GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists
& Environmental Specialists

August 29, 2022 G-5742

Mr. Charlie Chen
CC Design Homes
cchen@ccdesignhomes.com

Subject: **GEOTECHNICAL ENGINEERING REPORT**
PROPOSED SINGLE FAMILY RESIDENCE
3705 – 77TH PL SE
MERCER ISLAND, WASHINGTON

Dear Mr. Chen:

In accordance with our July 21, 2022 contract with you we have prepared the following geotechnical report for the proposed development at the subject site.

SITE AND PROJECT DESCRIPTION

The project site consists of a developed residential lot at the subject address, as shown on the attached **Plate 1 - Vicinity Map**.

The subject site is currently developed with a one-story home with daylight basement/attached garage as shown on the attached **Plate 2 – Topographic Survey**. Steep slopes are not located at the subject site.

Based upon the project plans by CC Design Homes as shown on the attached **Plate 3 – Site Plan, Plate 4 – Lower Floor Plan, Plate 5 – Main Floor Plan, Plate 6 – Upper Floor Plan and Plate 7 – Elevations** we understand that the proposed development consists of constructing a new wood-framed 2-story building with partial basement, having the lower garage bay daylighting at plan east. A garage bay will be located at both the lower level and main level. The plan indicates that the main floor elevation will be at 127.5-feet with the lower garage floor elevation at 118.4-feet. A crawlspace is proposed at a portion of the lower level as shown on the plans.

MERCER ISLAND GEOLOGICAL HAZARDOUS AREAS - CRITICAL AREA MAPPING

GEO Group Northwest has reviewed the City of Mercer Island GIS with regard to potential mapped Geological Hazardous Areas. The GIS mapping indicates that there are potential Landslide Hazard and Seismic Hazard risks with regard to site development. Geotechnical reports are required to evaluate such risks and recommend appropriate mitigation.

This report includes evaluations regarding these potential risks in the report sections: Landslide Hazard Evaluation and Seismic Evaluation and Seismic Design Criteria.

SUBSURFACE CONDITIONS

GEOLOGIC MAPPING

The geologic map[1] for the site indicates that the subject lot is underlain by Recessional Lacustrine deposits (Qvrl). The Recessional Lacustrine deposits consist of laminated silt and clay soils deposited in a glacial recessional environment. These soils may include local sand layers, peat and other organic sediments. Stratigraphically the Recessional Lacustrine deposit may be underlain by fine-grained Glacial deposits (Qpogf). These hard fine-grained silt and clay soils were consolidated by glacial ice.

SUBSURFACE INVESTIGATION

GEO Group Northwest investigated the subsurface soil and groundwater conditions by drilling one exploratory boring labeled B-1. The boring B-1 was located at the front yard for the existing residence as shown on the attached **Plate 2 – Topographic Survey** and **Plate 3 – Site Plan.**

The boring was drilled by a limited access tracked drill rig with hollow stem auger and sampled via the Standard Penetration Test (SPT) method. Soil samples were collected at regular intervals and the observed soils were logged by an engineer from our office.

At the boring B-1 stiff to medium stiff sandy SILT and SILT overlies stiff to hard SILT and sandy SILT at a depth of around 7.5-feet below ground surface (bgs). We interpret the overlying medium stiff to stiff soils as potentially being either fills related to the current development or the recessional lacustrine soil deposit (Qvrl). The underlying stiff to hard silty soils appear to match the description for the anticipated glacially consolidated fine-grained soil unit (Qpogf).

[1] "Geologic Map of Mercer Island, Washington", Mercer Island, ESS, UW, GeoMapNW, Troost et al, 2006.

Based upon observation of the drilling rods and soil sample moisture conditions during the drilling it appears that perched groundwater seepage levels at the time of our investigation at B-1 were at depths of 6, 9 and 12.5-feet below ground surface. It is important to note that groundwater seepage levels may change dependent upon precipitation and other conditions.

The results of our drilling subsurface investigation are schematically illustrated in the attached **Appendix A – Boring Log and USCS Soil Legend**.

LABORATORY TESTS

Moisture contents were measured for the samples collected at the borings via the standard oven drying method (ASTM-D2216-98). Note that moisture contents do not confirm the presence of significant organic soils or peat at the sampled depths.

MERCER ISLAND GEOLOGICALLY HAZARDOUS AREAS - EVALUATIONS

LANDSLIDE HAZARD EVALUATION

First, we note that the City of Mercer Island GIS indicates that no reported landslides have occurred within around 600 feet of the subject site. Also, scarps are not mapped for the subject site and groundwater seeps are not reported within a couple of residential blocks.

The subject site is relatively flat and not located adjacent to any steep slope areas. Per the topographic survey site slopes are around 10-percent or less and there are three rockeries having heights of less than 2-feet.

In summary, the subject site soils consist of medium stiff to hard silt soils and slope inclinations at the site and adjacent properties are relatively low. The risk of ground movement which may be termed a landslide or erosion event is not significant. We have provided appropriate recommendations for mitigation of ground movement risks relating to erosion/landsliding in the following report. It is our opinion that such risks can be mitigated if such recommendations are properly implemented. We include Mercer Island Geologic Hazard Statement of Risk in the Conclusions and Recommendations section of this report.

SEISMIC HAZARD EVALUATION & SEISMIC DESIGN CRITIERA

Based upon our review of City of Mercer Island geologic hazard mapping as discussed above, we understand that the subject site may be mapped as located in a potential Seismic Hazard area.

The Pacific Northwest is a geologically active region which has experienced earthquakes. Therefore, there are risks related to earthquakes and the recommendations in this report take into account the risks to the proposed development as a result of the anticipated design earthquake events.

The primary seismic risk to site development is related to ground shaking and earthquake induced soil settlements as a result of subduction zone intraslab related earthquakes, such as the most recent "Nisqually" earthquake of 2001. There have been nine significant earthquakes which have impacted the Seattle vicinity since the beginning of the historical record roughly 169 years ago. Reported seismic magnitudes for these earthquakes range from 5.5 to 7.3.

Based upon review of the WA DNR Geologic Portal the subject site is located around 70 to 150-feet from two fault traces at the south side of the Seattle Fault Zone. The subject site is located within the general area of the east-west running Seattle Fault Zone. There have not been any reported (historical) surface ruptures at the Seattle Fault Zone or other local shallow faults. Nor has there been a documented instance of shallow fault activation during larger subduction earthquakes.

In order to mitigate the risks related to soil settlement at the building we recommend:

1. Over-excavation of unsuitable soft to medium stiff soils and construction of shallow foundation type on top of the underlying stiff to hard site soils or structural fills placed on top of those competent soils;

2. Or, the building may be supported on a pile foundation which is installed into the underlying very stiff to hard site soils.

Please see the Conclusions and Recommendations section of the report for more information regarding building foundation support.

Based upon the subsurface investigation it is our opinion that the overlying 100-foot thickness of soils at the project site may be characterized as Site Class D soil (Stiff Soil) and may be designed accordingly for seismic loads per IBC and ASCE. According to an online Seismic Hazard tool the seismic coefficients are as follows:

$S_s = 1.412$ $S_1 = 0.491$

And the site modified peak ground acceleration during the design earthquake is $PGA_M = 0.665g$.

GEO Group Northwest has evaluated the proposed building development with regard to liquefaction and lateral spreading risks. Liquefaction is a phenomenon where earth shaking, such as that which occurs during earthquakes, causes an increase in pore water pressure within the soil matrix. For some soil conditions this increase in pore water pressure causes the soil to lose strength and move or settle. Structures which derive their support from the liquefying soil zone may then be damaged. Lateral spreading is generally a condition where the soil which is liquefied is able to move laterally (horizontally) due to a ground surface sloping condition or what may be termed a "free-face", located in proximity to the subject site. Typically, liquefaction induced settlements and related lateral spreading events occur where loose clean sands are located below the water table. Our subsurface investigation indicates that the underlying soils consist of primarily stiff to hard cohesive SILT and sandy SILT. These soils are not susceptible to liquefaction for anticipated seismic events. Perched groundwater seepage conditions were encountered presumably at the sandiest zones within the cohesive soil unit. This water zone is likely limited in its horizontal and vertical extent within the soil matrix and may only be present during periods of the year. It is our opinion that the observed soil and groundwater conditions do not present a significant liquefaction risk to the property. Additionally, our recommendations to over-excavate and remove medium stiff site soils or construct the building on a pile foundation effectively mitigates potential liquefaction related risk to the building if an actual groundwater level were to develop within the overlying site soils. Similarly, it is our opinion that there is not a significant lateral spreading risk to the proposed development.

