. Your residence consists of a main floor overlying a west-facing daylight basement that underlies the entire footprint. To the east of the house is a detached garage located approximately 6 feet above the main floor elevation of residence. North of the garage is a paved driveway that enters the property from Southeast 61st Street. Along the west side of the lot is a paved driveway extending from West Mercer Way to provide access to neighboring homes.

The ground surface on the property and in the area generally slopes downward toward the west. The relatively flat driveway on the northeastern portion of the lot has been created by filling over the sloped ground surface, with a rockery-protected slope along the western side of the filled area. Below this filled rockery is a paved footpath that extends along the east side of the house. To the south of the detached garage are multiple short rock walls that create landscape terraces. The rear, western yard is relatively flat, with the west edge being a backfilled retaining wall comprised of concrete or modular landscape blocks. The ground slopes downward toward the neighboring private access drive. In the southwest corner of the property, the ground is terraced with several rock walls.

We saw no indications of recent slope movement on the site. Previous landslides have been documented on the Mercer Island Landslide Hazard Assessment on the steeper slopes further to the west of the site leading down to West Mercer Way.

We are familiar with the native subsurface conditions on the property from review of published geologic maps, explorations that our firm has completed in close proximity to the site, and the results of explorations conducted previously on your southern neighbor's property by another geotechnical firm. The geologic mapping indicates that this area is underlain by glacial till or glacial drift, a highly-competent, glacially-compressed mixture of gravel, silt, and fine-grained sand. Our firm has completed explorations for improvements on the property immediately to the south, and found dense, glacially-compressed sands within 3 to 5 feet of the ground surface. The areas of the planned addition and auto/entry expansions are underlain by fill soils placed to create the driveway and the landscaped area between the house and garage.

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 site and surrounding area are underlain by competent, glacially-compressed native soils. However, the area of the planned improvements east of the house have loose fill soils overlying the dense native soils. Excavating to reach dense soils in this area does not appear practical, due to the limited space for equipment and excavation. As a result, we recommend that the addition between the house and garage, as well as the walls for the new auto court and entry court, be supported on driven pipe piles. It should be possible to use 3-inch pipe piles driven using a small excavator.

Any improvements to the western side of the house, including the potential moment frame, should also be supported on deep foundations. This will provide appropriate support without adding load to existing footings, which could cause noticeable settlement. These deep foundations could consist of pipe piles, or helical piles, depending on equipment access close to the west face of the structure.

The pile-supported retaining walls for the auto court and entry court should be backfilled using geofoam. The piles have very limited lateral capacity, and the use of geofoam backfill minimizes the lateral soil pressures that the pile foundations would have to be designed for. A typical detail for geofoam wall backfill is attached to this report.

> ***Seismic Hazard:*** The underlying glacially-compressed soils beneath the site are not susceptible to seismic liquefaction. The pipe piles will be driven through the fill and any loose upper soils and will be embedded into this dense, non-liquefiable native soil layer.

Potential Landslide Hazard: The planned addition is not close to any steep or tall slope areas. The dense to very dense, glacially-compressed soils that underlie the site are not susceptible to instability, even during a strong earthquake. The stability of the short slope on the western side of the site, over 30 feet west of the house, will not be adversely affected by the shallow excavations needed for the new development. This sloped area also does not pose a risk to the planned new construction. No buffer or other mitigation measures are required to address the Potential Landslide Hazard mapping of the site.

Erosion Hazard: The site disturbance for the proposed development will be limited, and will occur primarily on gently-slope ground. The mapped Erosion Hazard can be mitigated by implementing proper temporary erosion control measures that 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 work areas. Existing ground cover and landscaping should be left in place wherever possible to minimize the amount of exposed soil. Small soil stockpiles should be covered with plastic during wet weather. Soil and mud should not be tracked onto the adjoining streets, and silty water must be prevented from traveling off the site. It should be possible to complete the planned addition during the wet season without adverse impacts to the site and neighboring lots. As with any construction project, it can be necessary to periodically maintain or modify temporary erosion control measures to address specific site and weather conditions.

Once we have reviewed the final plans for the development incorporating the recommendations of this report, we can provide a "statement of risk" to satisfy City of Mercer Island conditions.

