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JN 21409

Xiaoxia Wu
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Mercer Island, Washington 98040
via email: *xiaoxiaee@gmail.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed New Residence
8480 – 85th Avenue Southeast
Mercer Island, Washington

Greetings:

We are pleased to present this geotechnical engineering report for the proposed new residence to be constructed on your property 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, stormwater infiltration feasibility, and design criteria for foundations, retaining walls, and temporary shoring. This work was authorized by your acceptance of our proposal, P-10991, dated October 6, 2021.

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.

Adam S. Moyer
Geotechnical Engineer

cc: **The Brandt Design Group** – Bree Medley
via email: *bree@brandtdesigninc.com*

ASM/MRM:kg

GEOTECHNICAL ENGINEERING STUDY
Proposed New Residence
8480 – 85th Avenue 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 on your property in Mercer Island.

We were provided with preliminary plans of the proposed new residence overlaid onto a topographic map of the subject site, which were developed by the Brandt Design Group. For clarity, this report will reference Project North when using cardinal directions as depicted on the provided plans and the attached Site Exploration Plan (Plate 2), in which the south side of the lot is bordered on Lake Washington. Based on the provided plans, we understand that the existing house and detached garage will be demolished and a new two-story residence with an attached garage will be constructed near the center of the property. The new residence and garage will generally be "T-shaped" and located in the same location as the existing house, but will extend slightly farther to the north, toward 85th Avenue Southeast. The main footprint of the residence will be rectangular and long in the east-west direction; a garage will be located north of the center of the of the main portion of the residence and be connected by an entryway and staircase. Both the proposed residence and attached garage will overlie a full basement that will daylight to the south with a finished floor elevation of 30 feet. The finished floor of the first (main) floor is proposed at an elevation of 41 feet. Excavation depths of approximately 17.5 and 12 feet beneath the existing ground surface will be required along the upslope side of the proposed basement beneath the garage and main residence, respectively. The excavation depths will diminish to the south along the eastern and western perimeters of the basement as the grade slopes down to the south. New decks and patios are proposed off the southern side of the new residence, and new walkways and staircases are proposed alongside the perimeter of the house. The existing driveway will keep its existing layout, but the width will be expanded. Finally, we understand that the existing sportscourt north of the existing garage will be demolished and replaced with landscaping; however, the steep slope in the northern corner of the property and the existing concrete retaining wall along the toe of the steep slope will remain undisturbed.

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 along the shoreline of the southeast tip of Mercer Island. A one-story residence overlying a basement is located near the center of the property with a main floor elevation of 40.7 feet; the basement daylights to the south and has a finished floor elevation of 31.5 feet. A small breezeway connects the house to a detached garage (with a slab elevation of 40.4 feet) north of the house's eastern half. An asphalt driveway extends northwest from the garage and rises moderately up to 85th Avenue Southeast. The ground surface steps up several feet north of the garage to an asphalt sportscourt with a surface elevation of 45.5 to 46 feet. Based on the provided topographic survey, North of the sportscourt, the grade rises 12 to 14 at an inclination of 50 to 57 percent. A 3-foot-tall concrete retaining wall is located along the toe of the steep slope; the wall "returns" to the south along the western and eastern perimeters of the sportscourt, where the court is cut into the moderately sloped grade and daylights to the south.

As a whole, the subject site slopes moderately downward to the south at an inclination of 15 to 20 percent with a total grade change of 34 to 40 feet. A rock bulkhead and concrete steps border the Lake Washington shoreline along the subject site's southern property line and a wooden dock extends into the lake from the eastern end of the property's southern shoreline. A grass lawn is located south of the residence, and the remainder of the property is covered by landscaping beds and several scattered mature trees.

As discussed above, the subject site is bordered by Lake Washington to the south and 85th Avenue Southeast to the north. Developed residential properties border the site to the east and west which both contain single-family residences. The eastern adjacent residence (#8474) has two floors overlying a basement that daylights to the south near an estimated elevation of 31 feet; an attached garage is located northwest of that residence. The neighboring residence and attached garage are offset 10.8 and 7.1 feet from the subject site, respectively. The western adjacent residence (#8431) has one story, which is bunkered into the sloping ground surface and daylights to the south towards the lake; Based on our site observations and the topographic survey, this neighboring residence has an estimated finished floor elevation near 27 feet and is offset at little as 2 feet from the subject site.

The City of Mercer Island's online geographic information system (GIS) tool maps the subject site within several geologic hazard areas. The property as a whole is located within both a Seismic Hazard area as well as a Potential Landslide Hazard area. Furthermore, the majority of the site is mapped as an Erosion Hazard area. The site lies within a large area along the southeast side of Mercer Island that was affected by an ancient landslide. The headscarp of this slide is the tall bluff located behind the homes that are on the opposite side of East Mercer Way. It is believed that this large landslide, as well as others around Mercer Island, occurred following a very large earthquake. No recent movement of this large landslide mass has occurred. However, periodic slides occur on the old headscarp, and on other steep slopes comprised of improperly placed fill or loose landslide deposits. We saw no indications of recent movement on the short steep slope located between the driveway and the existing house.

SUBSURFACE

The subsurface conditions were explored by drilling thee test borings 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 borings were drilled on October 22, 2021 using a track-mounted, hollow-stem auger drill. Samples were taken at approximate 2.5- and 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The Test Boring Logs are attached as Plates 3 through 5.

Soil Conditions

The test borings conducted on the site encountered loose/medium stiff, native, fractured, silt below the ground surface. In Test Boring 1, located in the driveway north of the existing house and west of the existing garage, the native silt became stiff to very stiff and massive below a depth of 15 feet; the silt became very stiff below 25 feet and extended to the maximum-explored depth of 36.5 feet.

Southeast of the house, the native silt soils became very stiff and massive below a depth of 15 feet and very stiff to hard below 20 feet in Test Boring 2. Off the southwest, downslope corner of the house, Test Boring 3 revealed 7 feet of loose/stiff upper fractured silt overlying the very stiff, massive silt that extended to the maximum-explore depth of 26.5 feet.

The loose/medium stiff, near-surface soils are disturbed deposits remaining from the ancient landslide that affected this portion of Mercer Island sometime after the recession of the last glaciers over 10,000 years ago. The glacially-compressed silt underlies the old landslide deposits.

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

No groundwater seepage was observed in our subsurface explorations. The test borings were left open for only a short time period. It should be noted that groundwater levels vary seasonally with rainfall and other factors. Surface water commonly becomes perched on top of the relatively impervious silts underlying the subject site; this is most frequently observed after periods of sustained heavy precipitation during the wetter winter and spring months in the Puget Sound region.