We include a Mercer Island Geologic Hazard Statement of Risk in the Conclusions and Recommendations section of this report.

CONCLUSIONS AND RECOMMENDATIONS

General

Based upon the results of our study, it is our professional opinion that the site is geotechnically suitable for the proposed development. However, some medium stiff, settlement prone soils were encountered at depths of less than 7.5-feet below ground surface. These soils have relatively low strength and present a risk with regard to soil settlement. It is possible that some of the observed overlying medium stiff soils may be related to backfill for the existing construction, in which case the following option #1 may be the preferred option. In order to mitigate the risk of settlement due to such overlying soils we recommend that one of the following design options is incorporated into the project planning:

1. Over-Excavation/Shallow Spread Footing Option. If shallow spread footing foundations are to be constructed than we recommend over-excavating to remove unsuitable medium stiff site soils at foundation subgrade areas and replace with compacted structural fills or construct the foundation directly on top of the underlying stiff to hard site soils. In this case, a significant amount of over-excavation may be required, dependent upon how much of the building footprint area contains the overlying medium stiff or softer soils. On the other hand, if the medium stiff soils were encountered due to backfill for the existing building, then perhaps, over-excavation may only be necessary at a few select locations. In either case, we recommend that GEO Group Northwest be retained to verify appropriate over-excavation has been completed and that all foundation subgrades consist of the underlying stiff to hard site soils at the time of construction.

2. Pile Foundation Option. An alternative option is to support the building on a pile foundation which is embedded into the underlying very stiff to hard site soils. We anticipate that such a foundation may consist of small diameter driven steel pipe (pin) piles which are driven to meet the refusal criteria and embedded within the very stiff to hard site soils.

Details regarding site development recommendations including foundation design parameters follow in the report sections.

Site Preparation and General Earthwork

The proposed development areas should be stripped and cleared of surface vegetation, organic soils (topsoil), loose soils and fill debris.

Silt fences should be installed around areas disturbed by construction activity to prevent sediment-laden surface runoff from being discharged off-site. Exposed soils that are subject to erosion should be compacted and covered with plastic sheeting. Stockpiled soils should be covered with plastic sheeting if work is done during wet weather in order to mitigate off-site sedimentation risks.

If perched groundwater seepage conditions are encountered at the building pad excavation areas or if the building pad development occurs during wet weather than we recommend placing clean crushed rock or quarry spalls in order to protect subgrades from water and wet weather impacts. During dry weather periods and when seepage conditions are not encountered compacted structural fills may be placed to fill over-excavation areas to the proposed foundation or slab subgrade level. The use of alternatives such as lean mix concrete or CDF (controlled density fills) may also be considered.

Temporary Excavation Slopes and Permanent Slopes

Under no circumstances should temporary excavation slopes be greater than the limits specified in local, state and national government safety regulations. Temporary cuts greater than four feet in height should be sloped at an inclination no steeper than 1H:1V (Horizontal:Vertical) in the overlying medium stiff to stiff soils. Excavations shall not fall below a 1H:1V imaginary plane projected downward from the base of any adjacent existing structures/buildings or property lines without review and approval by the geotechnical engineer. If seepage is encountered at the excavation, then the excavation work should be halted and the temporary excavation slopes should be evaluated by GEO Group Northwest prior to proceeding to a deeper level.

We recommend that permanent graded slopes shall be sloped no steeper than 3H:1V and that any fills placed at these areas where inclinations are steeper than 4H:1V are compacted to meet the structural fill compaction requirements. Where fills must be placed at any sloping inclination of steeper than 5H:1V we recommend that benches with typical depths (heights) of 3 or 4-feet are excavated into the existing grades such that fills may be placed in level lifts at each excavated bench, thereby creating a well-interlocked fill zone.

Structural Fill

All fill material used to achieve design site elevations below the building areas and below non-structurally supported slabs, parking lots, sidewalks, driveways, and patios, should meet the requirements for structural fill. During wet weather conditions, material to be used as structural fill should have the following specifications:

1. Be free draining, granular material containing no more than five (5) percent fines (silt and clay-size particles passing the No. 200 mesh sieve);

2. Be free of organic material and other deleterious substances, such as construction debris and garbage;

3. Have a maximum size of three (3) inches in diameter.

All fill material should be placed at or near the optimum moisture content. The optimum moisture content is the water content in soil that enables the soil to be compacted to the highest dry density for a given compaction effort.

Based upon our subsurface investigation, the overlying site soils consist of silty site soils. These soils are not recommended for use as structural fill due to their moisture sensitivity and the anticipated difficulty related to compaction of fine-grained soils. We recommend importing a granular fill material meeting the specifications noted above for use as structural fill.

Structural fill should be placed in thin horizontal lifts not exceeding ten inches in loose thickness. Structural fill under building areas (including foundation and slab areas) should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM Test Designation D-1557-91 (Modified Proctor).

Structural fill under driveways, parking lots and sidewalks should be compacted to at least 90 percent maximum dry density, as determined by ASTM Test Designation D-1557-91 (Modified Proctor). Fill placed within 12-inches of finish grade should meet the 92% requirement.

We recommend that GEO Group Northwest, Inc., be retained to evaluate the suitability of structural fill material and to monitor the compaction work during construction for quality assurance of the earthwork.

Building Pad Preparation

Over-excavation / Shallow Spread Foundation Option

As previously noted, an overlying thickness of stiff to medium stiff soils was observed at the boring location. The medium stiff soils if located below foundations present settlement related risks to shallow foundations, if not properly mitigated. Therefore, one option is to over-excavate, remove the medium stiff soils, as necessary at all foundation and slab subgrade areas. Once the excavation has been sufficiently deepened to expose the underlying competent stiff to hard soils than the footings may either be constructed at the over-excavated subgrade elevation or compacted structural fills may be placed up to the footing subgrade elevation(s). Based upon soils observed at the boring B-1 the competent underlying stiff soils were observed at and below depths of 7.5-feet below ground surface (bgs).

We recommend that GEO Group Northwest be retained at the time of construction in order to monitor the over-excavation at foundation and slab subgrade areas. This is to verify that the overlying medium stiff soils have been removed and that the native subgrade bearing surfaces consist of competent stiff to hard and non-yielding site soils.

Following approval of the over-excavated foundation subgrades it is recommended that structural fills are placed and compacted in level, maximum 10-inch loose lift thicknesses, up to the proposed base of footing elevation. Structural fills should be compacted to meet the minimum specifications noted in the section: Structural Fill. Alternatively lean mix concrete, clean crushed rock or CDF (controlled density fills) may be placed to fill over-excavation trenches. Conventional spread footing foundations may be constructed to bear on top of the approved structural fills placed on top of the approved native subgrades.

Pile Foundation – Building Pad Preparation

If the building is to be constructed on top of a pile foundation system than over-excavation or foundation/crawlspace deepening is not necessary. It may be beneficial to place quarry spalls or crushed rock material if work is to occur during wet weather in order to create working pads for the piling machinery.

Building Foundations

Spread Footing Foundations

Conventional spread footings may be constructed to bear on top the approved underlying stiff site soils or compacted and approved structural fill placed on top of the approved subgrades.

Individual spread footings may be used for supporting columns and strip footings for bearing walls. Our recommended minimum design criteria for foundations bearing on the approved stiff to hard site soils or compacted structural fill placed on top of these soils is as follows:

- Allowable bearing pressure, including all dead and live loads
  Competent underlying stiff site soils = 2,000 psf

  Compacted structural fill on top of the
  Competent underlying stiff site soils = 2,000 psf

- Minimum depth to bottom of perimeter footing below adjacent final exterior grade = 18 inches

- Minimum depth to bottom of interior footings below top of floor slab = 18 inches

- Minimum width of wall footings = 16 inches

- Minimum lateral dimension of column footings = 24 inches

A one-third increase in the above allowable bearing pressures can be used when considering short-term transitory wind or seismic loads.

For this option the total anticipated post-construction settlement is anticipated to be up to ½-inch with potentially ¼-inch differential settlement. The pile support option is recommended if these settlement amounts are unacceptable.

Lateral loads can also be resisted by friction between the foundation and the supporting compacted fill subgrade or by passive earth pressure acting on the buried portions of the foundations. For the latter, the foundations must be poured "neat" against the existing undisturbed soil or be backfilled with a compacted fill meeting the requirements for structural fill. Our recommended parameters are as follows:

- Passive Pressure (Lateral Resistance)
  - 350 pcf equivalent fluid weight for compacted structural fill
  - 300 pcf equivalent fluid weight for native stiff soil.

