Geotechnical Report
Hart Residence
6025 77th Avenue SE
Mercer Island, Washington

Project 1865-1
July 6, 2017

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1.0 INTRODUCTION

The Galli Group performed a geotechnical investigation on the property located at 6025 77th Avenue SE, Mercer Island, Washington. The purpose of our investigation was to identify the subsurface soil conditions on the site and to provide recommendations for site development and foundation support of a new residence on the parcel.

This geotechnical report summarizes observations from our research and subsurface exploration performed for the above referenced property. It also presents our recommendations for the geotechnical design elements of the project.

2.0 PROJECT DESCRIPTION

The project site is located on the lower side of a slope that descends from 77th Avenue SE westerly toward Lake Washington (see Vicinity Map, Figure 1). The site is accessed from a shared driveway that descends from 77th Avenue southwesterly to the parcel. The topography of the parcel ascends moderately from an existing rock bulkhead on the shore of Lake Washington about 80 horizontal feet to the toe of a steep slope that ascends about 16 vertical feet in 16 horizontal feet to the relatively flat rear yard of the parcel above. The inclinations of the slope from the lake to the toe and from the toe to the top of the steep slope are about 13 percent and 100 percent respectively. The parcel is located within Environmentally Critical Areas due to geologic hazards identified in the Mercer Island Code as Landslide Hazard Areas, Erosion Hazard Areas, and Seismic Hazard Areas. The existing topography and site features are shown on Figure 2A, Site Survey.

We understand that the proposed development plan calls for removal of the existing single-story cottage on the lot and replacing it with a two-story single-family residence with full basement and detached garage. The residence will be excavated into the toe of the hillside creating cuts on the order of about 15 to 24 feet in overall height at the back (east end) of the residence. Based upon preliminary schematic drawings we estimate that the amount of excavation for the project will be on the order of about 1700 cubic yards. The proposed buildings can be supported on conventional spread footings provided that the footings are founded in native undisturbed dense glacial soil or compacted fill. Soldier pile shoring walls or soil nail walls will be required on the east side of the excavations. Temporary unsupported cuts may be utilized for the west, north, and south edges of the basement excavation provided they remain confined within the lot lines. A catchment wall might be recommended if the wall is located at the toe of the steep slope. The glacially consolidated soil will be able to stand temporarily in oversteepened cuts allowing for possible use of ecology blocks for temporary shoring or to contain cuts less than 6 feet in height with no backslope. Perched groundwater immediately above the dense underlying soil will likely result in the need for more aggressive waterproofing and subdrainage measures.

3.0 GEOLOGIC HAZARDS DISCUSSION

3.1 GEOLOGIC HAZARD AREAS AND CODE REQUIREMENTS

A review of the Uniform Land Development Code of the Mercer Island City Code (MICC) indicates that the site will be governed by Geologically Hazardous Areas regulations (Chapter 19.07.060). The site likely contains erosion hazard areas, and includes steep slopes that meet the definition of landslide hazard areas. The site also contains potential seismic hazard areas. Below we have discussed the elements that apply to the project site with reference to MICC Geologic Hazard requirements.

3.1.1 Erosion Hazard Areas

The MICC defines Erosion Hazard Areas as the following (MICC 19.16.010)

> "Those areas greater than 15 percent slope and subject to a severe risk of erosion due to wind, rain, water, slope and other natural agents including those soil types and/or areas identified by the U.S. Department of Agriculture's Natural Resources Conservation Service as having a "severe" or "very severe" rill and inter-rill erosion hazard."

The moderate slope between the lake and the toe of the steep slope is inclined at about 13 percent; the steep portion of the slope appears inclined at about 100 percent. The Soil Conservation Service maps the area as underlain by Kitsap silt loam and rates it a "severe" erosion hazard on slopes between 15 and 30 percent. Where left exposed and especially when exposed to concentrated discharges from stormwater culverts or pipes, the soil can present severe risks of erosion.

Because of these topographic and soil conditions a portion of the work area will be classified as an Erosion Hazard Area, that portion being the hillside ascending steeply to the east from the building footprint below. However, the site evidenced no signs of concentrated discharges on the slope, or surficial erosion that we could find. Existing surface water appears to be collected in a private storm drain system that directs water toward the toe of the slope and Lake Washington. Given the soil disturbance be confined largely within the existing developed area at the toe of the slope, we anticipate that conventional BMPs and maintaining a vegetative buffer between the building footprint and the lake should be adequate to prevent erosion, sediment transport, and slope incision during construction. Permanent vegetative cover and stormwater runoff control will adequately reduce long term risks of erosion.

3.1.2 Landslide Hazard Areas

The topography of the steep portion of the slope is mapped as inclined at about 100 percent or 1H:1V for a vertical distance of 16 feet. Geologic mapping indicates that the hillside is likely comprised of pre-Olympia glacial deposits. Based upon sampling from our subsurface exploration and site reconnaissance, the steep slope appears comprised of very dense silty SAND mantled by a loose unit of organic-rich topsoil and loose silty sand about 18 inches thick. Dense soil was encountered in our exploratory borings within 5 feet below existing grade.

Chapter 19.16.010 of the MICC defines "Landslide Hazard Areas" as follows:

> "Those areas subject to landslides based on a combination of geologic, topographic, and hydrologic factors, including:
>
> 1. Areas of historic failures;
> 2. Areas with all three of the following characteristics:
>   a. Slopes steeper than 15 percent; and
>   b. Hillsides intersecting geologic contacts with a relatively permeable sediment overlying a relatively impermeable sediment or bedrock; and
>   c. Springs or ground water seepage;
> 3. Areas that have shown evidence of past movement or that are underlain or covered by mass wastage debris from past movements;
> 4. Areas potentially unstable because of rapid stream incision and stream bank erosion; or
> 5. Steep Slope. Any slope of 40 percent or greater calculated by measuring the vertical rise over any 30-foot horizontal run."

Geologic maps of the area indicate that the site is likely underlain by Pre-Olympic glacial diamict (*Geologic Map of Mercer Island, Washington*, Troost et. al., 2006). A portion of the geologic map is provided on Figure 3, Geologic Map. These deposits generally appear comprised of silt, sand, clay, and gravel and appear very similar to glacial till. The unit appears very dense and hard after being consolidated by subsequent glacial advances.

We did not encounter any evidence of recent slope movement on the site. We did encounter seepage near the toe of the slope where the toe had been excavated. We interpreted the seepage as near surface water perched within the looser topsoil and flowing along the underlying dense soil unit. Because the hillside is inclined at more than 40 percent, the site would be classified as a "landslide hazard area." Mitigation measures for the slopes should address the control of stormwater runoff, address the potential of downslope creep by providing lateral support, and should address potential for shallow colluvial or "skin slides" from inadequate control of stormwater runoff or other influences such as broken irrigation or water services from upslope properties.

3.1.3 Seismic Hazard Area

Seismic hazard areas are defined as:

> "Seismic hazard areas are areas subject to severe risk of damage as a result of earthquake induced ground shaking, slope failure, settlement, soil liquefaction or surface faulting."
> (MICC 19.16.010)

The project site appears underlain by dense glacially consolidated soil at depth and does not appear to have a permanent shallow groundwater table except at the lake. This dense material does not present a significant risk of deep-seated slope movement or seismic liquefaction. Provided the new foundations are supported on native undisturbed soil, the risk of seismic-induced settlement does not appear significant. The house will be protected by the proposed foundation and the soldier pile walls. In our opinion, the improvements as designed will not introduce risk of damage due to seismic induced ground shaking.

The topography of the site presents potential risk of near surface slope movement under seismic induced ground shaking. The potential for deep seated slope failures does not appear significant due to the very dense glacial till forming the core of the hillside. Given prolonged ground shaking and wet antecedent conditions, the upper foot or two of the slope surface might slough and migrate downslope. To reduce the risk of adverse impacts of this type as well as the potential risk of near surface failures from surface water or irrigation water upslope, we have recommended constructing a catchment wall to reduce risk of damage to the residence.

In the report sections that follow we have described the site soil conditions and the subsurface conditions. The site appears underlain by very dense glacially consolidated sediment blanketed by about 2.5 feet of loose silty SAND topsoil and another 3 feet of medium dense silty SAND. The work area does not appear to present significant risk of deep seated slope failures. In our opinion, the project area does not present a significant risk of seismic liquefaction, landslides, or erosion if conventional Best Management Practices are followed during the repair, and our recommendations for design and mitigation are followed during project development.

3.2 SURFACE CONDITIONS AND GEOLOGY

The site ascends from the lake at about 13 percent toward the steep hillside. From that point, the hillside continues to ascend easterly at an inclination of about 1H:1V. Existing vegetation consists of lawn and mature landscaping on the lower portion of the lot and several large Cedars and understory growth of Salal, ferns, and blackberries on the steep hillside. Small facing rockeries armor toe of the slope east of the existing residence.

Geologic maps of the area indicate that site is likely underlain by pre-Olympia deposits including till-like soil (*Geologic Map of Seattle – a Progress Report,* Troost, Booth et al, 1985). Pre-Olympia deposits generally appear comprised of layers of sediment deposited many thousands of years ago prior to the most recent glacial advance 15,000 years ago. The material was subsequently carved and compacted by tons of advancing ice and then incised again by meltwater runoff as the glacier retreated northward. The unit tends to appear stable except where left unprotected by vegetation, subjected to concentrated stormwater runoff, or where groundwater emerges on steep slopes.

Existing drainage on the site consists of sheet flow from impervious surfaces and patios, seepage and surficial runoff from the hillside collected in yard drains and a shallow interceptor trench at the toe of the steep hillside. These drains and the downspouts appear to be collected in a 6-inch drain pipe that is routed toward Lake Washington.

3.3 SITE SOIL AND GROUNDWATER CONDITIONS

On June 12, 2017, we conducted a subsurface investigation on the site utilizing a track-mounted drill rig. We drilled three borings to depths varying from 11 feet to 26 feet. We identified the soil samples in the field and documented the density at periodic intervals as the drilling progressed. The results of our subsurface exploration are provided on the boring logs in the attached Appendix.

Based upon our subsurface exploration the site and the hillside appears underlain by very dense silty SAND with gravel and cobbles, interpreted as Pre-Olympia glacial diamicts or glacial till. The soil became medium dense to dense within the upper 3 feet and appeared very dense consistently at and below 5 feet depth. The dense soil was mantled by a layer of loose to medium dense silty SAND in all holes.

No permanent groundwater was observed in our test holes except B-2, where we encountered a wet seam at 16 feet depth forming water on the rods at 25 feet depth. We observed seepage along the contact with the topsoil and the underlying very dense silty SAND unit at the cut near the toe of the hillside. We interpreted this as near-surface water perched on top of the dense soil that migrates downslope through the loose topsoil and organic material.

3.4 EROSION AND SLOPE MITIGATION MEASURES

The dense core of the hillside and the proposed building footprint appears mantled by a thin layer of loose to medium dense silty SAND about 3 to 5 feet thick. Glacially consolidated sediment was observed in our borings at about 5 feet below grade. To prevent adversely impacting the slope, adjacent properties, or increasing the risk of erosion and sediment transport we recommend the following mitigation measures.