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).

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 dense soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of the absence of near-surface groundwater.

PIPE PILES

A 2-inch-diameter pipe pile driven with a minimum 90-pound jackhammer or a 140-pound Rhino hammer to a final penetration rate of 1-inch or less for one minute of continuous driving may be assigned an allowable compressive load of 3 tons. Load tests are not required to verify this allowable capacity. Extra-strong steel pipe should be used for 2-inch-diameter pipe piles.

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

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

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 for 3-inch piles.

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.

Mercer Island has instituted load test requirements for pipe piles larger than 2-inches in diameter. Load tests are required on 3 percent of the installed piles up to a maximum of 5 piles, with a minimum of one pile load test on each project.

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 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.

A 3-inch pipe pile installed at a 1:5 (Horizontal:Vertical) batter can be assumed to have an allowable lateral capacity of 1,000 pounds.

HELICAL PILES

Helical piles could be used for vertical building loads in limited-access areas where it is necessary to support heavier loads than 2-inch-diameter pipe piles can handle.

Helical piles consist of single or multiple helixes that are rotated into the ground on the end of round or square metal shafts. These anchors can be used to support both compression and tension loads, but their lateral capacity is negligible due to the relatively small diameter of the metal shafts. The design capacity of single helix anchors is the allowable soil bearing capacity on the helix area. Multiple-helix anchors are typically assumed to have a design capacity equal to the sum of the allowable bearing capacity on each helix if they are separated more than three helix diameters.

The minimum diameter of a single helix anchor is 8 inches. Multiple helix patterns and diameters are available depending on soil conditions and structural demands, and we recommend that a specialty helical pile contractor be contacted early in the structural design to provide guidance regarding installation means and methods. The ultimate capacity of the helical pile in tension or compression can be estimated roughly by multiplying the installation torque by 10. However, different torque motors and installation equipment have a varying empirical correlation between torque and compressive capacity. A typical allowable helical pile capacity in either compression for an 8-inch/10-inch helix configuration is 12 to 15 kips, with an uplift capacity of 10 kips. In order to achieve these capacities, the anchor should be installed to a minimum torque of 3,000 foot-pounds.

The helical piles should be installed by a specialty contractor familiar with the design and installation of chance systems. The contractor can assist with refining the helical pile design and details and estimating capacities for different soil and anchor conditions. At least one helical anchor should be load tested to at least 200 percent of the design load to verify the allowable capacity. In addition, all remaining helical piles should be torque tested to at least 200 percent of the estimated empirical ultimate capacity in pounds, which is roughly equivalent to 10 times the applied torque in ft-pounds.

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) passive earth pressure of 300 pounds per cubic foot (pcf) for this resistance.

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. As discussed above in the ***General*** section, the lateral design pressure from geofoam backfill is very low. The following recommended parameters are for walls that restrain <u>level</u> backfill:

PARAMETER	VALUE
Active Earth Pressure *	40 pcf (Compacted Free-Draining Backfill) 5 pcf (Geofoam Backfill)
Passive Earth Pressure	300 pcf
Coefficient of Friction	Neglect
Soil Unit Weight	130 pcf (Compacted Free-Draining Backfill)

Where: pcf is Pounds per Cubic Foot, and Active 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 active equivalent fluid pressure. This applies only to walls with level backfill.**

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 active 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 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 also does not need to be applied to walls that are backfilled with geofoam, as the geofoam is lightweight and self-supporting.

For walls backfilled with compacted fill, the recommended seismic surcharge pressure for this project is 8**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.

Retaining Wall Backfill and Waterproofing

The attached Geofoam Backfill Detail provides general guidance for placement of the drainage and waterproofing, and the geofoam itself.

For walls not backfilled using geofoam, it is important that the backfill consists of 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.

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.

Where geofoam backfill is not used, 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 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.

LIMITATIONS

This report has been prepared for the exclusive use of Jonathan Spare, and 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.

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.