- Coefficient of Friction (Friction Factor)
  - 0.35 for compacted structural fill
  - 0.30 for native stiff soil

Pile Foundation – Driven Pin Piles

An alternative to the previously noted over-excavation/spread footing option is to support the building on top of a pile system which is embedded into the underlying very stiff to hard site soils. If the building is to be supported on top of a pile foundation, then we recommend that the building loads are transferred to the piling via a system of concrete grade beams which span between piles. Similarly, structural slabs are recommended to transfer loads to piling where it is not feasible, practical, or preferred to for slabs to be supported by the ground.

The proposed building may be supported on top of driven pipe piles embedded within the underlying very stiff to hard site soils. Pipe (pin) piles should be driven vertically into the underlying competent dense to very dense soils until the refusal criteria is reached. Refusal is reached when the rate of penetration for a pile, in seconds per inch, reaches or exceeds the refusal criteria (noted below). Pile sections should be joined with couplers, or welded together as the pile is advanced. Welding of pile sections and pile caps should be required for piles designed with a component of uplift for seismic resistance. Concrete grade beams and pile caps should be used to transfer building loads to the piles. The following are our recommendations for available pile hammers, pile sizes, recommended refusal criteria and allowable axial bearing capacities to be used by the designer/structural engineer in designing the pile plan:

Pipe Pile Diameter	Pile Specifications	Hammer Size	Hammer Type	Refusal Criteria (Seconds Per Inch)	Allowable Capacity
3-inch	Schedule 40	600 lb	TB-225 (hydraulic)	12	6 tons
4-inch	Schedule 40	850 lb	TB-325 (hydraulic)	16	10 tons

We recommend a load test (ASTM Quick Test - minimum requirement) be performed on at least 3% of the 3-inch or 4-inch piles (5 piles maximum and 1 pile minimum). We recommend that we are retained to be on-site to verify the proper installation of pipe piles including monitoring pile depths, refusal verification and pile load testing.

Provided the pipe piles are driven to the recommended refusal criteria, the estimated total post-construction settlement should be 1/4-inch or less, and the differential settlement across a pile supported structure should be 1/4-inch or less.

If the boring B-1 is typical for the soils at the site than we anticipate that potentially the driven piling may have depths ranging from 17 to 25-feet below existing grades. Typically, SPT N blow count material of 50 or higher or significant side friction within dense/hard soils is necessary to slow the piling to meet the refusal standard. We recommend driving test piles if it is necessary to accurately estimate pile driving costs prior to the installation. Typically, a minimum pile length is not recommended since the depth to the competent soils may vary, however, we recommend that the geotechnical engineer investigate and then determine approval for any piles which meet the refusal criteria but have a depth of less than 3-feet. These conditions may indicate that an obstruction is present which should be removed prior to pile driving.

By themselves vertical pipe piles do not generate lateral capacities. Lateral capacities can be developed by driving battered piles or by friction between the foundation and the supporting compacted fill or native subgrade or by passive earth pressure acting on the buried portions of the foundations. For the latter, the foundations must be backfilled with a compacted fill meeting the requirements for structural fill. Our recommended parameters for passive pressure and the coefficient of friction are the same as noted in the section: Spread Footing Foundations.

We recommend that footing drains be placed around all perimeter footings and concrete grade beams. The footing drains should be configured as described in the **Drainage Considerations** section of this report.

Conventional Retaining Walls and Basement Walls

Based upon the preliminary plans we assume that conventional concrete basement retaining walls may be constructed at plan south side of the proposed basement. These walls may be constructed on top of spread footing foundations or a pile foundation, provided that the building pad areas are prepared as noted in this report. Typically, we recommend that the designer specify two rows of piling at basement concrete grade beams; one pile line at the toe and one at the heel.

Permanent retaining walls restrained horizontally on top (such as basement walls) are considered unyielding and should be designed for a lateral soil pressure under the at-rest condition; while conventional reinforced concrete walls free to rotate on top should be designed for an active lateral soil pressure.

Active Earth Pressure
Conventional reinforced concrete walls that are designed to yield an amount equal to 0.002 times the wall height, should be designed to resist the lateral earth pressure imposed by an equivalent fluid with a unit weight of 35 pcf for level backfill.

At-Rest Earth Pressure
Walls supported horizontally by floor slabs are considered unyielding and should be designed for lateral soil pressure under the at-rest condition. The design lateral soil pressure should have an equivalent fluid pressure of 45 pcf for level backfill.

Seismic Surcharge
For the anticipated design seismic event a horizontal surcharge load of 10H psf should be applied;

Passive Earth Pressure
350 pcf equivalent fluid weight for compacted structural fill
300 pcf equivalent fluid weight for native stiff site soil;

Base Coefficient of Friction
0.35 for compacted structural fill
0.30 for native stiff site soil;

To prevent the buildup of hydrostatic pressure behind permanent concrete basement or conventional retaining walls, we recommend that a vertical drain mat, such as Miradrain 6000 or equivalent, be used to facilitate drainage behind such walls. The drain mat core should be placed against the wall(s) with the filter fabric side facing the backfill. The drain mat should extend from near the finished surface grade down to the footing drain system. Additionally, all backfill placed between the excavation slopes and the new basement/retaining walls should consist of free-draining fills having less than 5% passing the No. 200 sieve. We recommend that a waterproofing layer is installed between the drainage mat layer and the concrete wall, for moisture protection at all basement wall locations.

The top 12 inches of backfill behind retaining or basement walls should consist of compacted and relatively impermeable soil. This cap material can be separated from the underlying more granular drainage material by a geotextile fabric, if desired. Alternatively, the surface can be sealed with asphalt or concrete paving. Where possible the ground surface should be sloped to drain away from the wall.

GEO Group Northwest, Inc., recommends that backfill material which will support structures or improvements (such as patios, sidewalks, driveways, etc.) behind permanent concrete retaining walls and basement walls be placed and compacted consistent with the structural fill specifications in the **Structural Fill** section of this report.

Concrete Slab Floors

We recommend that the concrete slab subgrades consist of stiff to hard site soils or compacted structural fills placed on top of these soils. If soft or loose soils are encountered at slab subgrade areas these soils should be over-excavated until reaching the competent stiff soil layers and then filled with compacted structural fills. We recommend that slab-on-grade floors are reinforced with minimum #4 bar at 12-inches on center, placed in a grid (perpendicular). Alternatively, structural slabs may be supported on driven piling which is driven into the underlying very stiff to hard site soils.

To avoid moisture build-up below concrete floors a capillary break should be placed over the prepared subgrade. The capillary break should consist of a minimum of a six (6) inch thick layer of free-draining crushed rock or gravel containing no more than five (5) percent finer than the No. 4 sieve.

To reduce moisture vapor transmission through the slab we recommend installing a minimum 10-mil thick vapor retarder, such as Moistop Ultra® 10, by Fortifiber Building Systems Group®, between the capillary break and concrete floor slab. Moistop Ultra 10 is a polyolefin film with a water vapor permeance of 0.02 perms. It is puncture and tear resistant, meets ASTM E-1745 Class A, B and C requirements for underslab vapor retarders and is suitable for residential and commercial applications. Boots are available for sealing around pipes, conduit and other penetrations. We recommend it be installed in accordance with the manufacturers' recommendations.

Drainage Considerations

We recommend that subsurface drains (footing drains) be installed around the perimeter of the foundation footings/grade beams and tightlined to an approved discharge point or stormwater system. Footing drains should consist of a four-inch minimum diameter, perforated, rigid PVC drain pipe laid at the bottom of the footing/grade beam with a gradient sufficient to generate flow. The footing drain line should be bedded on and surrounded with drain rock, pea gravel, or other appropriate, free-draining, granular material. The drain rock should be wrapped in a layer of geotextile fabric such as Mirafi 180N or equivalent. After the footing drains are installed, the excavation should be backfilled with compacted structural fill material.

Under no circumstances should roof downspout drain lines be connected to the footing drainage system. All roof downspouts should be separately tight lined to an appropriate storm-water discharge point. We recommend that sufficient cleanouts be installed at strategic locations in each of the drainage systems to allow for periodic maintenance of and clearing of possible future blockages.

MERCER ISLAND GEOLOGIC HAZARD AREA – STATEMENT OF RISK

Per Section 19.07.060.B.3 of the Mercer Island City Code, development within geologic hazard areas require that a Geotechnical Engineer licensed within the State of Washington provide a statement of risk with supporting documentation. We evaluated and discussed the subject site potential geologic hazard areas in the earlier report sections titled: Landslide Hazard Evaluation and Seismic Evaluation and Seismic Design Criteria.