1. A shoring wall should be constructed against the toe of the hillside where excavation is planned. The wall should include a catchment wall with at least 5 feet of freeboard behind the wall to collect minor sloughing, shallow skin slides, or erosion from man-made causes or extreme runoff events if the entire slope height remains above the wall.
2. Conventional BMPs discussed in sections below should be employed during construction to control sediment transport and limit erosion.
3. Mass excavation and construction of the shoring wall must be accomplished during the drier season and avoided between October 1, and April 1. Once the shoring wall is

installed, additional excavation may occur during the wet season if a grading extension is obtained. Additional erosion control measures might be required.

4. Upon completion of the project, the exposed soils in the work area should be protected by a landscape plan that will permanently stabilize disturbed portions of the slope and the site against surficial erosion.

Provided the recommendations in our report below are followed during design and construction it is our opinion that the proposed repairs may safely be constructed on the project site and in keeping with the Mercer Island City Code regulations related to geologically hazardous areas.

3.5 RISK ASSESSMENT

Most of the proposed development activity occurs within the previously developed area of the existing building, decks, or flatwork. There will be additional excavation at the toe of the hillside requiring shoring. Provided the recommendations in our report are incorporated into the proposed design and construction the development activity will not adversely impact the adjacent properties or the geologic hazard areas.

In keeping with MICC code requirements, we provide the following statement of risk:
"The geologic hazard area will be modified, or the development has been designed so that the risk to the lot and adjacent property is eliminated or mitigated such that the site is determined to be safe."

More specifically, the proposed activity will provide a catchment wall to prevent soil from moving downslope to the residence. The residence will be supported on dense underlying soil and possibly the soldier pile wall. Best Management Practices will be incorporated into the construction erosion control and permanent site stabilization.

4.0 CONCLUSIONS AND RECOMMENDATIONS

The site appears underlain very dense silty SAND with gravel, mantled by a layer of loose silty SAND and topsoil about 3 to 5 feet thick. The hillside behind the proposed residence appears comprised of dense glacially consolidated till with a thin layer of silty SAND and topsoil about 2 feet thick. The proposed residence can be supported on conventional foundations constructed on the underlying undisturbed glacially consolidated soil. In the report sections that follow we have addressed the following geotechnical elements:

- The residence may be supported on conventional foundations. Care should be exercised to preserve the bearing conditions of the foundation soils by keeping the excavation dry and preventing traffic on the bearing surface.
- Temporary cuts may be oversteepened according to our recommendations but cuts at the toe of the steep slope should be shored.

- We recommend use of the shoring piles to construct a catchment wall on the east side of the house at the toe of the slope.

4.1 SITE GRADING AND EARTHWORK

Site development will result in a large excavation footprint exposing dense silty SAND with gravel. These soils will be difficult to compact when wet and disturbed by equipment traffic. Best Management Practices commonly observed should be employed during construction. We anticipate these will include the following:

1. A construction entrance near the existing garage should be provided for the site and to act as a staging area for construction materials. The entrance should be constructed from 4" – 6" quarry spalls placed over a woven geotextile fabric such as Mirafi 500X.
2. It is important to avoid tracking sediment onto the roadway and shared driveway. The contractor should monitor the tracking of sediment from the site and clean up as necessary. Sand and silt tracked from the site should be removed or cleaned by the contractor. If tracking onto the roadway becomes a problem, the contractor will need to construct a wheel-wash area on site.
3. A silt fence should be erected along the downslope limits of the construction area. A highly visible construction fence should be erected along the edge of areas intended to be preserved as vegetative buffers for stormwater runoff.
4. Stormwater runoff or seepage can be handled by a system of sumps and trenches within the excavation and discharged to a suitable dispersion area. During the wet season additional measures such as gravel sumps and wattles might be needed to avoid transport of sediment or turbid water from the site.
5. Spoils should be removed immediately from the site or protected during wet weather by use of plastic sheeting. Generally, stockpiles should not remain uncovered for more than 2 days during the wet season or 5 days during the drier summer months.
6. The contractor should monitor the performance of the erosion control measures and contact the geotechnical engineer if the TESC measures do not provide the intended function.

4.2 TEMPORARY EXCAVATIONS AND GRADING

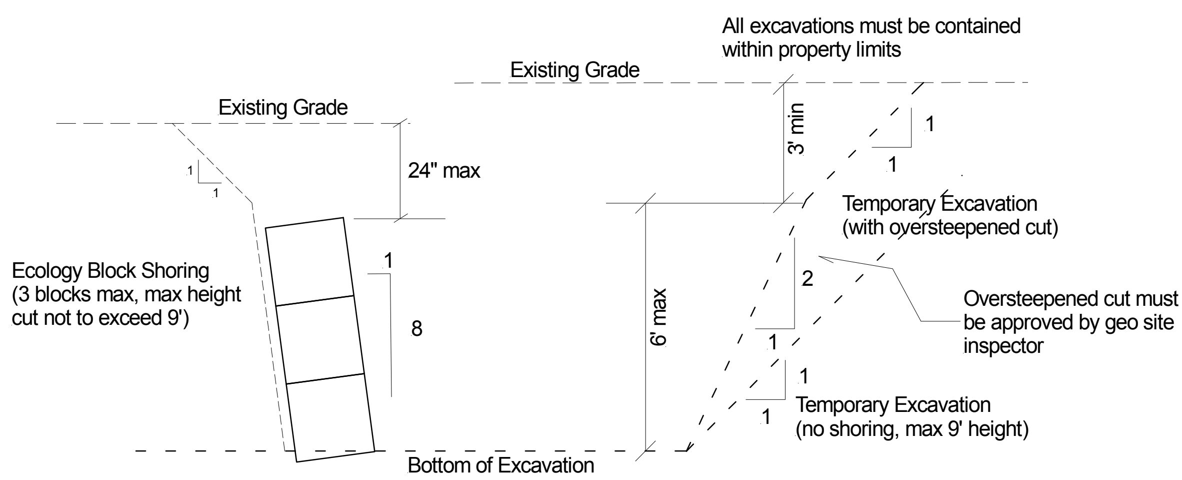
4.2.1 Fill Areas

The site soils will be unsuitable for fill beneath paved areas or structures. Generally, we anticipate that most of the site material will be removed from the site. The filled areas under proposed driveways or parking areas should consist of free draining sand and gravel with less than 7 percent passing the No. 200 sieve. It should be placed in thin lifts and compacted against horizontal surfaces in accordance with the criteria described in section 4.6 below. Permanent fill slopes should not exceed 2H:1V for ease of maintenance.

4.2.2 Unsupported Excavations

Temporary excavations should be shaped or benched to protect workers below. Generally, temporary construction cuts within site soils should be inclined no greater than 1H:1V (Horizontal to Vertical). The very dense consistency of the soil might allow for oversteepened cuts as shown in the illustration below where it is necessary to contain cuts within the property limits. This assumes that the excavation is undertaken during the dry summer months and that no seepage is encountered in the excavations. Oversteepened cuts will not be permitted if the excavation occurs during the wet season. The geotechnical engineer must monitor the excavation as it progresses where oversteepened cuts are planned. Once cuts are exposed, the soils must be protected during wet weather.

Where seepage is encountered or where the temporary cuts are not able to remain stable, or where cuts exceed 9 feet, additional measures such as ecology blocks or other temporary shoring might be required. Cuts may not be oversteepened without approval from the geotechnical engineer. Maintaining safe open excavations for workers and protecting the exposed cuts shall be the ongoing responsibility of the contractor.



Temporary Excavation Schematic

4.2.3 Temporary Shoring: Ecology Block Walls

If oversteepened excavations evidence signs of seepage or sloughing, temporary shoring should be used to protect the adjacent properties and the workers in the excavation. We anticipate that this can be accomplished by using ecology block shoring system for excavations up to 7 feet deep, with the obvious exception of the cut at the toe of the hillside. The following recommendations are provided for a temporary shoring system comprised of ecology blocks.

The following recommendations apply to the construction of an ecology block wall.

1. The ecology blocks should be placed as the excavation progresses. We recommend that no more than 20 horizontal feet of the excavation face be exposed at a time to place ecology block members. The blocks must be placed on native, undisturbed soil.
2. The temporary back cut shall be shaped and the blocks erected so that they have a face no steeper than 8V:1H. Cuts above the wall must not extend onto the adjacent property. The face of the base block shall be at least 18 inches from the proposed outside edge of the basement retaining wall footing to allow room to form the walls. A schematic is provided in Figure 5.
3. The blocks should be staggered and stacked so that they overlap the joints of the course immediately below.
4. Blocks may not be stacked any higher than three blocks without approval from the geotechnical engineer. Voids behind the wall should be filled with granular free draining material.
5. The geotechnical engineer should monitor the installation of the ecology block shoring system to verify that the anticipated conditions are encountered in the field and that these recommendations are followed.

4.2.4 Soldier Pile Shoring System

If property offsets are too tight or depth of excavation is too deep for unsupported cuts as described above, then measures such as soldier pile shoring system will be required. Soldier pile shoring must be installed at the toe of the hillside on the east side of the excavation.

The east side of the excavation for the garage and the residence should be shored. Shoring consists of temporary structural retaining elements that are designed to preserve stability of cuts and prevent movement of adjacent property, as well as to create safe conditions for workers within the excavation. This is of concern especially for the forming and stripping of subgrade walls, installation of perimeter footings drains, and waterproofing.

A variety of shoring methods are common in residential construction. These include ecology block shoring, drilled soldier pile walls, and soil nail walls. Ecology block walls are typically only appropriate for retained soil heights of about 7 feet maximum unless the soil is very dense.

We recommend shoring the east side of the excavation and other areas as needed to stabilize the toe of the hillside and create safe working conditions. An ecology block wall may be used to shore cuts that are not deeper than 7 feet where there is a 5-foot property setback, or 8 feet where there is no setback. We recommend using a soldier pile shoring system to support the toe of the steep hillside.

The cantilevered soldier pile shoring system should consist of drilled shafts, grouted and reinforced with steel beams. The vertical members shall consist of steel H or WF sections placed in a predrilled hole and then filled with lean mix concrete. As the site is excavated, the lean mix is chipped away and lagging is placed between the flanges of the H-beams and tight against the soil.

The drilled piers should be advanced to a depth sufficient to provide support for the anticipated excavation depth. The structural engineer should provide detailed calculations for piles after selecting precise locations, height, spacing, diameter of the hole, reinforcing elements, and material strength.