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. If a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the test boring logs are interpretive descriptions based on the conditions observed during drilling.

CRITICAL AREAS STUDY (MICC 19.07)

Seismic Hazard: The proposed development will be supported on deep foundation embedded into the glacially compressed soils which are not liquefiable, due to their dense nature and the absence of near-surface groundwater. This mitigates the Seismic Hazard.

Potential Landslide Hazard: As previously discussed, the core of the subject site consists of glacially compressed, native silt that has a low potential for deep-seated landslides. The mapping of the Potential Landslide Hazard Area is due to the inference by geologists that the site lies within an ancient landslide, which most likely occurred following the recession of the last glaciers approximately 13,000 years ago. No recent large-scale movement has been documented in this area.

The recommendations presented in this report are intended to prevent the planned development from adversely impacting stability of the site and the surrounding properties.

Steep Slope Hazard Areas: Based on the provided topographic map of the subject site, the northern corner of the property has an inclination of at least 40 percent over a horizontal distance of 30 feet (which the City of Mercer Island code defines as a Steep Slope), where the grade rises to 85th Avenue Southeast to the north. This steep slope has at least partially resulted from the grading for the street A Steep Slope is a qualification as a Landslide Hazard Area under the Mercer Island Code. As discussed, above, we understand the Steep Slope (including the existing concrete retaining wall along its toe) will be left undisturbed. The slope has a height of 12 to 14 feet and an inclination of 50 to 57 percent. The proposed new garage will be offset more than 25 feet from the toe of the Steep Slope. This setback is sufficient to protect the planned development in the event of any future shallow soil movement on the slope. Considering the height of, and the development's offset from, the steep slope, it is our professional opinion that no additional buffers re needed from this Steep Slope, provided the recommendations presented in this report are followed. No unshored excavations should occur within 10 feet of the slope's toe without first consulting the Geotechnical Engineer of Record.

Erosion Hazard Areas: The site also meets the City of Mercer Island's criteria for an Erosion Hazard Area. We have worked on numerous waterfront projects on Mercer Island that have avoided siltation of the lake and surrounding properties by exercising care and being proactive with the maintenance and potential upgrading of the erosion control system through the entire construction process. The location of the site on the shore of Lake Washington will make proper erosion control implementation important to prevent adverse impacts to the lake. 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 to the lake, so a temporary holding tank should be planned for wet weather earthwork until the bare soil is covered. 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 between the silt fence and the lake left in place. Typically, if wet weather construction is anticipated, two parallel silt fences should be installed along the shoreline. 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. It will also be important to cap any existing drain lines found running toward the lake until excavation is completed. This will reduce the potential for silty water finding an old pipe and flowing into the lake. 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. Utilities reaching between the house and the lake should not be installed during rainy weather, and any disturbed area caused by the utility installation should be minimized by using small equipment. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. 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: Under MICC 19.07.160(C), a prescriptive buffer of 25 feet is indicated from all sides of a shallow landslide-hazard area. As noted above, the entire site lies within a mapped Potential Landslide Hazard Area, and the prescriptive buffer would extend far beyond the boundaries of the property and the planned development area. The recommendations of this report consider that the development lies within the prescriptive landslide hazard buffer.

As discussed above, we believe no Steep Slope buffer is necessary based on the provided preliminary plans, and no buffer is required by the MICC for an Erosion Hazard Area.

We recognize that the planned development will occur within the designated critical areas. 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, shoring, and erosion control will mitigate any potential hazards to 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 alterations 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 borings conducted for this study encountered native medium-stiff, fractured silt beneath the ground surface. The upper silt soils are moderately compressible, and as a result, shallow foundations constructed on top of them could experience excessive long-term post-construction settlement. Considering this issue, we recommend the proposed residence be supported on a deep foundation system consisting of small-diameter pipe piles driven into the underlying very stiff/dense glacially-compressed silt. This is a typical foundation system that has been used for homes in the area. We recommend floor slabs and other settlement-sensitive elements such as decks, entryways, stairs, etc. be supported on driven pipe piles as well, to prevent differential settlement between them and the pile-supported residence.

The adjacent residences to the east and west are likely supported on conventional foundations that bear on compressible soils. As a result, it is likely that they have undergone excessive settlement already. There is always some risk associated with demolition and foundation construction near structures such as this. 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. It is imperative that un-shored excavations do not extend below a 3:1 (Horizontal:Vertical) imaginary bearing zone sloping downward from existing footings. The western neighboring property also contains several brick retaining walls near the shared property line with the subject site, which will also be very settlement-sensitive. Contractors working on the demolition and construction of your home must be cautioned to avoid strong ground vibrations, which could cause additional settlement in the neighboring foundations. While installation of pipe piles can be loud, it does not cause strong enough ground vibrations to cause settlement of nearby structures. During demolition, strong pounding on the ground with the excavator, which is often used to break up debris and concrete, should not occur. Large equipment and vibratory compactors should not be used close to the south property line. 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, in order to protect yourselves from unsubstantiated damage claims from the adjacent owners, 1) the existing condition of the foundation should be documented before starting demolition, and 2) the footings should be monitored for vertical movement during the demolition, excavation, and construction process. These are common recommendations for projects located close to existing structures that may bear on loose soil and have already experienced excessive settlement. We can provide additional recommendations for documentation and monitoring of the adjacent structures, if desired.

It is likely that some settlement of the ground surrounding pile-supported buildings will occur over time. In order to reduce the potential problems associated with this, we recommend the following:

- Fill to the desired site grades several months prior to constructing on-grade slabs, walkways, and pavements around the buildings. This allows the underlying soils to undergo some consolidation under the new soil loads before final grading is accomplished.

- Connect all in-ground utilities beneath the floor slabs to the pile-supported floors or grade beams. This is intended to prevent utilities, such as sewers, from being pulled out of the floor as the underlying soils settle away from the slab. Hangers or straps can be poured into the floors and grade beams to carry the piping. The spacing of these supporting elements will depend on the distance that the pipe material can span unsupported.

- Construct all entrance walkways as reinforced slabs that are doweled into the grade beam at the door thresholds. This will allow the walkways to ramp down and away from the building as they settle, without causing a downset at the threshold.

- Isolate on-grade elements, such as walkways or pavements, from pile-supported foundations and columns to allow differential movement.