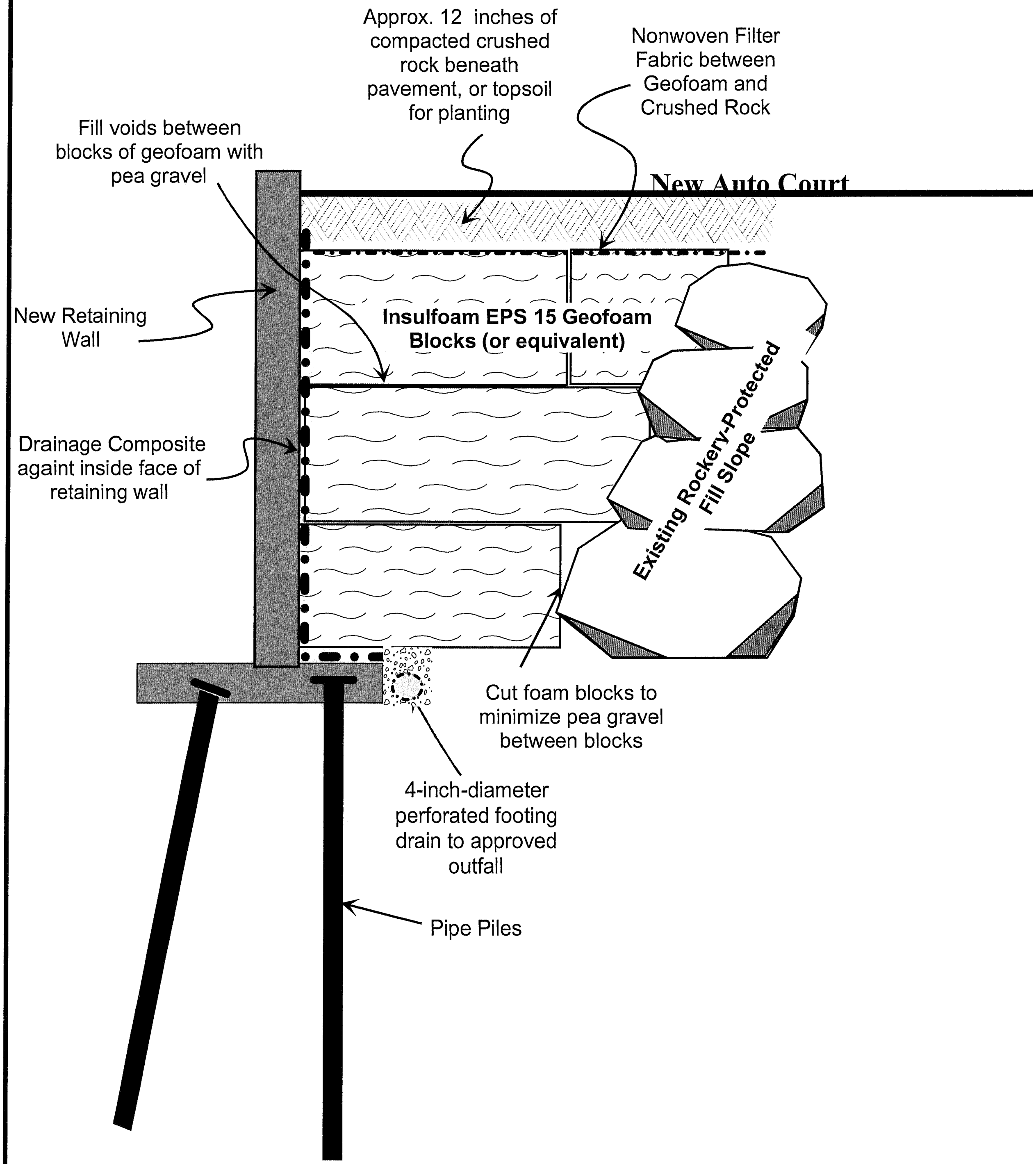
12/21/2023

Marc R. McGinnis, P.E.
Principal

Attachment: GeoFoam Backfill Detail

cc: **Ectypos Architecture** – Lucia Pirzio-Biroli
via email: lucia@ectypos.com

MRM:kg



GEOTECH
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GEOFOAM BACKFILL DETAIL

8265 S.E 61st Street
Mercer Island, Washington

Job	Date:	Scale:	Plate:
23440	Dec. 2023	Not to Scale	1