Accordingly, it is our opinion that with regard to the potential landslide hazard area designation the following statement applies:

"b. An evaluation of site specific subsurface conditions demonstrates that the proposed development is not located in a geologic hazard area;"

And also, it is our opinion that with regard to the potential seismic hazard area the following statement applies:

"c. Development practices are proposed for the alteration that would render the development as safe as if it were not located in a geologic hazard area;"

ADDITIONAL SERVICES

We recommend that GEO Group Northwest Inc. be retained to perform a general plan review of the final design and specifications for the proposed development to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design and in the construction documents. We also recommend that GEO Group Northwest Inc. be retained to provide monitoring and testing services for geotechnically-related work during construction. This is to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. We anticipate that geotechnical construction monitoring inspections may be necessary for the following construction tasks:

1. Grading for temporary excavation slopes;

2. Building Pad Improvement: Over-excavation at foundation/slab areas and structural fill placement/compaction;

3. Structural fill placement and compaction for utilities and pavements;

4. Installation of pile foundations (as necessary, based upon the plans);

5. Subsurface drainage installation;

6. Installation and maintenance of erosion mitigation measures.

LIMITATIONS

This report has been prepared for the specific application to this site for the exclusive use of CC Design Homes and their authorized representatives. Any use of this report by other parties is solely at that party's own risk. We recommend that this report be included in its entirety in the project contract documents for reference during construction.

Our findings and recommendations stated herein are based on field observations, our experience and judgement. The recommendations are our professional opinion derived in a manner consistent with the level of care and skill ordinarily exercised by other members of the profession currently practicing under similar conditions in this area and within the budget constraint. No warranty is expressed or implied. In the event that soil conditions not anticipated in this report are encountered during site development, GEO Group Northwest, Inc., should be notified and the above recommendations should be re-evaluated.

If you have any questions, or if we may be of further service, please do not hesitate to contact us.

Sincerely,
GEO GROUP NORTHWEST, INC.

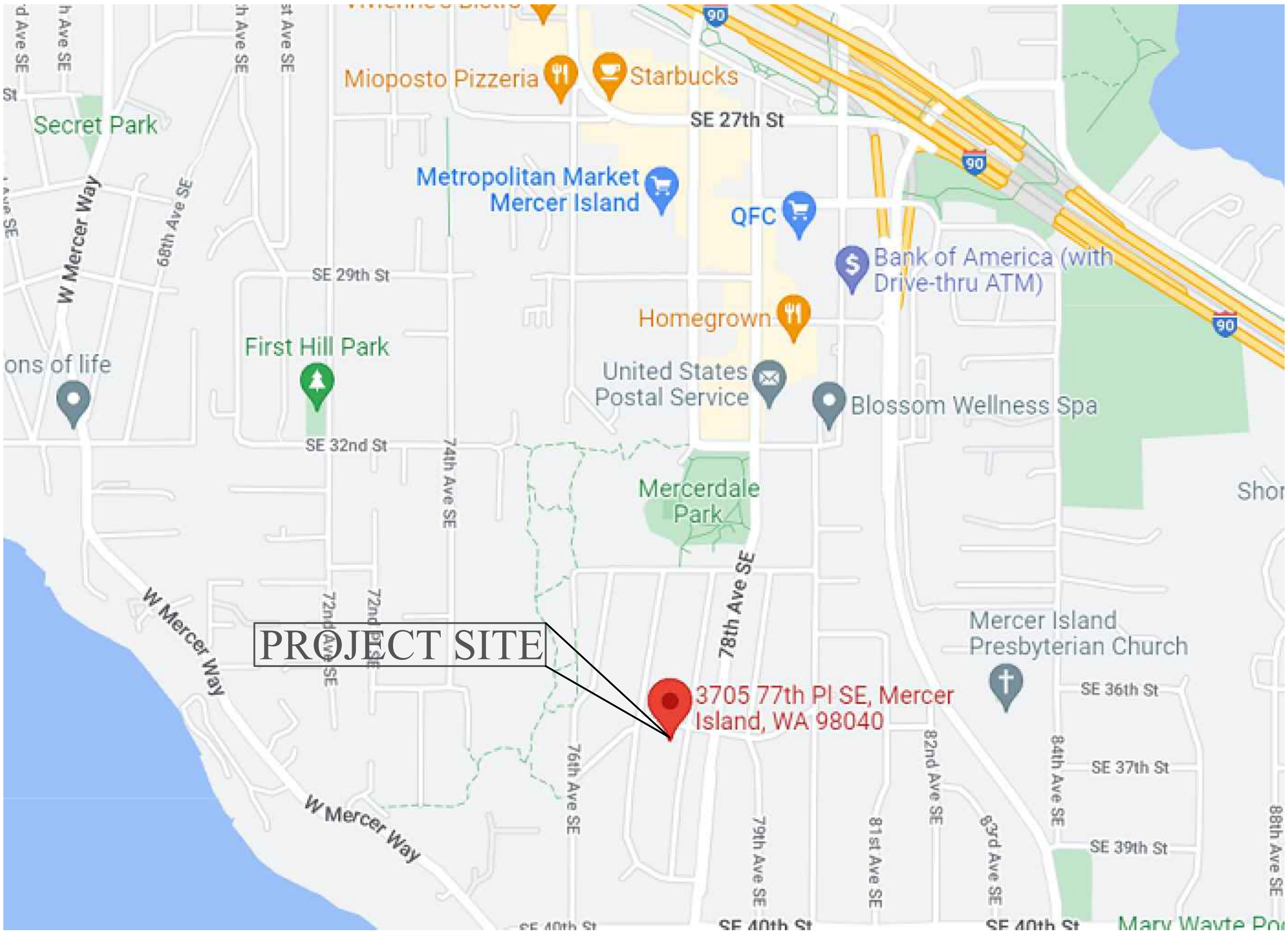
Adam Gaston
Project Engineer

8-29-2022

William Chang, P.E.
Principal

Attachments: Plate 1 – Vicinity Map
Plate 2 – Topographic Survey
Plate 3 – Site Plan
Plate 4 – Lower Floor Plan
Plate 5 – Main Floor Plan
Plate 6 – Upper Floor Plan
Plate 7 – Elevations

Appendix A – Boring Log and USCS Soil Legend



GEO Group Northwest, Inc.

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Phone 425/649-8757 FAX 425/649-8758
Email info@geogroupnw.com

VICINITY MAP
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

SCALE: NTS	DATE: 8-10-22	MADE: AG	JOB NO.: G-5742	PLATE: 1

BASED UPON THE TOPOGRAPHIC AND BOUNDARY SURVEY BY TERRANE, 3-9-21.