We recommend the following design parameters for the temporary shoring system:

- Based upon the consistency of the underlying soil encountered, we have recommended earth pressures as shown in Figure 6 for design of the shoring system with level backslopes. Forces above the base of the excavation should be considered to act on the spacing of the piles. Below the excavation, passive forces should be considered to act on 2 pile diameters. The bottom of the perimeter footings shall be the base of the excavation.
- Surcharge loads within 10 feet of the wall such as traffic loads, material stockpiles, equipment or structure loads should be included in the design of the wall. It does not appear that long term surcharge loads will be a factor on this site.
- A pressure equivalent to 80 percent of the design active pressure may be used to size the timber lagging, provided the pile spacing does not exceed 8 feet. Pressure treated timber lagging should be placed as excavation proceeds. Voids behind the lagging should be filled with free draining material such as pea gravel; CDF must be used where earth anchors are planned. Maximum height of the exposed cut should not exceed 4 feet before placing lagging. Lagging should be completed to the base of the excavation at the end of every working day.
- An active earth pressure of 32 pcf may be used for calculating the lateral earth pressure against the shoring walls. This assumes level backfill. An active pressure of 70 pcf should be used for inclined backslopes. The earth pressure should be assumed to act against the lagging for the spacing of the piles above the excavation. If the system is only temporary then the active pressure on the pile below the excavation may be ignored.
- A passive earth pressure of 460 pcf may be used to calculate the lateral earth pressure below the bottom of the excavation for drilled piers. This value is ultimate. A factor of

safety of 1.2 is often used for design. It should be assumed to act on 2 pile diameters. Minimum pile embedment should be at least 12 feet below the base of excavation.

- If anchors are required (for example on the catchment wall) then we recommend use of pressure grouted anchors installed at each pile location. The anchors should be designed assuming an ultimate capacity of 8 kips/foot of bonded length. The anchor should include a bond breaker in the no load zone as depicted in Figure 7. The anchor should be designed to resist 200 percent of the design values.
- There was evidence of possible seasonal perched water seepage in the upper few feet of soil. The lagging should have spaces between the boards sufficient to allow water to drain through the lagging into the excavation and to avoid hydrostatic pressure against the lagging.
- Temporary excavations for the remainder of the site should conform to requirements described in section 4.2.2 above. All excavations must be contained within the site.
- The Galli Group should monitor the installation of the shoring system.

The above parameters are shown on the attached earth pressure diagrams in Figure 6 and Figure 7. Figure 7 shows the earth pressure diagram for a cantilevered soldier pile shoring system and catchment wall with an inclined backslope.

The design of the shoring wall and the piles shall be the responsibility of the structural engineer, utilizing the design parameters provided in this report. Additional requirements related to concrete strength, grout, reinforcing elements, construction monitoring, and material specifications should be provided by the structural engineer.

4.2.5 Shoring Wall Tiebacks

Construction of the shoring system might require installation of tiebacks on the east side of the excavation for the soldier pile wall system. The tiebacks can be connected at each soldier pile. We recommend the following for design of the tiebacks:

1. Anchors must extend beyond the “no load” zone shown on Figure 7. The no load zone consists of the area immediately behind the shoring wall described as beginning at a point H/4 (where H is the height of the wall) beyond the base of the excavation and extending upward at 60 degrees away from the wall toward the existing grade. A minimum embedment of 15 feet beyond the no load zone is required.
2. The anchors should be inclined downward at 15 to 30 degrees from horizontal. The inclination can be determined by the structural engineer and architect as needed to avoid utility conflicts and other site-specific criteria. The anchors must not extend beyond the property line without obtaining easements.
3. Anchors can be either strand anchors or bars. Selection is up to the structural engineer in consultation with the contractor. The design load shall not exceed 60 percent of the specified minimum tensile strength (SMTS) of the steel members. Lock-off load shall not exceed 70 percent of the SMTS and the maximum test load shall not exceed 80 percent of the SMTS. The steel in the anchors shall be at least 150 ksi steel. If the selection of the anchor type is different

from the plan documents or requires a different anchor assembly than shown on the plans, the contractor must submit shop drawings to the structural engineer prior to installing the anchorage system.

4. The structural engineer shall provide the anchor head assembly detail including trumpet and connection to the pile. Web stiffeners may be required at the connection between the pile and the anchor head assembly.
5. All anchors should be designed to withstand at least 150 percent of the design load. A performance test shall be conducted on the first production anchor which might require additional capacity. Details regarding the performance test are provided in the section below. The remaining anchors should be proof tested.
6. A transfer load of 4000 psf per foot may be used for the design value of a pressure grouted anchor that is post grouted. The minimum diameter of the pressure grouted anchor shall be 5 inches.
7. The contractor should select the installation method and the method of grouting to develop the design loads indicated on the project plans. These capacities must be verified in accordance with the tieback testing program described below.

4.2.6 Tieback Testing

One of the production anchors shall be performance tested. The contractor shall be responsible for supplying the testing equipment. The geotechnical engineer shall monitor the testing. A performance test shall be conducted as follows:

Performance or Verification Test

1. An alignment load (AL) no more than 5 percent of the design load shall be applied to the anchor and the displacement equipment zeroed thereafter.
2. The anchors shall be loaded in 25 percent increments of the design load with the incremental movement of the anchor recorded at each loading cycle. Following each incremental load, the anchor load is reduced to the alignment load.
3. The anchor shall be reloaded in increasing increments until the test load is reached.
4. The performance test load shall be 200 percent of the design load. The load must be held for ten minutes with movements recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. The geotechnical engineer must monitor the performance testing.
5. Reduce the load to the design load and lock off the anchor.

Each additional anchor not subjected to a performance test shall be proof tested. The proof test shall be conducted as follows:

Proof Test

1. An initial alignment load shall be applied to the anchor and gauges adjusted to zero. The alignment load shall not exceed 5 percent of the design load (DL.)

2. Successively apply and record total movements for the following load increments: 0.25DL, 0.50DL, 0.75DL, 1.00DL, 1.20DL, and 1.33DL. The test load shall be 133 percent of the design load.
3. Hold the test load for 10 minutes and record total movement.
4. At the discretion of the geotechnical engineer, he or she might require some of the anchors to be unloaded to the alignment load to record residual movement.
5. Reduce the load on anchors that pass the acceptance criteria to the lock-off load.

Acceptance Criteria
Each performance tested anchor shall be considered acceptable if it passes the following:

- Creep of the anchor shall not exceed 1mm (0.045 inches) between 1 and 10 minutes. If it does not pass this test then the creep test shall be extended to 60 minutes. The anchor shall be considered acceptable if the total movement over the interval from 6 to 60 minutes does not exceed 2 mm, or 2mm per log cycle of time.

Lock-off Load and Lift-off Testing
The anchors shall be locked off at 80 percent of the design load. After the load has been transferred from the jack to the anchorage, the contractor should perform a lift-off test to verify the magnitude of the loaded anchor. The anchor shall be gradually stressed until the wedge plate lifts off the bearing plate. The load measured during lift-off should be within five percent of the lock-off load. If this criterion is not met, the anchor load should be adjusted and the lift-off test repeated.

4.2.7 Permanent Soldier Pile Walls and Catchment Walls
We recommend constructing the uphill wall as a catchment wall to reduce potential negative impacts of earth slides or debris in the event of a failure from upslope or upslope utilities. The catchment wall should be extended a minimum of 5 feet above finish grade on the eastern or uphill side of the residence. An additional impact load of 100 pcf should be used to design the wall for the potential earth or debris slide. We have a provided an earth pressure diagram for use in design of the catchment wall. It seems likely given the additional loading that the wall will require installation of anchors to resist the lateral forces. Design earth pressures and values are provided in Figure 7, Catchment Wall Earth Pressures.

4.2.8 Monitoring of Shoring System Performance

The contractor shall provide a monitoring program to evaluate the performance of the shoring system and the impact of the excavation on adjacent property. We recommend that horizontal and vertical survey points be established on the shoring piles. A licensed surveyor should establish the coordinates of the points and read the points at the following times: 1) prior to commencing excavation, 2) every other week during excavation activity, and 3) prior to commencing backfilling or construction of the retaining walls. If deflection of the piles exceeds ½ inch then more frequent readings might be required. The results of the performance monitoring should be supplied to the structural engineer and geotechnical site inspector in tabular form.

In addition, if applicable, we recommend documenting existing conditions of the adjacent building walls and footings by digital camera prior to commencing excavation and again prior to backfilling or construction of the walls. This helps protect all parties involved in the process.

4.2.9 Soil Nail Wall Alternative

Due to the anticipated height of the shoring wall on the east side of the excavation and the potential for tiered walls on the slope, it might be advantageous to consider a soil nail wall for stabilizing the east side of the project site. A soil nail wall consists of shorter earth anchors installed in a grid pattern along the face of the proposed wall. The wall is shotcreted as the excavation proceeds forming a temporary restraint system and then a permanent concrete wall caps the temporary wall. The advantages of the soil nail concept on this project relative to the soldier pile wall include shorten length of anchors, potential to have a tiered wall system that doesn't transfer all the loads to the lowermost wall, and it avoids the use of the large steel sections required for a soldier pile wall. The disadvantages include the number of anchors required for the system.

The following parameters may be used for design of the soil nail wall. We typically recommend contacting someone with both a structural engineering license and a geotechnical license to design the soil nail wall. Ground Support LLC is a firm that provides this type of structural and geotechnical design support for soil nail walls or hybrid walls.

We recommend using the following parameters for evaluating the feasibility of the soil nail wall system. These values may be adjusted by the geotechnical engineer based upon the results of our subsurface investigation.

Design Parameter	Value
Soil Unit Weight, γ	120 pcf
Internal Friction, φ	36
Cohesion, c	300 to 400 psf
FOS Pullout	2.0

4.3 LATERAL EARTH PRESSURES AND RETAINING WALLS

The proposed residence incorporates retaining elements. These include possible braced walls or cantilevered retaining walls. Site development might also include concrete walls that are constructed as landscape features or to protect walkways or grade changes.

The table below provides soil parameters used in the analyses for this project.

Table 1

Soil design parameters used in determination of lateral earth pressures

Soil Type	Unit Weight γ, pcf	Passive Resistance (EFW)	Active Earth Pressure (EFW)	At-Rest Earth Pressure (EFW)	Inclined Slope Condition
Dense silty SAND	120	460 pcf	36 pcf	53 pcf	70 pcf
M. dense silty SAND	120	460 pcf	36 pcf	53 pcf	70 pcf
Compacted Fill	125	300 pcf	35 pcf	53 pcf	NA

(EFW) = Equivalent Fluid Unit Weight in pounds per cubic foot

For the conventional concrete walls, we recommend the following:

1. Excavation for the walls must be accomplished in accordance with the recommendations supplied in section 4.2 above. The excavation should be benched so that compaction of backfill may take place against horizontal soil surfaces.
2. All walls must be supported on native undisturbed soil. We recommend using an allowable bearing capacity of 4500 psf for design of footings supported on the dense very silty SAND with gravel.
3. The walls should be designed to resist an active earth pressure equivalent to 32 pcf per foot of retained soil height. This assumes level drained backfill. Wall backslopes must not exceed 4H:1V. Walls with backslopes should be designed using 70 pcf active earth pressure.
4. For braced walls or restrained walls, a lateral at-rest earth pressure of 53 pcf should be used for design of the walls.
5. A uniform load equivalent to 8H where H is the retained height of the wall, may be used to calculate the lateral load contributed by seismic induced ground acceleration.
6. Lateral resistance for basement retaining walls may be calculated at 300 pcf per foot of overburden. The contribution from the uppermost 12 inches of soil should be ignored except for basement walls or where compacted structural fill is placed beneath a slab. A coefficient of friction of 0.3 may be used for design.