The depth of the excavation for the proposed basement will be an important geotechnical consideration, particularly due to its proximity to the subject site's eastern and western property lines, and the toe of the steep slope north of the residence. We anticipate a bottom-of-excavation elevation near 28 feet to construct the foundations beneath the basement slab. Based on the provided preliminary plans and topographic map, the excavation along the upslope side of the basement beneath the main portion of the house will be in the order of 12 feet below the ground surface; however, the excavation for the basement floor beneath the garage will be up to 17 to 18 feet beneath the existing sportscourt to the north. Temporary open cut slopes of more than a few feet in the loose upper silt soils will be challenging, particularly during wet weather or if active groundwater seepage is encountered. However, if the footprint of the new basement (beneath the main portion of the residence) will be located several feet to the south and downslope of the existing basement's upslope foundation walls, it may be possible to use the existing foundation walls as temporary shoring for a portion of the new basement excavation. The project structural engineer will need to design an internal bracing system to provide the existing foundation walls with adequate lateral support to be used as temporary shoring walls. Coordination with the project contractor will also be necessary, as careful demolition sequencing will also be an important factor. However, it appears temporary cantilevered soldier pile shoring will be required for at least portions of the main house excavation (along the eastern and western property lines. The deeper excavation for the basement beneath the new garage will need temporary soldier pile shoring, and may likely need to be laterally restrained with tieback anchors. We can assist in excavation and shoring options once more specific plans have been developed.

We understand the existing driveway will be expanded to the north, which will likely require a small permanent retaining wall, where it is cut into the sloping ground surface. Such a retaining wall would have to be pipe pile-supported to avoid excessive settlement and rotation. The onsite silt soils are very moisture-sensitive and extremely difficult to adequately re-compact. It will not be possible to re-use the onsite soils as structural fill. Exporting the onsite soils and imported granular structural fill should be anticipated.

Considering the very low permeability of the silt encountered in our test borings, it is our professional opinion that onsite stormwater infiltration is infeasible for the subject site. Attempting to infiltrate or disperse runoff from impervious surfaces would potentially create drainage problems not only for the planned residence, but also the adjacent properties.

Temporary erosion control measures are discussed above in the Erosion Hazard Areas sub-section. Wet weather construction on this site should be possible without adverse impacts to the surrounding properties. In preventing erosion control problems on any site, it is most important that any disturbed soil areas be immediately protected. This requires diligence and frequent communication on the part of the general contractor and earthwork subcontractor. As with all construction projects undertaken during potentially wet conditions, it is important that the contractor's on-site personnel are familiar with erosion control measures and that they monitor their performance on a regular basis. It is also appropriate for them to take immediate action to correct any erosion control problems that may develop, without waiting for input from the geotechnical engineer or representatives of the City.

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 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.

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 1.47g and 0.50g, respectively.

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) 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 proposed residence will be supported on small-diameter pipe piles embedded into the dense underlying soils which are not susceptible to seismic liquefaction.

PIPE PILES

Three-, 4-, or 6-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
4 inches	16 sec/inch	10 sec/inch	4 sec/inch	10 tons
6 inches	20 sec/inch	10 sec/inch	6 sec/inch	25 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.

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 250 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.

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
Active Earth Pressure * - Level Backfill - 2.5:1 (H:V) Sloped Backfill	 40 pcf 55 pcf
Passive Earth Pressure	300 pcf
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.**

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 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 for a distance of 1.5 times the wall height from corners or bends in the walls. 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 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.

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 percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The onsite soils are not free-draining and they have a low compacted strength. They should not be used as wall backfill.

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

FLOOR SLABS

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. This capillary break/drainage layer is not necessary if an underslab drainage system is installed.

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 also notes that vapor *retarders* such as 6-mil plastic sheeting have been used in the past, but are now recommending a minimum 10-mil thickness for better durability and long term performance. 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

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a 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 C. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1.5:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut. Unless approved by the geotechnical engineer of record, it is important that vertical cuts not be made where the overall depth of the temporary cut slopes is taller than 4 feet.

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 and/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.

All permanent cuts into native soil should be inclined no steeper than 2.5:1 (H:V). Fill slopes should not be constructed with an inclination greater than 3:1 (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.

SOLDIER PILE SHORING

Cantilevered soldier pile systems have proven to be an efficient and economical method for providing excavation shoring where the depth of excavation is less than approximately 15 feet.

Soldier pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. Based on the perched groundwater and loose soils encountered in our test borings, the contractor should be prepared to case the holes or use the slurry method if caving soil is encountered. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement on adjacent properties. If water is present in a hole at the time the soldier pile is poured, concrete must be tremied to the bottom of the hole.

As excavation proceeds downward, the space between the piles should be lagged with timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Treated lagging is usually required for permanent walls, while untreated lagging can often be utilized for temporary shoring walls. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure, if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

If permanent building walls are to be constructed against the shoring walls, drainage should be provided by attaching a geotextile drainage composite with a solid plastic backing, similar to Miradrain 6000, to the entire face of the lagging, prior to placing waterproofing and pouring the foundation wall. These drainage composites should be hydraulically connected to the foundation drainage system through weep holes placed in the foundation walls.

Soldier Pile Wall Design

Temporary soldier pile shoring that is cantilevered, and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 40 pounds per cubic foot (pcf). If soldier pile walls will permanently retain soil, they should be designed for earth pressures presented in the ***Permanent Foundation and Retaining Walls*** section.

Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. Existing adjacent buildings will exert surcharges on the proposed shoring wall, unless the buildings are underpinned. Slopes above the shoring walls will exert additional surcharge pressures. These surcharge pressures will vary, depending on the configuration of the cut slope and shoring wall. We can provide recommendations regarding slope and building surcharge pressures when the preliminary shoring design is completed.

It is important that the shoring design provides sufficient working room to drill and install the soldier piles, without needing to make unsafe, excessively steep temporary cuts. Cut slopes should be planned to intersect the backside of the drilled holes, not the back of the lagging.

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 300 pcf. A reduction factor is included in this passive pressure to account for strain compatibility in regards to pile deflection. For permanent walls, we recommend a minimum factor of safety of 1.5 be applied to overturning and sliding calculations when using this ultimate value (temporary installations may use a factor of safety of 1.2). This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two times the grouted pile diameter. Cut slopes made in front of shoring walls significantly decrease the passive resistance. This includes temporary cuts necessary to install internal braces or rakers. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up." Tied-back soldier piles should be embedded no less than 10 feet below the lowest point of the excavation, including footing and utility excavations. Plate 6 attached to this report presents a cantilevered soldier pile retaining wall detail.

The vertical capacity of soldier piles to carry the downward component of the tieback forces will be developed by frictional shaft resistance along the embedded length of the pile.

PARAMETER	DESIGN VALUE
Pile Shaft Friction	1,500 psf

Where: psf is Pounds per Square Foot.

