GEOTECH
CONSULTANTS, INC.

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January 4, 2024

JN 23443

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Subject: **Geotechnical Engineering Report**
Proposed Remodel/Expansion of Existing Residence
2411 – 60th Avenue S.E.
Mercer Island, Washington

Greetings:

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

Based on the preliminary plans prepared by Sturman Architects, we understand that the existing residence will undergo a substantial remodel. As a part of this work, a second floor will be added over the existing attached garage located off the southeastern corner of the house. The footprint of the house will be modified on the west side of the structure, including a reconstruction and expansion of existing decks west of the house. This will require construction of new foundations. As a part of the remodel, a swim spa will be installed in the north end of the existing basement. This will require excavation of 4 to 5 feet below the existing floor slab. New foundation walls and a floor slab will be needed to provide a below-grade “pit” in which the swim spa will sit. This will necessitate temporary excavation for the construction of the new pit foundations and walls.

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

SITE CONDITIONS

We visited the subject property on December 13 and 21, 2023 to observe the existing conditions and to conduct subsurface explorations in the areas of the proposed work. The subject lot is situated on the western shore of Lake Washington. It is accessed by a paved driveway extending from 60th Avenue Southeast through the southern edge of the neighboring eastern property.

Your residence consists of a main floor overlying a west-facing daylight basement that underlies the footprint of the living space. An attached garage located at the main floor elevation extends eastward from the southeast portion of the residence. The eastern, upslope, side of the garage is embedded 3 to 4 feet below the surrounding grade. The area to the west of the house is grass-covered yard extending to the bulkhead along the shore of Lake Washington.

The ground surface on the property and in the surrounding area generally slopes gently to moderately downward toward the west. We saw no steep slopes on, or around, the site. There are some short (less than 5 feet in height) landscape walls around the site.

We observed cracks and signs of previous settlement in the north and south foundation walls of the attached garage, extending at least 15 feet east from the basement wall of the house. This portion of the garage is at, or above, the pre-existing site grades.

The house to the north of the site sits approximately 25 feet away from the common property line. To the south is a house located only approximately 5 feet from the property line. We observed at least one crack in the north foundation wall of this residence, indicating the likelihood of previous excessive settlement.

We saw no indications of recent slope movement on the site. The ground surface on the property and in the vicinity is not steeply sloped, and the area is known to be underlain at a shallow depth by competent glacially-compressed soils.

We are familiar with the native subsurface conditions on the property from review of published geologic maps, explorations that our firm has completed in close proximity to the site, and the results of explorations conducted around the existing house. Geologic maps indicate that the site and surrounding properties are underlain by glacial drift, a glacially-compressed silty sand or sandy silt. The far western edge of the property may be underlain by old lake deposits, resulting from the higher level of Lake Washington that existed before the Montlake Cut was opened in 1916. Our firm conducted explorations and observed the foundation excavation for the new house under construction on the lot immediately to the southeast. That neighboring site is underlain by dense to very dense, sandy silt consistent with glacial drift. On December 21, 2023 we conducted hand-excavated test holes at the approximate locations shown on the attached Site Exploration Plan. Logs of these test holes are also attached. All of the test holes found dense, glacially-compressed silt at varying depths. Test Hole 1 was conducted alongside the existing western perimeter basement footing, and reached dense, native silt at the base of the footing. Test Holes 2 and 3 were excavated beneath the western edge of the existing elevated portions of the house. These explorations revealed dense, native silt at a depth of 1.5 to 3 feet below the existing ground surface. The uppermost soil was loose fill or brick debris. Test Holes 4, 5, and 6 were conducted alongside the south footing of the existing garage, through the portion that appears to have experienced previous settlement. Test Hole 4 could not be extended beyond a depth of 5 feet, and encountered only fill that had been placed as backfill behind the east wall of the house's basement. Immediately to the east of this, Test Holes 5 and 6 found fill extending to depths of approximately 2 feet below the existing footing. Dense, native silt was encountered below this fill. Based on our observations, we expect that the western portion of the entire garage was built over fill soils.

Groundwater seepage was only encountered in Test Hole 1. It is relatively common to find shallow subsurface water perched on top of the impervious glacially-compressed silt following extended wet weather.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE

CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The site and surrounding area are underlain at a shallow depth by competent, glacially-compressed native soils. The existing foundations of the main portion of the existing residence appear to have been placed on these dense soils, which are suitable to support conventional footings. Any new foundations that are constructed for the remodel/expansion of the house should be excavated to bear on this dense soil.