7. A backwall drainage system must be supplied for all newly constructed walls. The drainage system shall include at a minimum, a 4-inch perforated, smooth-walled pipe, enveloped in ¾” to 1½” washed gravel, and wrapped in Mirafi 140N filter fabric for separation from adjacent soils. On this site, we recommend installing sheet drains against the new concrete for the basement walls. The composite drain shall be Enkadrain, Delta Drain, or equivalent approved by the engineer.
8. Backfill placed behind the wall should be placed and compacted in thin enough lifts to achieve the compaction criteria listed in the report sections below.
9. The geotechnical engineer should verify that the drainage system, bearing conditions, and backfill compaction are in accordance with the report recommendations.

4.4 FOUNDATIONS

Foundations for the residence will consist of spread footings supported on the undisturbed silty SAND unit. We anticipate that this unit will be encountered at depths on the order of 3 feet near the proposed structures. It appears likely that the basement walls will be supported on the very dense silty SAND unit or compacted backfill.

4.4.1 Seismic Design Parameters

The site is underlain by glacially consolidated silty SAND with gravel. Based upon the density of the underlying soil we do not think seismic liquefaction or lateral spreading will be a significant risk factor to site development. Seismic liquefaction typically occurs in loose to medium dense clean sands. We recommend using site Class D for this project site.
The site is mapped within 2 miles of the Seattle Fault Zone.

The site appears underlain by very dense glacially consolidated SAND with gravel and glacially consolidated SILT. Based upon these site factors seismic liquefaction does not appear to be a significant concern. The risk of seismically induced slope movement does not represent a significant threat to the project site.

The following seismic design parameters may be used for the site.

Table 2
Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec (g)	Spectral Acceleration at 1.0 sec (g)	Site Coefficients		Design Spectral Response Parameters	
	S_S	S_1	F_a	F_v	S_{DS}	S_{D1}
D	1.465	0.563	1	1.5	0.977	0.563

4.4.2 Spread Footings and Wall Footings

Column or wall loads within the excavation may be supported on spread footings. For spread footings within the excavation we recommend the following:

1. An allowable bearing pressure of 4,500 psf may be used for footings bearing on undisturbed dense glacial soil. This may be increased by 1/3 for temporary loads such as wind loads or seismic loads.
2. The passive resistance for the footings may be calculated at 350 psf in the native soil.
3. A coefficient of friction of 0.3 may be used for the interface between the bottom of the footing and the soil.
4. The footing area must be free from loose or wet soil prior to placing reinforcing or pouring concrete. The geotechnical engineer should verify the bearing.
5. Perimeter footing drains should be provided around all footings and discharge to an approved storm drain.
6. Deck or porch footings should bear on native undisturbed soils to avoid settlement. These can be provided by pouring a footing and bringing the support to grade using a concrete pier.

4.5 SLAB-ON-GRADE FLOORS

Reinforced concrete floors which are beneath structures ringed with perimeter footings or walls can be supported on a 6-inch drain rock layer placed over properly prepared subgrade or granular fill soils. For slabs on grade, we recommend that granular import be placed as soon as the subgrade is prepared to protect the subgrade soil.

The following recommendations are provided for slabs constructed on the unyielding subgrade surface:

1. A four-inch layer of clean crushed rock (3\4" to 1 1/4" clean crushed rock works well) should be placed over the structural fill to provide a positive capillary moisture break and uniform slab support.
2. If the subgrade or crushed rock will be subject to equipment traffic we recommend placing a layer of 6-ounce non-woven geotechnical fabric such as Mirafi 160N to protect the subgrade and provide separation for the drainage zone beneath the slab.
3. An impermeable membrane, such as 10-mil plastic sheeting, should be placed over the crushed rock layer to further prevent upward migration of moisture vapor into and through the concrete slab.
4. To protect the membrane and provide more uniform curing of the slab, it is advisable to place one to two inches of chip rock on top of the membrane. The rock should be moistened prior to placing concrete.
5. Where insulation is required along the perimeter, the insulation may replace the 2-inch sand or chip rock layer.

We recommend that the contractor use deformed reinforcing steel for slab reinforcement rather than welded wire fabric. A minimum reinforcement scheme would be #3 or # 4 bars, 18 inches on center, both ways. Fibermesh may be used to help decrease drying shrinkage cracks, however it is not a replacement for structural reinforcing. All slabs tend to crack, therefore jointing at approximately 8 to 10 foot intervals, both directions, should significantly decrease random cracking in the open areas.

4.6 BACKFILL AND COMPACTION

Site soils are not suitable for backfill behind walls or under slabs. Imported fill soils or site soils used as backfill behind walls and under slabs should be moisture conditioned to within 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and compacted to at least 92 percent of the maximum dry density, as determined using ASTM D1557 (Modified Proctor). The 92 percent compaction criteria should apply to any material intended to support pavement or intended as backfill behind walls. If structures are planned to be supported on structural fill the compaction criteria should be 95 percent of the Modified Proctor. All structural fill areas supporting structures should be density tested to verify compaction criteria is achieved. In areas not constructed as fill slopes or not intended to support pavement or structures, fill material should be placed in loose lifts less than 12 inches in thickness and compacted to at least 90 percent of the maximum dry density.

4.7 PERMANENT EROSION CONTROL

Following backfill of the retaining walls, installation of the subsurface utilities and drainage system, and completion of the flat work, the site must be permanently stabilized. All exposed soils on site must either be covered with a thick layer of mulch (3 – 4 inches) that is incorporated into the final landscaping plan or vegetated with lawn or other groundcover. Additional requirements for soil amendment may be specified by the landscape designer.

5.0 ADDITIONAL SERVICES AND LIMITATIONS

5.1 ADDITIONAL SERVICES

Additional services by the geotechnical engineer are important to help insure that report recommendations are correctly interpreted in final project design and to help verify compliance with project specifications during the construction process. For this project, we anticipate additional services may include the following:

1. Review final design and construction drawings for conformance with geotechnical recommendations.
2. Monitor erosion control measures.
3. Monitor temporary excavations and evaluate need for temporary shoring.
4. Monitor installation of Ecology Block and/or Soldier Pile shoring system
5. Verify soil bearing for walls and footings.
6. Monitor installation of perimeter subdrains.
7. Monitor compaction of backfill and drainage behind the retaining walls.
8. Provide periodic construction field reports, as requested by the client and required by the building department.

We would provide these additional services on a time-and-expense basis in accordance with our Standard Fee Schedule and General Conditions already in place for this project.

5.2 LIMITATIONS

This geotechnical investigation was planned and conducted in accordance with generally accepted engineering standards practiced presently within this geographic area. Geotechnical investigations performed by these standards reveal with reasonable regularity soils that are representative of subsurface conditions throughout the site under consideration. Recommendations contained in this report are based upon the assumption that soil conditions encountered in explorations are representative of actual conditions throughout the building site. However, inconsistent conditions can occur between exploratory borings or test pits and not be detected by a geotechnical study. If, during construction or subsequent exploration, subsurface or slope conditions are encountered which differ from those anticipated based upon results of this investigation, The Galli Group should be notified so that we can review and revise our recommendations where necessary. If conditions change prior to the proposed construction, we should be consulted so that we may alter our recommendations if necessary.

This report is prepared for the exclusive use of the owner or the owner's consultants for specific application on this project at this site. Copies of this report should be made available to the design team, and should be included with the contract drawings issued to the contractor. Our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions on the site and should not be applied to neighboring sites. No warranty, expressed or implied is made. We recommend that geotechnical observation and testing be provided during the construction phases to verify that the recommendations provided in this report are incorporated into the actual construction.