The above value assumes that the excavation is level in front of the soldier pile and that the bottom of the pile is embedded a minimum of 10 feet below the floor of the excavation. The concrete surrounding the embedded portion of the pile must have sufficient bond and strength to transfer the vertical load from the steel section through the concrete into the soil.

Tieback Anchors

General considerations for the design of tied-back or braced soldier-pile walls are presented on Plate 7. We recommend installing tieback anchors at inclinations between 20 and 30 degrees below horizontal. The tieback will derive its capacity from the soil-grout strength developed in the soil behind the no-load zone. The minimum grouted anchor length should be 10 feet. The no-load zone is the area behind which the entire length of each tieback anchor should be located. To prevent excessive loss-of-ground in a drilled hole, the no-load section of the drilled tieback hole should be backfilled with a sand and fly ash slurry, after protecting the anchor with a bond breaker, such as plastic casing, to prevent loads from being transferred to the soil in the no-load zone. The no-load section could be filled with grout after anchor testing is completed.

During the design process, the possible presence of foundations or utilities close to the shoring wall must be evaluated to determine if they will affect the configuration and length of the tiebacks.

Based on the results of our analyses and our experience at other construction sites, we suggest using an adhesion value of 750 psf in the (very stiff massive silt) to design temporary anchors, if the mid-point of the grouted portion of the anchor is more than 10 feet below the overlying ground surface. This value applies to non-pressure-grouted anchors. Pressure-grouted or post-grouted anchors can often develop adhesion values that are two to three times higher than that for non-pressure-grouted anchors. These higher adhesion values must be verified by load testing.

Soil conditions, soil-grout adhesion strengths, and installation techniques typically vary over any site. This sometimes results in adhesion values that are lower than anticipated. Therefore, we recommend substantiating the anchor design values by load-testing all tieback anchors. At least two anchors in each soil type encountered should be performance-tested to 200 percent of the design anchor load to evaluate possible anchor creep. Wherever possible, the no-load section of these tiebacks should not be grouted until the performance tests are completed. Unfavorable results from these performance tests could require increasing the lengths of the tiebacks. The remaining anchors should be proof-tested to at least 135 percent of their design value before being "locked off." After testing, each anchor should be locked off at a prestress load of 80 to 100 percent of its design load.

If caving or water-bearing soil is encountered, the installation of tieback anchors will be hampered by caving and soil flowing into the holes. It will be necessary to case the holes, if such conditions are encountered. Alternatively, the use of a hollow-stem auger with grout pumped through the stem as the auger is withdrawn would be satisfactory, provided that the injection pressure and grout volumes pumped are carefully monitored.

All drilled installations should be grouted and backfilled immediately after drilling. No drilled holes should be left open overnight.

EXCAVATION AND SHORING MONITORING

As with any shoring system, there is a potential risk of greater-than-anticipated movement of the shoring and the ground outside of the excavation. This can translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring or commencing 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, the shoring walls and any adjacent foundations should be monitored during 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. At least every other soldier pile should be monitored by taking readings at the top of the pile. Additionally, benchmarks installed on the surrounding buildings should be monitored for at least vertical movement. We suggest taking the readings at least once a week, until it is established that no deflections are occurring. The initial readings for this monitoring should be taken before starting any demolition or excavation on the site.

DRAINAGE CONSIDERATIONS

We anticipate that permanent foundation walls may be constructed against the shoring walls. Where this occurs, a plastic-backed drainage composite, such as Miradrain, Battledrain, or similar, should be placed against the entire surface of the shoring prior to pouring the foundation wall. Weep pipes located no more than 6 feet on-center should be connected to the drainage composite and poured into the foundation walls or the perimeter footing. A footing drain installed along the inside of the perimeter footing will be used to collect and carry the water discharged by the weep pipes to the storm system. Plate 8 presents typical considerations for a shoring drainage system. Isolated zones of moisture or seepage can still reach the permanent wall where groundwater finds leaks or joints in the drainage composite. Formal waterproofing is typically necessary in areas where wet conditions at the face of the permanent wall will not be tolerable. If this is a concern, the permanent drainage and waterproofing system should be designed by a specialty consultant familiar with the expected subsurface conditions and proposed construction.

Footing drains placed inside the building or behind backfilled walls should consist of 4-inch, perforated PVC pipe surrounded by at least 6 inches of 1-inch-minus, washed rock wrapped in a 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 level of a crawl space or the bottom of a floor slab, and it should be sloped slightly for drainage. All roof and surface water drains must be kept separate from the foundation drain system.

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 9. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains.

As a minimum, a vapor retarder, as defined in the ***Floor Slabs*** 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 even a few inches of free draining gravel underneath the vapor retarder limits the potential for seepage to build up on top of the vapor retarder.

Perched 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 a building should slope away at least 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, behind permanent retaining or foundation walls, 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. As discussed in the ***General*** section, the on-site soils are not suitable for reuse as structural fill, due to their high fines content and moisture sensitivity.

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. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can 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 relative compactions for structural 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).

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 test borings 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 borings. 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 Xiaoxia Wu 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 - 5	Test Boring Logs
Plate 6	Cantilevered Soldier Pile Detail
Plate 7	Tied-Back Shoring Detail
Plate 8	Typical Shoring Drain Detail
Plate 9	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.