The existing foundations for the western portion of the attached garage are underlain by several feet of fill and loose soils. These foundations should not support additional loads, as more settlement will result. We recommend that the affected portions of these footings be underpinned using small-diameter pipe piles before any new loads are added. Two-inch-diameter pipe piles can be driven to refusal alongside the existing footings using hand-held jackhammers. The piles would then be structurally connected to the footings. Any new footings installed in the western approximately 15 feet of the garage should also be supported on pipe piles, in order to avoid the extensive excavation that would be necessary to reach dense soils.

Temporary excavations are not expected to be excessively tall. Sloped cuts can be used where there is sufficient space for a 1:1 (Horizontal:Vertical) cut inclination. Considering the proximity of the existing house to the northern property line, a small pipe pile shoring wall may be necessary for the cuts to construct the deepened foundation for the swim spa. A typical detail for a short temporary pipe pile shoring wall is attached to the end of this report. We have successfully utilized this type of a shoring system for short temporary excavations for other similar projects, where the shoring does not need to support surcharge loads, such as from adjacent foundations.

Providing adequate subsurface drainage around the “pit” for the swim spa will be important, as perched seepage on top of the dense glacial drift should be anticipated. These drains will have to be connected to a suitable outlet, preferably draining by gravity.

Seismic Hazard: The underlying glacially-compressed soils beneath the area of the residence and garage are not susceptible to seismic liquefaction. The foundations for the remodeled house and garage will all bear on these non-liquefiable soils.

Potential Landslide Hazard: The planned addition is not close to any steep or tall slopes. The dense to very dense, glacially-compressed soils that underlie the site are not susceptible to instability, even during a strong earthquake. The stability of the gently- to moderately-inclined ground on, and around, the site will not be adversely affected by the shallow excavations needed for the new development. This sloped area also does not pose a risk of potential slope instability to the planned new construction. No buffer or other mitigation measures are required to address the Potential Landslide Hazard mapping of the site.

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

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

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

SEISMIC CONSIDERATIONS

In accordance with the International Building Code (IBC), the site class within 100 feet of the ground surface is best represented by Site Class Type D (Stiff Soil).

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) during an earthquake be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period). The dense soils beneath the site are not susceptible to seismic liquefaction under the ground motions of the MCE because of the absence of near-surface groundwater.

CONVENTIONAL FOUNDATIONS

An allowable bearing pressure of 2,500 pounds per square foot (psf) is appropriate for new and existing footings supported on dense, native soil. A one-third increase in this design bearing pressure can 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 will be less than one inch, with differential settlements on the order of one-quarter-inch in a distance of 25 feet along a continuous footing with a uniform load.

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

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

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

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

PIPE PILE FOUNDATIONS

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

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

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

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

As a minimum, Schedule 40 pipe should be used for 3-inch piles.

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

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

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

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level compacted fill. We recommend using an ultimate passive earth pressure of 300 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate.

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

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 *	40 pcf (Compacted Free-Draining Backfill)
Passive Earth Pressure	300 pcf
Coefficient of Friction	0.40
Soil Unit Weight	130 pcf (Compacted Free-Draining Backfill)

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

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

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

The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized the wall and reinforcing design for a distance of 1.5 times the wall height from corners or bends in the walls, or from other points of restraint. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

Per IBC Section 1803.5.12, a seismic surcharge load need only be considered in the design of walls with a retention height of 6 feet or more.

For walls backfilled with compacted fill, the recommended seismic surcharge pressure for this project is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

It is important that the backfill consists of coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4

sieve should be between 25 and 70 percent. The on-site soils are not free-draining and will have a low compacted strength. They should not be reused as wall backfill.

A footing drain construction in general accordance with the attached detail should be installed at the base of backfilled 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.

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

LIMITATIONS

This report has been prepared for the exclusive use of Jody and Stephanie Biggs, 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

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

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

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

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

1/4/2024

Marc R. McGinnis, P.E.
Principal

Attachments:

- Vicinity Map
- Site Exploration Plan
- Test Hole Logs
- Footing Drain Detail
- Pipe Pile Shoring Detail

cc: **Sturman Architects** – Brad Sturman
via email: brad@sturmanarchitects.com

MRM:kg

(Source: Microsoft MapPoint, 2013)