Respectfully submitted,

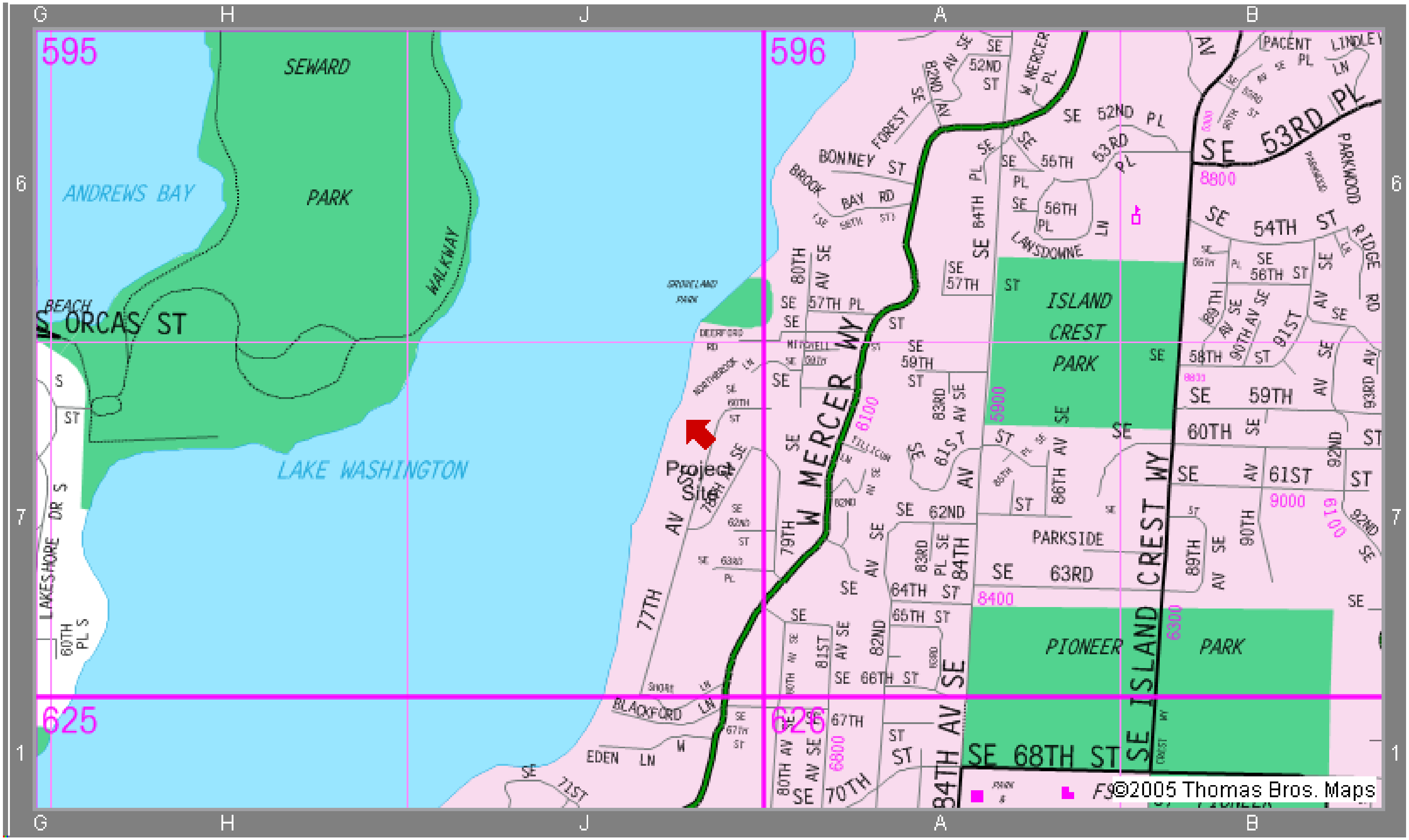
THE GALLI GROUP

Paul L. Stoltenberg, P.E.
Project Geotechnical Engineer

07-06-2017

Appendix

Logs of Exploratory Borings



Ref: Thomas Guide, 2005

The Galli Group
PO Box 30759
Seattle, WA 98113

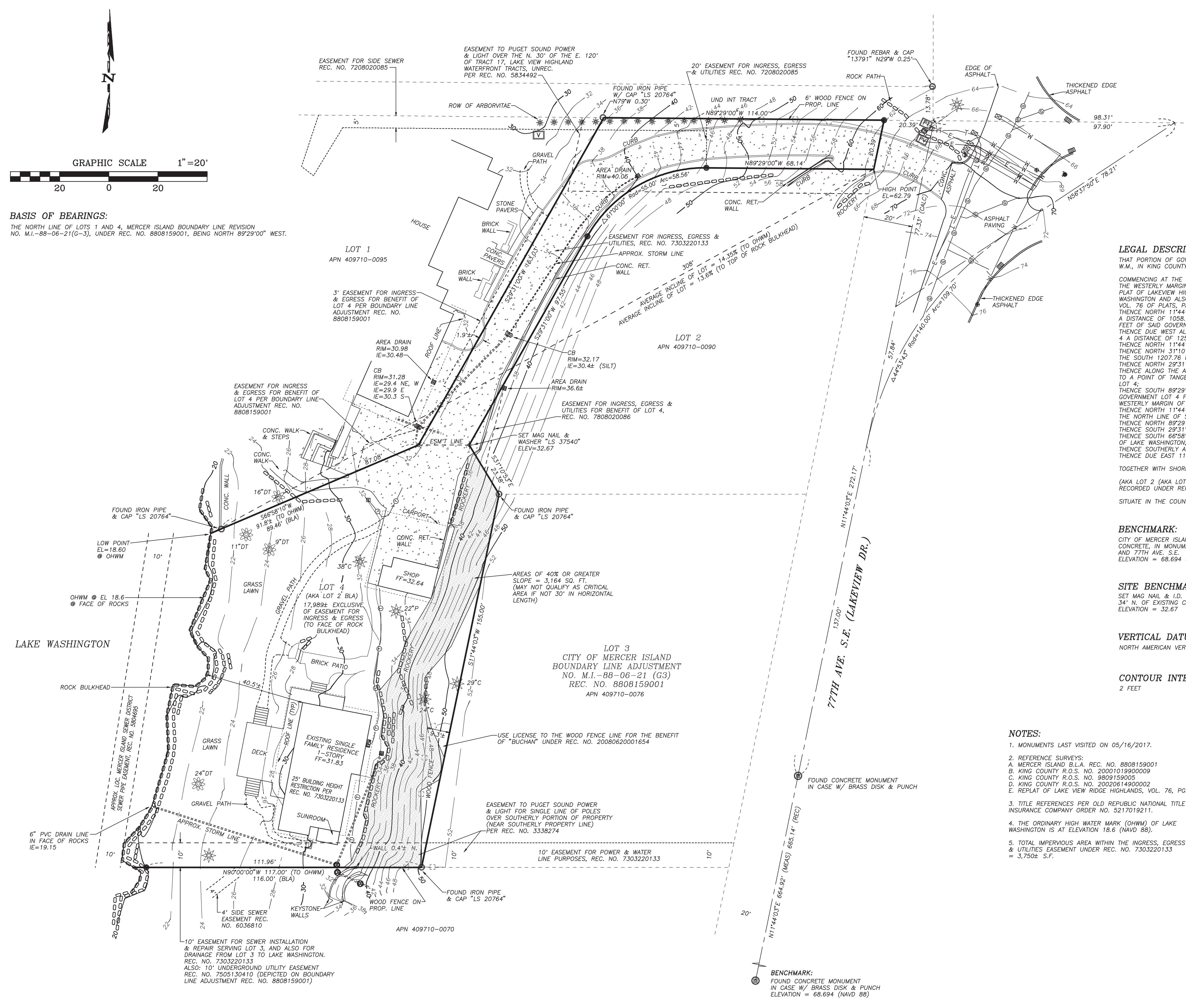
HART RESIDENCE
6025 77TH Avenue SE
Mercer Island, Washington

VICINITY MAP
FIGURE 1

TOPOGRAPHICAL SITE SURVEY

LOCATED IN THE S.W. 1/4, OF THE S.E. 1/4,
OF SECTION 24, TOWNSHIP 24 NORTH, RANGE 4 EAST, W.M.,
KING COUNTY, WASHINGTON

SITE SURVEY
Figure 2A



BASIS OF BEARINGS:

THE NORTH LINE OF LOTS 1 AND 4, MERCER ISLAND BOUNDARY LINE REVISION NO. M.I.-88-06-21(G-3), UNDER REC. NO. 8808159001, BEING NORTH 89°29'00" WEST.

LEGAL DESCRIPTION:

THAT PORTION OF GOVERNMENT LOT 4, SECTION 24, TOWNSHIP 24 NORTH, RANGE 4 EAST, W.M., IN KING COUNTY, WASHINGTON, MORE PARTICULARLY DESCRIBED AS FOLLOWS:

COMMENCING AT THE INTERSECTION OF THE SOUTH LINE OF SAID GOVERNMENT LOT 4 WITH THE WESTERLY MARGIN OF LAKE VIEW DRIVE (NOW KNOWN AS 77TH AVE. S.E.) ACCORDING TO PLAT OF LAKEVIEW HIGHLANDS, RECORDED IN VOLUME 33 OF PLATS, PAGE 34, IN KING COUNTY, WASHINGTON AND ALSO ACCORDING TO THE REPLAT OF LAKEVIEW HIGHLANDS, RECORDED IN VOL. 76 OF PLATS, PAGES 41 AND 42, IN KING COUNTY, WASHINGTON;
THENCE NORTH 11°44'03" EAST ALONG THE WESTERLY MARGIN OF SAID 77TH AVE. S.E. FOR A DISTANCE OF 1058.11 FEET TO AN INTERSECTION WITH THE NORTH LINE OF THE SOUTH 1036 FEET OF SAID GOVERNMENT LOT 4;
THENCE DUE WEST ALONG A LINE PARALLEL WITH THE SOUTH LINE OF SAID GOVERNMENT LOT 4 A DISTANCE OF 125 FEET TO THE TRUE POINT OF BEGINNING;
THENCE NORTH 11°44'03" EAST 155 FEET;
THENCE NORTH 31°10'53" WEST 23.38 FEET TO AN INTERSECTION WITH THE NORTH LINE OF THE SOUTH 1207.76 FEET OF SAID GOVERNMENT LOT 4;
THENCE NORTH 29°31'00" EAST 97.55 FEET TO A POINT OF CURVATURE TO THE RIGHT;
THENCE ALONG THE ARC OF A CURVE HAVING A RADIUS OF 55 FEET FOR A DISTANCE OF 58.56 FEET TO A POINT OF TANGENCY ON THE SOUTH LINE OF THE NORTH 20 FEET OF SAID GOVERNMENT LOT 4;
THENCE SOUTH 89°29'00" EAST ALONG A LINE PARALLEL TO THE NORTH LINE OF SAID GOVERNMENT LOT 4 FOR A DISTANCE OF 68.14 FEET TO AN INTERSECTION WITH THE SAID WESTERLY MARGIN OF 77TH AVE. S.E.;
THENCE NORTH 11°44'03" EAST ALONG SAID ROAD MARGIN 20.39 FEET, MORE OR LESS, TO THE NORTH LINE OF SAID GOVERNMENT LOT 4;
THENCE NORTH 89°29'00" WEST A DISTANCE OF 114.00 FEET;
THENCE SOUTH 29°31'00" WEST A DISTANCE OF 153.03 FEET;
THENCE SOUTH 66°58'10" WEST A DISTANCE OF 89.5 FEET, MORE OR LESS, TO THE SHORELINE OF LAKE WASHINGTON;
THENCE SOUTHERLY ALONG SAID SHORELINE TO A POINT WEST OF THE POINT OF BEGINNING;
THENCE DUE EAST 116 FEET, MORE OR LESS, TO THE TRUE POINT OF BEGINNING.

TOGETHER WITH SHORELANDS OF THE SECOND CLASS ADJOINING THERETO.

(AKA LOT 2 (AKA LOT 4) CITY OF MERCER ISLAND BOUNDARY LINE ADJUSTMENT MI-88-06-21 (G-3), RECORDED UNDER RECORDING NO. 8808159001.)

SITUATE IN THE COUNTY OF KING, STATE OF WASHINGTON

BENCHMARK:

CITY OF MERCER ISLAND BENCHMARK NO. 3113, BEING A 3/8" BRASS PLUG IN CONCRETE, IN MONUMENT CASE, AT THE INTERSECTION OF 78TH AVE. S.E AND 77TH AVE. S.E
ELEVATION = 68.694 (NAVD 88)

SITE BENCHMARK:

SET MAG NAIL & I.D. WASHER "LS 37540" IN CONCRETE DRIVEWAY, APPROX. 34' N. OF EXISTING CARPORT AND 4.0' W. OF AN EXISTING CONCRETE WALL.
ELEVATION = 32.67

VERTICAL DATUM:

NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88)

CONTOUR INTERVAL:

2 FEET

NOTES:

1. MONUMENTS LAST VISITED ON 05/16/2017.

2. REFERENCE SURVEYS:
A. MERCER ISLAND B.L.A. REC. NO. 8808159001
B. KING COUNTY R.O.S. NO. 20001019900009
C. KING COUNTY R.O.S. NO. 9809159005
D. KING COUNTY R.O.S. NO. 20020614900002
E. REPLAT OF LAKE VIEW RIDGE HIGHLANDS, VOL. 76, PG. 41

3. TITLE REFERENCES PER OLD REPUBLIC NATIONAL TITLE INSURANCE COMPANY ORDER NO. 5217019211.

4. THE ORDINARY HIGH WATER MARK (OHWM) OF LAKE WASHINGTON IS AT ELEVATION 18.6 (NAVD 88).

5. TOTAL IMPERVIOUS AREA WITHIN THE INGRESS, EGRESS & UTILITIES EASEMENT UNDER REC. NO. 7303220133 = 3,750± S.F.

LEGEND:

- FOUND CONCRETE MONUMENT IN CASE
- SET 1/2" REBAR & CAP "CASCADE LS 37540"
- FOUND IRON PIPE OR REBAR & CAP
- SET MAG NAIL & WASHER "LS 37540"
- CATCH BASIN
- AREA DRAIN
- STORM DRAIN CLEANOUT
- FIRE HYDRANT
- WATER VALVE
- WATER METER
- TELEPHONE RISER
- CTV CABLE TV BOX
- V UTILITY VAULT
- EPM ELECTRIC POWER METER
- PV POWER VAULT
- GAS VALVE
- CONIFER TREE
- SMALL CONIFER TREE
- DECIDUOUS TREE
- DT DECIDUOUS TREE
- C CEDAR
- P PINE
- ROCKERY
- W WATER LINE
- E UNDERGROUND POWER
- G GAS LINE
- T TELEPHONE OR COMM LINE
- WOOD FENCE

