GEOTECH
CONSULTANTS, INC.

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August 9, 2024

JN 24273

Kimberly and Paul Florence
8818 Southeast 62nd Street
Mercer Island, Washington 98040
via email: kimberly.spears@gmail.com

Subject: **Geotechnical Engineering Study**
Proposed Residential Project
8818 Southeast 62nd Street
Mercer Island, Washington

Greetings:

We are pleased to submit this geotechnical engineering report for the proposed residential project to be constructed on Mercer Island, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, obtaining geological information from an adjacent property, and then developing this report to provide recommendations for the geotechnical engineering aspects of the project. This work was authorized by your acceptance of our Contract for Professional Services dated June 27, 2024.

We were provided with architectural plans by First Lamp Architects/Builders dated June 27, 2024. Based on the plans and conversations with First Lamp personnel, we understand that the project will generally consist of three additions: 1) a one-story, main-level addition on the northeastern side of the existing residence, 2) a second-story addition added to much of the existing detached garage, and 3) a covered pedestrian entrance on the eastern side of the residence. The finish floor of the northeastern residence addition will be the same as the existing residence. The addition will have a crawl space below the finish floor of the northeastern addition.

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 site is located on near the middle of Mercer Island, just east of Island Crest Way. The site is located near the corner of Southeast 62nd Street and 89th Avenue Southeast. The site is somewhat northwest of this corner, but for ease of description, this report assumes that Southeast 62nd Street is on the southern side of the site.

The site is nearly flat with just a very gentle slope down to the east/southeast. The adjacent properties have similar grading. The residence and detached garage are located in the central portion of the site, while an asphalt driveway is located in the southeastern corner. Based on a topography survey we received, prepared by Terrane, the driveway and the garage slab have a grade of approximately 314.5 feet. The one-story residence has a main floor grade at approximately elevation 316.7 feet. There is a patio and landscape area that is between the residence and garage (essentially located where the new residence addition will be located) that is about 1 to 2 feet above the ground north and east of it).

Based on the Mercer Island GIS portal, the eastern edge of the site is considered a Geologic Hazard Area because it is a potential Seismic Hazard Area. This Area continues onto the adjacent eastern property. It appears it is designated as such because there is a ravine/creek located on the eastern side of the adjacent eastern property.

SUBSURFACE

Our firm has provided geotechnical engineering services for a residential project on the adjacent property to the east. The subsurface conditions at that site were explored by excavating three test pits and two test holes. These explorations generally revealed approximately 2 to 4 feet of loose fill soil overlying native, gravelly silty sand. The native sand was initially loose but became dense at depths of about 5 feet in the test pits. In some of the explorations, the native, loose silty sand was revealed at the ground surface. The dense silty sand was revealed at depths of about 1.5 to 3.5 feet in the explorations where loose fill wasn't revealed at the ground surface. The dense soil is geologically known as Glacial Till; this soil is typical in upland areas of the Puget Sound region.

Two test holes were hand-excavated in the proposed development portion of the subject property. The locations of the test holes are indicated on the attached Site Exploration Plan. One test hole was excavated in the area of the proposed addition, and about 3 feet of loose fill was revealed overlying native, gravelly silty sand; dense Glacial Till was revealed at approximately 6 feet. In the test hole excavated adjacent to the existing garage, about 3.5 feet of loose silty sand was revealed overlying the dense Glacial till. The depth to the Glacial Till is indicated on the attached Plan.

No groundwater was revealed in the onsite test holes, and only minor groundwater was revealed in the explorations done on the adjacent eastern property.

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.