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VICINITY MAP

2411 - 60th Avenue S.E.
Mercer Island, Washington

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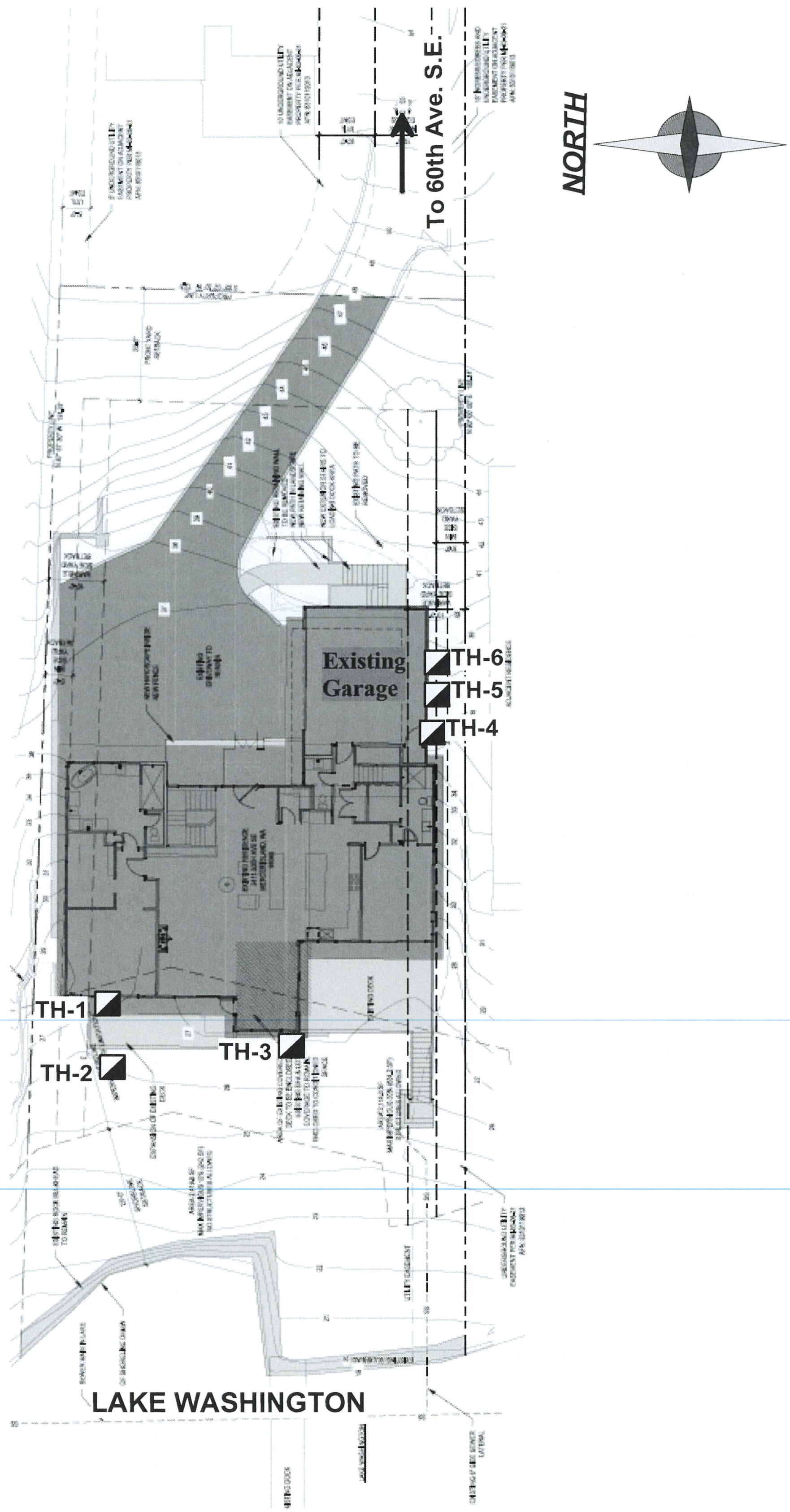

Legend:

Test Hole Location

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SITE EXPLORATION PLAN

2411 - 60th Avenue S.E.
Mercer Island, Washington

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TEST HOLE 1

Depth	Soil Description
Bottom of Footing at 21”	Footing Bears on Gray, very Dense, sandy SILT (Glacial Drift)

Groundwater seepage was encountered perched on Glacial Drift.

TEST HOLE 2

Depth (feet)	Soil Description
0 – 12”	Brown, silty SAND, fine-grained, very moist, loose (FILL)
12” - 16”	Layer of Brick (Old Patio?)
16” – 20”	Gray, slightly sandy SILT, non-plastic, very moist, dense (Glacial Drift)

No groundwater seepage was observed.

TEST HOLE 3

Depth (feet)	Soil Description
0 – 36”	Brown to gray, sandy SILT, fine-grained, very moist, loose (FILL)
36” – 42”	Gray, slightly sandy SILT, non-plastic, very moist, dense (Glacial Drift)

No groundwater seepage was observed.