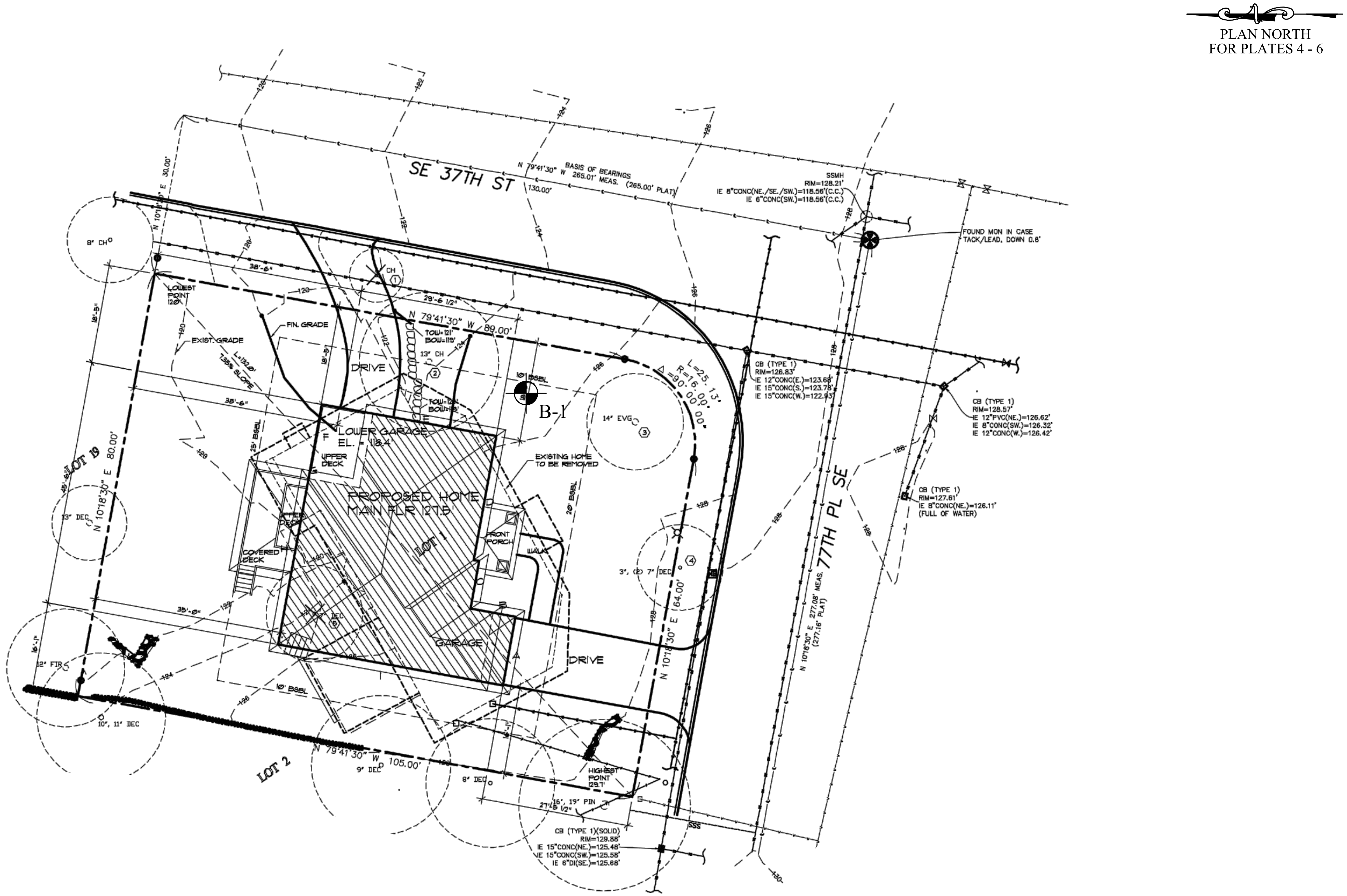
LEGEND

B-1 = BORING NUMBER AND APPROXIMATE LOCATION

GEO Group Northwest, Inc.
13705 Bel-Red Rd, Bellevue, WA 98005
Phone 425/649-8757 FAX 425/649-8758
Email info@geogroupnw.com

TOPOGRAPHIC SURVEY
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

PROJECT #: G-5742	
DATE: 8-10-22	
DRAWN: AG	
CHECKED: WC	
SCALE: ~ 1" = 20'	
PLATE: 2	



LEGEND

= BORING NUMBER AND APPROXIMATE LOCATION

B-1

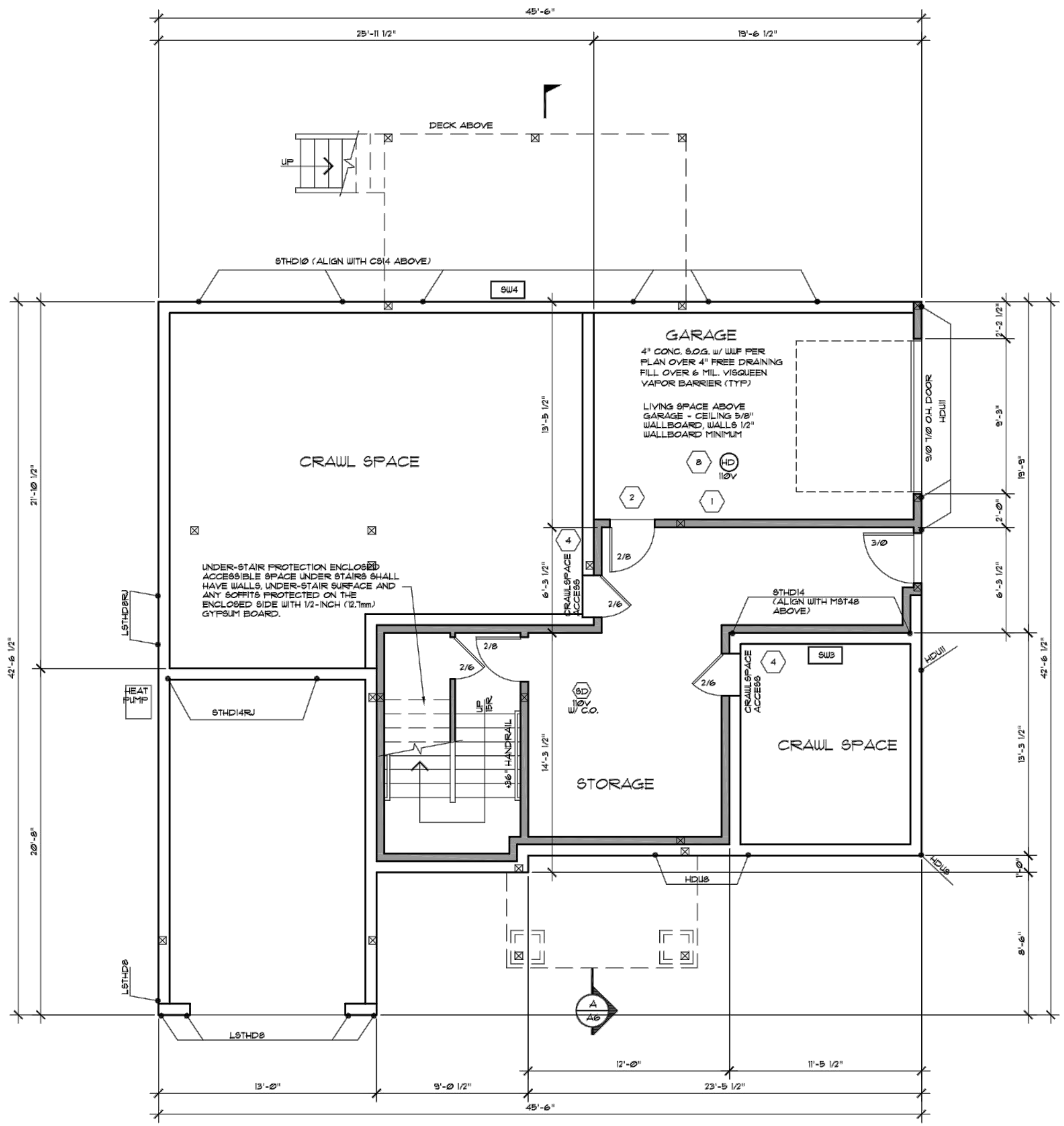
GEO Group Northwest, Inc.

13705 Bel-Red Rd, Bellevue, WA 98005
Phone 425/649-8757 FAX 425/649-8758
Email info@geogroupnw.com

SITE PLAN
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

PROJECT #: G-5742
DATE: 8-10-22
DRAWN: AG
CHECKED: WC
SCALE: ~ 1" = 20'
PLATE: 3

BASED UPON THE UNDATED SITE PLAN, SHEET 1.0, BY CC DESIGN HOMES



PLAN NORTH

BASED UPON SHEET A2, BY CC DESIGN HOMES, 6-13-22.

GEO Group Northwest, Inc.