SURVEYOR'S CERTIFICATE

THIS MAP CORRECTLY REPRESENTS A SURVEY MADE BY ME OR UNDER MY DIRECTION IN CONFORMANCE WITH THE REQUIREMENTS OF THE SURVEY RECORDING ACT AT THE REQUEST OF GREG & KRISTIN HART IN Jun. 2017

P.L.S. CERTIFICATE NO. 37540

JEFFREY ALLEN OTTERSON
STATE OF WASHINGTON
37540
REGISTERED PROFESSIONAL LAND SURVEYOR

TOPOGRAPHICAL SITE SURVEY FOR GREG AND KRISTIN HART

CASCADE LAND SURVEYING
Complete Land Surveying Services
23257 SE 284th ST, Maple Valley, Washington 98038
(253) 820-4016 or (360) 897-1017
1-(800) 728-4993 (toll free) Email: jeff@cascadels.com

DRAWN BY: JAO CHECKED BY: JAO SHEET: 1 of 1 DATE: Wed., Jun. 7, 2017
JOB NO.: 2017-013 SCALE: 1"=20'

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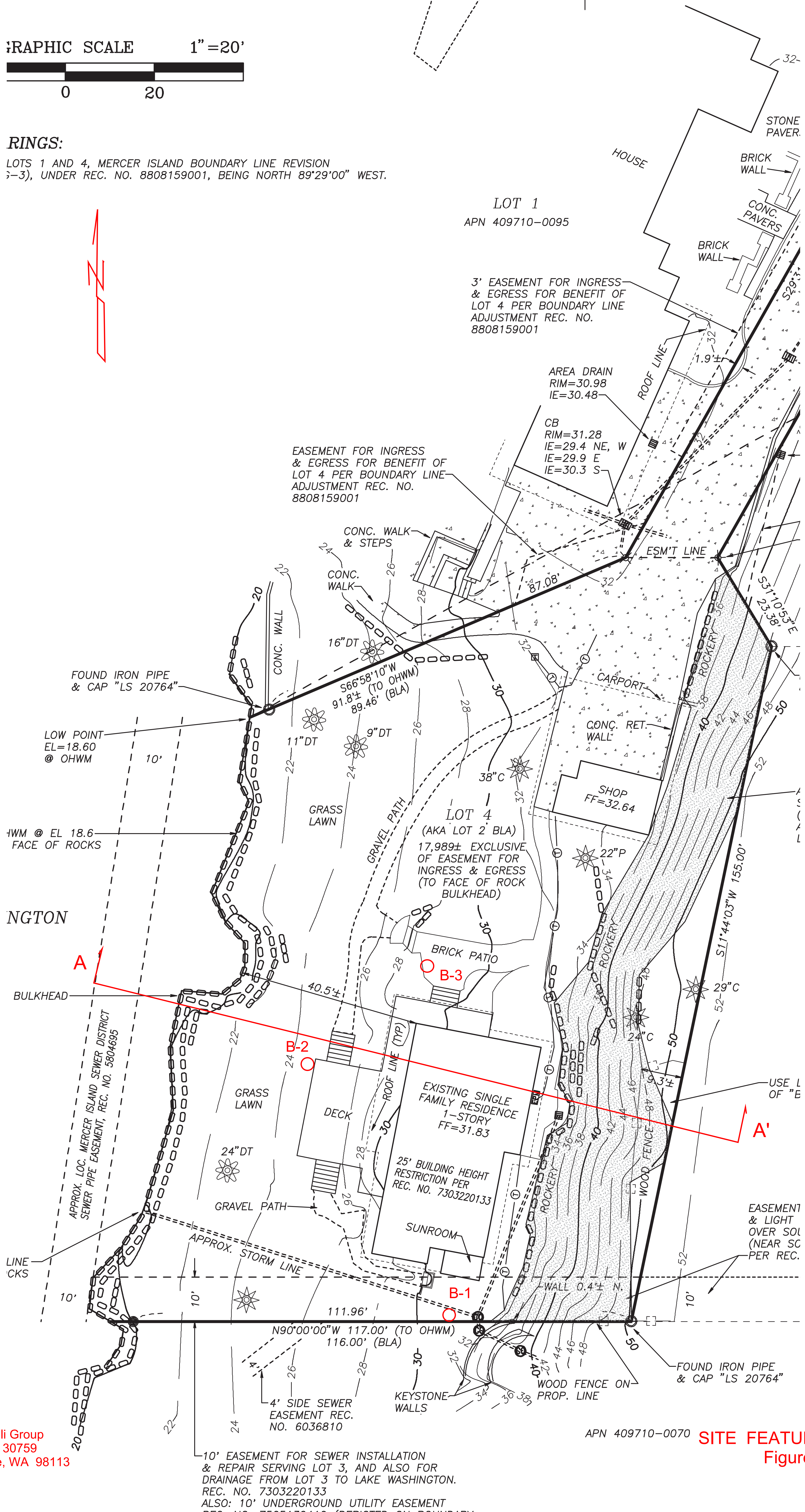

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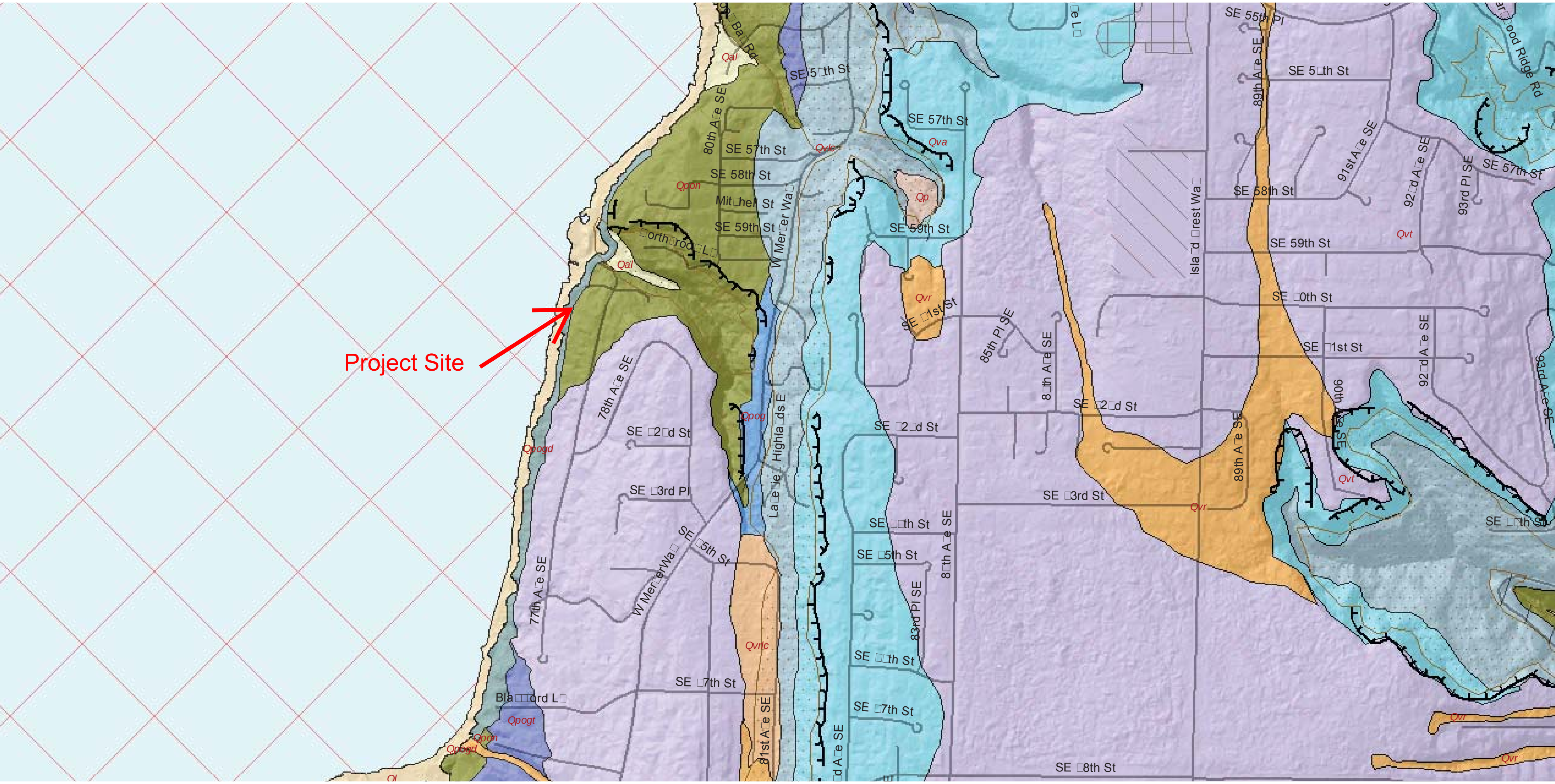
SITE FEATURES
Figure 2B

	LEGEND
Ql	Lake deposits
Qpogd	Pre-Olympia glacial deposits
Qpon	Pre-Olympia non-glacial deposits
Qal	Alluvial Deposits
Qvt	Vashon Glacial Till