TEST HOLE 4

Depth	Soil Description
Bottom of Footing at 12”	Brown, loose, sandy SILT (FILL) to >60” below footing

Groundwater seepage was encountered perched on Glacial Drift.

TEST HOLE 5

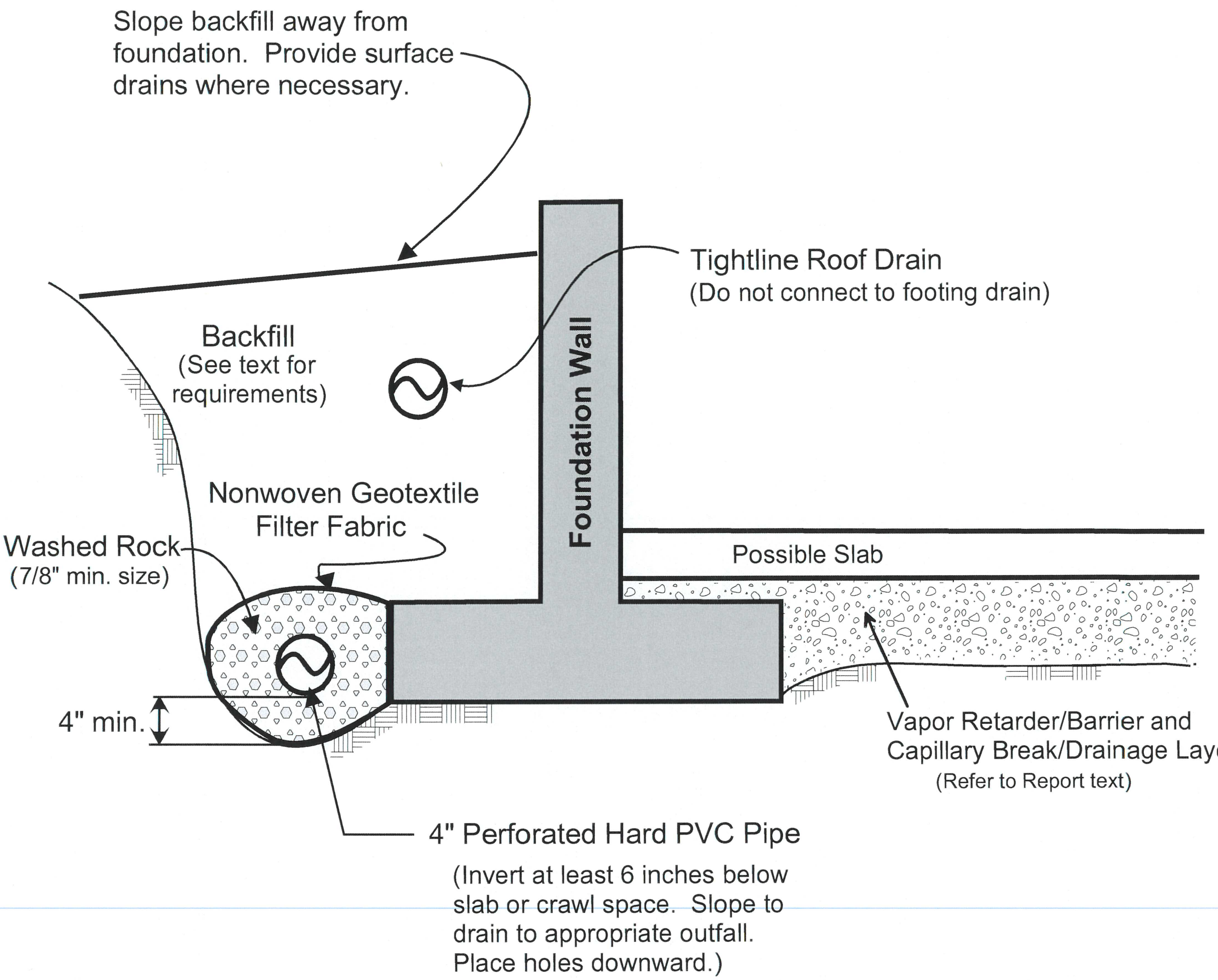
Depth	Soil Description
Bottom of Footing at 12”	Brown, loose, sandy SILT (FILL) to 24” below footing, dense Glacial Drift at 24” below footing

No groundwater seepage was encountered.

TEST HOLE 6

Depth	Soil Description
Bottom of Footing at 12”	Brown, loose, sandy SILT (FILL) to 24” below footing, dense Glacial Drift at 24” below footing

No groundwater seepage was encountered.