Adam S. Moyer
Geotechnical Engineer

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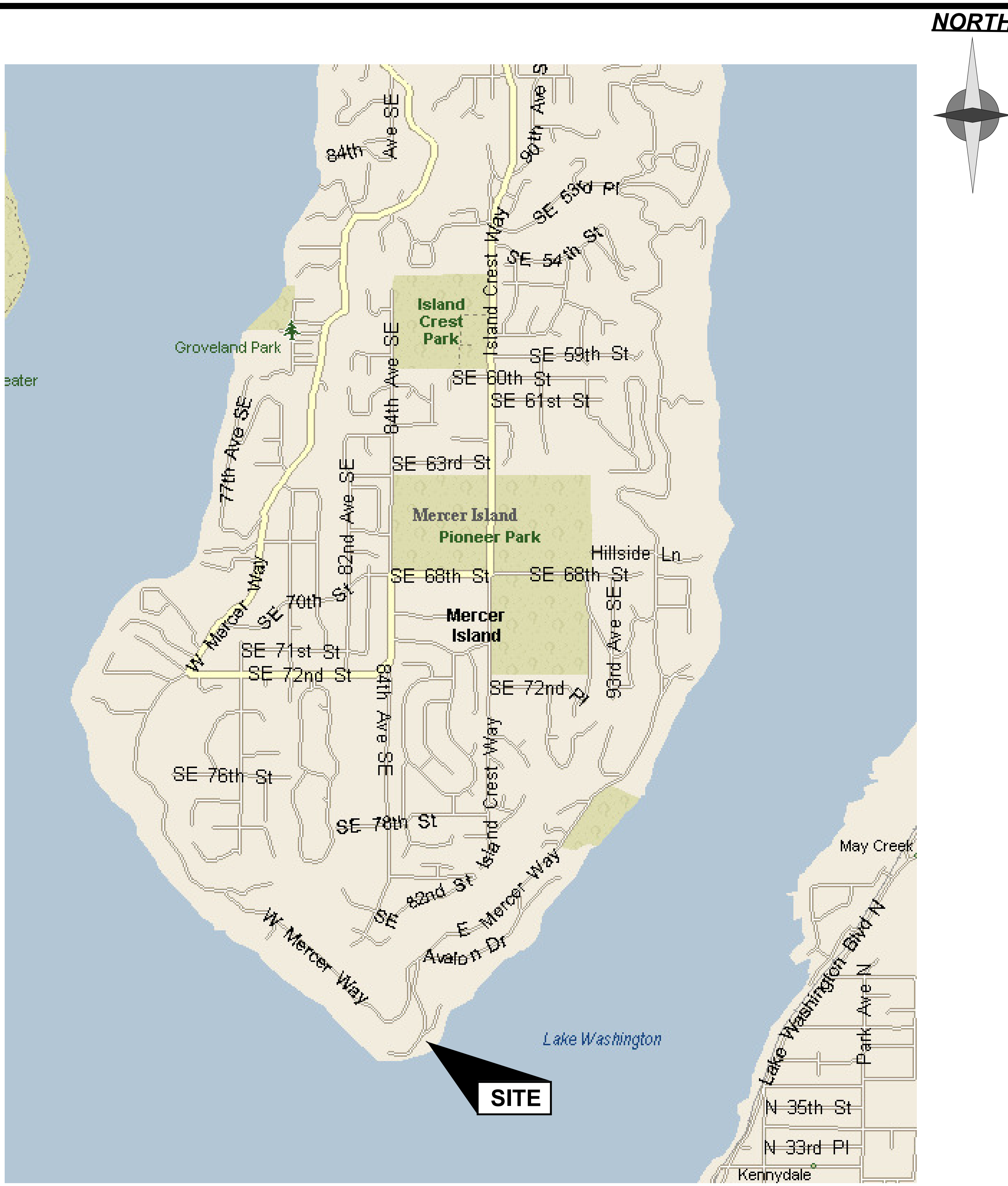
REGISTERED
PROFESSIONAL ENGINEER

11/16/2021

Marc R. McGinnis, P.E.
Principal

ASM/MRM:kg



(Source: Microsoft MapPoint, 2013)

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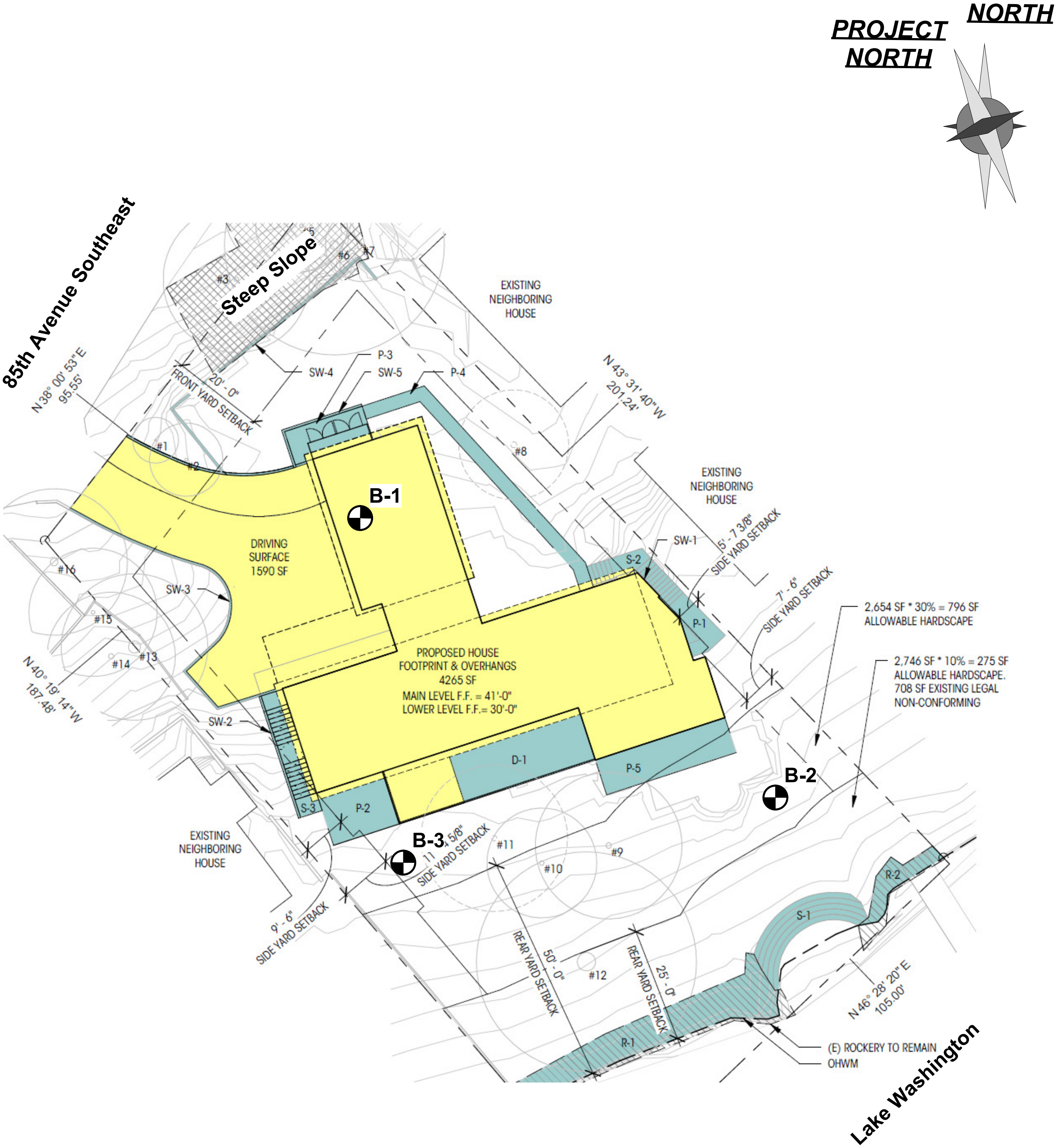
VICINITY MAP