Ref: Geologic Map of Mercer Island, Troost & Wisher, 2006



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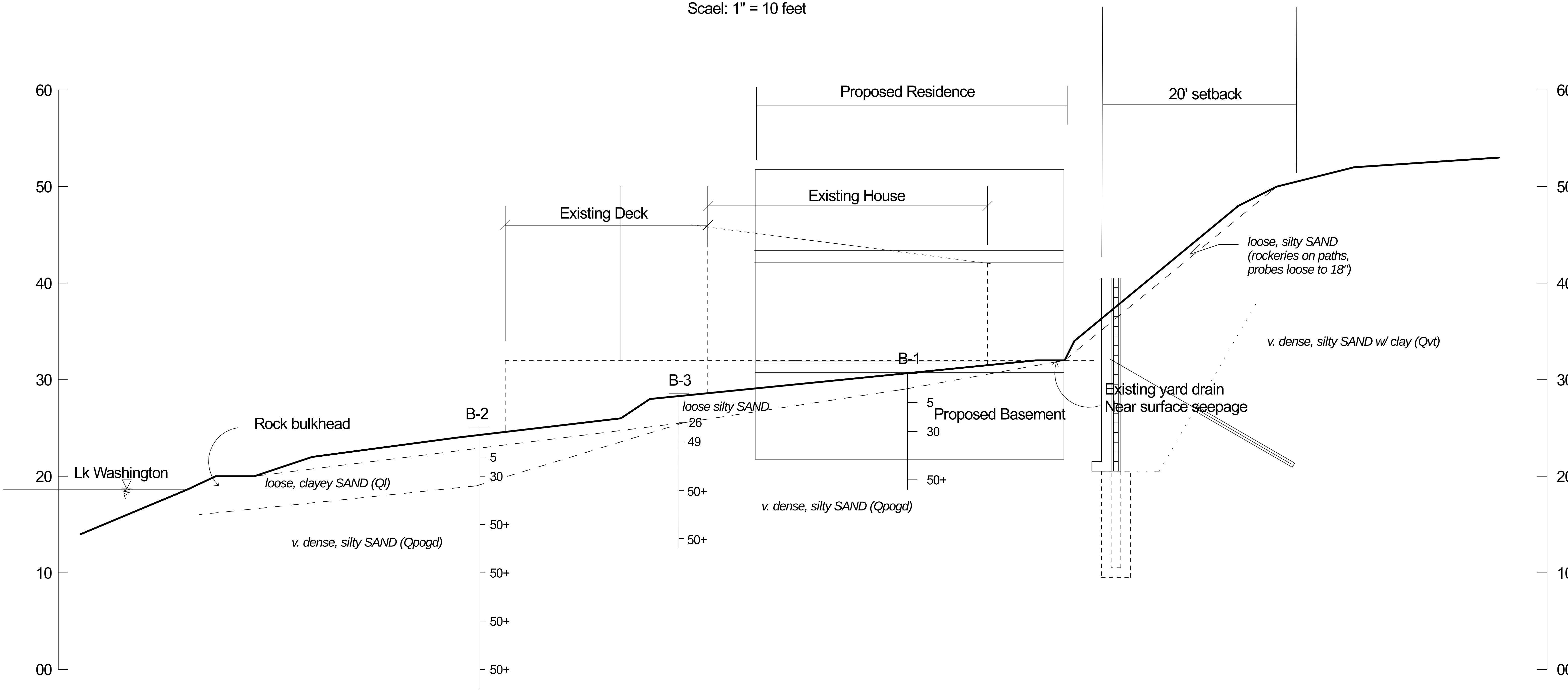
Mercer Island, Washington

GEOLOGIC MAP
Figure 3

Section A - A'
Scael: 1" = 10 feet



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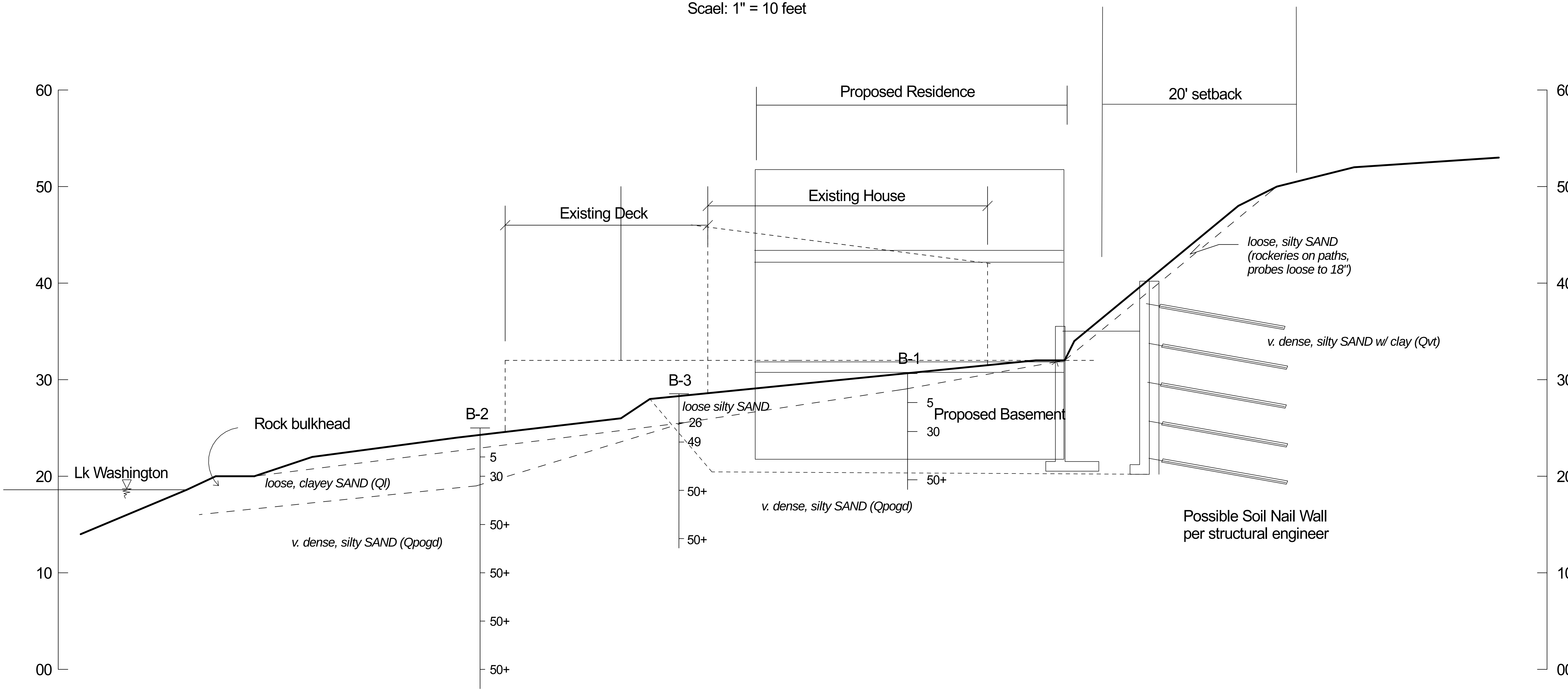
Generalized Subsurface Stratigraphy
Figure 4A

Section A - A'

Scael: 1" = 10 feet



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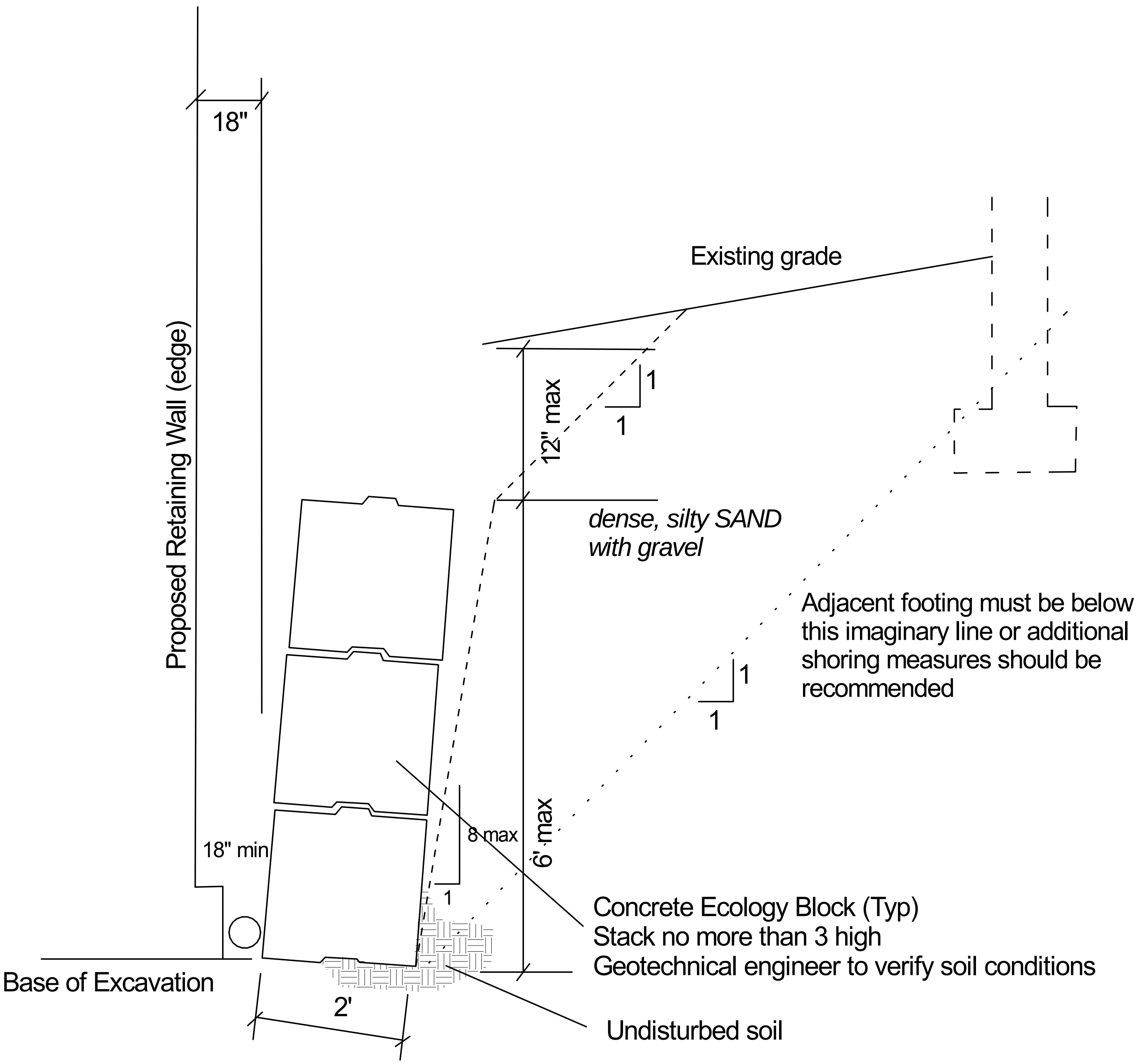
Generalized Subsurface Stratigraphy
Figure 4B

Ecology Block Shoring Schematic

No Scale



Note:
Adjacent footing elevations must be confirmed
prior to final drawings and construction.
Contractor to confirm in field.