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

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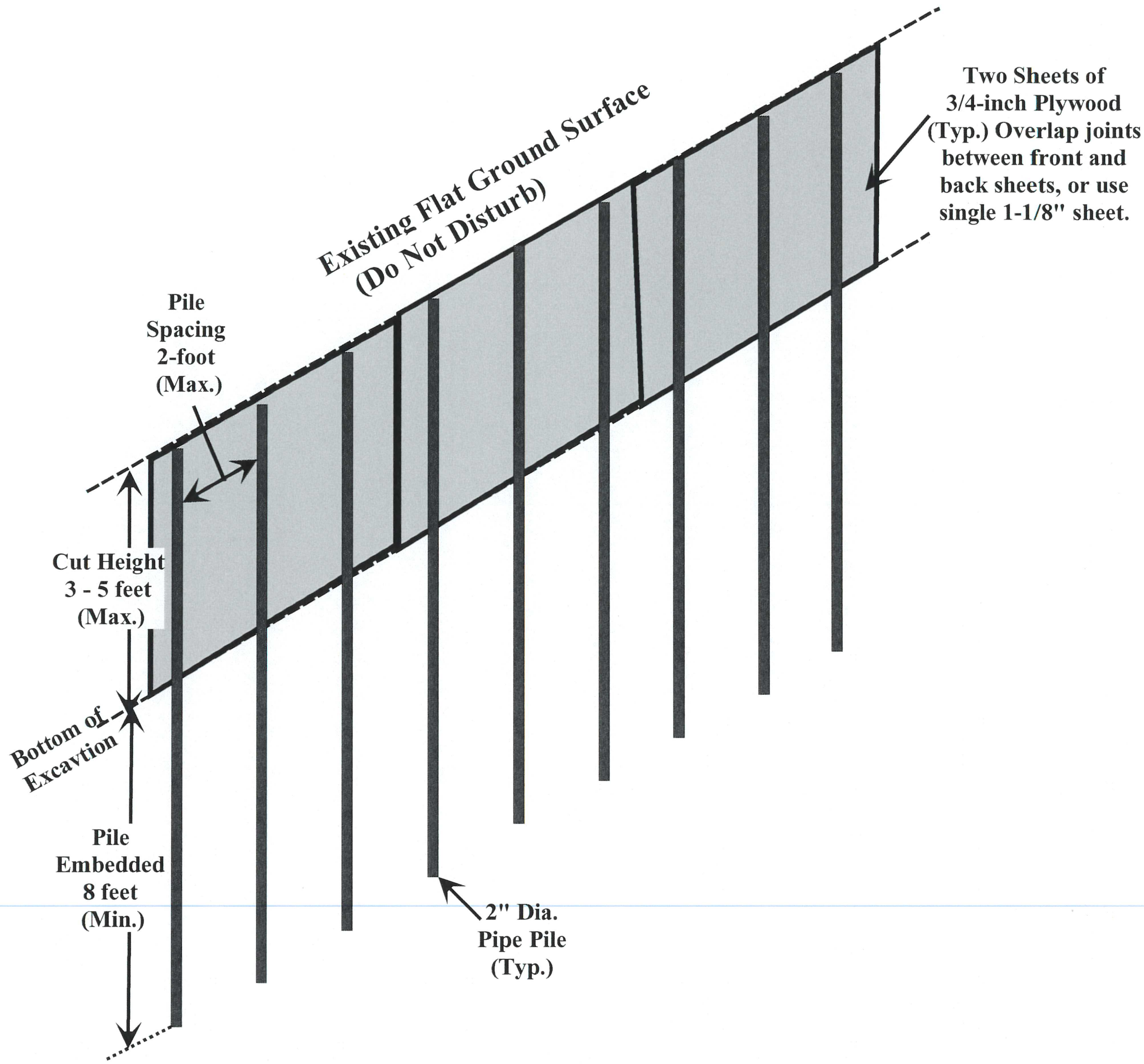

General Notes

1) Pipe piles shall be 2-inch inside diameter Schedule 80 steel pile. Non-galvanized pipe is acceptable.
2) Piles shall be driven to at least 8 feet below the planned excavation level prior to starting the excavation. The pile contractor is responsible for using a large enough hammer to install the piles to the design embedment depth.
3) The ground surface behind the pipe pile shoring wall shall remain undisturbed.
4) After pile installation, two sheets of 3/4-inch plywood or a single 1-1/8 inch sheet shall be slid behind the piles as the excavation proceeds downward. Use wood shims as necessary to brace plywood to backsides of the piles.

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PIPE PILE SHORING

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