8480 - 85th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
21409	Nov. 2021		1



Legend:

Test Boring Location

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SITE EXPLORATION PLAN

8480 - 85th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
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BORING 1

Depth	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			6	1	ML	Gray-brown with rust, sandy SILT with occasional hard silt inclusions and trace roots, non-plastic, fine-grained, moist, loose
5			10	2	ML / SP	Gray, very sandy SILT / very fine-grained SAND, non-plastic, moist to very moist, loose (Colluvium)
			10	3		
10			5	4	ML	Gray with rust SILT, slightly plastic, jumbled texture, moderately-fractured, moist, medium stiff (Colluvium)
15			13	5		Gray SILT, plastic, massive, moist, stiff to very stiff (Glacially-Compressed)
20			13	6		-slickenside
25			17	7	ML	-becomes very stiff
30			22	8		
35			26	9		
40						

*** Test boring was terminated at 36.5 feet on October 22, 2021.**
*** No groundwater seepage was encountered during drilling.**

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BORING LOG
8480 - 85th Avenue Southeast
Mercer Island, Washington

Job No:	*Date:*	*Logged by:*	*Plate:*
21409	Nov. 2021	ASM	3

BORING 2

Depth (ft)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			7	1	ML	Gray with occasional rust SILT, plastic, jumbled texture, moderately-fractured, moist, medium stiff (Colluvium)
5			14	2		
						Gray SILT, plastic, slightly fractured, moist, stiff to very stiff (Colluvium)
			11	3	ML	-becomes sandy, jumbled, with occasional hard silt inclusions
10			10	4		Gray SILT, plastic, slightly-fractured, moist, stiff
15			14	5	ML	-becomes massive, very stiff, with trace thin sand lenses (Glacially-Compressed)
20			27	6		-becomes very stiff to hard
25			36	7		-becomes non-plastic, dense
30						
35						
40						

*** Test boring was terminated at 26.5 feet on October 22, 2021.**
*** No groundwater seepage was encountered during drilling.**

GEOTECH CONSULTANTS, INC.

BORING LOG
8480 - 85th Avenue Southeast
Mercer Island, Washington

Job No:	*Date:*	*Logged by:*	*Plate:*
21409	Nov. 2021	ASM	4

BORING 3

Depth (ft)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
			8	1	ML	Gray-brown, sandy SILT with trace roots, non-plastic, jumbled, moderately-fractured, moist, loose (Colluvium)
5			6	2		-with occasional hard silt inclusions
			17	3		Gray SILT, plastic, massive, very stiff (Glacially-Compressed)
10			20	4		-slickenside
15			17	5	ML	
20			22	6		
25			28	7		-becomes very stiff to hard -with occasional very thin sand seams
30						
35						
40						

*** Test boring was terminated at 26.5 feet on October 22, 2021.**
*** No groundwater seepage was encountered during drilling.**

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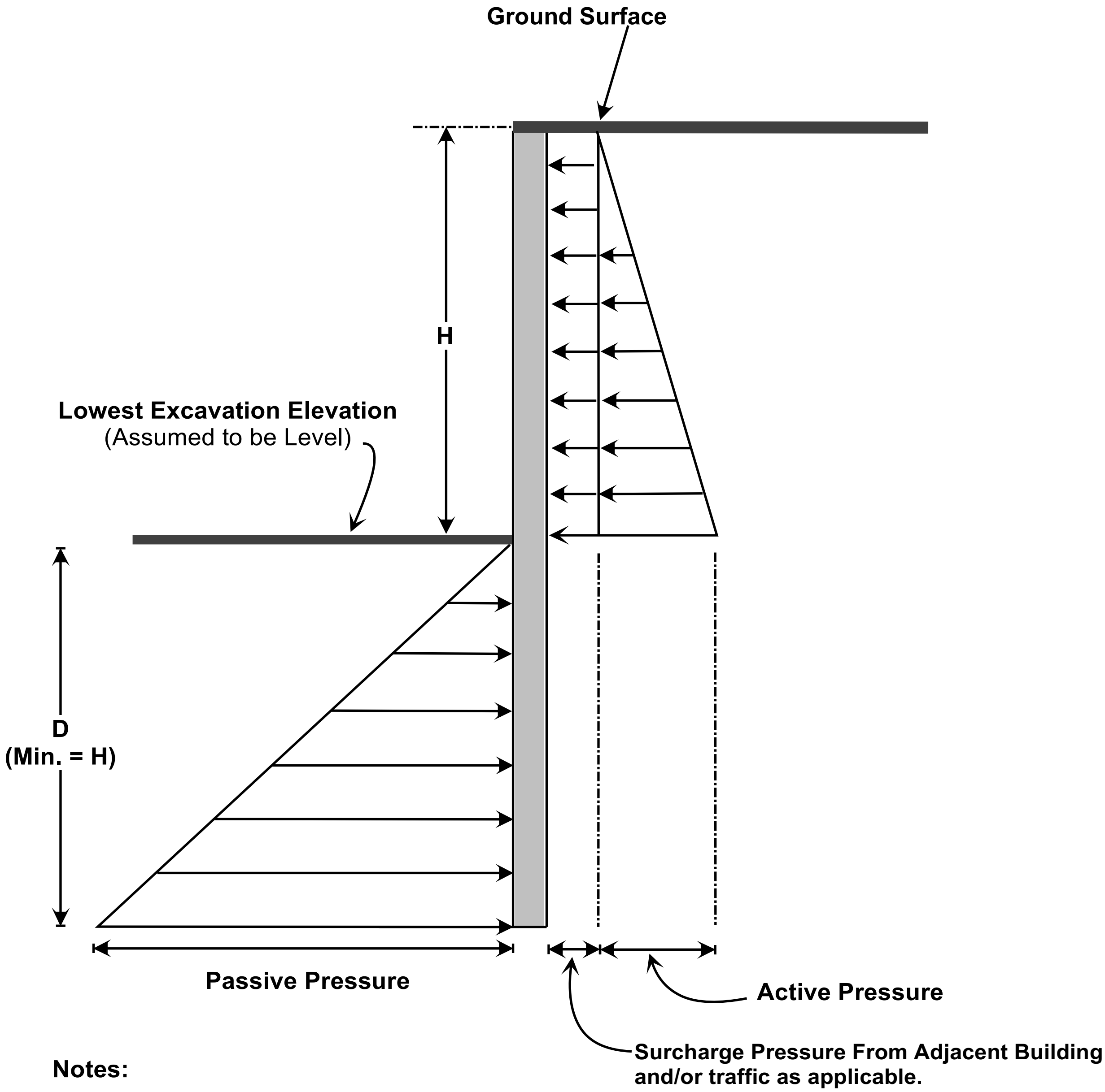
BORING LOG

8480 - 85th Avenue Southeast
Mercer Island, Washington

Job No:	*Date:*	*Logged by:*	*Plate:*
21409	Nov. 2021	ASM	5



Notes:

(1) The report should be referenced for specifics regarding design and installation.
(2) Active pressures act over the pile spacing.
(3) Passive pressures act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller.
(4) It is assumed that no hydrostatic pressures act on the back of the shoring walls.
(5) Cut slopes or adjacent structures positioned above or behind shoring will exert additional pressures on the shoring wall.