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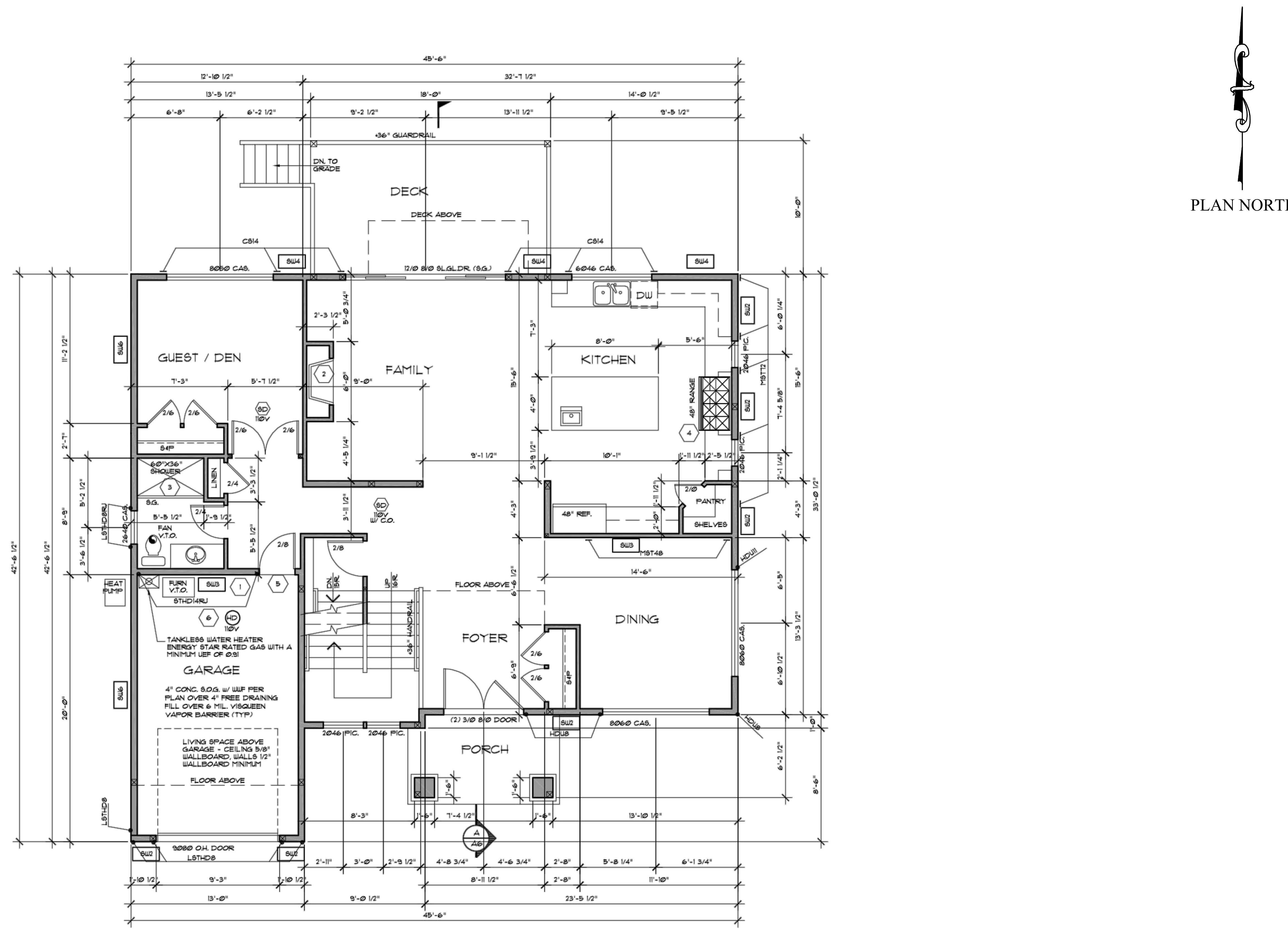
LOWER FLOOR PLAN
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

PROJECT #: G-5742
DATE: 8-10-22
DRAWN: AG
CHECKED: WC
SCALE: ~ 1" = 8'
PLATE: 4



BASED UPON SHEET A3, BY CC DESIGN HOMES, 6-13-22.

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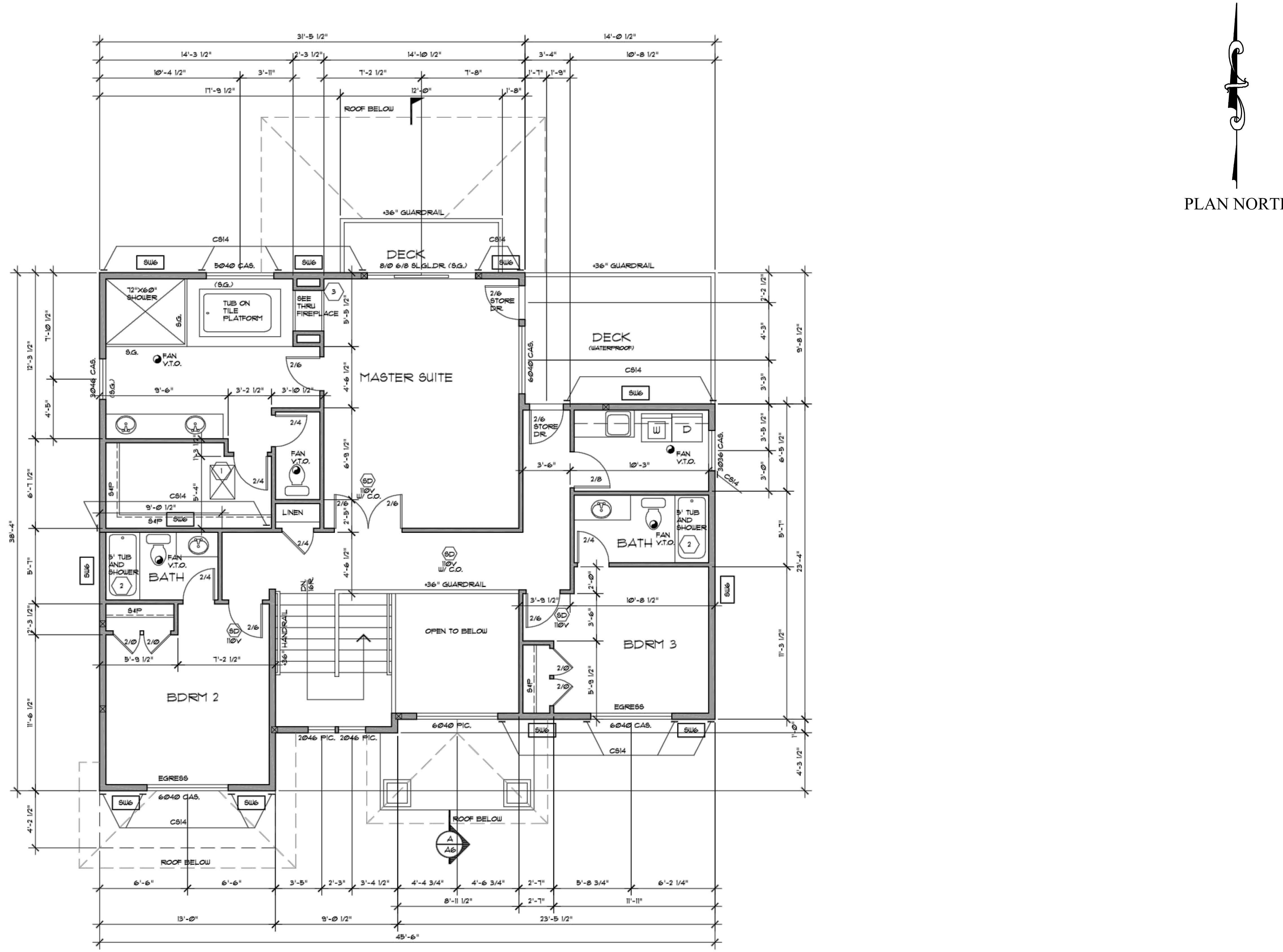
MAIN FLOOR PLAN
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