Stack ecology blocks so that adjoining ends of upper
blocks are not immediately above adjoining ends of
lower blocks

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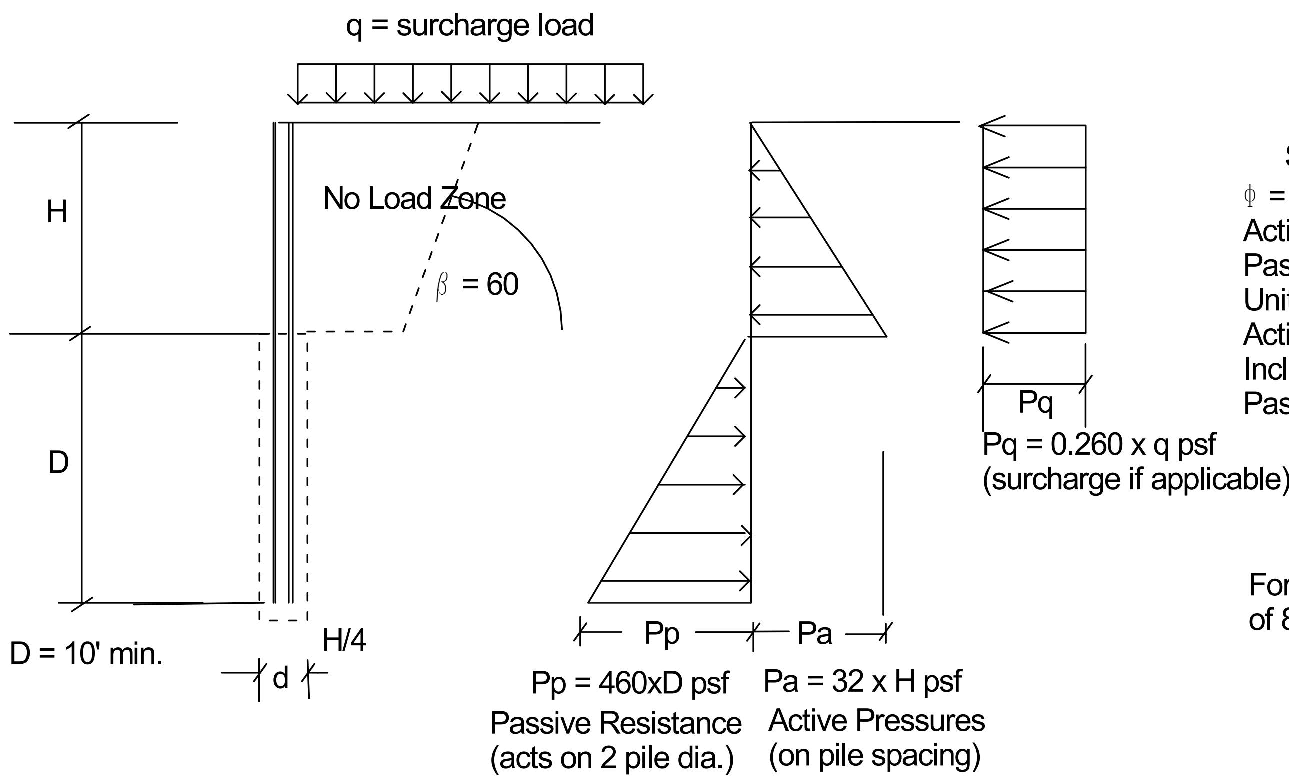
206-525-5097

6025 77th Avenue SE
Mercer Island, Washington

Figure 5

Earth Pressure Diagrams
Cantilevered Soldier Pile Shoring System
(Level Backslope)



Soil Design Parameters
ϕ = 36 degrees
Active Ka = 0.26
Passive Kp = 3.85
Unit weight, γ = 120 pcf
Active Pressure = 32 pcf (EFW)
Inclined backslope = 40 pcf
Passive Pressure = 460 pcf (EFW) x 2d

For permanent walls add seismic force
of 8H uniform pressure against retaining wall.

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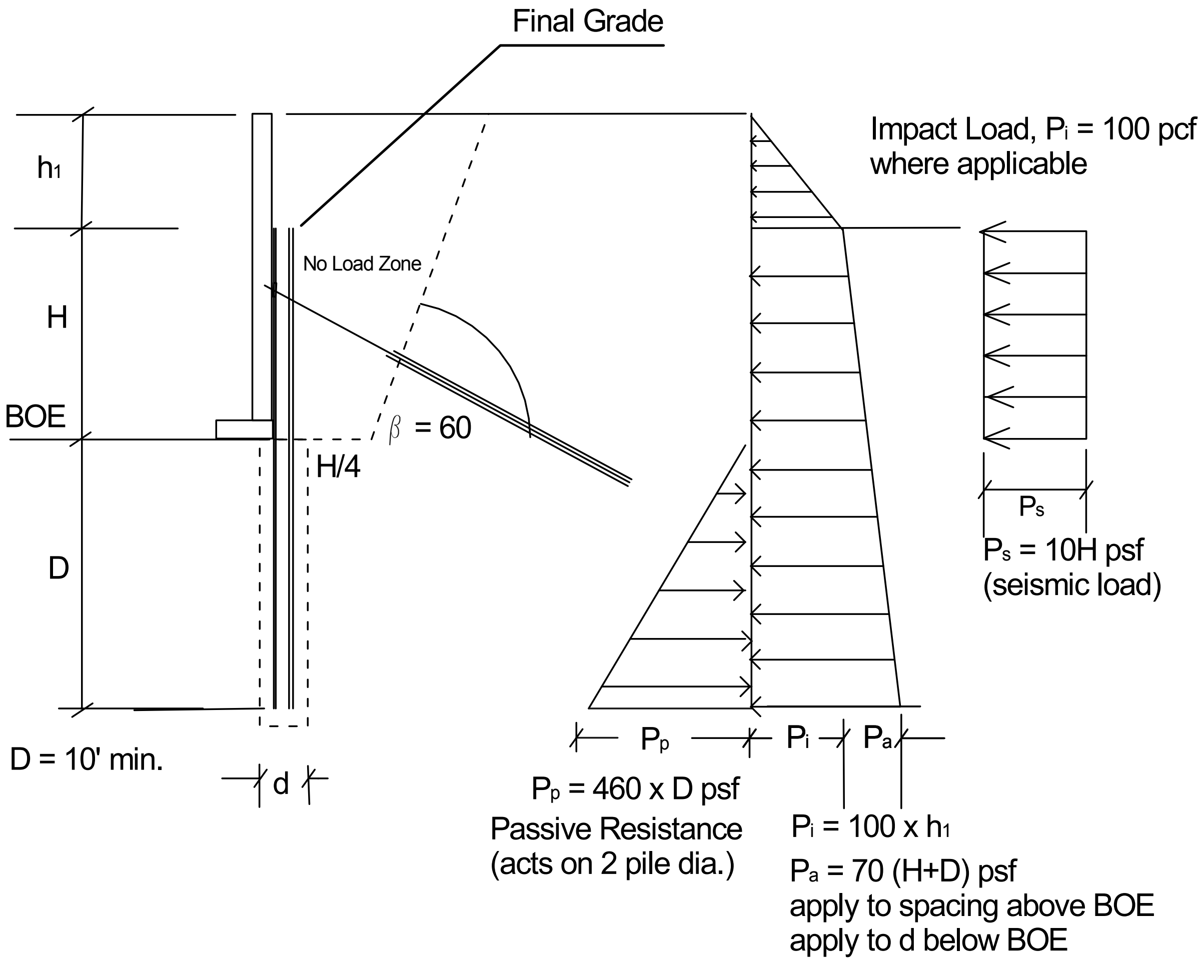
Mercer Island, Washington

Earth Pressure Diagrams
Figure 6

Earth Pressure Diagrams
Permanent Soldier Pile Catchment Wall
(Inclined Backslope Condition)



Soil Design Parameters

ϕ = 36 degrees
Active Ka = 0.283
Passive Kp = 3.53
Unit weight, γ = 120 pcf
Active Pressure = 32 pcf (EFW)
Active Inclined Slope = 40 pcf
Passive Pressure = 460 pcf (EFW) x 2d
(values ultimate)

For permanent walls use seismic load or impact loads, whichever controls. Impact load controls on east side of residence.

<u>Pressure Grouted Tieback Design Parameters</u>
Ulitmate Capacity = 8 kips/ft bonded length
Unbonded length in no load zone = 14' min.
Number and spacing per structural engineer
Steel elements per structural engineer
Performance spec per structural engineer
Apply FOS = 2 for design values

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Catchment Wall Earth Pressures
Figure 7

Appendix A: Logs of Exploratory Borings and Test Pits

Unified Soil Classification System; from American Society for Testing and Materials, 1985

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO.200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO.4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS MORE THAN 50% PASSES NO.200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

FOR SAND AND GRAVELS

DENSITY	STANDARD PENETRATION RESISTANCE (SPT) BLOWS/FT.
VERY LOOSE	0 – 4
LOOSE	4 – 10
MEDIUM DENSE	10 – 30
DENSE	30 – 50
VERY DENSE	> 50

FOR SILTS AND CLAYS

CONSISTENCY	STANDARD PENETRATION RESISTANCE (SPT) BLOWS/FT.
VERY SOFT	0 – 2
SOFT	2 - 4
MEDIUM STIFF	4 – 8
STIFF	8 - 16
VERY STIFF	16 – 32
HARD	> 32

Project 1954 - 01

Boring Log B-1

6-13-2017

Log	Soil Description	Depth	Sample	SPT	Comments
	Loose, silty SAND; wet (Topsoil)				Surface soil wet from seepage
	Very dense, gray, silty SAND (GLACIAL TILL) -gray, silty SAND w/ gravel; moist		S1	3-3-2	Perched water seems to seep into the hole after drilling. Water at 5'4" after one hour.
	-gray, silty SAND w/ gravel; moist	5'	S2	3-3-27	
	-gray, silty SAND w/ gravel; moist clean SAND in tip of sampler	10'	S3	25-44-50	
	Bottom of boring at 11.5' depth (refusal) Seepage into the hole after drilling Drilled with mini-track drill rig	15'			
		20'			
		25'			
		30'			
		35'			
		40'			
		45'			

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6025 77th Ave SE
Mercer Island, Washington

Figure A2

Project 1954 - 01

Boring Log B-2

6-13-2017

Soil Description	Depth	Sample	SPT	Comments
Loose, silty SAND; moist (Topsoil)				
Loose, brown, SAND w/ fines (Lake Deposit) -brn, f-m SAND w/ silt; moist; topsoil in upper 4"		S1	3-3-2	
-brn, clayey SAND w/ organics; wet (lake deposit) gray silty SAND w/ gravel; moist (till-like)	5'	S2	3-3-27	
Very dense, gray, silty SAND (GLACIAL TILL)				
-gray, silty SAND w/ gravel; moist	10'	S3	25-44-50	
	15'	S4	40-50/5"	
Very dense, interbedded silty SAND and SAND -gray, silty SAND w/ gravel; moist 3" brn, weathered SAND at 16.2'; wet				Sand zones appear to seep into the hole after drilling
Hard, gray, SILT (Glaciolacustrine) -gray, fine sandy SILT; moist	20'	S5	16-30-37	
-gray, SILT; moist (water on rods at 25')	25'	S6	14-28-40	
Bottom of boring at 26.5' depth Seepage into the hole after drilling Drilled with mini-track drill rig				
	30'			
	35'			
	40'			
	45'			

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Figure A-3

Project 1954 - 01

Boring Log B-3

6-13-2017

Log	Soil Description	Depth	Sample	SPT	Comments
	brick pavers in sand bed				
	Med dense, interbedded silty SAND and SAND; moist				
	-4" brn, silty SAND w/ organics (topsoil 6" brn, med. SAND; wet; 6" brn, silty SAND w/ gravel		S1	3-5-21	
	Very dense, interbedded SAND and silty SANDw/ gravel	5'			
	-12" brn, silty SAND w/ gravel; moist 6" brn, med. SAND w/ silt; weathered; wet		S2	25-27-22	Sand zones appear to seep into the hole after drilling
	-gray/brn SAND w/ silt; some gravel; wet gray in tip	10'	S3	16-24-33	
	-6" silty SAND w/ gravel; moist 12" brn, f-m SAND w/ silt; moist to wet	15'	S4	20-50/5"	
	Bottom of boring at 16.5' depth Seepage into the hole after drilling Drilled with mini-track drill rig				
		20'			
		25'			
		30'			
		35'			
		40'			
		45'			

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Figure A-4