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CANTILEVERED SOLDIER PILE SHORING

8480 - 85th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
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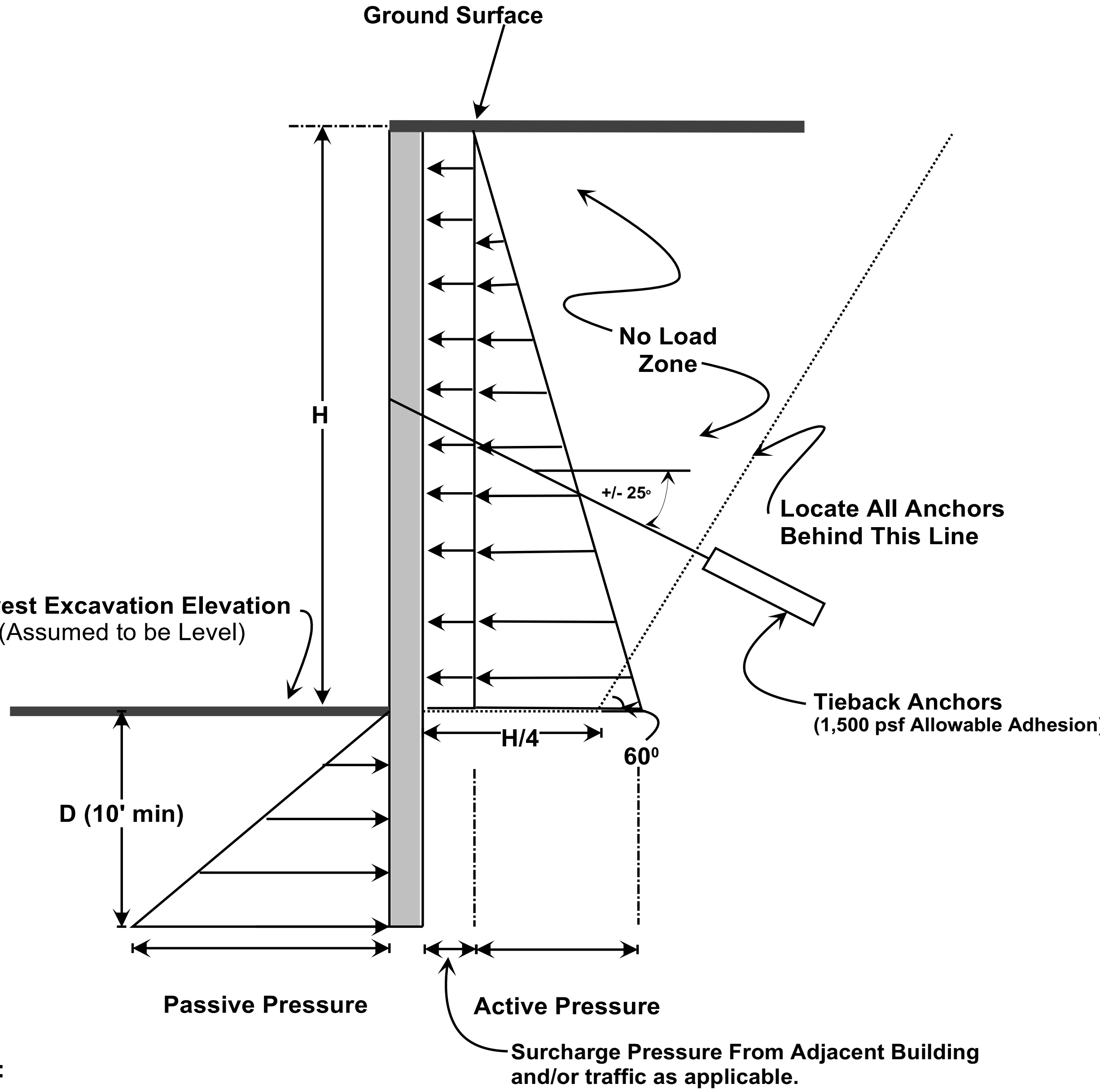

Notes:

(1) The report should be referenced for specifics regarding design and installation.
(2) Active pressures act over the pile spacing.
(3) Passive pressures act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller.
(4) It is assumed that no hydrostatic pressures act on the back of the shoring walls.
(5) Cut slopes or adjacent structures positioned above or behind shoring will exert additional pressures on the shoring wall.