PROJECT #:	G-5742
DATE:	8-10-22
DRAWN:	AG
CHECKED:	WC
SCALE:	~ 1" = 8'
PLATE:	5



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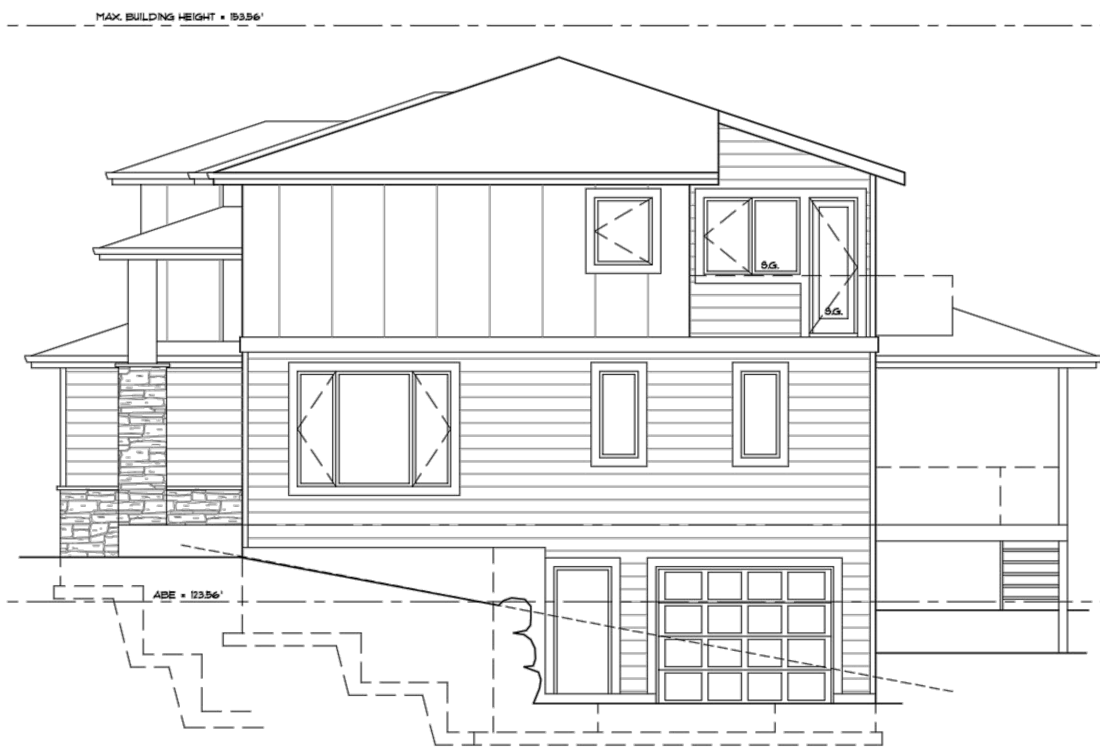
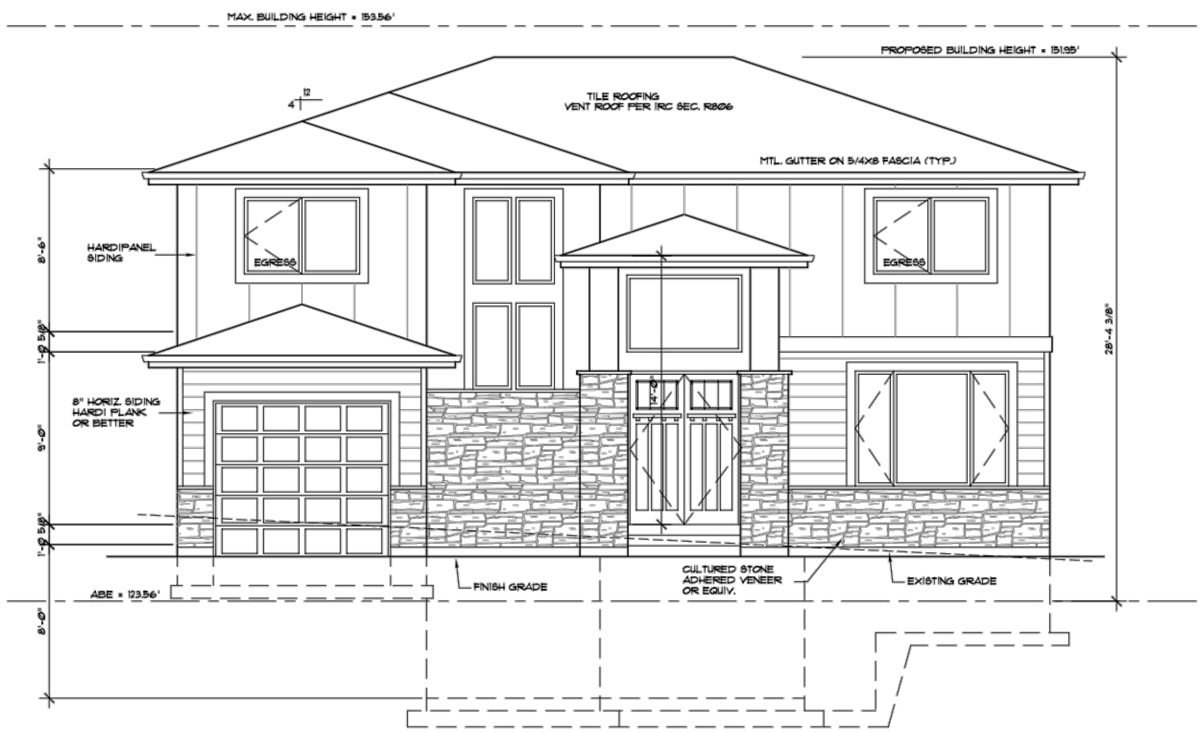
UPPER FLOOR PLAN
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

PROJECT #: G-5742	
DATE: 8-10-22	
DRAWN: AG	
CHECKED: WC	
SCALE: ~ 1" = 8'	
PLATE: 6	

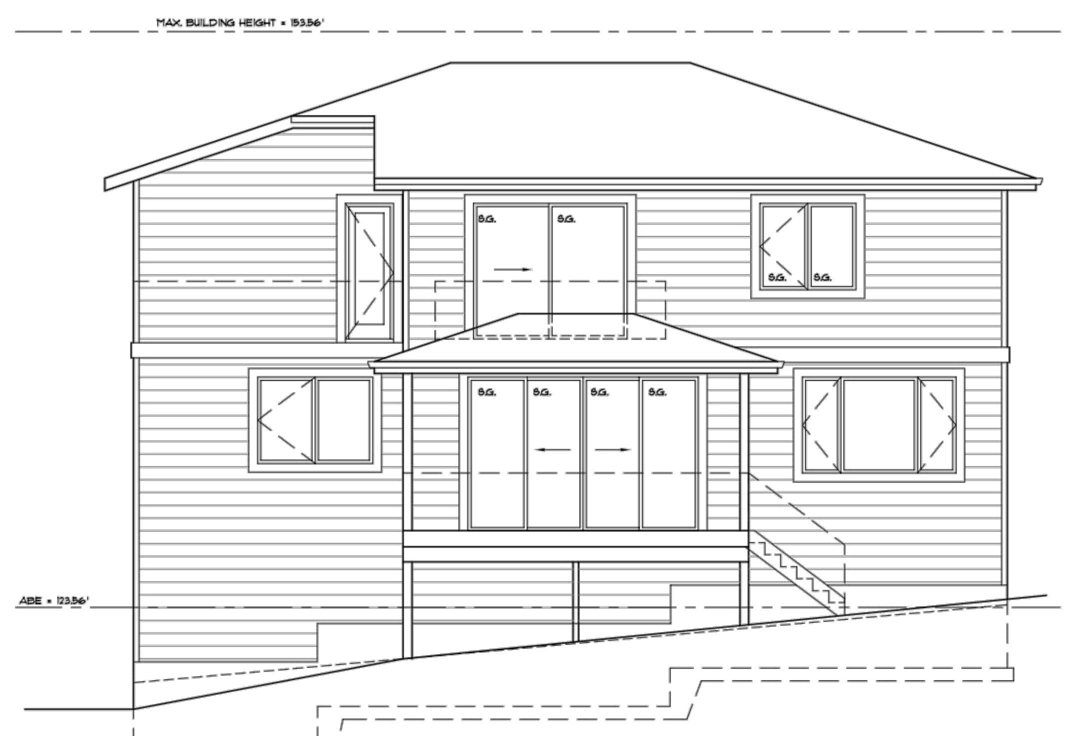
BASED UPON SHEET A4, BY CC DESIGN HOMES, 6-13-22.



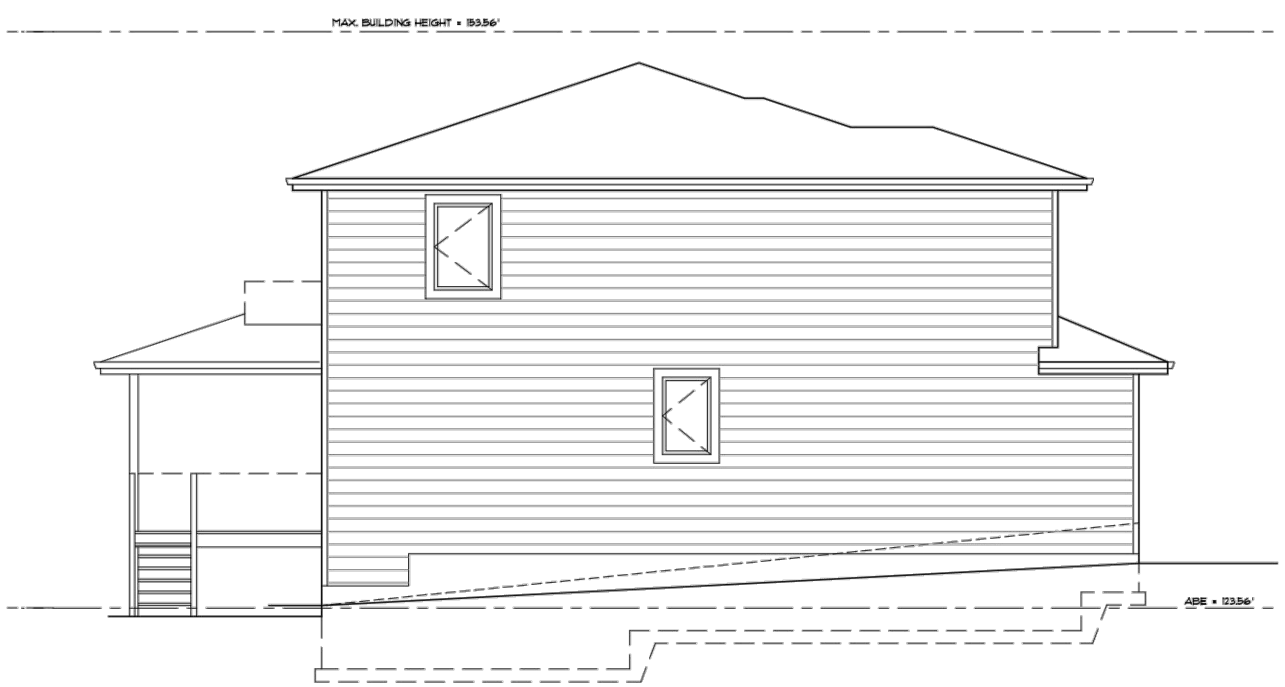
RIGHT ELEVATION



FRONT ELEVATION



REAR ELEVATION



LEFT ELEVATION

BASED UPON SHEET A5, BY CC DESIGN HOMES, 6-13-22.