This report provides geotechnical engineering conclusions and recommendations based on the information noted above. This report also addresses Geologic Hazard Areas per Mercer Island Code (MICC 19.07). The test holes excavated in the proposed development areas of this project site encountered dense Glacial Till soil at depths ranging from about 3.5 to greater than 6 feet, with the greater depth in the area of the proposed northeastern addition (the ground level in that addition area is about 2 feet higher than the grade where the depth was less). Loose soil was revealed above the Glacial Till. All of the building loads of this project need to bear on or into the Glacial Till. Conventional footings can be used as the foundation, but they need to either bear on the Glacial Till or on structural fill placed on the Glacial Till. However, excavations as deep as about 6 feet are needed to reach the Glacial Till in the addition area and approximately 3.5 feet at the existing garage. If this amount of excavation is deemed excessive, driven pipe piles are the next best alternative as a foundation in our opinion.

It appears that the existing garage structure is just supported on a slab, not a formal foundation. Therefore, we believe new foundations are needed under the slab to support all building loads associated with the upper-story addition over the garage.

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. 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.

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.

CRITICAL AREAS STUDY (MICC 19.07)

Seismic Hazard Area: As noted earlier, the eastern edge of the property, as well as the property to the east that we previously provided services for, is mapped on the Mercer Island GIS portal as a Potential Seismic Hazard Area. However, also as noted earlier, the soils revealed in explorations at the site (including the western edge) and the east property consist of some looser fill and native soils overlying dense Glacial Till. No groundwater or evidence was noted in the explorations on the site or adjacent property. Because of these site conditions, there is not a potential of liquefaction of soil at the site. Therefore, the western side of the site should not be considered a Geologic Hazard Area/Seismic Hazard Area. The proposed development will not be subjected to seismic liquefaction and will be very suitable from a geotechnical engineering standpoint if the recommendations in this report are followed.

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 C (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 1.43g and 0.50g, 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 0.68g. The soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of their dense nature and/or the absence of near-surface groundwater.

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 0.45g.

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.

Three- or 4-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 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.

Extra-strong, Schedule 80 steel pipe should be used for 2-inch-diameter piles. As a minimum, Schedule 40 pipe should be used for the larger 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.

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 a 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. We recommend a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate passive value.

CONVENTIONAL FOUNDATIONS

Structures can be supported on conventional continuous and spread footings bearing on undisturbed, dense, native Glacial Till soil, or on structural fill placed above this competent native soil. See the section entitled ***General Earthwork and Structural Fill*** for recommendations regarding the placement and compaction of structural fill beneath structures. Adequate compaction of structural fill should be verified with frequent density testing during fill placement. 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 12 and 16 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.

An allowable bearing pressure of 3,000 pounds per square foot (psf) is appropriate for footings supported on soils as noted above. 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 inch, with differential settlements on the order of one-half inch in a distance of 40 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	0.50
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.

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 <u>level</u> backfill:

PARAMETER	VALUE
Active Earth Pressure *	35 pcf
Passive Earth Pressure	300 pcf
Coefficient of Friction	0.50
Soil Unit Weight	130 pcf

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. 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

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure 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

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 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.

Wall backfill should 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.

The ***General***, ***Slabs-On-Grade***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

SLABS-ON-GRADE

The building floors can be constructed as slabs-on-grade atop non-organic, firm existing 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.

The ***General***, ***Permanent Foundation and Retaining Walls***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

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 upper, loose 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 sand or 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.

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 are only needed 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. 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.

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 buildings 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 fill 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.

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 footings, slabs or walkways	95%
Filled slopes and behind retaining walls	90%

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).

Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

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 explorations 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 explorations. 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 Kimberly and Paul Florence, and their 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

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.