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TIED-BACK SHORING DETAIL

8480 - 85th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
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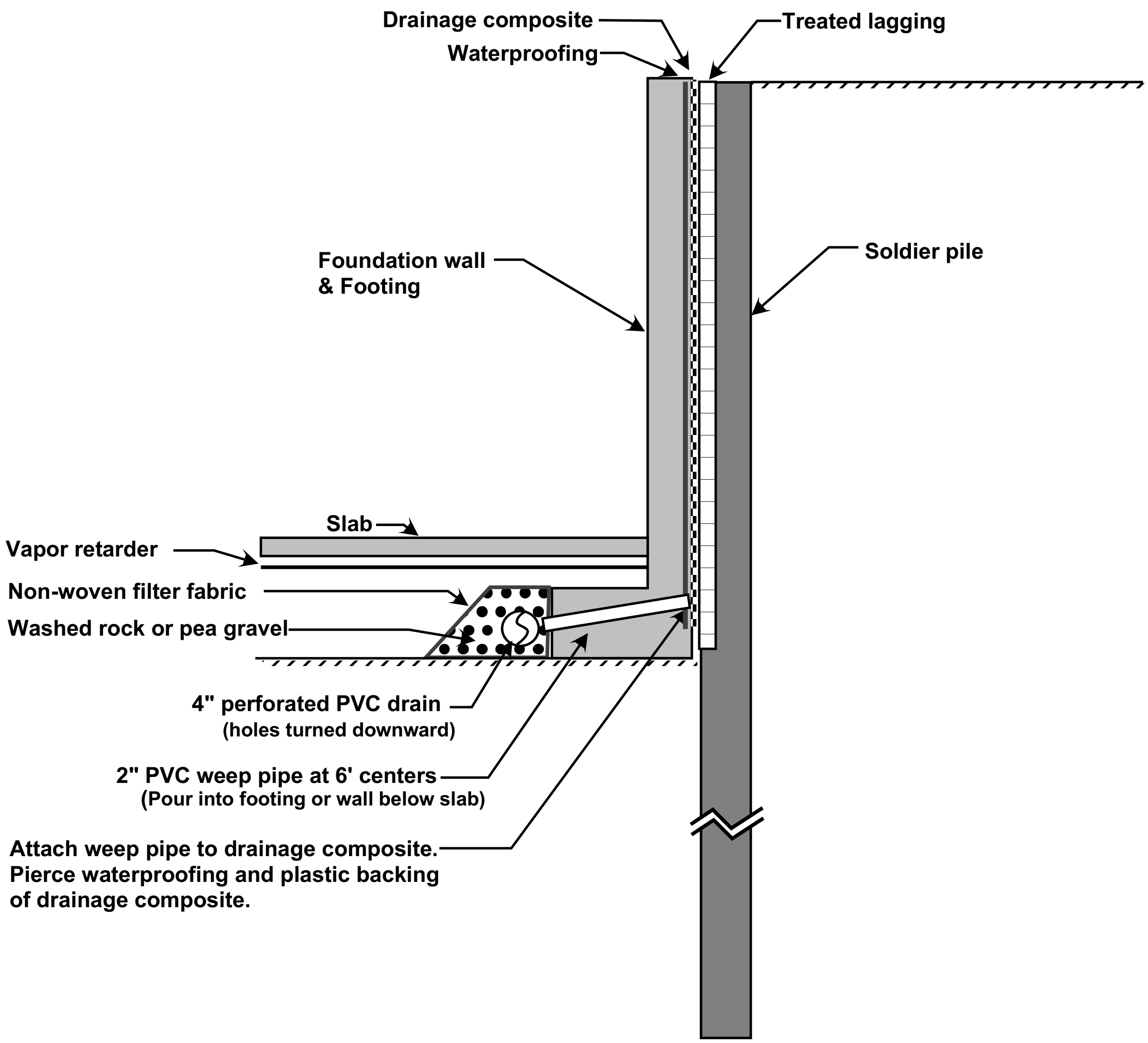

<u>Note</u> - Refer to the report for additional considerations related to drainage and waterproofing.

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SHORING DRAIN DETAIL

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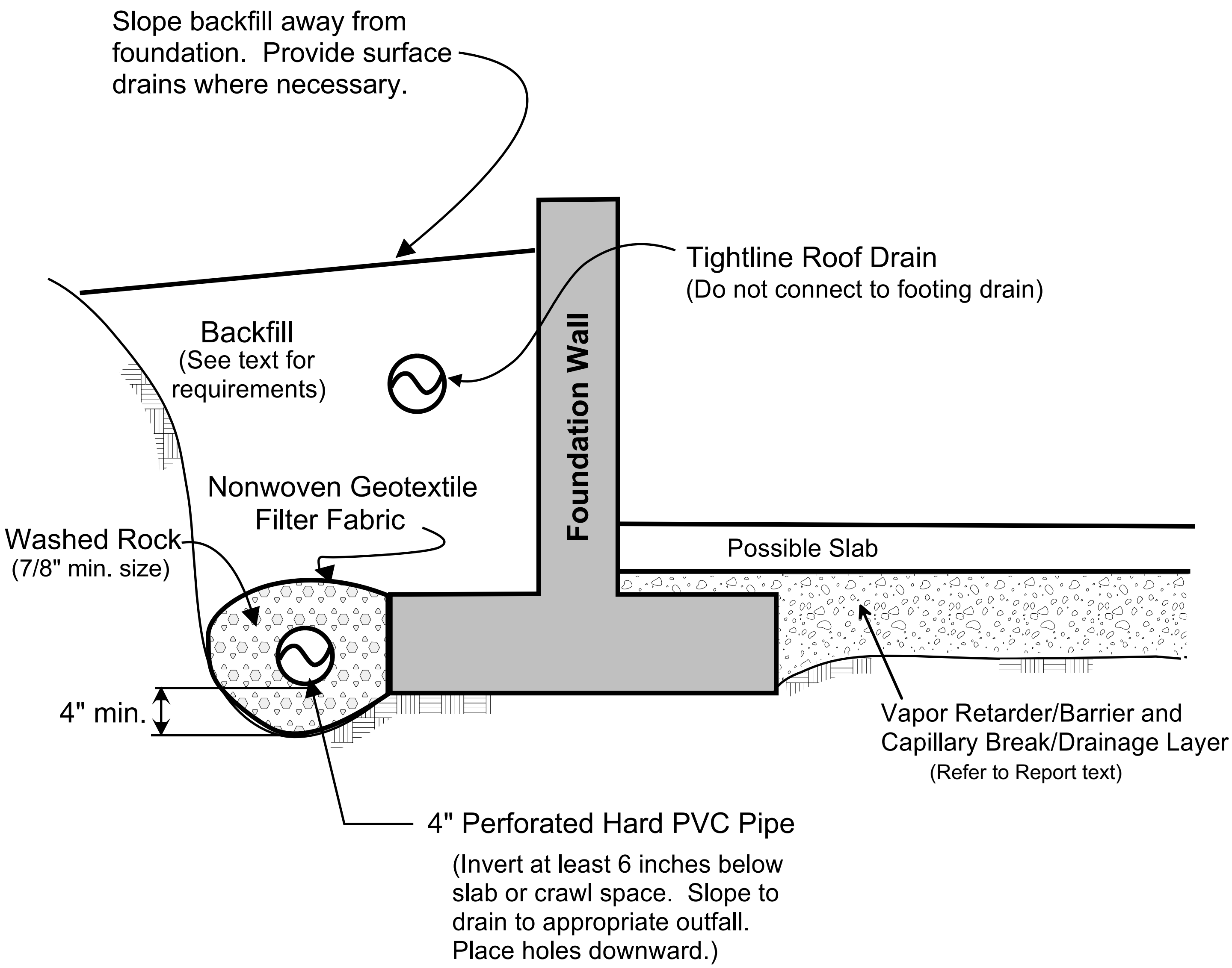
Mercer Island, Washington

Job No:	Date:		Plate:
21409	Nov. 2021		8



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

8480 - 85th Avenue Southeast
Mercer Island, Washington

Job No:	Date:		Plate:
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