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ELEVATIONS
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

PROJECT #: G-5742
DATE: 8-10-22
DRAWN: AG
CHECKED: WC
SCALE: ~ 1" = 10'
PLATE: 7

APPENDIX A

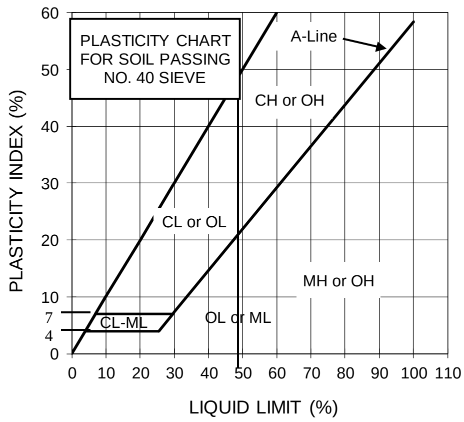
BORING LOG & USCS SOIL LEGEND

G-5742

LEGEND OF SOIL CLASSIFICATION AND PENETRATION TEST

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA		
COARSE-GRAINED SOILS More Than Half by Weight Larger Than No. 200 Sieve	GRAVELS (More Than Half Coarse Grains Larger Than No. 4 Sieve)	CLEAN GRAVELS (little or no fines)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURE, LITTLE OR NO FINES	DETERMINE PERCENTAGES OF GRAVEL AND SAND FROM GRAIN SIZE DISTRIBUTION CURVE	Cu = (D60 / D10) greater than 4 Cc = (D302) / (D10 * D60) between 1 and 3	
			GP	POORLY GRADED GRAVELS, AND GRAVEL-SAND MIXTURES LITTLE OR NO FINES		NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (with some fines)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES		CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE. or P.I. LESS THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	COARSE GRAINED SOILS ARE CLASSIFIED AS FOLLOWS:		ATTERBERG LIMITS ABOVE "A" LINE. or P.I. MORE THAN 7
	SANDS (More Than Half Coarse Grains Smaller Than No. 4 Sieve)	CLEAN SANDS (little or no fines)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LIITLE OR NO FINES		Cu = (D60 / D10) greater than 6 Cc = ($D30^2$) / (D10 * D60) between 1 and 3	
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	< 5% Fine Grained: GW, GP, SW, SP	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (with some fines)	SM	SILTY SANDS, SAND-SILT MIXTURES	> 12% Fine Grained: GM, GC, SM, SC	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE with P.I. LESS THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES	5 to 12% Fine Grained: use dual symbols		ATTERBERG LIMITS ABOVE "A" LINE with P.I. MORE THAN 7
FINE-GRAINED SOILS More Than Half by Weight Smaller Than No. 200 Sieve	SILTS (Below A-Line on Plasticity Chart, Negligible Organic)	Liquid Limit < 50%	ML	INORGANIC SILTS, ROCK FLOUR, SANDY SILTS OF SLIGHT PLASTICITY			
		Liquid Limit > 50%	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOIL			
	CLAYS (Above A-Line on Placticity Chart, Negligible Organic)	Liquid Limit < 30%	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, CLEAN CLAYS			
		Liquid Limit > 50%	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
	ORGANIC SILTS & CLAYS (Below A-Line on Placticity Chart)	Liquid Limit < 50%	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY			
		Liquid Limit > 50%	OH	ORGANIC CLAYS OF HIGH PLASTICITY			
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS			



SOIL PARTICLE SIZE

FRACTION	U.S. STANDARD SIEVE			
	Passing		Retained	
	Sieve	Size (mm)	Sieve	Size (mm)
SILT / CLAY	#200	0.075		
SAND				
FINE	#40	0.425	#200	0.075
MEDIUM	#10	2	#40	0.425
COARSE	#4	4.75	#10	2
GRAVEL				
FINE		19	#4	4.75
COARSE		76		19
COBBLES	76 mm to 203 mm			
BOULDERS	> 203 mm			
ROCK FRAGMENTS	> 76 mm			
ROCK	>0.76 cubic meter in volume			

GENERAL GUIDANCE OF SOIL ENGINEERING PROPERTIES FROM STANDARD PENETRATION TEST (SPT)

SANDY SOILS				SILTY & CLAYEY SOILS		
Blow Counts N	Relative Density %	Friction Angle ϕ, degree	Description	Blow Counts N	Unconfined Strength q_u, tsf	Description
0 - 4	0 -15		Very Loose	< 2	< 0.25	Very soft
4 - 10	15 - 35	26 - 30	Loose	2 - 4	0.25 - 0.50	Soft
10 - 30	35 - 65	28 - 35	Medium Dense	4 - 8	0.50 - 1.00	Medium Stiff
30 - 50	65 - 85	35 - 42	Dense	8 - 15	1.00 - 2.00	Stiff
> 50	85 - 100	38 - 46	Very Dense	15 - 30	2.00 - 4.00	Very Stiff
				> 30	> 4.00	Hard

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PLATE A1

BORING NO. B - 1

Logged By: AG Date Drilled: 8/2/22
Drilled By: Geologic Drill Partners

Depth ft.	Elevation	USCS Code	Description	Sample Loc.	Sample No.	SPT Blow Counts	Water Content %	Other Tests/ Comments
		ML	Tan sandy SILT, dry, stiff			6,6,7 (N=13)	14.8	
		ML	Tan sandy SILT becoming gray with brown mottling SILT, moist, stiff			8,5,6 (N=11)	31.5	
5		ML	Tan and gray SILT and sandy SILT with occasional fibrous wood in soil matrix, moist, wet at sampler tip, medium stiff			4,2,4 (N=6)	32.8	
		ML	Gray with tan mottling SILT, massive, moist to wet, stiff			5,5,8 (N=13)	34.1	
10		ML	Tan SILT, massive, moist, very stiff			6,9,15 (N=24)	30.2	
		ML	Tan sandy SILT, wet, very stiff			9,10,19 (N=29)	28.4	
15		ML	Tan becoming gray SILT, moist, hard			18,21,25 (N=46)	32.1	
20		ML	Gray SILT with trace very fine sand, moist, hard			27,33,44 (N=77)	24.8	
25			Continued on next page (Plate A3)					

LEGEND: 2" O.D. SPT Sampler; 3" O.D. California Sampler; Water Level noted during drilling; Water Level estimated at later time, as noted

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BORING LOG

PROPOSED RESIDENCE
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

JOB NO. G-5742 **DATE** 8/19/22 **PLATE** A2

BORING NO. B - 1

Logged By: AG   Date Drilled: 8/2/22
Drilled By: Geologic Drill Partners

Depth ft.	Elevation	USCS Code	Description	Sample Loc.	Sample No.	SPT Blow Counts	Water Content %	Other Tests/ Comments
		ML	- Continued from first page (Plate A2) Gray SILT with trace very fine sand, moist, hard			22,33,44 (N=77)	24.8	
25						50/6" (N=100)		No Recovery
			Depth of boring: 25.5' below ground surface (bgs) - Perched groundwater seepage first observed (wet soils) at 6' bgs. Also observed at 9' and 12.5' bgs. - After drill withdrawal (ATD) hole caved to 22.5-ft bgs and water at bottom of hole no higher than 22-ft bgs. - Drilling Method: Hollow-stem auger - Sampling Method: 2-inch-O.D. standard penetration sampler driven using a 140 lb. hammer with a 30-inch drop (cathead).					
30								
35								
40								
45								

LEGEND:
- 2" O.D. SPT Sampler
- 3" O.D. California Sampler
- Water Level noted during drilling
- Water Level estimated at later time, as noted

GEO Group Northwest, Inc.
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BORING LOG

PROPOSED RESIDENCE
3705 - 77TH PL SE
MERCER ISLAND, WASHINGTON

JOB NO. G-5742 **DATE** 8/19/22 **PLATE** A3