8/09/2024

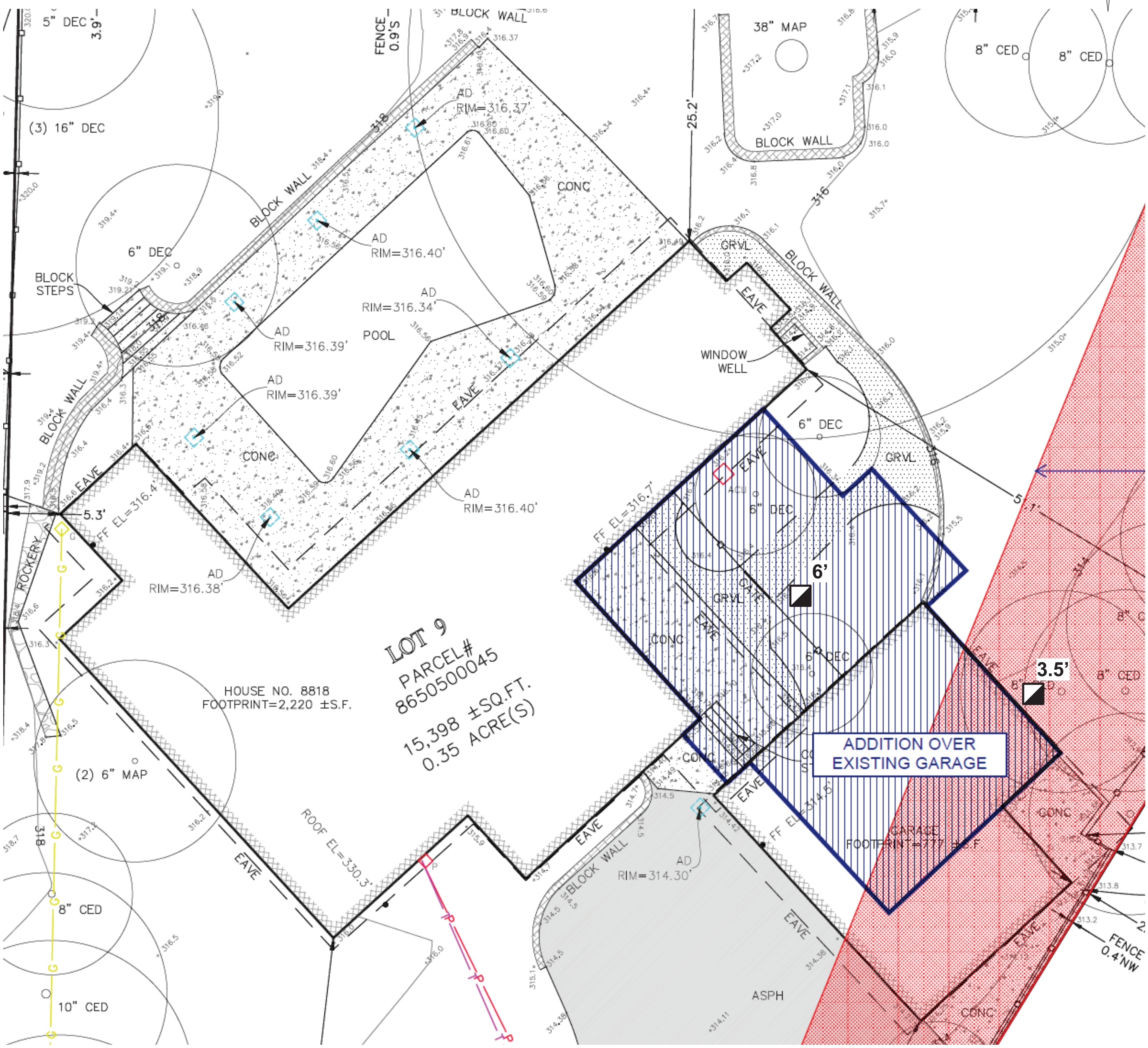
D. Robert Ward, P.E.
Principal

Attachment: Site Exploration Plan

cc: **First Lamp** – Mark Dorsey
via email: mark@firstlamp.net

DRW:kg

NORTH



Legend:

Test Hole Location with Depth to Glacial Till

GEOTECH CONSULTANTS, INC.

SITE EXPLORATION PLAN

8818 Southeast 62nd Street
Mercer Island, Washington

Job No:	Date:		Plate:
24273	Aug. 2024	No Scale	